Volume III
Hydrologic Analysis and Flow Control BMPs
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## Volume III

### Hydrologic Analysis and Flow Control BMPs

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Chapter 1 - Introduction

1.1 Purpose of this Volume

Best Management Practices (BMPs) are schedules of activities, prohibitions of practices, maintenance procedures, managerial practices, or structural features that prevent or reduce adverse impacts to waters of Washington State. As described in Volume I of this stormwater manual, BMPs for long-term management of stormwater at developed sites can be divided into three main categories:

- BMPs addressing the volume and timing of stormwater flows,
- BMPs addressing prevention of pollution from potential sources.
- BMPs addressing treatment of runoff to remove sediment and other pollutants.

This volume of the stormwater manual focuses mainly on the first category. It presents techniques of hydrologic analysis, and BMPs related to management of the amount and timing of stormwater flows from developed sites. The purpose of this volume is to provide guidance on the estimation and control of stormwater runoff quantity.

BMPs for preventing pollution of stormwater runoff and for treating contaminated runoff are presented in Volumes IV and V, respectively.

1.2 Content and Organization of this Volume

Volume III of the stormwater manual contains three chapters. Chapter 1 serves as an introduction. Chapter 2 reviews methods of hydrologic analysis, covers the use of hydrograph methods for designing BMPs, and provides an overview of various computerized modeling methods and analysis of closed depressions. Chapter 3 describes flow control BMPs and provides design specifications for roof downspouts and detention facilities. It also provides design considerations of infiltration facilities for flow control.

This volume includes three appendices. Appendix A has isopluvial maps for western Washington. Appendix B has information and assumptions on the Western Washington Hydrology Model (WWHM). Appendix C includes detailed information concerning how to represent various Low Impact Development (LID) techniques in continuous runoff models so that the models predict lower surface runoff rates and volumes.

Design considerations for conveyance systems are included in Appendix I-F. Stormwater conveyance systems shall be designed in accordance with Chapter 5 of the City of Olympia Engineering Design and Development Standards (EDDS).

1.3 How to Use this Volume

Volume I should be consulted to determine Core Requirements for flow management (e.g. Core Requirements #4, #5, #7, and #8 in Chapter 2 of Volume I). After the Core Requirements have been determined, this volume should be consulted to design flow management facilities. These facilities can then be included in Drainage Control Plans (see Volume I, Chapter 3).
Chapter 2 - Hydrologic Analysis

The broad definition of hydrology is “the science which studies the source, properties, distribution, and laws of water as it moves through its closed cycle on the earth (the hydrologic cycle).” As applied in this manual, however, the term “hydrologic analysis” addresses and quantifies only a small portion of this cycle. That portion is the relatively short-term movement of water over the land resulting directly from precipitation and called surface water or stormwater runoff. Localized and long-term ground water movement must also be of concern, but generally only as this relates to the movement of water on or near the surface, such as stream base flow or infiltration systems.

The purpose of this chapter is to define the minimum computational standards required, to outline how these may be applied, and to reference where more complete details may be found, should they be needed. This chapter also provides details on the hydrologic design process; that is, what are the steps required in conducting a hydrologic analysis, including flow routing.

2.1 Minimum Computational Standards

The minimum computational standards depend on the type of information required and the size of the drainage area to be analyzed, as follows:

1. For the purpose of designing most types of runoff treatment BMPs, a calibrated continuous simulation hydrologic model based on the EPA’s HSPF (Hydrologic Simulation Program-Fortran) program, or an approved equivalent model, must be used to calculate runoff and determine the water quality design flow rates and volumes. Models must use the most recent recorded 15-minute precipitation data (time series) for Thurston County. Synthetically generated precipitation time series are not allowed when using any continuous simulation model.

For the purpose of designing wetpool treatment facilities, there are two acceptable methods: an approved continuous runoff model to estimate the 91st percentile, 24-hour runoff volume, or the NRCS (Natural Resources Conservation Service) curve number method to determine a water quality design storm volume. The water quality design storm volume is the amount of runoff predicted from the 6-month, 24-hour storm.

For the purpose of designing flow control BMPs, a calibrated continuous simulation hydrologic model, based on the EPA’s HSPF, must be used. Models must use the most recent 15-minute precipitation data for Thurston County.

The circumstances under which different methodologies apply are summarized below in Table 2.1.1.
2. If a basin plan is being prepared, then a hydrologic analysis should be performed using a continuous simulation model such as the EPA's HSPF model, the EPA's Stormwater Management Model (SWMM), or an equivalent model as approved by the City of Olympia (City).

   a. Significant progress has been made in the development and availability of HSPF-based continuous runoff models for Western Washington. The Department of Ecology has coordinated the development of the Western Washington Hydrology Model (WWHM). It uses rainfall/runoff relationships developed for specific basins in the Puget Sound region to all parts of western Washington. Where field monitoring establishes basin-specific rainfall/runoff parameter calibrations, those can be entered into the model, superseding the default input parameters. Use of other continuous simulation models should receive prior approval from Ecology.

   b. Where large master-planned developments are proposed, the City may require a basin-specific calibration of HSPF rather than use of the default parameters in the above-referenced models. Ecology suggests such basin-specific calibrations should be considered for projects that will occupy more than 320 acres.

2.1.1 Discussion of Hydrologic Analysis Methods Used for Designing BMPs

This section provides a discussion of the methodologies to be used for calculating stormwater runoff from a project site. It includes a discussion of estimating stormwater runoff with single event models, such as the Santa Barbara Unit Hydrograph (SBUH), versus continuous simulation models.
Single Event and Continuous Simulation Model

A continuous simulation model has considerable advantages over the single event-based methods such as the SCSUH, SBUH, or the Rational Method. HSPF is a continuous simulation model that is capable of simulating a wider range of hydrologic responses than the single event models such as the SBUH method. Single event models cannot take into account storm events that may occur just before or just after the single event (the design storm) that is under consideration. In addition, the runoff files generated by the HSPF models are the result of a considerable effort to introduce local parameters and actual rainfall data into the model and therefore produce better estimations of runoff than the SCSUH, SBUH, or Rational methods.

Ecology has developed a continuous simulation hydrologic model (WWHM) based on the HSPF for use in western Washington (see Section 2.2). Continuous rainfall records/data files have been obtained and appropriate adjustment factors were developed as input to HSPF. Input algorithms (referred to as IMPLND and PERLND) have been developed for a number of watershed basins in King, Pierce, Snohomish, and Thurston counties. These rainfall files and model algorithms are used in the HSPF in western Washington. Local counties and cities are encouraged to develop basin-specific calibrations of HSPF that can be input into the WWHM. However, until such a calibration is developed for a specific basin, the input data mentioned above must be used throughout western Washington.

Concerns with SBUH

A summary of the concerns with SBUH and other single event models is in order.

While SBUH may give acceptable estimates of total runoff volumes, it tends to overestimate peak flow rates from pervious areas because it cannot adequately model subsurface flow (which is a dominant flow regime for pre-development conditions in western Washington basins). One reason SBUH overestimates the peak flow rate for pervious areas is that the actual time of concentration is typically greater than what is assumed. Better flow estimates could be made if a longer time of concentration was used. This would change both the peak flow rate (i.e., it would be lower) and the shape of the hydrograph (i.e., peak occurs somewhat later) such that the hydrograph would better reflect actual pre-developed conditions.

Another reason for overestimation of the runoff is the curve numbers (CN) in the 1992 Manual. These curve numbers were developed by US-Natural Resources Conservation Service (NRCS), formerly the Soil Conservation Service (SCS) and published as the Western Washington Supplemental Curve Numbers. These CN values are typically higher than the standard CN values published in Technical Release 55, June 1986. In 1995, the NRCS recalled the use of the western Washington CNs for floodplain management and found that the standard CNs better describe the hydrologic conditions for rainfall events in western Washington. However, based on runoff comparisons with the KCRTS better estimates of runoff are obtained when using the western Washington CNs for the developed areas such as parks, lawns, and other landscaped areas. Accordingly, the CNs in this manual (see Table 2.3.2) are changed to those in the Technical Release 55 except for the open spaces category for the developed areas which include, lawn, parks, golf courses, cemeteries, and landscaped areas. For these areas, the western Washington...
CNs are used. These changes are intended to provide better runoff estimates using the SBUH method.

Another major weakness of SBUH is that it is used to model a 24-hour storm event, which is too short to model longer-term storms in western Washington. The use of a longer-term (e.g. 3- or 7-day storm) is perhaps better suited for western Washington.

Related to the last concern is the fact that single event approaches, such as SBUH, assume that flow control ponds are empty at the start of the design event. Continuous runoff models are able to simulate a continuous long-term record of runoff and soil moisture conditions. They simulate situations where ponds are not empty when another rain event begins.

Finally, single event models do not allow for estimation and analyses of flow durations nor water level fluctuations. Flow durations are necessary for discharges to streams. Estimates of water level fluctuations are necessary for discharges to wetlands and for tracking influent water elevations and bypass quantities to properly size treatment facilities.

2.2 Western Washington Hydrology Model

This section summarizes the assumptions made in creating the western Washington Hydrology Model (WWHM) and discusses limitations of the model. Appendix III-B contains more information on the assumptions and on WWHM. However, since the first version of WWHM was developed and released to public in 2001, the WWHM program has gone through several upgrades incorporating new features and capabilities including low impact development (LID modeling capability. WWHM users should periodically check Ecology’s WWHM web site for the latest releases of WWHM, user manual, and any supplemental instructions. The web address for WWHM is: www.ecy.wa.gov/programs/wq/stormwater/wwhmtraining/index.html.

2.2.1 Limitations to the WWHM

Ecology created WWHM for the specific purpose of sizing stormwater control facilities for new developments in western Washington. WWHM can be used for a range of conditions and developments; however, certain limitations are inherent in this software.

WWHM uses the EPA HSPF software program to do all of the rainfall-runoff and routing computations. Therefore, HSPF limitations are included in WWHM. For example, backwater or tailwater control situations are not explicitly modeled by HSPF. This is also true in WWHM.

Earlier versions of WWHM, WWHM1 and WWHM2 had limited routing capabilities. The routing capabilities of WWHM3 and WWHM2012 have improved and the user can input multiple stormwater control facilities and runoff is routed through them. If the proposed development site involves routing through a natural lake or wetland in addition to multiple stormwater control facilities, WWHM2012 can be used to do the routing computations and additional analysis.

2.2.2 Assumptions made in creating the WWHM

Precipitation Data:

- WWHM uses long-term (over 50 years) precipitation data to simulate the potential impacts of land use development in western Washington. A minimum period of 20 years is required to simulate enough peak flow events to produce accurate flow frequency results.
• For Thurston County, WWHM uses data from 12 separate precipitation stations throughout the county.
• The precipitation stations represent rainfall at elevations below 1500 feet. WWHM does not include snowfall and snowmelt.
• The primary source for precipitation data is National Weather Service stations, although Thurston County-managed stations are also included.

The base computational time step used in WWHM is 15-minutes using Thurston County precipitation data. The 15-minute time step better represents the temporal variability of actual precipitation. This data is used in WWHM2012 computations to generate runoff hydrograph. The computations include generating design flows and volumes for sizing water quality treatment facilities. The 15-minute water quality design flows are used for the design of water quality treatment facilities that are expected to have a hydraulic residence time of less than one hour.

**Precipitation Multiplication Factors:**

• WWHM uses precipitation multiplication factors to increase or decrease recorded precipitation data to better represent local rainfall conditions.
• The factors are based on the ratio of the 24-hour, 25-year rainfall intensities for the representative precipitation gage and the surrounding area represented by that gage’s record.
• The factors have been placed in the WWHM database and linked to each county’s map. They will be transparent to the general user, however, the advanced user has the ability to change the coefficient for a specific site where justified and approved by the reviewing jurisdiction. Changes made by the user will be recorded in the WWHM output. By default, WWHM does not allow the precipitation multiplication factor to go below 0.8 or above 2.

**Pan Evaporation Data:**

• WWHM uses pan evaporation coefficients to compute the actual evapotranspiration potential (AET) for a site, based on the potential evapotranspiration (PET) and available moisture supply. AET accounts for the precipitation that returns to the atmosphere without becoming runoff.
• The pan evaporation coefficients have been placed in the WWHM database and linked to each county’s map. They will be transparent to the general user. Advanced users have the ability to change the coefficient for a specific site where justified and approved by the reviewing jurisdiction. These changes will be recorded in the WWHM output.

**Soil Data:**

• WWHM uses three predominant soil types to represent the soils of western Washington: till, outwash, and saturated.
• The user determines actual local soil conditions for the specific development planned and inputs that data into WWHM. The user inputs the number of acres of outwash (A/B), till (C/D), and saturated (wetland) soils for the site conditions.
• Additional soils will be included in WWHM if appropriate HSPF parameter values are found to represent other major soil groups.

**Vegetation Data:**

• WWHM represents the vegetation of western Washington with three predominant vegetation categories: forest, pasture, and lawn (also known as grass).
• The predevelopment land conditions are generally assumed as forest (the default condition), however, the user has the option of specifying pasture if there is documented evidence that pasture vegetation was native to the pre-development site. In highly urbanized basins (see Core
Requirement #7 in Volume I, Chapter 2), it is possible to use the existing land cover as the pre-developed land condition.

Development Land Use Data:

- Development land use data are used to represent the type of development planned for the site and are used to determine the appropriate size of the required stormwater mitigation facility.
- Earlier versions of WWHM included a Standard residential development option, which made specific assumptions about the amount of impervious area per lot and its division between driveways and rooftops. Streets and sidewalk areas were input separately. Ecology had selected a standard impervious area of 4200 square feet per residential lot, with 1000 square feet of that as driveway, walkways, and patio area, and the remainder as rooftop area.
- The more recent versions of WWHM (e.g., WWHM3 through WWHM2012) no longer have the Standard residential development category. Users can use the above land use assumptions for a modeling runoff from Standard residential development or, where better land use information is available, use that information to model and estimate runoff from the residential development.
- WWHM distinguishes between effective impervious area and non-effective impervious area in calculating total impervious area.
- Credits are given for infiltration and dispersion of roof runoff and for use of porous pavement for driveway areas. WWHM2012 includes LID modeling features that calculate credits directly in the model. Refer to the WWHM2012 user manual for modeling instructions for LIDs.
- Forest and pasture vegetation areas are only appropriate for separate undeveloped parcels dedicated as open space, wetland buffer, or park within the total area of the development. Development areas must only be designated as forest or pasture where legal restrictions can be documented that protect these areas from future disturbances.
- WWHM can model bypassing a portion of the runoff from the development area around a stormwater detention facility and/or having off-site inflow enter the development area.

Application of WWHM in Redevelopments Projects

- Redevelopment requirements may allow, for some portions of the redevelopment project area, the predeveloped condition to be modeled as the existing condition rather than forested or pasture condition. For instance, where the replaced impervious areas do not have to be served by updated flow control facilities because area or cost thresholds in Section 2.4.2 of Volume I are not exceeded.

Pervious and Impervious Land Categories (PERLND and IMPLND parameter values):

- In WWHM (and HSPF) pervious land categories are represented by PERLNDs; impervious land categories by IMPLNDs.
- WWHM provides over 20 unique PERLND parameters that describe various hydrologic factors that influence runoff and 4 parameters to represent IMPLND.

These values are based on regional parameter values developed by the U.S. Geological Survey for watersheds in western Washington (Dinicola, 1990) plus additional HSPF modeling work conducted by AQUA TERRA Consultants.

Surface runoff and interflow are computed based on the PERLND and IMPLND parameter values. Ground water flow can also be computed and added to the total runoff from a development if there is a reason to believe that ground water would be surfacing (such as where there is a cut in a slope). However, the default condition in WWHM assumes that no ground water flow from small catchments reaches the surface to become runoff.
2.2.3 Guidance for flow-related standards

Flow-related standards are used to determine whether or not a proposed stormwater facility will provide a sufficient level of mitigation for the additional runoff from land development. There are three flow-related standards stated in Volume I of this Manual: Core Requirement #5 – On-site Stormwater Management; Core Requirement #7 - Flow Control; and Core Requirement #8 - Wetlands Protection).

Core Requirement #5 allows the user to demonstrate compliance with the LID Performance Standard of matching developed discharge durations to pre-developed durations for the range of pre-developed discharge rates from 8% of the 2-year peak flow to 50% of the 2-year peak flow. If the postdevelopment flow duration values exceed any of the predevelopment flow levels between 8% and 50% of the 2-year predevelopment peak flow values, then the LID performance standard has not been met.

Core Requirement #7 specifies that stormwater discharges to streams shall match developed discharge durations to predeveloped durations for the range of predeveloped discharge rates from 50% of the 2-year peak flow up to the full 50-year peak flow.

WWHM computes the predevelopment 2- through 100-year flow frequency values and computes the post-development runoff 2- through 100-year flow frequency values from the outlet of the proposed stormwater facility.

The model uses pond discharge data to compare the predevelopment and post development durations and determines if the flow control standards have been met.

There are three criteria by which flow duration values are compared:

1. If the post development flow duration values exceed any of the predevelopment flow levels between 50% and 100% of the 2-year predevelopment peak flow values (100 Percent Threshold) then the flow duration requirement has not been met.

2. If the post development flow duration values exceed any of the predevelopment flow levels between 100% of the 2-year and 100% of the 50-year predevelopment peak flow values more than 10 percent of the time (110 Percent Threshold) then the flow duration requirement has not been met.

3. If more than 50 percent of the flow duration levels exceed the 100 percent threshold then the flow duration requirement has not been met.

Core Requirement #8 specifies that total discharge to a wetland must not deviate by more than 20% on a daily basis, and must not deviate by more than 15% on a monthly basis. Flow components feeding the wetland under both pre- and post-development scenarios are assumed to be the sum of the surface, interflow, and ground water flows from the project site. Ecology has added the capability to model flows to wetlands and analyze the daily and monthly flow deviations per Core Requirements #8 (above) in WWHM2012.

2.3 Single Event Hydrograph Method

Hydrograph analysis utilizes the standard plot of runoff flow versus time for a given design storm, thereby allowing the key characteristics of runoff such as peak, volume, and phasing to be considered in the design of drainage facilities. Because the only utility for single event methods in this manual is to size wet pool treatment facilities, only the subjects of design storms, curve numbers and calculating runoff volumes are presented. If single event methods are used to size temporary and permanent conveyances, the reader should reference other texts and software for assistance.

2.3.1 Water Quality Design Storm

The design storm for sizing wetpool treatment facilities is the 6-month, 24-hour storm. Unless amended to reflect local precipitation statistics, the 6-month, 24-hour precipitation amount may be assumed to
be 72 percent of the 2-year, 24-hour amount. Precipitation estimates of the 6-month and 2-year, 24-hour storms for certain towns and cities are listed in Appendix I-B of Volume I. For other areas, interpolating between isopluvials for the 2-year, 24-hour precipitation and multiplying by 72% yields the appropriate storm size.

The total depth of rainfall (in tenths of an inch) for storms of 24-hour duration and 2, 5, 10, 25, 50, and 100-year recurrence intervals are published by the National Oceanic and Atmospheric Administration (NOAA). The information is presented in the form of “isopluvial” maps for each state. Isopluvial maps are maps where the contours represent total inches of rainfall for a specific duration. Isopluvial maps for the 2, 5, 10, 25, 50, and 100-year recurrence interval and 24-hour duration storm events can be found in the NOAA Atlas 2, “Precipitation - Frequency Atlas of the Western United States, Volume IX-Washington.” Appendix III-A provides the isopluvials for the 2, 10, and 100-year, 24-hour design storms. Other precipitation frequency data may be obtained through the Western Regional Climate Center (WRCC) at Tel: (775) 674-7010. WRCC can generate 1-30 day precipitation frequency data for the location of interest using data from 1880 to present (currently June 2012).

2.3.2 Runoff Parameters

All storm event hydrograph methods require input of parameters that describe physical drainage basin characteristics. These parameters provide the basis from which the runoff hydrograph is developed. This section describes only the key parameter of curve number that is used to estimate the runoff from the water quality design storm.

**Curve Number**

The NRCS (formerly SCS) has, for many years, conducted studies of the runoff characteristics for various land types. After gathering and analyzing extensive data, NRCS has developed relationships between land use, soil type, vegetation cover, interception, infiltration, surface storage, and runoff. The relationships have been characterized by a single runoff coefficient called a “curve number.” The National Engineering Handbook - Section 4: Hydrology (NEH-4, SCS, August 1972) contains a detailed description of the development and use of the curve number method.

NRCS has developed “curve number” (CN) values based on soil type and land use. They can be found in *Urban Hydrology for Small Watersheds, Technical Release 55 (TR-55), June 1986*, published by the NRCS. The combination of these two factors is called the “soil-cover complex.” The soil-cover complexes have been assigned to one of four hydrologic soil groups, according to their runoff characteristics. NRCS has classified over 4,000 soil types into these four soil groups. Table 2.3.1 shows the hydrologic soil group of most soils in the state of Washington and provides a brief description of the four groups. For details on other soil types refer to the NRCS publication mentioned above (TR-55, 1986).
<table>
<thead>
<tr>
<th>Soil Type</th>
<th>Hydrologic Soil Group</th>
<th>Soil Type</th>
<th>Hydrologic Soil Group</th>
</tr>
</thead>
<tbody>
<tr>
<td>Agnew</td>
<td>C</td>
<td>Hoko</td>
<td>C</td>
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<tr>
<td>Ahl</td>
<td>B</td>
<td>Hoodspor</td>
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<td>Baldhill</td>
<td>B</td>
<td>Jumpe</td>
<td>B</td>
</tr>
<tr>
<td>Barneston</td>
<td>C</td>
<td>Kalaloche</td>
<td>C</td>
</tr>
<tr>
<td>Baumgard</td>
<td>B</td>
<td>Kapowsin</td>
<td>C/D</td>
</tr>
<tr>
<td>Beausite</td>
<td>B</td>
<td>Katula</td>
<td>C</td>
</tr>
<tr>
<td>Belfast</td>
<td>C</td>
<td>Kilchis</td>
<td>C</td>
</tr>
<tr>
<td>Bellingham</td>
<td>D</td>
<td>Kitsap</td>
<td>C</td>
</tr>
<tr>
<td>Bellingham variant</td>
<td>C</td>
<td>Klaus</td>
<td>C</td>
</tr>
<tr>
<td>Boisfort</td>
<td>B</td>
<td>Klone</td>
<td>B</td>
</tr>
<tr>
<td>Bow</td>
<td>D</td>
<td>Lates</td>
<td>C</td>
</tr>
<tr>
<td>Briscot</td>
<td>D</td>
<td>Lebam</td>
<td>B</td>
</tr>
<tr>
<td>Buckley</td>
<td>C</td>
<td>Lummi</td>
<td>D</td>
</tr>
<tr>
<td>Bunker</td>
<td>B</td>
<td>Lynnwood</td>
<td>A</td>
</tr>
<tr>
<td>Cagey</td>
<td>C</td>
<td>Lystair</td>
<td>B</td>
</tr>
<tr>
<td>Carlsborg</td>
<td>A</td>
<td>Mal</td>
<td>C</td>
</tr>
<tr>
<td>Casey</td>
<td>D</td>
<td>Manley</td>
<td>B</td>
</tr>
<tr>
<td>Cassolary</td>
<td>C</td>
<td>Mashel</td>
<td>B</td>
</tr>
<tr>
<td>Cathcart</td>
<td>B</td>
<td>Maytown</td>
<td>C</td>
</tr>
<tr>
<td>Centralia</td>
<td>B</td>
<td>McKenna</td>
<td>D</td>
</tr>
<tr>
<td>Chehalis</td>
<td>B</td>
<td>McMurray</td>
<td>D</td>
</tr>
<tr>
<td>Chesaw</td>
<td>A</td>
<td>Melbourne</td>
<td>B</td>
</tr>
<tr>
<td>Cinebar</td>
<td>B</td>
<td>Menzel</td>
<td>B</td>
</tr>
<tr>
<td>Clallam</td>
<td>C</td>
<td>Mixed Alluvial</td>
<td>variable</td>
</tr>
<tr>
<td>Clayton</td>
<td>B</td>
<td>Molson</td>
<td>B</td>
</tr>
<tr>
<td>Coastal beaches</td>
<td>variable</td>
<td>Mukilteo</td>
<td>C/D</td>
</tr>
<tr>
<td>Colter</td>
<td>C</td>
<td>Naff</td>
<td>B</td>
</tr>
<tr>
<td>Custer</td>
<td>D</td>
<td>Nargar</td>
<td>A</td>
</tr>
<tr>
<td>Custer, Drained</td>
<td>C</td>
<td>National</td>
<td>B</td>
</tr>
<tr>
<td>Dabob</td>
<td>C</td>
<td>Neilton</td>
<td>A</td>
</tr>
<tr>
<td>Delphi</td>
<td>D</td>
<td>Newberg</td>
<td>B</td>
</tr>
<tr>
<td>Dick</td>
<td>A</td>
<td>Nisqually</td>
<td>B</td>
</tr>
<tr>
<td>Dimal</td>
<td>D</td>
<td>Nooksack</td>
<td>C</td>
</tr>
<tr>
<td>Dupont</td>
<td>D</td>
<td>Norma</td>
<td>C/D</td>
</tr>
<tr>
<td>Earlmont</td>
<td>C</td>
<td>Ogarty</td>
<td>C</td>
</tr>
<tr>
<td>Edgewick</td>
<td>C</td>
<td>Olete</td>
<td>C</td>
</tr>
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<td>Eld</td>
<td>B</td>
<td>Olomount</td>
<td>C</td>
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<tr>
<td>Elwell</td>
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<td>Olympic</td>
<td>B</td>
</tr>
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<td>B</td>
<td>Orcas</td>
<td>B</td>
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<td>Everett</td>
<td>A</td>
<td>Oridia</td>
<td>D</td>
</tr>
<tr>
<td>Eversen</td>
<td>D</td>
<td>Orting</td>
<td>D</td>
</tr>
<tr>
<td>Galvin</td>
<td>D</td>
<td>Oso</td>
<td>C</td>
</tr>
<tr>
<td>Getchell</td>
<td>A</td>
<td>Olden</td>
<td>C</td>
</tr>
<tr>
<td>Giles</td>
<td>B</td>
<td>Pastik</td>
<td>C</td>
</tr>
<tr>
<td>Godfrey</td>
<td>D</td>
<td>Pheeney</td>
<td>C</td>
</tr>
<tr>
<td>Greenwater</td>
<td>A</td>
<td>Phelan</td>
<td>D</td>
</tr>
<tr>
<td>Grove</td>
<td>C</td>
<td>Pilchuck</td>
<td>C</td>
</tr>
<tr>
<td>Harstine</td>
<td>C</td>
<td>Potchub</td>
<td>C</td>
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<tr>
<td>Hartnit</td>
<td>C</td>
<td>Poulbo</td>
<td>C</td>
</tr>
<tr>
<td>Hoh</td>
<td>B</td>
<td>Prather</td>
<td>C</td>
</tr>
<tr>
<td>Puget</td>
<td>D</td>
<td>Solleks</td>
<td>C</td>
</tr>
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<td>Puyallup</td>
<td>B</td>
<td>Spana</td>
<td>D</td>
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<td>Quients</td>
<td>B</td>
<td>Spanaway</td>
<td>A/B</td>
</tr>
<tr>
<td>Quilcene</td>
<td>C</td>
<td>Springdale</td>
<td>B</td>
</tr>
<tr>
<td>Ragnar</td>
<td>B</td>
<td>Sulsavar</td>
<td>B</td>
</tr>
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<td>Soil Type</td>
<td>Hydrologic Soil Group</td>
<td>Soil Type</td>
<td>Hydrologic Soil Group</td>
</tr>
<tr>
<td>---------------------------</td>
<td>-----------------------</td>
<td>---------------------------</td>
<td>-----------------------</td>
</tr>
<tr>
<td>Rainier</td>
<td>C</td>
<td>Sultan</td>
<td>C</td>
</tr>
<tr>
<td>Raught</td>
<td>B</td>
<td>Sultan variant</td>
<td>B</td>
</tr>
<tr>
<td>Reed</td>
<td>D</td>
<td>Sumas</td>
<td>C</td>
</tr>
<tr>
<td>Reed, Drained or Protected</td>
<td>C</td>
<td>Swantown</td>
<td>D</td>
</tr>
<tr>
<td>Renton</td>
<td>D</td>
<td>Tacoma</td>
<td>D</td>
</tr>
<tr>
<td>Republic</td>
<td>B</td>
<td>Tanwax</td>
<td>D</td>
</tr>
<tr>
<td>Riverwash</td>
<td>variable</td>
<td>Tanwax, Drained</td>
<td>C</td>
</tr>
<tr>
<td>Rober</td>
<td>C</td>
<td>Tealwhit</td>
<td>D</td>
</tr>
<tr>
<td>Salal</td>
<td>C</td>
<td>Tenino</td>
<td>C</td>
</tr>
<tr>
<td>Salkum</td>
<td>B</td>
<td>Tisch</td>
<td>D</td>
</tr>
<tr>
<td>Sammamish</td>
<td>D</td>
<td>Tokul</td>
<td>C</td>
</tr>
<tr>
<td>San Juan</td>
<td>A</td>
<td>Townsend</td>
<td>C</td>
</tr>
<tr>
<td>Scamman</td>
<td>D</td>
<td>Triton</td>
<td>D</td>
</tr>
<tr>
<td>Schneider</td>
<td>B</td>
<td>Tukwila</td>
<td>D</td>
</tr>
<tr>
<td>Seattle</td>
<td>D</td>
<td>Tukey</td>
<td>C</td>
</tr>
<tr>
<td>Sekiu</td>
<td>D</td>
<td>Urbana</td>
<td>C</td>
</tr>
<tr>
<td>Semiahmoo</td>
<td>D</td>
<td>Vailton</td>
<td>B</td>
</tr>
<tr>
<td>Shalcar</td>
<td>D</td>
<td>Verlot</td>
<td>C</td>
</tr>
<tr>
<td>Shano</td>
<td>B</td>
<td>Wapato</td>
<td>D</td>
</tr>
<tr>
<td>Shelton</td>
<td>C</td>
<td>Warden</td>
<td>B</td>
</tr>
<tr>
<td>Si</td>
<td>C</td>
<td>Whidbey</td>
<td>C</td>
</tr>
<tr>
<td>Sinclair</td>
<td>C</td>
<td>Wilkeson</td>
<td>B</td>
</tr>
<tr>
<td>Skipopa</td>
<td>D</td>
<td>Winston</td>
<td>A</td>
</tr>
<tr>
<td>Skykomish</td>
<td>B</td>
<td>Woodinville</td>
<td>B</td>
</tr>
<tr>
<td>Snahopish</td>
<td>B</td>
<td>Yelm</td>
<td>C</td>
</tr>
<tr>
<td>Snohomish</td>
<td>D</td>
<td>Zynbar</td>
<td>B</td>
</tr>
<tr>
<td>Solduc</td>
<td>B</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Notes:

Hydrologic Soil Group Classifications, as Defined by the Soil Conservation Service:

A = (Low runoff potential) Soils having low runoff potential and high infiltration rates, even when thoroughly wetted. They consist chiefly of deep, well to excessively drained sands or gravels and have a high rate of water transmission (greater than 0.30 in/hr.).

B = (Moderately low runoff potential). Soils having moderate infiltration rates when thoroughly wetted and consist chiefly of moderately deep to deep, moderately well to well drained soils with moderately fine to moderately coarse textures. These soils have a moderate rate of water transmission (0.15-0.3 in/hr.).

C = (Moderately high runoff potential). Soils having low infiltration rates when thoroughly wetted and consist chiefly of soils with a layer that impedes downward movement of water and soils with moderately fine to fine textures. These soils have a low rate of water transmission (0.05-0.15 in/hr.).

D = (High runoff potential). Soils having high runoff potential. They have very low infiltration rates when thoroughly wetted and consist chiefly of clay soils with a high swelling potential, soils with a permanent high water table, soils with a hardpan or clay layer at or near the surface, and shallow soils over nearly impervious material. These soils have a very low rate of water transmission (0-0.05 in/hr.).


Additional Note: Where field infiltration tests indicate a measured (initial) infiltration rate less than 0.30 in/hr, the WWHM user may model the site as a C soil.
Table 2.3.2 shows the CNs, by land use description, for the four hydrologic soil groups. These numbers are for a 24-hour duration storm and typical antecedent soil moisture condition preceding 24-hour storms.

The following are important criteria/considerations for selection of CN values:

Many factors may affect the CN value for a given land use. For example, the movement of heavy equipment over bare ground may compact the soil so that it has a lesser infiltration rate and greater runoff potential than would be indicated by strict application of the CN value to developed site conditions.

CN values can be area weighted when they apply to pervious areas of similar CNs (within 20 CN points). However, high CN areas should not be combined with low CN areas. In this case, separate estimates of $S$ (potential maximum natural detention) and $Q_d$ (runoff depth) should be generated and summed to obtain the cumulative runoff volume unless the low CN areas are less than 15 percent of the subbasin.

Separate CN values must be selected for the pervious and impervious areas of an urban basin or subbasin. For residential districts the percent impervious area given in Table 2.3.2 must be used to compute the respective pervious and impervious areas. For proposed commercial areas, planned unit developments, etc., the percent impervious area must be computed from the site plan. For all other land uses the percent impervious area must be estimated from best available aerial topography and/or field reconnaissance. The pervious area CN value must be a weighted average of all the pervious area CNs within the subbasin. The impervious area CN value shall be 98.

Example: The following is an example of how CN values are selected for a sample project.

Select CNs for the following development:

- **Existing Land Use:** forest (undisturbed)
- **Future Land Use:** residential plat (3.6 DU/GA)
- **Basin Size:** 60 acres
- **Soil Type:** 80 percent Alderwood, 20 percent Ragnor

Table 2.3.1 shows that Alderwood soil belongs to the “C” hydrologic soil group and Ragnor soil belongs to the “B” group. Therefore, for the existing condition, CNs of 70 and 55 are read from Table 2.3.2 and areal weighted to obtain a CN value of 67. For the developed condition with 3.6 DU/GA the percent impervious of 39 percent is interpolated from Table 2.3.2 and used to compute pervious and impervious areas of 36.6 acres and 23.4 acres, respectively. The 36.6 acres of pervious area is assumed to be in Fair condition (for a conservative design) with residential yards and lawns covering the same proportions of Alderwood and Ragnor soil (80 percent and 20 percent respectively). Therefore, CNs of 90 and 85 are read from Table 2.3.2 and areal weighted to obtain a pervious area CN value of 89. The impervious area CN value is 98. The result of this example is summarized below:

<table>
<thead>
<tr>
<th>On-Site Condition</th>
<th>Existing</th>
<th>Developed</th>
</tr>
</thead>
<tbody>
<tr>
<td>Land Use</td>
<td>Forest</td>
<td>Residential</td>
</tr>
<tr>
<td>Pervious Area</td>
<td>60 ac.</td>
<td>36.6 ac.</td>
</tr>
<tr>
<td>CN of pervious area</td>
<td>67</td>
<td>89</td>
</tr>
<tr>
<td>Impervious area</td>
<td>0 ac.</td>
<td>23.4 ac.</td>
</tr>
<tr>
<td>CN of Impervious area</td>
<td>--</td>
<td>98</td>
</tr>
</tbody>
</table>
### Table 2.3.2

**Runoff Curve Numbers for Selected Agricultural, Suburban, and Urban Areas**

(Sources: TR 55, 1986. See Section 2.1.1 for explanation)

<table>
<thead>
<tr>
<th>Cover type and hydrologic condition.</th>
<th>CNs for hydrologic group</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>A</td>
</tr>
<tr>
<td><strong>Curve Numbers for Pre-Development Conditions</strong></td>
<td></td>
</tr>
<tr>
<td>Pasture, grassland, or range-continuous forage for grazing:</td>
<td></td>
</tr>
<tr>
<td>Fair condition (ground cover 50% to 75% and not heavily grazed).</td>
<td>49</td>
</tr>
<tr>
<td>Good condition (ground cover &gt;75% and lightly or only occasionally grazed).</td>
<td>39</td>
</tr>
<tr>
<td>Woods:</td>
<td></td>
</tr>
<tr>
<td>Fair (Woods are grazed but not burned, and some forest litter covers the soil).</td>
<td>36</td>
</tr>
<tr>
<td>Good (Woods are protected from grazing, and litter and brush adequately cover the soil).</td>
<td>30</td>
</tr>
<tr>
<td><strong>Curve Numbers for Post-Development Conditions</strong></td>
<td></td>
</tr>
<tr>
<td>Open space (lawns, parks, golf courses, cemeteries, landscaping, etc.)</td>
<td></td>
</tr>
<tr>
<td>Fair condition (grass cover on 50% - 75% of the area).</td>
<td>77</td>
</tr>
<tr>
<td>Good condition (grass cover on &gt;75% of the area).</td>
<td>68</td>
</tr>
<tr>
<td>Impervious areas:</td>
<td></td>
</tr>
<tr>
<td>Open water bodies: lakes, wetlands, ponds etc.</td>
<td>100</td>
</tr>
<tr>
<td>Paved parking lots, roofs(^1), driveways, etc. (excluding right-of-way)</td>
<td>98</td>
</tr>
<tr>
<td><strong>Permeable Pavement (See Appendix III-C to decide which condition below to use)</strong></td>
<td></td>
</tr>
<tr>
<td>Landscaped area</td>
<td>77</td>
</tr>
<tr>
<td>50% landscaped area/50% impervious</td>
<td>87</td>
</tr>
<tr>
<td>100% impervious area</td>
<td>98</td>
</tr>
<tr>
<td>Paved</td>
<td>98</td>
</tr>
<tr>
<td>Gravel (including right-of-way)</td>
<td>76</td>
</tr>
<tr>
<td>Dirt (including right-of-way)</td>
<td>72</td>
</tr>
<tr>
<td><strong>Pasture, grassland, or range-continuous forage for grazing:</strong></td>
<td></td>
</tr>
<tr>
<td>Poor condition (ground cover &lt;50% or heavily grazed with no mulch).</td>
<td>68</td>
</tr>
<tr>
<td>Fair condition (ground cover 50% to 75% and not heavily grazed).</td>
<td>49</td>
</tr>
<tr>
<td>Good condition (ground cover &gt;75% and lightly or only occasionally grazed).</td>
<td>39</td>
</tr>
<tr>
<td>Woods:</td>
<td></td>
</tr>
<tr>
<td>Poor (Forest litter, small trees, and brush are destroyed by heavy grazing or regular burning).</td>
<td>45</td>
</tr>
<tr>
<td>Fair (Woods are grazed but not burned, and some forest litter covers the soil).</td>
<td>36</td>
</tr>
<tr>
<td>Good (Woods are protected from grazing, and litter and brush adequately cover the soil).</td>
<td>30</td>
</tr>
<tr>
<td><strong>Single family residential</strong>:</td>
<td></td>
</tr>
<tr>
<td>Dwelling Unit/Gross Acre</td>
<td>Should only be used for subdivisions &gt; 50 acres</td>
</tr>
<tr>
<td>1.0 DU/GA</td>
<td></td>
</tr>
<tr>
<td>1.5 DU/GA</td>
<td></td>
</tr>
<tr>
<td>2.0 DU/GA</td>
<td></td>
</tr>
<tr>
<td>2.5 DU/GA</td>
<td></td>
</tr>
<tr>
<td>3.0 DU/GA</td>
<td></td>
</tr>
<tr>
<td>3.5 DU/GA</td>
<td></td>
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<td>4.0 DU/GA</td>
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<td>6.0 DU/GA</td>
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<tr>
<td>6.5 DU/GA</td>
<td></td>
</tr>
<tr>
<td>7.0 DU/GA</td>
<td></td>
</tr>
<tr>
<td>7.5 DU/GA</td>
<td></td>
</tr>
<tr>
<td>PUD’s, condos, apartments, commercial businesses, industrial areas &amp; subdivisions &lt; 50 acres</td>
<td>%impervious must be computed</td>
</tr>
</tbody>
</table>

* Composite CN’s may be computed for other combinations of open space cover type.

\(^1\) Where roof runoff and driveway runoff are infiltrated or dispersed according to the requirements in Chapter 3, the average percent impervious area may be adjusted in accordance with the procedure described under “Flow Credit for Roof Downspout Infiltration” (Section 3.1.1), and “Flow Credit for Roof Downspout Dispersion” (Section 3.1.2).

\(^2\) Assumes roof and driveway runoff is directed into street/storm system.

\(^3\) All the remaining pervious area (lawns) are considered to be in good condition for these curve numbers.
NRCS Curve Number
Equations for
determination of
runoff depths and
volumes

The rainfall-runoff equations of the NRCS curve number method relates a land area’s runoff depth (precipitation excess) to the precipitation it receives and to its natural storage capacity, as follows:

\[ Q_d = \frac{(P - 0.2S)^2}{(P + 0.8S)} \]  for \( P \geq 0.2S \)

and \[ Q_d = 0 \]  for \( P < 0.2S \)

Where:

- \( Q_d \) = runoff depth in inches over the area,
- \( P \) = precipitation depth in inches over the area, and
- \( S \) = potential maximum natural detention, in inches over the area, due to infiltration, storage, etc.

The area’s potential maximum detention, \( S \), is related to its curve number, \( CN \):

\[ S = \frac{1000}{CN} - 10 \]

The combination of the above equations allows for estimation of the total runoff volume by computing total runoff depth, \( Q_d \), given the total precipitation depth, \( P \). For example, if the curve number of the area is 70, then the value of \( S \) is 4.29. With a total precipitation for the design event of 2.0 inches, the total runoff depth would be:

\[ Q_d = \frac{[2.0 - 0.2(4.29)]^2}{[2.0 + 0.8(4.29)]} = 0.24 \text{ inches} \]

This computed runoff represents inches over the tributary area. Therefore, the total volume of runoff is found by multiplying \( Q_d \) by the area (with necessary conversions):

Calculating the
design volume for
wetpool treatment
facilities

Total runoff

Volume \[ = \frac{3,630 \times Q_d \times A}{\text{(cu. ft.)}} \]

\[ \text{(cu. ft./ac. in.)} \] \[ \text{(in)} \] \[ \text{(ac)} \]

If the area is 10 acres, the total runoff volume is:

3,630 cu. ft./ac. in. x 0.24 in. x 10 ac. = 8,712 cu. ft.

This is the design volume for treatment BMPs for which the design criterion is based on the volume of runoff.

2.4 Closed Depression Analysis

The analysis of closed depressions requires careful assessment of the existing hydrologic performance in order to evaluate the impacts a proposed project will have. The applicable requirements (see Core Requirement #7) and the City of Olympia’s Critical Areas Ordinance and Rules (if applicable) should be thoroughly reviewed prior to proceeding with the analysis.

Closed depressions generally facilitate infiltration of runoff. If a closed depression is classified as a wetland, then the Core Requirement #8 for wetlands applies. If there is an outflow from this wetland to a surface water (such as a creek), then the flow from this wetland must also meet the Core Requirement #7 for flow control. A calibrated continuous simulation hydrologic model must be used for closed depression analysis and design of mitigation facilities. If a closed depression is not classified as a wetland, model the ponding area at the bottom of the closed depression as an infiltration pond using WWHM or an approved equivalent runoff model.

A calibrated continuous simulation hydrologic model, such as the latest version of the WWHM with Thurston County enhancements, shall be used for closed depression analysis and design of mitigation facilities. The procedures below may be followed.
**Analysis and Design Criteria:**
The infiltration rates used in the analysis of closed depressions shall be determined according to the procedures in Volume III Chapter 3. For closed depressions containing standing water, soil texture tests must be performed on dry land adjacent to, and on opposite sides of the depression (as is feasible). A minimum of two tests must be performed to estimate an average surface infiltration rate. Wet-season water level fluctuations, measured using a datalogger, are also useful in estimating infiltration rates, especially if the depression currently receives runoff. Projects proposing to modify or compensate for replacement storage in a closed depression must meet the design criteria for detention ponds as described in this volume.

**Method of Analysis:**
Closed depressions are analyzed using WWHM. In assessing the impacts of a proposed project on the performance of a closed depression, two cases dictate different approaches to meeting Core Requirement #7 and applicable local requirements. Note that where there is a flooding potential, concern about rising ground water levels, property rights/ownership/use issues, or there are local critical areas ordinances and rules, this analysis may not be sufficient and the local permitting authorities may require more stringent analysis and impose more stringent requirements.

**Case 1:** The 100-year storm flow from the drainage basin tributary to the closed depression is routed into the closed depression, using only infiltration as outflow. Under this scenario, there is no overflow from closed depression. Determine the predevelopment high water level. The post-development high water level, assuming full build-out of the contributing watershed, shall be no more than 0.1 feet higher than the predevelopment level, unless the development has acquired ownership or discharge rights to the closed depression. (If ownership rights are acquired, the closed depression may be flooded, subject to any applicable government requirements or conditions, such as for wetlands.)

Absent ownership or discharge rights, excavate additional storage volume in the closed depression (subject to all applicable requirements, for example, access rights and providing a defined overflow system) or in an upland area, as needed to achieve the development’s contribution to the 0.1-foot maximum water level increase standard.

**Case 2:** The 100-year storm flow from the drainage basin tributary to the closed depression is routed into the closed depression, using only infiltration as outflow. Under this scenario, predevelopment runoff causes overflows from closed depression. In this case, use WWHM to match pre- and post-development flows and durations, determining how much storage must be added to the closed depression. Design an appropriate flow control and overflow structure.
Chapter 3 - Flow Control Design

Note: Figures in Chapter 3 courtesy of King County, except as noted

This chapter presents methods, criteria, and details for hydraulic analysis and design of flow control facilities and roof downspout controls. Flow control facilities are detention or infiltration facilities engineered to meet the flow control standards specified in Volume I. Roof downspout controls include infiltration trenches, dry wells, and partial dispersion systems for use in individual lots, proposed plats, and short plats. Roof downspout controls are used in conjunction with, and in addition to, any flow control facilities that may be necessary. They are included in the list of BMPs to consider for compliance with Core Requirement #5. Implementation of roof downspout controls may reduce the total effective impervious area and result in less runoff from these surfaces. Ecology’s Western Washington Hydrology Model (WWHM) incorporates flow credits for implementing two types of roof downspout controls. These are:

- If roof runoff is *infiltrated* according to the requirements of this section, the roof area may be discounted from the total project area used for sizing stormwater facilities.
- If roof runoff is *dispersed* according to the requirements of this section on single-family lots greater than 22,000 square feet, and the *vegetative flow* path is 50 feet or larger through *undisturbed* native landscape or lawn/landscape area that meets BMP T5.13, the roof area may be modeled as grassed surface.

This chapter also provides design procedures, criteria, and field tests methods concerning infiltration facilities used for flow control or treatment. Section 3.4 covers design of bioretention and permeable pavement facilities. Additional design considerations for bioretention facilities, a type of infiltration design, are covered in Chapter 7 of Volume V.

Stormwater management facilities located within public right-of-way shall be designed to manage stormwater runoff only from the public right-of-way. Stormwater runoff from private property shall be managed on private property.

3.1 Roof Downspout Controls

This section presents the criteria for design and implementation of roof downspout controls. Roof downspout controls are simple pre-engineered designs for infiltrating and/or dispersing runoff from roof areas for the purposes of increasing opportunities for ground water recharge and reduction of runoff volumes from new developments.

**Selection of Roof Downspout Controls**

Large lots in rural areas (5 acres or greater) typically have enough area to disperse or infiltrate roof runoff. Lots created in urban areas will typically be smaller (about 8,000 square feet) and have a limited amount of area in which to site infiltration or dispersion trenches. Downspout infiltration should be used in those soils that readily infiltrate. Dispersion BMPs should be used for urban lots located in less permeable soils, where infiltration is not feasible. Where dispersion is not feasible because of very small lot size, or where there is a potential for creating drainage problems on adjacent lots, connect downsputs with perforated stub-out connections to the street storm drain system, which directs the runoff to a stormwater management facility.

*Vegetative flow* path is measured from the downspout or dispersion system discharge point to the downstream property line, stream, wetland, or other impervious surface.
Where roof downspout controls are planned, the following types must be considered in descending order of preference:

1. Full Dispersion in accordance with BMP T5.30 in Chapter 5 of Volume V, or Downspout Full Infiltration Systems in accordance with BMP T5.10A in Section 3.1.1.
2. Rain Gardens in accordance with BMP T5.14 in Chapter 5 of Volume V, or if the project area is subject to Core Requirements #6 and/or #7, Bioretention in accordance with BMP T7.30 in Chapter 7 of Volume V.
3. Downspout Dispersion Systems in accordance with BMP T5.10B in Section 3.1.2.
4. Perforated Stub-out Connections in accordance with BMP T5.10C in Section 3.1.3.

Figure 3.1.1 illustrates, in general, how roof downspout controls are selected and applied in single-family subdivision projects. Where supported by appropriate soil infiltration tests, downspout full infiltration in finer soils may be practical using a larger infiltration system.

Roof downspout controls can be applied to individual commercial lot developments when the total site impervious area and pollutant characteristics are comparable to those from residential lots.

Note: Other innovative downspout control BMPs such as rain barrels, ornamental ponds, downspout cisterns, or other downspout water storage devices may be used to supplement any of the above BMPs if approved by the City of Olympia (City).
Figure 3.1.1 - Flow Diagram Showing Selection of Roof Downspout Controls

- Large native area set aside or high on-site infiltration? YES → Use Full Dispersion or Full Infiltration Systems
- NO
- Lots suitable for infiltration? YES → Use Bioretention or Rain Garden depending on project size
- NO
- Criteria for Downspout Dispersion met? YES → Use Downspout Dispersion Systems
- NO
- Connect downspouts to street drainage system with perforated stub-outs (see Section 3.1.3)

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Volume III – Hydrologic Analysis and Flow Control BMPs
3.1.1 Downspout Full Infiltration Systems (BMP T5.10A)

Downspout full infiltration systems are trench or drywell designs intended only for use in infiltrating runoff from roof downspout drains. They are not designed to directly infiltrate runoff from pollutant-generating impervious surfaces.

On lots or sites with more than 3 feet of permeable soil from the proposed final grade to the seasonal high groundwater table, downspout infiltration is considered feasible if soils and the infiltration trench can be designed to meet the minimum design criteria specified below.

Application

Projects subject to Core Requirement #5 (Section 2.5.5, Volume I) must provide for individual downspout full infiltration systems or full dispersion if feasible. Evaluate the feasibility, or applicability, of downspout full infiltration unless full dispersion is proposed. Use the evaluation procedure below to determine the feasibility of downspout full infiltration.

The design criteria in this section is applicable only to single family residences. Downspout full infiltration for commercial and multi-family structures shall be designed per Section 3.3.11 – Infiltration Trenches.

Runoff Modeling for Roof Downspout Full Infiltration

If roof runoff is infiltrated according to the requirements of this section, the roof area may be discounted from the project area used for sizing stormwater facilities.

Procedure for Evaluating Feasibility

1. An engineer, soil scientist, or other licensed or certified professional with appropriate training shall evaluate soils to determine if they are suitable for infiltration.

2. On lots or sites with more than 3 feet or more of permeable soil from the proposed final grade to the seasonal high groundwater table, downspout infiltration is considered feasible if the infiltration trench can be designed to meet the minimum design criteria specified below.

Design Criteria for Infiltration Trenches

Figure 3.1.2 shows a typical downspout infiltration trench system, and Figure 3.1.3 presents an alternative infiltration trench system for sites with coarse sand and cobble soils. These systems are designed as specified below.

General

1. The following minimum total trench volumes, including rock backfill, are required per 1,000 square feet of roof area. For subdivisions, calculate sizes and provide a schedule, by lot number, with the engineered plans:
   • Hydrologic Group A and B (sand, loamy sand, sandy loam), 125 cubic feet
   • Hydrologic Group C (loam, silt loam, sandy clay loam, layered sandy loam soils, till soils with Group A or B surface soils), 250 cubic feet
   • Hydrologic Group D soils (silt, clays, rock outcroppings, till soils with Group C or D surface soils, most fill materials), 750 cubic feet. Infiltration is not recommended in these soils.

2. Maximum length of trench shall not exceed 100 feet from the inlet sump.

3. Minimum spacing between trench centerlines shall be 6 feet.

4. Filter fabric shall be placed over the drain rock as shown on Figure 3.1.2 prior to backfilling. Do not place fabric on trench bottom.
5. Minimum infiltration trench setbacks are as follows:
   - Water supply wells, building crawl spaces, or basements shall be at least 10 feet upgradient or 30 feet downgradient from the trench.
   - Infiltration trenches in till or layered soils must be located downgradient of crawl spaces or basements.
   - Septic systems shall be per Thurston County Health Department requirements.
   - Top of slopes over 15 percent, 25 feet. May be increased if landslide hazards are present. A geotechnical analysis and report may be required on slopes over 15 percent or if located within 200 feet of the top of slope steeper than 40%, or in a landslide hazard area.
   - In case of conflict among setback requirements, the more stringent shall prevail.

6. Infiltration trenches may be placed in fill material if the fill is placed and compacted under the direct supervision of a geotechnical engineer or professional civil engineer with geotechnical expertise, and if the measured infiltration rate is at least 8 inches per hour. Trench length in fill must be 60 linear feet per 1,000 square feet of roof area. Infiltration rates can be tested using the methods described in Section 3.3.

7. Infiltration trenches should not be built on slopes steeper than 25% (4:1). A geotechnical analysis and report may be required on slopes over 15 percent or if located within 200 feet of the top of slope steeper than 40%, or in a landslide hazard area.

8. Trenches may be located under pavement if a small yard drain or catch basin with grate cover is placed at the end of the trench pipe such that overflow would occur out of the catch basin at an elevation at least one foot below that of the pavement, and in a location which can accommodate the overflow without creating a significant adverse impact to downhill properties or drainage systems. This is intended to prevent saturation of the pavement in the event of system failure.

**Design Criteria for Infiltration Drywells**

Figure 3.1.2 shows a typical downspout infiltration drywell system. These systems are designed as specified below.

**General**

1. Drywell bottoms must be a minimum of 1 foot above seasonal high ground water level or impermeable soil layers.

2. The following minimum total trench volumes, including rock backfill, are required per 1,000 square feet of roof area. For subdivisions, calculate sizes and provide a schedule, by lot number, with the engineered plans:
   - Hydrologic Group A and B (sand, loamy sand, sandy loam), 125 cubic feet
   - Hydrologic Group C (loam, silt loam, sandy clay loam, layered sandy loam soils, till soils with Group A or B surface soils), 250 cubic feet
   - Hydrologic Group D soils (silt, clays, rock outcroppings, till soils with Group C or D surface soils, most fill materials), 750 cubic feet. Infiltration is not recommended in these soils.
3. Drywells must be at least 48 inches in diameter (minimum) and deep enough to contain the gravel amounts specified above for the soil type and impervious surface served.

4. Filter fabric (geotextile) must be placed on top of the drain rock and on trench or drywell sides prior to backfilling.

5. Spacing between drywells must be a minimum of 10 feet.

6. Downspout infiltration drywells must not be built on slopes greater than 25% (4:1). Drywells may not be placed on or above a landslide hazard area or on slopes greater than 15% without evaluation by a professional engineer with geotechnical expertise or a licensed geologist, hydrogeologist, or engineering geologist, and with jurisdiction approval.
Figure 3.1.2 - Typical Downspout Infiltration Trench

NOTES:
1. PIPE MATERIAL SHALL BE AS APPROVED IN UNIFORM PLUMBING CODE AND BE INSTALLED ACCORDING TO MANUFACTURER SPECIFICATIONS.
2. TRENCH LENGTH IS GREATER THAN OR EQUAL TO WIDTH.
3. BENDS AND TEES ALLOWED IN PIPE.
4. TOP OF TRENCH FOR VOLUME CALCULATION IS HORIZONTAL LINE EXTENDED FORM LOWEST TOP OF WASHED ROCK.
5. CATCH BASIN/YARD DRAIN TO BE MIN. 18 INCH DIA. AND 30 INCH DEEP, MUST HAVE REMOVABLE COVER, AND SOLID BASE. STRUCTURE MAY BE MADE OF CONCRETE, HOCPE OR PVC. USE HANCOR SUMP PUMP WELL (CASW018000C0) AND LID (CAW018000C0) OR EQUAL. DRILL HOLES IN LID AS NEEDED TO ALLOW WATER ENTRY.
6. WASHED ROUND ROCK 1 1/2"-3/4" 100% PASSING 1 1/2" SIEVE. LESS THAN 10% PASSING 3/4" SIEVE. ROUND ROCK TO BE WASHED BEFORE INSTALLATION.

Refer to City of Olympia Engineering Design and Development Standards for additional details.
Setbacks

The following setbacks shall apply:

1. Water supply wells, building crawl spaces, or basements shall be at least 10 feet upgradient or 30 feet downgradient from the trench. This includes adjacent properties.

2. Infiltration trenches in till or layered soils must be located downgradient of crawl spaces or basements.

3. Top of slopes over 15 percent, 25 feet. Top of slopes over 40 percent, 50 feet. This setback may be reduced based on a geotechnical evaluation, but in no instances may it be less than the buffer width. Setbacks may be increased if landslide hazards are present.

4. Septic system setbacks shall be per Thurston County Health Department requirements.

5. In case of conflict among setback requirements, the more stringent shall prevail.

3.1.2 Downspout Dispersion Systems (BMP T5.10B)

Downspout dispersion systems are splash blocks or gravel-filled trenches, which serve to spread roof runoff over vegetated pervious areas. Dispersion attenuates peak flows by slowing the runoff entering into the conveyance system, allowing some infiltration, and providing some water quality benefits.

**Applications & Limitations**

Downspout dispersion may be used in all subdivision lots where downspout full infiltration, full dispersion, and bioretention/rain gardens are not feasible.

**Flow Credit for Roof Downspout Dispersion**

In WWHM, roof areas may be modeled as grassed surfaces (landscape) if roof runoff is dispersed according to the requirements of this section on lots greater than 22,000 square feet, and the vegetated flowpath is 50 feet or larger through undisturbed native landscape or lawn/landscape area that meets BMP T5.13. If the available vegetated flowpath is 25 to 50 feet, use of a dispersion trench allows modeling the roof as 50% impervious/50% landscape. This is done in WWHM on the Mitigated Scenario screen by entering the roof area into one of the entry options for dispersal of impervious area runoff. For WWHM 2012, see Appendix III-C in this Volume.

**Design Criteria**

1. Use downspout trenches designed as shown in Figures 3.1.5 and 3.1.6 for all downspout dispersion applications except where splash blocks are allowed below.

2. Splash blocks shown in Figure 3.1.7 may be used for downspouts discharging to a vegetated flowpath at least 50 feet in length as measured from the downspout to the downstream property line, structure, slope over 15%, stream, wetland, or other impervious surface. Sensitive area buffers may count toward flowpath lengths.

3. Cover the vegetated flowpath with well-established lawn or pasture, 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.
4. If the vegetated flowpath (measured as defined above) is less than 25 feet, a perforated stub-out connection per Section 3.1.3 may be used in lieu of downspout dispersion. A perforated stub-out may also be used where implementation of downspout dispersion might cause erosion or flooding problems, either on site or on adjacent lots. For example, this provision might be appropriate for lots constructed on steep hills where downspout discharge could culminate and might pose a potential hazard for lower lying lots, or where dispersed flows could create problems for adjacent off-site lots. This provision does not apply to situations where lots are flat and on-site downspout dispersal would result in saturated yards. Perforated stub-outs are not appropriate when seasonal water table is <1 foot below trench bottom.

5. For sites with septic systems, the discharge point of all dispersion systems must be downgradient of the drainfield. This requirement may be waived if site topography clearly prohibits flows from intersecting the drainfield.

Design Criteria for Dispersion Trenches

1. 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 slope steeper than 15%. Sensitive area buffers may count towards flowpath lengths.

2. Trenches serving up to 700 square feet of roof area may be simple 10-foot-long by 2-foot wide gravel filled trenches as shown in Figure 3.1.5. For roof areas larger than 700 square feet, a dispersion trench with notched grade board as shown in Figure 3.1.6 or alternative material approved by the City of Olympia may be used. The total length of this design must not exceed 50 feet and must provide at least 10 feet of trench per 700 square feet of roof area.

3. Maintain a setback of at least 5 feet between any edge of the trench and any structure or property line.

4. No erosion or flooding of downstream properties may result.

5. Have a geotechnical engineer or a licensed geologist, hydrogeologist, or engineering geologist evaluate runoff discharged towards landslide hazard areas. Do not place the discharge point on or above slopes greater than 15% or above erosion hazard areas without evaluation by a geotechnical engineer or qualified geologist and jurisdiction approval.

6. For purposes of maintaining adequate separation of flows discharged from adjacent dispersion devices, the outer edge of the vegetated flowpath segment for the dispersion trench must not overlap with other flowpath segments, except those associated with sheet flow from a non-native pervious surface.
Figure 3.1.5 - Typical Downspout Dispersion Trench

Source: King County
Figure 3.1.6 - Standard Dispersion Trench with Notched Grade Board

NOTES:
1. This trench shall be constructed so as to prevent point discharge and/or erosion.
2. Trenches may be placed no closer than 50 feet to one another. (100 feet along flowline)
3. Trench and grade board must be level. Align to follow contours of site.
4. Support post spacing as required by soil conditions to ensure grade board remains level.
**Design Criteria for Splashblocks**

A typical downspout splashblock is shown in Figure 3.1.7. 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, as long as the runoff can travel through a well-vegetated area as described below.

The following apply to the use of splashblocks:

1. Maintain a vegetated flowpath of at least 50 feet between the discharge point and any property line, structure, slope steeper than 15%, stream, wetland, lake, or other impervious surface. Sensitive area buffers may count toward flowpath lengths.

2. A maximum of 700 square feet of roof area may drain to each splashblock.

3. For purposes of maintaining adequate separation of flows discharged from adjacent dispersion devices, the vegetated flowpath segment for the splashblock must not overlap with other flowpath segments, except those associated with sheet flow from a non-native pervious surface.

4. Place a splashblock or a pad of crushed rock (2 feet wide by 3 feet long by 6 inches deep) at each downspout discharge point.

5. No erosion or flooding of downstream properties may result.

6. Have a geotechnical engineer or a licensed geologist, hydrogeologist, or engineering geologist evaluate runoff discharged towards landslide hazard areas. Do not place splashblocks on or above slopes greater than 15% or above erosion hazard areas without evaluation by a professional engineer with geotechnical expertise or a licensed geologist, hydrogeologist, or engineering geologist, and approval by the City of Olympia.

7. For sites with septic systems, the discharge point must be downslope of the primary and reserve drainfield areas. This requirement may be waived if site topography clearly prohibits flows from intersecting the drainfield or where site conditions (soil permeability, distance between systems, etc) indicate that this is unnecessary.
3.1.3 Perforated Stub-Out Connections (BMP T5.10C)

A perforated stub-out connection is a length of perforated pipe within a gravel-filled trench that is placed between roof downspouts and a stub-out to the local drainage system. Figure 3.1.8 illustrates a perforated stub-out connection. These systems are intended to provide some infiltration during drier months. During the wet winter months, they may provide little or no flow control.
**Applications & Limitations** Perforated stub-outs are not appropriate when seasonal water table is less than one foot below trench bottom.

In projects subject to Core Requirement #5 (see Volume I), perforated stub-out connections may be used only when all other higher priority on-site stormwater management BMPs are not feasible, per the criteria for each of those BMPs.

Select the location of the connection to allow a maximum amount of runoff to infiltrate into the ground (ideally a dry, relatively well drained, location). To facilitate maintenance, do not locate the perforated pipe portion of the system under impervious or heavily compacted (e.g., driveways and parking areas) surfaces. Use the same setbacks as for infiltration trenches in Section 3.1.1.

Have a licensed geologist, hydrogeologist, or engineering geologist evaluate potential runoff discharges towards landslide hazard areas. Do not place the perforated portion of the pipe on or above slopes greater than 20% or above erosion hazard areas without evaluation by a professional engineer with geotechnical expertise or qualified geologist and jurisdiction approval.

For sites with septic systems, the perforated portion of the pipe must be downgradient of the drainfield primary and reserve areas. This requirement can be waived if site topography will clearly prohibit flows from intersecting the drainfield or where site conditions (soil permeability, distance between systems, etc) indicate that this is unnecessary.

**Design Criteria** Perforated stub-out connections consist of at least 10 feet of perforated pipe per 5,000 square feet of roof area laid in a level, 2-foot wide trench backfilled with washed drain rock. Extend the drain rock to a depth of at least 8 inches below the bottom of the pipe and cover the pipe. Lay the pipe level and cover the rock trench with filter fabric and 6 inches of fill (see Figure 3.1.8).

**Runoff Model Representation** Any flow reduction is variable and unpredictable. No computer modeling techniques are allowed that would predict any reduction in flow rates and volumes from the connected area.
3.2 Detention Facilities

This section presents the methods, criteria, and details for design and analysis of detention facilities. These facilities provide for the temporary storage of increased surface water runoff resulting from development pursuant to the performance standards set forth in Core Requirement #7 for flow control (Volume I).

There are three primary types of detention facilities described in this section: detention ponds, tanks, and vaults.

3.2.1 Detention Ponds

The design criteria in this section are for detention ponds. However, many of the criteria also apply to infiltration ponds (Section 3.3 and Volume V), and water quality wetponds and combined detention/wetponds (Volume V).
Stormwater detention facilities that can impound 10 acre-feet (435,600 cubic feet; 3.26 million gallons) or more with the water level measured 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. Peak flows from dam failures are typically much larger than the 100-year flows which these ponds are typically designed to accommodate.

The Dam Safety Office of the Department of Ecology uses consequence dependent design levels for critical project elements. There are eight design levels with storm recurrence intervals ranging from 1 in 500 for design step, 1 to 1 in 1,000,000 for design step 8. The specific design step for a particular project depends on the downstream population and other resources that would be at risk from a failure of the dam. Precipitation events more extreme than the 100-year event may be rare at any one location, but have historically occurred somewhere within Washington State every few years on average.

With regard to the engineering design of stormwater detention facilities, the primary effect of the state’s dam safety requirements is in sizing the emergency spillway to accommodate the runoff from the dam safety design storm without overtopping the dam. The hydrologic computation procedures are the same as for the original pond design, except that the computations must use more extreme precipitation values and the appropriate dam safety design storm hyetographs. This information is described in detail within guidance documents developed by and available from the Dam Safety Office. In addition to the other design requirements for stormwater detention BMPs described elsewhere in this manual, dam safety requirements should be an integral part of planning and design for stormwater detention ponds. It is most cost-effective to consider these requirements right from the beginning of the project.

In addition to the hydrologic and hydraulic issues related to precipitation and runoff, other dam safety requirements include geotechnical issues, construction inspection and documentation, dam breach analysis, inundation mapping, emergency action planning, and periodic inspections by project owners and by Dam Safety engineers. All of these requirements, plus procedural requirements for plan review and approval and payment of construction permit fees are described in detail in guidance documents developed by and available from the Dam Safety Office.

In addition to the written guidance documents, Dam Safety engineers are available to provide technical assistance to project owners and design engineers in understanding and addressing the dam safety requirements for their specific project. In the interest of providing a smooth integration of dam safety requirements into the stormwater detention project and streamlining Dam Safety’s engineering review and issuance of the construction permit, it is recommended and requested that Dam Safety be contacted early in the facilities planning process. 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.
Design Criteria

Standard details for detention ponds are shown in Figure 3.2.1 through Figure 3.2.3. Control structure details are provided in Section 3.2.4.

General

1. Ponds must be designed as flow-through systems (however, parking lot storage may be utilized through a back-up system; see Section 3.2.5). Developed flows must enter through a conveyance system separate from the control structure and outflow conveyance system. Maximizing distance between the inlet and outlet is encouraged to promote sedimentation.

2. Pond bottoms shall be level and include one foot of dead storage below the inlet and outlet to provide sediment storage and to maximize small storm/dry season capture and infiltration.

3. Design guidelines for outflow control structures are specified in Section 3.2.4.

4. A geotechnical analysis and report must be prepared for slopes over 15%, or if located within 200 feet of the top of a slope steeper than 40%, or landslide hazard area. The scope of the geotechnical report should include the assessment of impoundment seepage on the stability of the natural slope where the facility will be located within the setback limits set forth in this section.

Side Slopes

1. Interior side slopes up to the emergency overflow water surface shall not be steeper than 3H:1V.

2. Exterior side slopes must not be steeper than 2H:1V unless analyzed for stability by a geotechnical engineer.

3. Where ponds abut a right-of-way, a 5-foot wide bench shall be provided between ponds and the edge of right-of-way. The bench shall be landscaped to screen ponds from the street. If a fence is provided, it shall be located between the pond and the vegetated screening.

4. Pond walls may be vertical retaining walls, provided:
   - They are constructed of an architectural faced reinforced concrete per Section 3.2.3, Material.
   - A fence is provided along the top of the wall.
   - The retaining wall(s) is less than 50% of the perimeter of the pond; the
   - The design is stamped by a licensed civil engineer with structural expertise. Other retaining walls such as rockeries, concrete, masonry unit walls, and keystone type wall may be used if designed by a geotechnical engineer or a civil engineer with structural expertise;
   - There are no structures or roadways placed above the structural elements of the walls; and,
   - The wall height is limited to 4 feet unless a 4 feet wide bench is used between sections of the wall and then each section must be limited to 4 feet high. (Structural elements of the wall include all the elements of the wall needed to maintain their structural integrity-this includes reinforcing material placed behind).
**Embankments**

1. Have a professional engineer with geotechnical expertise design pond berm embankments higher than 6 feet.

2. For berm embankments 6 feet or less, the minimum top width should be 6 feet or as recommended by a geotechnical engineer.

3. Construct pond berm embankments 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.

4. Construct pond berm embankments greater than 4 feet in height by excavating a key equal to 50 percent of the berm embankment cross-sectional height and width, unless specified otherwise by a geotechnical engineer.

5. Embankment compaction should be accomplished in such a manner as to produce a dense, low permeability engineered fill that can tolerate post-construction settlements with a minimum of cracking. Place the embankment fill on a stable subgrade and compact to a minimum of 95% of the Standard Proctor Maximum Density, ASTM Procedure D698. Placement moisture content should lie within 1% dry to 3% wet of the optimum moisture content.

6. Construct the berm embankment of soils with the following characteristics: a minimum of 20% silt and clay, a maximum of 60% sand, a maximum of 60% silt, with nominal gravel and cobble content. Soils outside this specified range can be used, provided the design satisfactorily addresses the engineering concerns posed by these soils. The paramount concerns with these soils are their susceptibility to internal erosion or piping and to surface erosion from wave action and runoff on the upstream and downstream slopes, respectively. *Note: In general, excavated glacial till is well suited for berm embankment material.*

7. Place anti-seepage filter-drain diaphragms on outflow pipes in berm embankments impounding water with depths greater than 8 feet at the design water surface. For more information, refer to Department of ecology Dam Safety Guidelines, Part IV, Section 3.3.B. An electronic version of the Dam Safety Guidelines is available in PDF format at [https://fortress.wa.gov/ecy/publications/summarypages/9255d.html](https://fortress.wa.gov/ecy/publications/summarypages/9255d.html).
Overflow

1. Provide a primary overflow (usually a riser pipe within the control structure; see Section 3.2.4) in all ponds, tanks, and vaults to bypass the 100-year developed peak flow over or around the restrictor system. This assumes the facility will be full due to plugged orifices or high inflows; the primary overflow is intended to protect against breaching of a pond embankment (or overflows of the upstream conveyance system in the case of a detention tank or vault). The design must provide controlled discharge directly into the downstream conveyance system or another acceptable discharge point.

2. Provide a secondary inlet to the control structure in ponds as additional protection against overtopping should the inlet pipe to the control structure become plugged. A slated opening (“jailhouse window”) in the control structure manhole functions as a weir (see Figure 3.2.2) when used as a secondary inlet.

   Note: The maximum circumferential length of this opening must not exceed one-half the control structure circumference. The “birdcage” overflow structure as shown in Figure 3.2.3 may also be used as a secondary inlet.

Emergency Overflow Spillway

1. In addition to the above overflow provisions, 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 above). 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 acceptable discharge point.

2. Provide emergency overflow spillways 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 II manhole fitted with a birdcage as shown in Figure 3.2.3. The emergency overflow structure must be designed to pass the 100-year developed peak flow, with a minimum 10% overvolume and 1 foot of freeboard, directly to the downstream conveyance system or another acceptable discharge point. Where an emergency overflow spillway would discharge to a slope steeper than 15%, consideration should be given to providing an emergency overflow structure in addition to the spillway.

3. Armor the emergency overflow spillway with riprap in conformance with BMP C209: Outlet Protection in Volume II. 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 3.2.2).

4. 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 3.2.2 may be used.
5. Design the emergency overflow spillway to allow a minimum of 1 foot of freeboard.

Access
The following guidelines for access may be used.

1. Provide maintenance access road(s) to the control structure and other drainage structures associated with the pond (e.g., inlet or bypass structures). It is recommended that manhole and catch basin lids be in or at the edge of the access road and at least three feet from a property line.

2. An access ramp is needed for removal of sediment with a trackhoe and truck. Extend the ramp to the pond bottom if the pond bottom is greater than 1,500 square feet (measured without the ramp). If the pond bottom is less than 1,500 square feet (measured without the ramp), the ramp may end at an elevation 4 feet above the pond bottom.

On large, deep ponds, provide truck access to the pond bottom via an access ramp 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 wetpond or combined pond (trackhoes can negotiate interior pond side slopes).

3. The internal berm of a wetpond, or combined detention and wetpond, may be used for access if all of the following apply:
   - The internal 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 internal berm is designed to support a loaded truck, considering the berm is normally submerged and saturated.

4. Access ramps must meet the requirements for design and construction of access roads specified below.

5. If a fence is required, access should be limited by a double-posted gate or by bollards – two fixed bollards on each side of the access road and two removable bollards equally located between the fixed bollards.

Design of Access Roads
The design guidelines for access roads:

1. A maximum grade of 15%.
2. A minimum of 40 feet outside turning radius.
3. Locate fence gates only on straight sections of road.
4. 15 feet in width on curves and 12 feet on straight sections.
5. Provide a paved apron where access roads connect to paved public roadways.

Construction of Access Roads
Construct access roads with permeable pavement, gravel surface, or modular grid pavement. All surfaces must conform to the jurisdictional standards and manufacturer's specifications.
**Fencing**

1. A fence is needed at the emergency overflow water surface elevation, or higher, where a pond interior side slope is steeper than 3H:1V, or where the impoundment is a wall greater than 24 inches in height. The fence need only be constructed for those slopes steeper than 3H:1V. Other applicable regulations such as the International Building Code or Uniform Building Code may require fencing of vertical walls. If more than 10 percent of slopes are steeper 3H:1V, it is recommended that the entire pond be fenced.

Detention ponds on school sites will need to comply with safety standards developed by the Department of Health (DOH) and the Superintendent for Public Instruction (SPI). These standards include what is called a ‘non-climbable fence.’ One example of a non-climbable fence is a chain-link fence with a tighter mesh, so children cannot get a foot-hold for climbing. For school sites, and possibly for parks and playgrounds, the designer should consult the DOH’s Office of Environmental Programs.

Fences discourage access to portions of a pond where steep side slopes (steeper than 3:1) increase the potential for slipping into the pond. Fences also serve to guide those who have fallen into a pond to side slopes that are flat enough (flatter than 3:1 and unfenced) to allow for easy escape.

2. For example fence designs, see WSDOT Standard Plan L-2, Type 1 or Type 3 chain link fence. The fence may be a minimum of 4 feet in height if the depth of the impoundment (measured from the lowest elevation in the bottom of the impoundment, directly adjacent to the bottom of the fenced slope, up to the emergency overflow water surface) is 5 feet or less. For example designs, see WSDOT Standard Plan L-2, Type 4 or Type 6 chain link fence.

3. Access road gates may be 16 feet in width consisting of two swinging sections 8 feet in width. Provide additional vehicular access gates as needed to facilitate maintenance access.

4. Pedestrian access gates (if needed) should be 4 feet in width.

5. Vertical metal balusters or 9 gauge galvanized steel fabric with bonded vinyl coating can be used as fence material. For steel fabric fences, consider the following aesthetic features:
   a) Vinyl coating that is compatible with the surrounding environment (e.g., green in open, grassy areas and black or brown in wooded areas). All posts, cross bars, and gates may be painted or coated the same color as the vinyl clad fence fabric.
   b) Fence posts and rails that conform to WSDOT Standard Plan L-2 for Types 1, 3, or 4 chain link fence.

6. For metal baluster fences, applicable building code standards apply.

7. Wood fences may be used in subdivisions where the fence will be maintained by homeowners associations or adjacent lot owners.

8. Wood fences should have pressure treated posts (ground contact rated) either set in 24-inch deep concrete footings or attached to footings by galvanized brackets. Rails and fence boards may be cedar, pressure-treated fir, or hemlock.
9. Where only short stretches of the pond perimeter (< 10 percent) have side slopes steeper than 3:1, split rail fences (3 foot minimum height) or densely planted thorned hedges (e.g., tall Oregon grape, Nootka rose) may be used in place of a standard fence.

10. Fences installed along public rights-of-way and adjoining public spaces shall be located between the pond and vegetated screening.

**Signage**

Detention ponds, infiltration ponds, wetponds, and combined ponds should have a sign placed for maximum visibility from adjacent streets, sidewalks, and paths. Contact City of Olympia for required sign specifications.

**Right-of-Way**

Right-of-way may be needed for detention pond maintenance. Any tract not abutting public right-of-way shall have a 15-20 foot wide extension of the tract to an acceptable access location.

**Setbacks**

Facilities shall be setback a minimum of 20 feet from any structure, property line, and any required vegetative buffer. The detention pond water surface at the pond outlet invert elevation must be set back 100 feet from proposed or existing septic system drainfields. Refer to Thurston County Environmental Health codes and applicable building codes for additional setback requirements. All facilities must be a minimum of 50 feet from the top of any steep (greater than 15%) slope. A geotechnical analysis and report must be prepared addressing the potential impact of the facility on a slope steeper than 15%.

**Seeps and Springs**

Intermittent seeps along cut slopes are typically fed by a shallow ground water source (interflow) flowing along a relatively impermeable soil stratum. These flows are storm driven and should discontinue after a few weeks of dry weather. However, more continuous seeps and springs, which extend through longer dry periods, are likely from a deeper ground water source. When continuous flows are intercepted and directed through flow control facilities, adjustments to the facility design may have to be made to account for the additional base flow.

**Planting Requirements**

The entire pond site, including the interior side slopes and below the design water surface elevation, shall be amended with compost or topsoil 12 inches deep to BMP T5.13 or equivalent, and planted and sodded or seeded with an appropriate seed mixture.

**Landscaping**

Landscaping is required for most stormwater tract areas (see below for areas not to be landscaped). However, if provided, landscaping should 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.
**Follow these guidelines if landscaping is proposed for facilities:**

1. Do not plant trees or shrubs on berms meeting the criteria of dams regulated for safety.
2. Do not plant trees or shrubs within 10 feet of inlet or outlet pipes or manmade drainage structures such as spillways or flow spreaders. Avoid using species with roots that seek water, such as willow or poplar, within 50 feet of pipes or manmade structures.
3. Restrict planting on berms that impound water permanently or temporarily during storms. This restriction does not apply to cut slopes that form pond banks, only to berms.
   a) Do not plant trees or shrubs on portions of water-impounding berms taller than four feet high. Plant only grasses on berms taller than four feet. Grasses allow unobstructed visibility of berm slopes for detecting potential dam safety problems such as animal burrows, slumping, or fractures in the berm.
   b) 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 3.2.1 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.
      - **Note:** The internal berm in a wetpond is not subject to this planting restriction since the failure of an internal berm would be unlikely to create a safety problem.
4. Plant all landscape material, including grass, in good topsoil. Make native underlying soils suitable for planting by amending with 4 inches of mature composted material (as defined in Chapter 173-350 WAC) tilled into the subgrade. Refer to BMP T5.13.
5. Soil in which trees or shrubs are planted may need additional enrichment or additional compost top-dressing. Consult a nursery, landscape professional, or arborist for site-specific recommendations.
6. For a naturalistic effect as well as ease of maintenance, plant trees in clumps to form “landscape islands” rather than spacing evenly.
7. The landscaped islands should 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 six feet 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.
8. Evergreen or columnar deciduous trees along the west and south sides of ponds are recommended to reduce thermal heating. Evergreen trees or shrubs are preferred to avoid problems associated with leaf drop. Columnar deciduous trees (e.g., hornbeam, Lombardy poplar) typically have fewer leaves than other deciduous trees.
   In addition to shade, trees and shrubs also discourage waterfowl use and the attendant phosphorus enrichment problems they cause. Setback trees so the branches will not extend over the pond.
9. Drought tolerant species are recommended.
**Table 3.2.1**
Small Trees and Shrubs with Fibrous Roots

<table>
<thead>
<tr>
<th>Small Trees / High Shrubs</th>
<th>Low Shrubs</th>
</tr>
</thead>
<tbody>
<tr>
<td>*Red twig dogwood</td>
<td>*Snowberry</td>
</tr>
<tr>
<td><em>(Cornus stolonifera)</em></td>
<td><em>(Symproricarpus albus)</em></td>
</tr>
<tr>
<td>*Serviceberry</td>
<td>*Salmonberry</td>
</tr>
<tr>
<td><em>(Amelanchier alnifolia)</em></td>
<td><em>(Rubus spectabilis)</em></td>
</tr>
<tr>
<td>*Filbert</td>
<td>Rosa rugosa</td>
</tr>
<tr>
<td><em>(Corylus cornuta, others)</em></td>
<td><em>(avoid spreading varieties)</em></td>
</tr>
<tr>
<td>Highbush cranberry</td>
<td>Rock rose</td>
</tr>
<tr>
<td><em>(Vaccinium opulus)</em></td>
<td><em>(Cistus spp.)</em></td>
</tr>
<tr>
<td>Blueberry</td>
<td>Ceanothus spp.</td>
</tr>
<tr>
<td><em>(Vaccinium spp.)</em></td>
<td><em>(choose hardier varieties)</em></td>
</tr>
<tr>
<td>Fruit trees on dwarf rootstock</td>
<td>New Zealand flax</td>
</tr>
<tr>
<td><em>(Phormium penax)</em></td>
<td><em>(Phormium penax)</em></td>
</tr>
<tr>
<td>Rhododendron <em>(native and ornamental varieties)</em></td>
<td>Ornamental grasses</td>
</tr>
<tr>
<td>*Native species</td>
<td><em>(e.g., Miscanthis, Pennisetum)</em></td>
</tr>
</tbody>
</table>

**Guidelines for Naturalistic Planting.** Stormwater facilities may sometimes be located within open space tracts if “natural appearing.” Two generic kinds of naturalistic planting are outlined below, but other options are also possible. Native vegetation is preferred in naturalistic plantings.

**Open Woodland.** In addition to the general landscaping guidelines above, the following are recommended.

1. Landscaped islands (when mature) should cover a minimum of 30 percent or more of the tract, exclusive of the pond area.
2. Underplant tree clumps with shade-tolerant shrubs and groundcover plants. The goal is to provide a dense understory that need not be weeded or mowed.
3. Place landscaped islands at several elevations rather than “ring” the pond, and vary the size of clumps from small to large to create variety.
4. Not all islands need to have trees. Shrub or groundcover clumps are acceptable, but lack of shade should be considered in selecting vegetation.

**Note:** Landscaped islands are best combined with the use of wood-based mulch (hog fuel) or chipped on-site vegetation for erosion control (only for slopes above the flow control water surface). It is often difficult to sustain a low-maintenance understory if the site was previously hydrosedeed. Compost or composted mulch (typically used for constructed wetland soil) can be used below the flow control water surface (materials that are resistant to and preclude flotation). The method of construction of soil landscape systems can also cause natural selection of specific plant species. Consult a soil restoration or wetland soil scientist for site-specific recommendations.

**Northwest Savannah or Meadow.** In addition to the general landscape guidelines above, the following are recommended.

1. Landscape islands (when mature) should cover 10 percent or more of the site, exclusive of the pond area.
2. Planting groundcovers and understory shrubs is encouraged to eliminate the need for mowing under the trees when they are young.

3. Place landscape islands at several elevations rather than “ring” the pond.

Plant the remaining site area with an appropriate grass seed mix, which may include meadow or wildflower species. Native or dwarf grass mixes are preferred. Table 3.2.2 below gives an example of dwarf grass mix developed for central Puget Sound. Apply grass seed at 2.5 to 3 pounds per 1,000 square feet.

*Note: Amended soil or good topsoil is required for all plantings.*

Creation of areas of emergent vegetation in shallow areas of the pond is recommended. Native wetland plants, such as sedges (Carex sp.), bulrush (Scirpus sp.), water plantain (Alisma sp.), and burreed (Sparganium sp.) are recommended. If the pond does not hold standing water, a clump of wet-tolerant, non-invasive shrubs, such as salmonberry or snowberry, is recommended below the detention design water surface.

*Note: This landscape style is best combined with the use of grass or sod for site stabilization and erosion control.*

**Seed Mixes.** The seed mixes listed below were developed for central Puget Sound.

<table>
<thead>
<tr>
<th>Table 3.2.2 Stormwater Tract “Low Grow” Seed Mix</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Seed Name</strong></td>
</tr>
<tr>
<td>Dwarf tall fescue</td>
</tr>
<tr>
<td>Dwarf perennial rye “Barclay”*</td>
</tr>
<tr>
<td>Red fescue</td>
</tr>
<tr>
<td>Colonial bentgrass</td>
</tr>
</tbody>
</table>

* If wildflowers are used and sowing is done before Labor Day, the amount of dwarf perennial rye can be reduced proportionately to the amount of wildflower seed used.
Figure 3.2.1 - Typical Detention Pond

Note:
This detail is a schematic representation only. Actual configuration will vary depending on specific site constraints and applicable design criteria.

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Figure 3.2.2 - Typical Detention Pond Sections

SECTION A-A

NTS

SECTION a-a

NTS

SECTION B-B has 2 options

NTS

SECTION B-B

Emergency Overflow Spillway

NTS

SECTION C-C

NTS
Figure 3.2.3 - Overflow Structure

PLAN VIEW

- 3/4" diameter smooth bars equally spaced (4" O.C. max.)
- 4 hook clamps evenly placed see detail below
- Provide maintenance access by welding 4 crossbars to 4 vertical bars as shown. Hinge upper ends with flanges/bolts and provide locking mechanism (padlock) on lower end. Locate steps directly below
- lower steel band 3/4" x 4" wide formed to fit in groove of C.B. riser
- 3/4" dia. smooth round bars welded equally spaced. Bars shall be welded to upper & lower bands (24 bands evenly spaced see note 1)

SECTION A-A

- Type 2 CB
- Standard galvanized steps or ladder

DETAIL HOOK CLAMP

NOTES:
1. Dimensions are for illustration on 54" diameter CB. For different diameter CB's adjust to maintain 45° angle on "vertical" bars and 7" o.c. maximum spacing of bars around lower steel band.
2. Metal parts must be corrosion resistant; steel bars must be galvanized.
3. This debris barrier is also recommended for use on the inlet to roadway cross-culverts with high potential for debris collection (except on type 2 streams)
**Maintenance**

General. Maintenance is of primary importance if detention ponds are to continue to function as originally designed. The facility owner (government, a designated group such as a homeowners’ association, or some individual) must accept the responsibility for maintaining the structures and the impoundment area. Formulate a specific maintenance plan outlining the schedule and scope of maintenance operations. Achieve debris removal in detention basins by using trash racks or other screening devices.

Design with maintenance in mind. Good maintenance will be crucial to successful use of the impoundment. Hence, build in provisions to facilitate maintenance operations into the project when it is installed. Maintenance must be a basic consideration in design and in determination of first cost. See Volume IV, Chapter 4 for specific maintenance requirements.

Handle any standing water and sediments removed during the maintenance operation in a manner consistent with Appendix IV-G in Volume IV.

Vegetation. If a shallow marsh is established, then periodic removal of dead vegetation may be necessary. Since decomposing vegetation can release pollutants captured in the wet pond, especially nutrients, it may be necessary to harvest dead vegetation annually prior to the winter wet season. Otherwise the decaying vegetation can export pollutants out of the pond and also can cause nuisance conditions to occur. If harvesting is to be done in the wetland, have a wetland scientist prepare a written harvesting procedure and submitted it with the drainage design to the City.

Sediment. Maintenance of sediment forebays and attention to sediment accumulation within the pond is extremely important. Continually monitor sediment deposition in the basin. Owners, operators, and maintenance authorities should be aware that significant concentrations of metals (e.g., lead, zinc, and cadmium) as well as some organics such as pesticides, may be expected to accumulate at the bottom of these treatment facilities. Regularly conduct testing of sediment, especially near points of inflow, to determine the leaching potential and level of accumulation of potentially hazardous material before disposal.

**Methods of Analysis**

Detention Volume and Outflow. Design volumes and outflows for detention ponds in accordance with Core Requirements #7 in Volume I and the hydrologic analysis and design methods in Chapter 2 of this Volume. Design guidelines for restrictor orifice structures are given in Section 3.2.4.

Note: The design water surface elevation is the highest elevation which occurs in order to meet the required outflow performance for the pond.

Detention Ponds in Infiltrative Soils. Detention ponds may occasionally be sited on till soils that are sufficiently permeable for a properly functioning infiltration system (see Section 3.3). 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 3.3 for infiltration ponds, including a soils report, testing, ground water protection, presettling, and construction techniques.
**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 3.2.4, for example, would be:

\[
Q_{100} = C \left(2g\right)^{1/2} \left[ \frac{2}{3} LH^{3/2} + \frac{8}{15} \left(\tan \theta \right) H^{5/2} \right] \text{ (equation 1)}
\]

Where

- \( Q_{100} \) = 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)
- \( \theta \) = angle of side slopes

\( Q_{100} \) is either the peak volumetric flow rate calculated using a 10-minute time step from the 100-year, 24-hour storm and a Type 1A distribution, or the 100-year, 1-hc flow, indicated by an approved continuous runoff model, multiplied by a factor of 1.6.

Assuming \( C = 0.6 \) and \( \tan \theta = 3 \) (for 3:1 slopes), the equation becomes:

\[
Q_{100} = 3.21 \left[ LH^{3/2} + 2.4 H^{5/2} \right] \text{ (equation 2)}
\]

To find width \( L \) for the weir section, the equation is rearranged to use the computed \( Q_{100} \) and trial values of \( H \) (0.2 feet minimum):

\[
L = \left[ Q_{100} / (3.21 H^{3/2}) \right] - 2.4 H \text{ or } 6 \text{ feet minimum} \text{ (equation 3)}
\]

---

### 3.2.2 Detention Tanks

Detention tanks are underground storage facilities typically constructed with large diameter corrugated metal pipe. Standard detention tank details are shown in Figure 3.2.5 and Figure 3.2.6. Control structure details are shown in Section 3.2.4.
**Design Criteria**

**General.** Typical design guidelines are as follows:

1. Tanks may be designed as flow-through systems with manholes in line (see Figure 3.2.5) to promote sediment removal and facilitate maintenance. Tanks may be designed as back-up systems if preceded by water quality facilities, since little sediment should reach the inlet/control structure and low head losses can be expected because of the proximity of the inlet/control structure to the tank.

2. Locate the detention tank bottom 1.0 feet below the inlet and outlet to provide dead storage for sediment.

3. Use a 36-inch minimum pipe diameter.

4. Tanks larger than 36 inches may be connected to each adjoining structure with a short section (2-foot maximum length) of 36-inch minimum diameter pipe.

5. Refer to the details of outflow control structures in Section 3.2.4.

*Note: Control and access manholes should have additional ladder rungs to allow ready access to all tank access pipes when the catch basin sump is filled with water (see Figure 3.2.8, plan view).*

**Materials.** 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, use of galvanized materials in stormwater facilities and conveyance systems is discouraged. Where other metals, such as aluminum or stainless steel, or plastics are available, they should be used.

Pipe material, joints, and protective treatment for tanks should be in accordance with Section 9.05 of the WSDOT/APWA Standard Specifications.

**Structural Stability.** Tanks must meet structural requirements for overburden support and traffic loading if appropriate. Accommodate H-20 live loads for tanks lying under parking areas and access roads. Design metal tank end plates for structural stability at maximum hydrostatic loading conditions. Flat end plates generally require thicker gage material than the pipe and/or require reinforcing ribs. Place tanks on stable, well consolidated native material with a suitable bedding. Do not place tanks in fill slopes, unless analyzed in a geotechnical report for stability and constructability.

**Buoyancy.** In moderately pervious soils where seasonal ground water may induce flotation, balance buoyancy tendencies by either ballasting with backfill or concrete backfill, providing concrete anchors, increasing the total weight, or providing subsurface drains to permanently lower the ground water table. Calculations that demonstrate stability must be documented.

**Access.** The following guidelines for access may be used.

1. The maximum depth from finished grade to tank invert should be 20 feet.

2. Position access openings a maximum of 50 feet from any location within the tank.

3. All tank access openings may have round, solid locking lids (usually 1/2 to 5/8-inch diameter Allen-head cap screws).

4. Thirty-six-inch minimum diameter CMP riser-type manholes (Figure 3.2.7) of the same gage as the tank material may be used for access along the length of the tank and at the upstream terminus of the tank in a backup system.
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 tank.

5. Make all tank access openings readily accessible by maintenance vehicles.

6. Tanks must comply with the OSHA confined space requirements, which includes 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.

**Access Roads.** Access roads are needed to all detention tank control structures and risers. Design and construct access roads as specified for detention ponds in Section 3.2.1.

**Right-of-Way.** Right-of-way may be needed for detention tank maintenance. It is recommended that any tract not abutting public right-of-way have a 15 to 20-foot wide extension of the tract to accommodate an access road to the facility.

**Setbacks.** It is recommended that facilities be a minimum of 20 feet from any structure, property line, and any vegetative buffer required by the local government and from any septic drainfield. However, the setback requirements are generally specified by the local government, uniform building code, or other statewide regulation and may be different from those mentioned above.

All facilities must be a minimum of 50 feet from the top of any steep (greater than 15%) slope. A geotechnical analysis and report must be prepared addressing the potential impact of the facility on a slope steeper than 15%.

**Maintenance.** Build in provisions to facilitate maintenance operations into the project when it is installed. Maintenance must be a basic consideration in design and in determination of first cost. See Volume IV, Chapter 4 for specific maintenance requirements.

**Methods of Analysis**

**Detention Volume and Outflow**

The volume and outflow design for detention tanks must be in accordance with Core Requirement #7 in Volume I and the hydrologic analysis and design methods in Chapter 2. Restrictor and orifice design are given in Section 3.2.4.
Figure 3.2.5 - Typical Detention Tank

PLAN VIEW

"Flow-through" system shown solid.
Designs for "flow backup" system and parallel tanks shown dashed

SECTION A-A

"Flow through" system shown solid.

NOTE:
All metal parts corrosion resistant.
Steel parts galvanized and asphalt coated (Treatment 1 or better).
Notes:
1. Use adjusting blocks as required to bring frame to grade.
2. All materials to be aluminum or galvanized and asphalt coated (Treatment 1 or better).
3. Must be located for access by maintenance vehicles.
4. May substitute WSDOT special Type IV manhole (RCP only).

3.2.3 Detention Vaults

Detention vaults are box-shaped underground storage facilities typically constructed with reinforced concrete. A standard detention vault detail is shown in Figure 3.2.7. Control structure details are shown in Section 3.2.4.
Design Criteria

General. Typical design guidelines are as follows:

1. Detention vaults may be designed as flow-through systems with bottoms level (longitudinally) or sloped toward the inlet to facilitate sediment removal. Maximize the distance between the inlet and outlet as feasible.

2. The detention vault bottom may slope at least 5 percent from each side towards the center, forming a broad “v” one foot below the outlet to facilitate sediment removal. More than one “v” may be used to minimize vault depth. However, the vault bottom may be flat with 0.5-1 foot of sediment storage if removable panels are provided over the entire vault. It is recommended that the removable panels be at grade, have stainless steel lifting eyes, and weigh no more than 5 tons per panel.

3. Elevate the invert elevation of the outlet above the bottom of the vault to provide an average 6 inches of sediment storage over the entire bottom. Also, elevate the outlet a minimum of 2 feet above the orifice to retain oil within the vault.

4. Details of outflow control structures are given in Section 3.2.4.

Materials. Minimum 3,000 psi structural reinforced concrete may be used for detention vaults. Provide all construction joints with water stops.

Structural Stability. All vaults must meet structural requirements for overburden support and H-20 traffic loading (See Standard Specifications for Highway Bridges, latest edition, American Association of State Highway and Transportation Officials). Vaults located under roadways must meet any live load requirements of the City of Olympia. Design cast-in-place wall sections as retaining walls. Structural designs for cast-in-place vaults must be stamped by a licensed civil engineer with structural expertise. Place vaults on stable, well-consolidated native material with suitable bedding. Do not place vaults in fill slopes, unless analyzed in a geotechnical report for stability and constructability.

Buoyancy. In moderately pervious soils where seasonal groundwater may induce floatation, buoyancy tendencies must be balanced either by ballasting with backfill or concrete backfill, providing concrete anchors, increasing the total weight, or providing subsurface drains to permanently lower the groundwater table. Calculations that demonstrate stability must be documented.

Access. Provide access over the inlet pipe and outlet structure. Use the following guidelines for access.

1. Position access openings a maximum of 50 feet from any location within the tank. Additional access points may be needed on large vaults. Provide access to each “v” if more than one “v” is provided in the vault floor.

2. For vaults with greater than 1,250 square feet of floor area, provide a 5' by 10' removable panel over the inlet pipe (instead of a standard frame, grate and solid cover). Or, provide a separate access vault as shown in Figure 3.2.7.

3. For vaults under roadways, locate the removable panel outside the travel lanes. Or, provide multiple standard locking manhole covers. Ladders and hand-holds need only be provided at the outlet pipe and inlet pipe, and as needed to meet OSHA confined space requirements. Vault providing
manhole access at 12-foot spacing need not provide corner ventilation pipes as specified in Item 10 below.

4. All access openings, except those covered by removable panels, may have round, solid locking lids, or 3-foot square, locking diamond plate covers.

5. Vaults with widths 10 feet or less must have removable lids.

6. The maximum depth from finished grade to the vault invert should be 20 feet.

7. Provide internal structural walls of large vaults with openings sufficient for maintenance access between cells. Size and situate the openings to allow access to the maintenance “v” in the vault floor.

8. The minimum internal height should be 7 feet from the highest point of the vault floor (not sump), and the minimum width should be 4 feet. However, 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. Also, the minimum internal height requirement may not be needed for any areas covered by removable panels.

9. Vaults must comply with the OSHA confined space requirements, which includes 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.

10. Provide ventilation pipes (minimum 12-inch diameter or equivalent) in all four corners of vaults to allow for artificial ventilation prior to entry of maintenance personnel into the vault. Or, provide removable panels over the entire vault.

Access Roads. Access roads are needed to the access panel (if applicable), the control structure, and at least one access point per cell, and they may be designed and constructed as specified for detention ponds in Section 3.2.1.

Right-of-Way. Right-of-way is needed for detention vaults maintenance. It is recommended that any tract not abutting public right-of-way should have a 15 to 20-foot wide extension of the tract to accommodate an access road to the facility.

Setbacks. It is recommended that facilities be a minimum of 20 feet from any structure, property line, and any required vegetative buffer and from any septic drainfield. Refer to Thurston County Environmental Health codes and the uniform building code for additional setback requirements.

All facilities must be a minimum of 50 feet from the top of any steep (greater than 15%) slope. A geotechnical analysis and report must be prepared addressing the potential impact of the facility on a slope steeper than 15%.

Maintenance. Build in provisions to facilitate maintenance operations into the project when it is installed. Maintenance must be a basic consideration in design and in determination of first cost. See Volume IV, Chapter 4 for specific maintenance requirements.
Methods of Analysis  

Detention Volume and Outflow

Design the volume and outflow for detention vaults in accordance with Core Requirement #7 in Volume I and the hydrologic analysis and design methods in Section 2.2.3. Restrictor and orifice design are given in Section 3.2.4.

Figure 3.2.7 - Typical Detention Vault

- All vault areas must be within 50' of an access point.
- Optional 5' x 10' access vault may be used in lieu of top access.
- Frames, grates, and round solid covers marked "DRAIN" with locking bolts.
- Capacity of outlet pipe not less than developed 100-yr design flow.
- Floor grate with 2' x 2' hinged access door (1" x 1/4" galvanized metal bars).

NOTES:
1. All metal parts must be corrosion resistant. Steel parts must be galvanized and asphalt coated (Treatment I or better).
2. Provide water stop at all cast-in-place construction joints.
3. Precast vaults shall have approved rubber gasket system.
4. Vaults ≤ 10' wide must use removable lids.
5. Prefabricated vault sections may require structural modifications to support 5' x 10' opening over main vault. Alternatively, access can be provided via a side vestibule as shown.
3.2.4 Control Structures

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 Figure 3.2.8 through Figure 3.2.10.

**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.

1. Minimum orifice diameter is 0.5 inches. *Note:* 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.

2. Orifices may be constructed on a tee section as shown in Figure 3.2.8 or on a baffle as shown in Figure 3.2.9.

3. 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 3.2.12).

4. Consider 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.

**Riser and Weir Restrictor**

1. Properly designed weirs may be used as flow restrictors (see Figure 3.2.10 and Figure 3.2.12 through Figure 3.2.14). However, they must be designed to provide for primary overflow of the developed 100-year peak flow discharging to the detention facility.

2. 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 3.2.15 can be used to calculate the head in feet above a riser of given diameter and flow.

**Access.** The following guidelines for access may be used.

1. Provide an access road to the control structure for inspection and maintenance. Design and construct the access road as specified for detention ponds in Section 3.2.1.

2. Manhole and catch basin lids for control structures must be locking, and rim elevations must match proposed finish grade.
3. Manholes and catch-basins must meet the OSHA 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, just under the access lid.

**Information Plate.** It is recommended that a brass or stainless steel plate 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
- Outflow performance criteria
- Release mechanism size, type, and invert elevation
- List of stage, discharge, and volume at one-foot increments
- Elevation of overflow
- Recommended frequency of maintenance.
Figure 3.2.8 - Flow Restrictor (TEE)

NOTES:
1. Use a minimum of a 54" diameter type 2 catch basin.
2. Outlet Capacity: 100-Year developed peak flow.
4. Frame and ladder or steps offset so:
   A. Cleanout gate is visible from top.
   B. Climb-down space is clear of riser and cleanout gate.
   C. Frame is clear of curb.
5. If metal outlet pipe connects to cement concrete pipe; outlet pipe to have smooth O.D. equal to concrete pipe I.D. less 1/4".
6. Provide at least one 3" X .090" inches support bracket anchored to concrete wall. (maximum 3'-0" vertical spacing)
7. Locate elbow restrictor(s) as necessary to provide minimum clearance as shown.
8. Locate additional ladder rungs in structures used as access to tanks or vaults to allow access when catch basin is filled with water.
Figure 3.2.9 - Flow Restrictor (Baffle)

**Section A-A NTS**
- Frames, grates and round solid covers marked “drain” with locking bolts.
-的设计水深
- elbow restrictors: see detail below
- handholds, steps or ladder
- 流量

**Section B-B NTS**
- attach shear gate control rod to support bracket on inside of access opening
- shear gate with control rod for drain
- orifice plate 10 gage minimum galvanized steel with orifice diameter 1” minimum less than diameter of concrete hole

**Plan View NTS**
- removable water-tight coupling
- plate welded to elbow with orifice as specified

**Isometric NTS**
- NOTES:
  - outlet capacity: 100 year developed peak flow
  - metal parts: corrosion resistant steel parts galvanized and asphalt coated
  - catch basin: type 2 minimum 72” diameter
- orifices: sized and located as required with lowest orifice a minimum of 2’ from base

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Figure 3.2.10 - Flow Restrictor (Weir)

NOTES:
Outlet Capacity: 100-year developed peak flow.
Metal Parts: corrosion resistant steel parts galvanized and asphalt coated.
Catch Basin: type 2 Min. 72" diameter

Baffle Wall: to be designed with concrete reinforcing as required.
Spill containment must be provided to temporarily detain oil or floatable pollutants in runoff due to accidental spill.
**Maintenance.** Control structures and catch basins have a history of maintenance-related problems and it is imperative to establish a good maintenance program for them to function properly. Typical sediment builds up inside the structure, which blocks or restricts flow to the inlet. To prevent this problem routinely clean out these structures at least twice per year. Conduct regular inspections of control structures to detect the need for non-routine cleanout, especially if construction or land-disturbing activities occur in the contributing drainage area.

Install a 15-foot wide access road to the control structure for inspection and maintenance.

Volume IV, Chapter 4 provides maintenance recommendations for control structures and catch basins.

**Methods of Analysis**

This section presents the methods and equations for design of control structure restrictor devices. Included are details for the design of orifices, rectangular sharp-crested weirs, v-notch weirs, sutro weirs, and overflow risers.

**Orifices.** Flow-through orifice plates in the standard tee section or turn-down elbow may be approximated by the general equation:

\[ Q = C A \sqrt{2gh} \]  
\[ \text{equation 4} \]

where

- \( Q = \text{flow (cfs)} \)
- \( C = \text{coefficient of discharge (0.62 for plate orifice)} \)
- \( A = \text{area of orifice (ft}^2\)\)
- \( h = \text{hydraulic head (ft)} \)
- \( g = \text{gravity (32.2 ft/sec}^2\)\)

Figure 3.2.11 illustrates this simplified application of the orifice equation.

**Figure 3.2.11 - Simple Orifice**

![Figure 3.2.11 - Simple Orifice](image)
The diameter of the orifice is calculated from the flow. The orifice equation is often useful when expressed as the orifice diameter in inches:

\[
d = \sqrt[2]{\frac{36.88Q}{h}}
\]

(equation 5)

where  
- \(d\) = orifice diameter (inches)  
- \(Q\) = flow (cfs)  
- \(h\) = hydraulic head (ft)

**Rectangular Sharp-Crested Weir.** The rectangular sharp-crested weir design shown in Figure 3.2.12 may be analyzed using standard weir equations for the fully contracted condition.

![Figure 3.2.12 - Rectangular, Sharp-Crested Weir](image)

**V-Notch Sharp-Crested Weir**

V-notch weirs as shown in Figure 3.2.13 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 3.2.14). 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):

\[ \frac{x}{b} = 1 - \frac{2}{\pi} \tan^{-1} \left( \frac{Z}{a} \right) \]  (equation 7)

where a, b, x and Z are as shown in Figure 3.2.14. The head-discharge relationship is:

\[ Q = (C_d) (b) \left( \sqrt{2ga}(h_i - \frac{a}{3}) \right) \]  (equation 8)

Values of Cd for both symmetrical and non-symmetrical sutro weirs are summarized in Table 3.2.3.

*Note: When b > 1.50 or a > 0.30, use Cd=0.6.*
### Table 3.2.3
Values of $C_d$ for Sutro Weirs

**Cd Values, Symmetrical**

<table>
<thead>
<tr>
<th>a (ft)</th>
<th>0.50</th>
<th>0.75</th>
<th>1.0</th>
<th>1.25</th>
<th>1.50</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.02</td>
<td>0.608</td>
<td>0.613</td>
<td>0.617</td>
<td>0.6185</td>
<td>0.619</td>
</tr>
<tr>
<td>0.05</td>
<td>0.606</td>
<td>0.611</td>
<td>0.615</td>
<td>0.617</td>
<td>0.6175</td>
</tr>
<tr>
<td>0.10</td>
<td>0.603</td>
<td>0.608</td>
<td>0.612</td>
<td>0.6135</td>
<td>0.614</td>
</tr>
<tr>
<td>0.15</td>
<td>0.601</td>
<td>0.6055</td>
<td>0.610</td>
<td>0.6115</td>
<td>0.612</td>
</tr>
<tr>
<td>0.20</td>
<td>0.599</td>
<td>0.604</td>
<td>0.608</td>
<td>0.6095</td>
<td>0.610</td>
</tr>
<tr>
<td>0.25</td>
<td>0.598</td>
<td>0.6025</td>
<td>0.6065</td>
<td>0.608</td>
<td>0.6085</td>
</tr>
<tr>
<td>0.30</td>
<td>0.597</td>
<td>0.602</td>
<td>0.606</td>
<td>0.6075</td>
<td>0.608</td>
</tr>
</tbody>
</table>

**Cd Values, Non-Symmetrical**

<table>
<thead>
<tr>
<th>a (ft)</th>
<th>0.50</th>
<th>0.75</th>
<th>1.0</th>
<th>1.25</th>
<th>1.50</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.02</td>
<td>0.614</td>
<td>0.619</td>
<td>0.623</td>
<td>0.6245</td>
<td>0.625</td>
</tr>
<tr>
<td>0.05</td>
<td>0.612</td>
<td>0.617</td>
<td>0.621</td>
<td>0.623</td>
<td>0.6235</td>
</tr>
<tr>
<td>0.10</td>
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<td>0.614</td>
<td>0.618</td>
<td>0.6195</td>
<td>0.620</td>
</tr>
<tr>
<td>0.15</td>
<td>0.607</td>
<td>0.6115</td>
<td>0.616</td>
<td>0.6175</td>
<td>0.618</td>
</tr>
<tr>
<td>0.20</td>
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<td>0.614</td>
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</tr>
<tr>
<td>0.25</td>
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<td>0.6125</td>
<td>0.614</td>
<td>0.6145</td>
</tr>
<tr>
<td>0.30</td>
<td>0.603</td>
<td>0.608</td>
<td>0.612</td>
<td>0.6135</td>
<td>0.614</td>
</tr>
</tbody>
</table>

**Riser Overflow.** The nomograph in Figure 3.2.15 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).
Figure 3.2.15 - Riser Inflow Curves

\[ Q_{inl}=9.739 \, DH^{3/2} \]
\[ Q_{orifice}=3.782 \, D^2H^{1/2} \]

\( Q \) in cfs, \( D \) and \( H \) in feet
Slope change occurs at weir-orifice transition
3.2.5 Other Detention Options

This section presents other design options for detaining flows to meet flow control facility requirements.

**Design Options**

**Use of Parking Lots for Additional Detention**

Private parking lots may be used to provide additional detention volume for runoff events greater than the 2-year runoff event provided all of the following are met:

1. The depth of water detained does not exceed 1 foot at any location in the parking lot for runoff events up to and including the 100-year event.
2. The gradient of the parking lot area subject to ponding is 1 percent or greater.
3. 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.
4. Fire lanes used for emergency equipment are free of ponding water for all runoff events up to and including the 100-year event.

**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:

1. The roof support structure is analyzed by a structural engineer to address the weight of ponded water.
2. The roof area subject to ponding is sufficiently waterproofed to achieve a minimum service life of 30 years.
3. The minimum pitch of the roof area subject to ponding is 1/4-inch per foot.
4. An overflow system is included in the design to safely convey the 100-year peak flow from the roof.
5. 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.

3.3 Infiltration Facilities for Flow Control and for Treatment

3.3.1 Purpose

The purpose of this section is to describe the steps required to: evaluate the suitability of a site for infiltration facilities; establish a design infiltration rate; and design facilities for infiltration.

This section applies to infiltration ponds/basins, trenches, vaults and tanks. It does not apply to downspout infiltration trenches. This section only applies to the design of Bioretention facilities, permeable pavements, and filter media devices where specific references are made in:

- Section 3.4.
- BMP T7.30 – Bioretention (see Volume V).
- BMP T5.15 – Permeable Pavement (see Volume V).

This section also highlights design criteria that are applicable to infiltration facilities serving a treatment function.
3.3.2 Description

An infiltration facility is typically an open basin (pond), trench, or buried perforated pipe used for distributing the stormwater runoff into the underlying soil (See Figure 3.3.1). Stormwater dry-wells receiving uncontaminated or properly treated stormwater can also be considered as infiltration facilities. (See Underground Injection Control Program, Chapter 173-218 WAC).

Coarser more permeable soils can be used for complying with the LID performance standard (an option in Core Requirement #5), and the flow control requirement (Core Requirement #7) provided that the infiltrated stormwater does not cause a violation of groundwater quality standards. At a minimum, pre-treatment for removal of TSS is necessary prior to discharge to the infiltration facility if any runoff comes from a pollution-generating surface. An oil control facility is also necessary for “high use” sites. Pre-treatment facilities that have the capability for removal of soluble pollutants, particularly, petroleum-related pollutants and bacteria, are advisable if Site Suitability Criterion SSC-6 is not met at the infiltration facility.

Use of the soil for treatment purposes is an option as long as it is preceded by a pre-settling basin or a basic treatment BMP. This pre-treatment should reduce the incidence of plugging and extend operational times between major maintenance.

3.3.3 Applications

Infiltration facilities for complying with the LID performance standard and the flow control requirement are used to convey stormwater runoff from new development or redevelopment to the ground and groundwater after appropriate treatment. Infiltration facilities for treatment purposes rely on the soil profile to provide treatment. In either case, manage runoff in excess of the infiltration capacity of the facilities to comply with the flow control requirement in Volume I, if flow control applies to the project.

Infiltration facilities can help accomplish the following:

- Ground water recharge.
- Discharge of uncontaminated or properly treated stormwater to dry-wells in compliance with Ecology’s UIC regulations (Chapter 173-218 WAC).
- Retrofits in limited land areas: Infiltration trenches can be considered for residential lots, commercial areas, parking lots, and open space areas.
- Flood control.
- Streambank erosion control.
Figure 3.3.1 - Typical Infiltration Pond/Basin

NOTE:
Detail is a schematic representation only. Actual configuration will vary depending on specific site constraints and applicable design criteria.
3.3.4 Steps for the Design of Infiltration Facilities - Simplified Approach

The simplified approach for the design of infiltration facilities was derived from high ground water and shallow pond sites in western Washington, and in general will produce conservative designs. This approach can be used when determining the trial geometry of the infiltration facility, for small facilities serving sites less than 1 acre in size. Designs of infiltration facilities for larger projects should use the detailed approach and may have to incorporate the results of a ground water mounding analysis as described in Section 3.3.8. Note: A ground water mounding analysis is advisable for facilities with drainage areas smaller than 1 acre if the depth to a low permeability layer (e.g., less than 0.1 inches per hour) is less than 10 feet.

The simplified approach is applicable to ponds and trenches and includes the process in Figure 3.3.2 and the following steps:

1. **Select a location:**
   
   This will be based on the ability to convey flow to the location and the expected soil conditions of the location. Conduct a preliminary surface and sub-surface characterization study (Section 3.3.5). Do a preliminary check of Site Suitability Criteria (Section 3.3.7) to initially estimate feasibility of locating an infiltration facility on the site.

2. **Estimate volume of stormwater, \( V_{\text{design}} \):**

   Estimate the volume of stormwater by using a continuous hydrograph and an approved continuous runoff model such as WWHM, MGSFlood, or KCRTS for the calculations. The runoff file developed for the project site serves as input to the infiltration basin.

   For infiltration basins sized simply to meet treatment requirements, the basin must successfully infiltrate 91% of the influent runoff file. The remaining 9% of the influent file can bypass the infiltration facility.

   For infiltration basins sized to meet the flow control standard, the basin must infiltrate either all of the influent file, or a sufficient amount of the influent file such that any overflow/bypass meets the flow duration standard. In addition, the overflow/bypass must meet the LID performance standard if it is the option chosen to meet Core Requirement #5, or if it is required of the project.

3. **Develop trial infiltration facility geometry:**

   To develop the trial facility geometry assume an infiltration rate based on previously available data, or a default infiltration rate of 0.5 inches/hour. Use this trial facility geometry to help locate the facility and for planning purposes in developing the geotechnical subsurface investigation plan.

4. **Complete More Detailed Site Characterization Study and Consider Site Suitability Criteria:**

   Information gathered during initial geotechnical and surface investigations is necessary to know whether infiltration is feasible. The geotechnical investigation evaluates the suitability of the site for infiltration, establishes the infiltration rate for design, and evaluates slope stability, foundation capacity, and other geotechnical design information needed to design and assess constructability of the facility.

   See sections 3.3.5 and 3.3.7.

5. **Determine the design infiltration rate as follows:**

   Estimate the design (long-term) infiltration rate as follows:

   - For stormwater facilities designed to fulfill Core Requirement #7, the measured saturated hydraulic conductivity rate \( K_{\text{sat}} \) shall be estimated using the grain size analysis method.
in Section 3.3.6. For stormwater facilities designed to fulfill only Core Requirement #5, the Large Scale or Small Scale Pilot Infiltration Test (PIT) method (or other local-approved method) as described in Section 3.3.6 may be used to estimate a measured (initial) saturated hydraulic conductivity ($K_{sat}$). Testing should occur between December 1 and April 1. Alternatively, the measured saturated hydraulic conductivity rate ($K_{sat}$) may be estimated using the grain size analysis method in Section 3.3.6.

- Assume that the $K_{sat}$ is the measured (initial) infiltration rate for the facility.
- Adjust this rate using the appropriate correction factors, as explained in Section 3.3.6 for the Gradation Analysis or the PIT results, to obtain the design infiltration rate ($K_{sat\ design}$).

6. **Size the facility:**

   The maximum ponded water depth should be between 2 and 6 feet with at least one foot of freeboard.

   If sizing a treatment facility, use the output files from an approved continuous runoff model to document: 1) that the facility can infiltrate 91 percent of the influent runoff file; and 2) that the Water Quality Design Storm Volume (indicated by WWHM or MGS Flood) can infiltrate through the infiltration basin surface within 48 hours. The latter can be calculated by multiplying a horizontal projection of the infiltration basin mid-depth dimensions by the estimated long-term infiltration rate; and multiplying the result by 48 hours.

   If sizing a facility to meet the flow control requirement, use the output files of an approved continuous runoff model to document that the total of any bypass and overflow meets the applicable flow control standard.

   If choosing, or required, to comply with the LID performance standard use the output files to document that the facility’s total of bypass and overflow meets the LID performance standard. **Note:** Use of distributed LID facilities dispersed throughout the project site will help achieve the LID performance standard.

7. **Construct the facility & Conduct Performance Testing:**

   Test and monitor the constructed facility per the verification testing guidelines in Section 3.3.7 to demonstrate that the facility performs as designed. Submit the results and comparisons to the pre-project measured (initial) and design rates to the City of Olympia. If the rates are lower than the design saturated hydraulic conductivity, the applicant shall implement measures to improve infiltration capability within the footprint of the constructed facility and re-test. If less intensive measures prove unsuccessful, methods identified in the contingency plan for underperformance (Section 3.3.7) shall be implemented.
Figure 3.3.2 - Steps for Design of Infiltration Facilities – Simplified Approach

Perform subsurface characterization and collection, including location of water.

Estimate infiltration rate:
- Soil gradation
- Field measurement

Estimate stormwater quantities using continuous hydrograph models.

Choose trial based on site constraints or assume $f = \text{in./hr.}$

Re-size infiltration basin using continuous model and the estimated design infiltration rate.

Check compliance with drawdown, resizing facility as necessary.

Size facility to maximum depth/minimum freeboard to accommodate $V_{\text{design}}$.

Construct facility

Maintain facility and verify performance. Retrofit facility if performance is inadequate.
3.3.5 Site Characterization Criteria

One of the first steps in siting and designing infiltration facilities is to conduct a characterization study that includes the following:

*Note: Information gathered during initial geotechnical investigations can be used for the site characterization.*

**Surface Features Characterization:**
1. Topography within 500 feet of the proposed facility.
2. Anticipated site use (street/highway, residential, commercial, high-use site).
3. Location of water supply wells within 500 feet of proposed facility.
4. Location of ground water protection areas and/or 1, 5 and 10 year time of travel zones for municipal well protection areas (if available).
5. A description of local site geology, including soil or rock units likely to be encountered, the ground water regime, and geologic history of the site.

**Subsurface Characterization:**
1. Subsurface explorations (test holes or test pits) to a depth below the base of the infiltration facility of at least 5 times the maximum design depth of ponded water proposed for the infiltration facility, but not less than 10 feet below the base of the facility. However, at sites with shallow ground water (less than 15 feet from the estimated base of facility), if a ground water mounding analysis is necessary, determine the thickness of the saturated zone.

   Continuous sampling (representative samples from each soil type and/or unit within the infiltration receptor) to a depth below the base of the infiltration facility of 3 times the maximum design ponded water depth, but not less than 10 feet. For large infiltration facilities serving drainage areas of 10 acres or more, perform soil grain size analyses on layers up to 50 feet deep (or no more than 10 feet below the water table).

   To estimate the infiltration rate using the soil grain size analysis method, obtain samples adequate for the purposes of that gradation/classification testing.
   - For basins, at least one test pit or test hole per 5,000 ft² of basin infiltrating surface (in no case less than two per basin).
   - For trenches, at least one test pit or test hole per 200 feet of trench length (in no case less than two per trench).

   *Note: The depth and number of test holes or test pits, and samples should be increased, if in the judgment of a licensed engineer with geotechnical expertise (P.E.), a licensed geologist, engineering geologist, hydrogeologist, or other licensed professional acceptable to the City of Olympia, the conditions are highly variable and such increases are necessary to accurately estimate the performance of the infiltration system. The exploration program may also be decreased if, in the opinion of the licensed engineer or other professional, the conditions are relatively uniform and the borings/test pits omitted will not influence the design or successful operation of the facility. In high water table sites, the subsurface exploration sampling need not be conducted lower than two (2) feet below the ground water table.*

2. Prepare detailed logs for each test pit or test hole and a map showing the location of the test pits or test holes. Logs must include at a minimum, depth of pit or hole, soil descriptions, depth to water, presence of stratification. *(Note: Logs must substantiate whether stratification does or does not exist. The licensed professional may consider additional methods of analysis to substantiate the presence of stratification that will significantly impact the design of the infiltration facility).*

3. Absent a shallow very low permeability layer (confining layer) and/or apparent wet-season high groundwater level defined through other subsurface explorations, install groundwater monitoring...
wells to at least 5 times the maximum design water depth, minimum 20 feet and maximum 50 feet depth, to locate the ground water table and establish its gradient, direction of flow, and seasonal variations, considering both confined and unconfined aquifers. For facilities serving a drainage area less than an acre, establish that the depth to ground water or other hydraulic restriction layer will be at least 10 feet below the base of the facility. Use subsurface explorations or information from nearby wells.

In general, a minimum of three wells per infiltration facility, or three hydraulically connected surface or ground water features, are needed to determine the direction of flow and gradient. If in the assessment of the site professional, the surrounding site conditions indicate that gradient and flow direction are not critical (e.g., there is low risk of down-gradient impacts) one monitoring well may be sufficient. Alternative means of establishing the ground water levels may also be considered. If the ground water in the area is known to be greater than 50 feet below the proposed facility, detailed investigation of the ground water regime is not necessary.

Monitoring through at least one wet season is required, unless substantially equivalent site historical data regarding ground water levels is available.

5. Complete laboratory (sieve) analysis of soil grain size for each infiltration facility. Grain Size Analysis Method for estimating infiltration rates: laboratory testing as necessary to establish the soil gradation characteristics and other properties as necessary, to complete the infiltration facility design. At a minimum, conduct one-grain size analysis per soil stratum in each test hole within 3 times the maximum design water depth, but not less than 10 feet. When assessing the hydraulic conductivity characteristics of the site, soil layers at greater depths must be considered if the licensed professional conducting the investigation determines that deeper layers will influence the rate of infiltration for the facility, requiring soil gradation/classification testing for layers deeper than indicated above.

6. For each soil layer, plot a particle size distribution curve highlighting the following factors: D_{10} (particle size in millimeters, for which 10% of particles, by weight, are smaller), D_{60}, D_{90}, and fraction of soil, by weight, that passes the Number 200 sieve (defined as f_{ines}).

**Soil Testing:**

Soil characterization for each soil unit (soils of the same texture, color, density, compaction, consolidation and permeability) encountered should include:

- Grain-size distribution (ASTM D422 or equivalent AASHTO specification) (If using the grain size analysis method to estimate infiltration rates)
- Visual grain size classification
- Percent clay content (include type of clay, if known)
- Color/mottling
- Variations and nature of stratification

If the infiltration facility will provide treatment as well as flow control, the soil characterization should also include:

- Cation exchange capacity (CEC) and organic matter content for each soil type and strata where distinct changes in soil properties occur, to a depth below the base of the facility of at least 2.5 times the maximum design water depth, but not less than 6 feet.
- For soils with low CEC and organic content, deeper characterization of soils may be warranted (refer to Section 3.3.7 Site Suitability Criteria)
Infiltration Receptor:
Infiltration receptor (unsaturated and saturated soil receiving the stormwater) characterization should include:

1. The information obtained from ground water monitoring in #4 of the Subsurface Characterization above.
2. An assessment of the ambient ground water quality, if that is a concern.
3. An estimate of the volumetric water holding capacity of the infiltration receptor soil. This is the soil layer below the infiltration facility and above the seasonal high-water mark, bedrock, hardpan, or other low permeability layer. Conduct this analysis at a conservatively high infiltration rate based on vadose zone porosity, and the water quality runoff volume to be infiltrated. This, along with an analysis of ground water movement, will be useful in determining if there are volumetric limitations that would adversely affect drawdown, and if a ground water mounding analysis should be conducted.
4. Determination of:
   • Depth to ground water table and to bedrock/impermeable layers.
   • Seasonal variation of ground water table based on well water levels and observed mottling.
   • Existing ground water flow direction and gradient.
   • Lateral extent of infiltration receptor.
   • Horizontal hydraulic conductivity of the saturated zone to assess the aquifer’s ability to laterally transport the infiltrated water.
   • Impact of the infiltration rate and volume at the project site on ground water mounding, flow direction, and water table; and the discharge point or area of the infiltrating water. Conduct a ground water mounding analysis at all sites where the depth to seasonal ground water table or low permeability stratum is less than 15 feet from the estimated bottom elevation of the infiltration facility, and the runoff to the infiltration facility is from more than one acre.

3.3.6 Saturated Potential Hydraulic Conductivity and Design Infiltration Rate Determination
This section provides guidelines for determining the saturated potential hydraulic conductivity. For stormwater facilities designed to fulfill Core Requirement #7, the measured (initial) saturated hydraulic conductivity rate (Ksat) shall be estimated using the grain size analysis method. For stormwater facilities designed to fulfill only Core Requirement #5, measured saturated hydraulic conductivity (Ksat) rates can be determined using in-situ field measurements, or by a correlation to grain size distribution from soil samples. The grain size analysis method uses the ASTM soil size distribution test procedure (ASTM D422), which considers the full range of soil particle sizes, to develop soil size distribution curves. Using the Simplified Approach in Section 3.3.4, the estimate obtained for the measured (initial) Ksat is used as the initial infiltration rate. Using the Detailed Approach in Section 3.3.8, the initial Ksat is combined with other information to compute an estimate for an initial infiltration rate.

1. Soil Grain Size Analysis Method
For each defined soil layer/stratum below the infiltration facility on which a soil grain-size sieve analysis was performed, estimate the potential saturated hydraulic conductivity using the following empirical equation (see Massmann et al. 2003):

\[ K_{\text{sat}} = 2835 \times 10^{(1.57 + 1.90D_{10}^{0.015D_{60}} - 0.013D_{90}^{9.90} - 2.08f_{\text{fines}})} \]  

(Equation 1)

where D values are in millimeters, f is a fraction between 0 and 1, and K_{\text{sat}} is in feet per day. The D_{10}, D_{60}, and D_{90} values are the grain size diameters for which 10%, 60% and 90% of the sample is
finer (smaller), and $f_{\text{finer}}$ is the fraction of the soil (by weight) that passes the number-200 sieve. For bioretention facilities, analyze each defined layer below the top of the final bioretention area subgrade to a depth of at least 3 times the maximum ponding depth, but not less than 3 feet (1 meter). For permeable pavement, analyze for each defined layer below the top of the final subgrade to a depth of at least 3 times the maximum ponding depth within the base course, but not less than 3 feet (1 meter).

If the soils professional conducting the investigation determines that deeper layers will influence the rate of infiltration for the facility, soil layers at greater depths must be considered when assessing the site’s hydraulic conductivity characteristics. Massmann (2003) indicates that where the water table is deep, soil or rock strata up to 100 feet below an infiltration facility can influence the rate of infiltration. Generally, only the layers near and above the water table or a laterally extensive very low permeability layer (e.g., a silt or clay, dense glacial till, or rock layer) need to be considered, as the layers below the water table or low permeability zone do not significantly influence the rate of infiltration. Also note that this equation for estimating $K_{\text{sat}}$ assumes minimal compaction consistent with the use of tracked (i.e., low to moderate ground pressure) excavation equipment.

If the soil layer being characterized has been exposed to heavy compaction (e.g., due to heavy equipment with narrow tracks, narrow tires, or large lugged, high pressure tires) the hydraulic conductivity for the layer could be approximately an order of magnitude less than what would be estimated based on grain size characteristics alone (Pitt, 2003). In such cases, compaction effects must be taken into account when estimating hydraulic conductivity.

For clean, uniformly graded sands and gravels, the reduction in $K_{\text{sat}}$ due to compaction will be much less than an order of magnitude. For well graded sands and gravels with moderate to high silt content, the reduction in $K_{\text{sat}}$ will be close to an order of magnitude. For soils that contain clay, the reduction in $K_{\text{sat}}$ could be greater than an order of magnitude.

Once the saturated potential hydraulic conductivity ($K_{\text{sat}}$) for each soil layer has been obtained, determine the effective average saturated potential hydraulic conductivity below the infiltration facility. Hydraulic conductivity estimates from different layers can be combined using the harmonic mean:

$$K_{\text{equiv}} = \frac{d}{\sum d_n \frac{K_n}{K_n}}$$

(Equation 2)

where $d$ is the total depth of the sampled soil column below the stormwater facility, $d_n$ is the thickness of layer “n” in the soil column, and $K_n$ is the saturated potential hydraulic conductivity of layer “n” in the soil column. The depth of the soil column, $d$, typically would include all layers between the facility bottom and the water table or laterally extensive very low permeability layer. However, for sites with very deep water tables or confining layers (deeper than 100 feet) where ground water mounding to the base of the facility is unlikely to occur, it is recommended that the total depth of the soil column in Equation 2 be limited to approximately 20 times the maximum water depth of the facility. This is to ensure that the most important and relevant layers are included in the hydraulic conductivity calculations. Deep layers that are not likely to affect the infiltration rate near the facility bottom should not be included in Equation 2.

Equation 2 may over-estimate the effective hydraulic conductivity value at sites with laterally extensive low conductivity layers immediately beneath the infiltration facility. For sites where the lowest conductivity layer is within five feet of the base of the facility, it is suggested that this lowest hydraulic conductivity value be used as the equivalent hydraulic conductivity, rather than the value from Equation 2. The harmonic mean given by Equation 2 is the appropriate effective hydraulic
conductivity for flow that is perpendicular to stratigraphic layers, and will produce conservative results when flow has a significant horizontal component such as could occur due to ground water mounding.

- For unusually complex or critical design cases (such as areas with fluctuating groundwater elevations that approach the ground surface, or where infiltration facilities are very large or may affect adjacent property’s developability), it may be necessary to develop input data for a groundwater flow model such as MODFLOW, including trial infiltration facility geometry, continuous hydrograph data, soil stratigraphy data, groundwater data, hydraulic conductivity data, and reduction in hydraulic conductivity due to siltation or biofouling on the surface of the facility. It is expected that this unusually complex modeling approach will be applied only in rare situations. Otherwise, skip this step and develop the data needed to estimate the hydraulic gradient as explained in the following steps.

2. **Large Scale Pilot Infiltration Test (PIT)**

Large-scale in-situ infiltration measurements, using the Pilot Infiltration Test (PIT) described below is acceptable for estimating the measured (initial) saturated hydraulic conductivity ($K_{sat}$) for stormwater facilities designed to fulfill only Core Requirement #5. The PIT reduces some of the scale errors associated with relatively small-scale double ring infiltrometer or “stove-pipe” infiltration tests. It is not a standard test but rather a practical field procedure recommended by Ecology’s Technical Advisory Committee.

**Infiltration Test**

- Excavate the test pit to the estimated surface elevation of the proposed infiltration facility. Lay back the slopes sufficiently to avoid caving and erosion during the test. Alternatively, consider shoring the sides of the test pit.
- The horizontal surface area of the bottom of the test pit should be approximately 100 square feet. Accurately document the size and geometry of the test pit.
- Install a vertical measuring rod (minimum 5-ft. long) marked in half-inch increments in the center of the pit bottom.
- Use a rigid 6-inch diameter pipe with a splash plate on the bottom to convey water to the pit and reduce side-wall erosion or excessive disturbance of the pond bottom. Excessive erosion and bottom disturbance will result in clogging of the infiltration receptor and yield lower than actual infiltration rates.
- Add water to the pit at a rate that will maintain a water level between 6 and 12 inches above the bottom of the pit. A rotameter can be used to measure the flow rate into the pit.

*Note: The depth should not exceed the proposed maximum depth of water expected in the completed facility. For infiltration facilities serving large drainage areas, designs with multiple feet of standing water can have infiltration tests with greater than 1 foot of standing water.*

Every 15-30 min, record the cumulative volume and instantaneous flow rate in gallons per minute necessary to maintain the water level at the same point on the measuring rod.

Keep adding water to the pit until one hour after the flow rate into the pit has stabilized (constant flow rate; a goal of 5% variation or less variation in the total flow) while maintaining the same pond water level. The total of the pre-soak time plus one hour after the flow rate has stabilized should be no less than 6 hours.

- After the flow rate has stabilized for at least one hour, turn off the water and record the rate of infiltration (the drop rate of the standing water) in inches per hour from the measuring rod data, until the pit is empty. Consider running this falling head phase of the test several times to estimate the dependency of infiltration rate with head.
At the conclusion of testing, over-excavate the pit to see if the test water is mounded on shallow restrictive layers or if it has continued to flow deep into the subsurface. The depth of excavation varies depending on soil type and depth to hydraulic restricting layer, and is determined by the engineer or certified soils professional. Mounding is an indication that a mounding analysis is necessary.

**Data Analysis**

Calculate and record the saturated hydraulic conductivity rate in inches per hour in 30 minutes or one-hour increments until one hour after the flow has stabilized.

*Note: Use statistical/trend analysis to obtain the hourly flow rate when the flow stabilizes. This would be the lowest hourly flow rate.*

Apply appropriate correction factors to determine the site-specific design infiltration rate. See the discussion of correction factors for infiltration facilities in this Section 3.3, and the discussion of correction factors for bioretention facilities and permeable pavement in Section 3.4.

**Example**

The area of the bottom of the test pit is 8.5-ft. by 11.5-ft.

Water flow rate was measured and recorded at intervals ranging from 15 to 30 minutes throughout the test. Between 400 minutes and 1,000 minutes the flow rate stabilized between 10 and 12.5 gallons per minute or 600 to 750 gallons per hour, or an average of \((9.8 + 12.3) / 2 = 11.1\) inches per hour.

3. **Small-Scale Pilot Infiltration Test**

A smaller-scale PIT can be substituted for the large-scale PIT only for stormwater facilities designed to fulfill only Core Requirement #5.

**Infiltration Test**

- Excavate the test pit to the estimated surface elevation of the proposed infiltration facility. In the case of bioretention, excavate to the estimated elevation at which the imported soil mix will lie on top of the underlying native soil. For permeable pavements, excavate to the elevation at which the imported subgrade materials, or the pavement itself, will contact the underlying native soil. If the native soils (road subgrade) will have to meet a minimum subgrade compaction requirement, compact the native soil to that requirement prior to testing. Note that the permeable pavement design guidance recommends compaction not exceed 90% - 92%. Finally, lay back the slopes sufficiently to avoid caving and erosion during the test. Alternatively, consider shoring the sides of the test pit.
- The horizontal surface area of the bottom of the test pit should be 12 to 32 square feet. It may be circular or rectangular, but accurately document the size and geometry of the test pit.
- Install a vertical measuring rod adequate to measure the ponded water depth and that is marked in half-inch increments in the center of the pit bottom.
- Use a rigid pipe with a splash plate on the bottom to convey water to the pit and reduce side-wall erosion or excessive disturbance of the pond bottom. Excessive erosion and bottom disturbance will result in clogging of the infiltration receptor and yield lower than actual infiltration rates. Use a 3-inch diameter pipe for pits on the smaller end of the recommended surface area, and a 4-inch pipe for pits on the larger end of the recommended surface area.
- Pre-soak period: Add water to the pit so that there is standing water for at least 6 hours. Maintain the pre-soak water level at least 12 inches above the bottom of the pit.
- At the end of the pre-soak period, add water to the pit at a rate that will maintain a 6-12 inch water level above the bottom of the pit over a full hour. The depth should not exceed the...
proposed maximum depth of water expected in the completed facility.

- Every 15 minutes, record the cumulative volume and instantaneous flow rate in gallons per minute necessary to maintain the water level at the same point (between 6 inches and 1 foot) on the measuring rod. The specific depth should be the same as the maximum designed ponding depth (usually 6 – 12 inches).
- After one hour, turn off the water and record the rate of infiltration (the drop rate of the standing water) in inches per hour from the measuring rod data, until the pit is empty.
- A self-logging pressure sensor may also be used to determine water depth and drain-down.
- At the conclusion of testing, over-excavate the pit to see if the test water is mounded on shallow restrictive layers or if it has continued to flow deep into the subsurface. The depth of excavation varies depending on soil type and depth to hydraulic restricting layer, and is determined by the engineer or certified soils professional. The soils professional should judge whether a mounding analysis is necessary.

**Data Analysis**

See the explanation under the guidance for large-scale pilot infiltration tests.

**Correction Factors**

**Correction Factors for Grain Size Method and PIT results** - The $K_{sat}$ obtained from the PIT test or Grain Size Method is a measured (initial) rate. This measured rate must be reduced through correction factors that are appropriate for the design situation to produce a design infiltration rate. This adjustment is made in Step 5 of the Design of Infiltration Facilities (Section 3.3.4).

Correction factors account for site variability, number of tests conducted, uncertainty of the test method, and the potential for long-term clogging due to siltation and bio-buildup. Table 3.3.1 summarizes the typical range of correction factors to account for these issues. The specific correction factors used shall be determined based on the professional judgment of the licensed engineer or other site professional considering all issues that may affect the infiltration rate over the long term, subject to the approval of the local jurisdictional authority.

<table>
<thead>
<tr>
<th>Issue</th>
<th>Partial Correction Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>Site variability and number of locations tested</td>
<td>$CF_v = 0.33$ to $1.0$</td>
</tr>
<tr>
<td>Test Method</td>
<td></td>
</tr>
<tr>
<td>Large-scale PIT</td>
<td>$CF_t = 0.75$</td>
</tr>
<tr>
<td>Small-scale PIT</td>
<td>$CF_t = 0.50$</td>
</tr>
<tr>
<td>Grain Size Method</td>
<td>$CF_t = 0.40$</td>
</tr>
<tr>
<td>Degree of influent control to prevent siltation and bio-buildup</td>
<td>$CF_m = 0.9$</td>
</tr>
</tbody>
</table>

Total Correction Factor, $CF_T = CF_v \times CF_t \times CF_m$

$CF_T$ is used in Step 5 of the Design of Infiltration Facilities (Section 3.3.4) to adjust the measured (initial) saturated hydraulic conductivity.

$$K_{sat\ design} = K_{sat\ initial} \times CF_T$$

Where $K_{sat\ initial}$ is either the $K_{equiv}$ for the grain size method or $K_{sat}$ for the PIT test methods.
Site variability and number of locations tested (CFv) - The number of locations tested must be capable of producing a picture of the subsurface conditions that fully represents the conditions throughout the facility site. The partial correction factor used for this issue depends on the level of uncertainty that adverse subsurface conditions may occur. If the range of uncertainty is low - for example, conditions are known to be uniform through previous exploration and site geological factors - one pilot infiltration test (or grain size analysis location) may be adequate to justify a partial correction factor at the high end of the range.

If the level of uncertainty is high, a partial correction factor near the low end of the range may be appropriate. This might be the case where the site conditions are highly variable due to conditions such as a deposit of ancient landslide debris, or buried stream channels. In these cases, even with many explorations and several pilot infiltration tests (or several grain size test locations), the level of uncertainty may still be high.

A partial correction factor near the low end of the range could be assigned where conditions have a more typical variability, but few explorations and only one pilot infiltration test (or one grain size analysis location) is conducted. That is, the number of explorations and tests conducted do not match the degree of site variability anticipated.

Uncertainty of test method (CFt) accounts for uncertainties in the testing methods. For the full scale PIT method, CF_t = 0.75; for the small-scale PIT method, CF_t = 0.50; for smaller-scale infiltration tests such as the double-ring infiltrometer test, CF_t = 0.40; for grain size analysis, CF_t = 0.40. These values are intended to represent the difference in each test’s ability to estimate the actual saturated hydraulic conductivity. The assumption is the larger the scale of the test, the more reliable the result.

Degree of influent control to prevent siltation and bio-buildup (CF_m) Even with a pre-settling basin or a basic treatment facility for pre-treatment, the soil’s initial infiltration rate will gradually decline as more and more stormwater, with some amount of suspended material, passes through the soil profile. The maintenance schedule calls for removing sediment when the facility is infiltrating at only 90% of its design capacity. Therefore, a correction factor, CF_m, of 0.9 is called for.

This correction is used in Step 5 of the Design of Infiltration Facilities (Section 3.3.4).

3.3.7 Site Suitability Criteria (SSC)

This section provides criteria that must be considered for siting infiltration systems. When a site investigation reveals that any of the applicable criteria cannot be met appropriate mitigation measures must be implemented so that the infiltration facility will not pose a threat to safety, health, and the environment.

For site selection and design decisions a geotechnical and hydrogeologic report should be prepared by a licensed engineer with geotechnical and hydrogeologic experience, or a licensed geologist, hydrogeologist, or engineering geologist. The design engineer may utilize a team of certified or registered professionals in soil science, hydrogeology, geology, and other related fields.

SSC-1 Setback Criteria

Setback requirements are generally required by local regulations, uniform building code requirements, or other state regulations.

These Setback Criteria are provided as guidance.

- Stormwater infiltration facilities should be set back at least 100 feet from drinking water wells, septic tanks or drainfields, and springs used for public drinking water supplies. Infiltration facilities upgradient of drinking water supplies and within 1, 5, and 10-year time of travel zones must comply

- Additional setbacks must be considered if roadway deicers or herbicides are likely to be present in the influent to the infiltration system.
- From building foundations; ≥ 20 feet downslope and ≥100 feet upslope. Ensure that the infiltration facility’s line of saturation at design depth falls below the basement, crawl space, or foundation.
- From the top of slopes >15%; ≥ 50 feet. The City may require a geotechnical analysis to assess slope stability, such as in known or suspected landslide hazard areas.
- Evaluate on-site and off-site structural stability due to extended subgrade saturation and/or head loading of the permeable layer, including the potential impacts to downgradient properties, especially on hills with known side-hill seeps.
- Comply with all other applicable requirements, such as health, building, and plumbing codes.

**SSC-2 Ground Water Protection Areas**

A site is not suitable if the infiltration facility will cause a violation of Ecology's Ground Water Quality Standards (Chapter 173-200 WAC). (See SSC-3 through SSC-6, and SSC-8 for measures to protect ground water quality. Consult local jurisdictions for applicable pollutant removal requirements upstream of the infiltration facility, and to determine whether the site is located in an aquifer sensitive area, sole source aquifer, or a wellhead protection zone.

**SSC-3 High Vehicle Traffic Areas**

An infiltration BMP may be considered for runoff from areas of industrial activity and the high vehicle traffic areas described below. For such applications, provide sufficient pollutant removal (including oil removal) upstream of the infiltration facility to ensure that ground water quality standards will not be violated and that the infiltration facility is not adversely affected.

High Vehicle Traffic Areas are:

- Commercial or industrial sites subject to an expected average daily traffic count (ADT) ≥100 vehicles/1,000 ft² gross building area (trip generation).
- Road intersections with an ADT of ≥ 25,000 on the main roadway and ≥ 15,000 on any intersecting roadway.

**SSC-4 Soil Infiltration Rate/Drawdown Time**

Infiltration Rates: measured (initial) and design (long-term):

For infiltration facilities used for treatment purposes, the measured (initial) soil infiltration rate should be 9 in./hour, or less. Design (long-term) infiltration rates up to 3.0 inches/hour can also be considered, if the infiltration receptor is not a sole-source aquifer, and in the judgment of the site professional, the treatment soil has characteristics comparable to those specified in SSC-6 to adequately control the target pollutants.

The design infiltration rate should also be used for maximum drawdown time and routing calculations. **Drawdown time:**

For infiltration facilities designed strictly for flow control purposes, there isn’t a maximum drawdown time. If sizing a treatment facility, document that the water quality design storm volume (indicated by
WWHM or MGS Flood, or runoff from a 6-month, 24-hour rain event) can infiltrate through the infiltration basin surface within 48 hours. This can be calculated multiplying the horizontal projection of the infiltration basin mid-depth dimensions by the estimated design infiltration rate, and multiplying the result by 48 hours.

This drawdown restriction is intended to meet the following objectives:

- Aerate vegetation and soil to keep the vegetation healthy.
- Enhance the biodegradation of pollutants and organics in the soil.

Note: This is a check procedure, not a method for determining basin size. If the design fails the check procedure, redesign the basin.

SSC-5 Depth to Bedrock, Water Table, or Impermeable Layer

The base of all infiltration basins or trench systems shall be \( \geq 5 \) feet above the seasonal high-water mark, bedrock (or hardpan) or other low permeability layer. A separation down to 3 feet may be considered if the ground water mounding analysis, volumetric receptor capacity, and the design of the overflow and/or bypass structures are judged by the site professional to be adequate to prevent overtopping and meet the site suitability criteria specified in this section.

SSC-6 Soil Physical and Chemical Suitability for Treatment

(Applies to infiltration facilities used as treatment facilities not to facilities used for flow control).

Consider the soil texture and design infiltration rates along with the physical and chemical characteristics specified below to determine if the soil is adequate for removing the target pollutants. The following soil properties must be carefully considered in making such a determination:

- Cation exchange capacity (CEC) of the treatment soil must be \( \geq 5 \) milliequivalents CEC/100 g dry soil (USEPA Method 9081). Consider empirical testing of soil sorption capacity, if practicable. Ensure that soil CEC is sufficient for expected pollutant loadings, particularly heavy metals. CEC values of \( >5 \) meq/100g are expected in loamy sands (Buckman and Brady, 1969). Lower CEC content may be considered if it is based on a soil loading capacity determination for the target pollutants that is accepted by the local jurisdiction.

- Depth of soil used for infiltration treatment must be a minimum of 18 inches. Depth of soil below permeable pavements serving as pollution-generating hard surfaces may be reduced to one foot if the permeable pavement does not accept run-on from other surfaces.

- Organic Content of the treatment soil (ASTM D 2974): Organic matter can increase the sorptive capacity of the soil for some pollutants. A minimum of 1.0 percent organic content is necessary.

- Waste fill materials shall not be used as infiltration soil media nor shall such media be placed over uncontrolled or non-engineered fill soils.

Engineered soils may be used to meet the design criteria in this chapter and the performance goals in Chapters 3 and 4 of Volume V. Field performance evaluation(s), using protocols cited in this manual, would be needed to determine feasibility and acceptability by the local jurisdiction.

SSC-7 Seepage Analysis and Control

Determine whether there would be any adverse effects caused by seepage zones on nearby building foundations, basements, roads, parking lots or sloping sites.

SSC-8 Cold Climate and Impact of Roadway Deicers

Consider the potential impact of roadway deicers on potable water wells in the siting determination. Implement mitigation measures if the infiltration of roadway deicers could cause a violation of ground water quality standards.
SSC-9  Contingency Planning

The empirical infiltration assessment methods provided above are expected to yield accurate estimates of ultimate infiltration rates. However, soils, shallow geology, and groundwater conditions can be extremely complex and highly variable, which may cause inaccuracies. Therefore, it is necessary to have a plan for fixing underperformance discovered after facilities are installed (see Verification Testing, below).

All projects using infiltration facilities to fulfill Core Requirement #7, including permeable pavements and bioretention, shall provide a contingency plan for underperformance. The plan shall include a reasonable “worst-case” projection of long-term infiltration performance (based on 50 percent or less of the design infiltration rate) and describe methods and costs for improving/restoring performance and/or expanding facility size. These costs shall provide one basis for required performance/operation and maintenance bonding (see Volume I, Section 2.6).

SSC-10  Verification Testing of the Completed Facility

Verification testing of completed full-scale infiltration facilities is required to confirm that the design infiltration rate is being attained. After project completion, the applicant shall submit a facility monitoring and evaluation report. A licensed civil engineer shall prepare and seal the report. The report shall document field work and assess stormwater infiltration facility performance versus design (e.g., infiltration rates). All field work shall be done under the engineer’s direction and supervision.

For infiltration facilities designed to fulfill Core Requirement #7, testing shall consist of automated continuous water level monitoring (datalogging pressure transducer) over a sufficient number of storms to provide an accurate “long-term” infiltration rate. Testing shall either have a minimum of 30 days’ test results with two or more events exceeding 30% of facility volume, or one full wet season’s data (November 1 to March 30).

For facilities designed to fulfill only Core Requirement #5, testing shall consist of monitoring the facilities to verify drawdown within 24 hours of the end of a storm event resulting in a 6-inch ponding depth or using the Pilot Infiltration Test (PIT) method (or other small-scale testing allowed by the City of Olympia). If using the PIT method, do not excavate Bioretention Soil Mix (conduct test at level of finished Bioretention Soil Mix elevation), use a maximum of 6 inch ponding depth and conduct test before plants are installed.

The report shall specify any actions needed to restore performance, such as sediment removal or facility expansion. The City will retain guarantees until the facility’s measured and design infiltration rates are substantially equivalent.

For the purposes of stormwater facility infiltration rate verification substantially equivalent shall be considered to be either above the design infiltration rate or a rate greater than the design infiltration rate minus 25%.

For infiltration trenches to be located under pavement, the City may consider alternative infiltration verification testing methods prior to paving.

The long term infiltration rate determination shall be based upon the direct measurement of the change in water level with time. In detention/retention facilities the discharge of water through the outlet structure shall be considered to determine the facility infiltration rate. In trench facilities the change in water level in drain rock shall be converted to the change in free water level by multiplying by the void content of the drain rock to determine the infiltration rate. Hydrologic modeling cannot be used to estimate flows into a facility in order to verify the infiltration rate.
The verification report shall include the following list of items:

- The stamp and certification of the professional engineer directing the work.
- A description of the infiltration facility as it was designed, including the design infiltration rate.
- A description of the infiltration facility as it was constructed.
- A description of how and where the performance monitoring was performed.
- Raw water level data.
- Plots of water level in the facility with time.
- Infiltration rate calculations based upon the observed data.
- A statement and conclusions about the performance of the facility and whether it meets its original design.

Recommendations of modifications to the facility, if required, to either ensure continued operation or comply with the original design requirements.

### 3.3.8 Steps for Designing Infiltration Facilities - Detailed Approach

This detailed approach was obtained from Massmann (2003). The detailed approach includes the process in Figure 3.3.3 and the steps in the following pages.
Figure 3.3.3 - Engineering Design Steps for Final Design of Infiltration Facilities Using the Detailed Method

1. Estimate volume of stormwater, Vdesign
   - Continuous Hydrograph

2. Choose trial geometry based on site constraints of assume $f = 0.5$ in./hr.

3. Perform computer design infiltration facility using WWHM or MGSFLOOD with continuous hydrograph, soil stratigraphy, ground water data, hydraulic conductivity, and CF as input.

4. Calculate infiltration rate using a stage-discharge relationship using MODRET.

5. Perform subsurface site characterization and data collection, including location of water table.

6. Estimate saturated hydraulic conductivity:
   - Soil grain size & $C_{F_t}$
   - Field tests & $C_{F_t}$

7. Calculate hydraulic gradient using Equation 3. If the calculated value is greater than 1.0, consider water table to be deep and use $i = 1.0$ max. Since $I$ is a function of water depth in pond, I must be embedded in the stage discharge relationship used in a runoff model.

8. Estimate the infiltration rate for the stage-discharge relationship (Equation 5).

9. Adjust infiltration rates for pond aspect ratio to estimate long-term infiltration rate (Equations 6 & 7).

10. Size facility to maximum depth/minimum freeboard to accommodate $V_{design}$


1 – 5. Steps 1 through 5 are the same as indicated for the Simplified Approach – Section 3.3.4

6. Calculate the hydraulic gradient as follows:

   Calculate the steady state hydraulic gradient as follows:

   
   \[
   i = \frac{(D_{\text{wt}} + D_{\text{pond}}) \ CF_{\text{size}}}{138.62 \ (K_{\text{sat \ design}}^{0.1})} \quad (\text{Equation 3a for ponds})
   \]

   \[
   i = \frac{(D_{\text{wt}} + D_{\text{trench}})}{78 \ (K_{\text{sat \ design}}^{0.05})} \quad (\text{Equation 3b for trenches})
   \]

   Note: The units in this equation vary from the units normally used in this manual.

   Where, \( D_{\text{wt}} \) is the depth from the base of the infiltration facility to the apparent wet season high groundwater level or confining layer/stratum in feet; \( K_{\text{sat \ design}} \) is the saturated hydraulic conductivity in feet/day (Section 3.3.6), \( D_{\text{pond}} \) and \( D_{\text{trench}} \) are one-quarter of the maximum depth of water in the facility in feet (see Massmann et al., 2003, for the development of this equation), and \( CF_{\text{size}} \), is the correction for pond size. The correction factor was developed for ponds with bottom areas between 0.6 and 6 acres in size. The correction factor has a maximum value of 1.0. For small ponds (ponds with an area equal to 2/3 acre or less) the \( CF_{\text{area}} \) correction factor is equal to one, as shown in Equation 4.

   \[
   CF_{\text{size}} = 0.73 \ (A_{\text{pond}})^{-0.76} \quad (\text{Equation 4})
   \]

   Where, \( A_{\text{pond}} \) is the area of pond bottom in acres. This equation generally will result in a calculated gradient of less than 1.0 for moderate to shallow ground water depths (or to a low permeability layer) below the facility, and conservatively accounts for the development of a ground water mound. A more detailed ground water mounding analysis using a program such as MODFLOW will usually result in a gradient that is equal to or greater than the gradient calculated using Equation 3. If the calculated gradient is greater than 1.0, the water table is considered to be deep, and a maximum gradient of 1.0 must be used. Typically, a depth to ground water of 100 feet or more is required to obtain a gradient of 1.0 or more using this equation. Since the gradient is a function of depth of water in the facility, the gradient will vary as the pond fills during the season. The gradient could be calculated as part of the stage-discharge calculation used in the continuous runoff models. As of the date of this update, neither the WWHM nor MGSFlood have that capability. However, updates to those models may soon incorporate the capability. Until that time, use a steady-state hydraulic gradient that corresponds with a ponded depth of \( \frac{1}{4} \) of the maximum ponded depth – as measured from the basin floor to the overflow.
7. **Calculate the preliminary design infiltration rate using Darcy’s Law as follows:**

\[ f = K_{\text{sat design}} \cdot i \]  

(Equation 5)

where \( f \) is the functional saturated hydraulic conductivity (in length per unit time, L/t, typically inches/hour) corrected for effects of groundwater mounding; \( K_{\text{sat design}} \) (Section 3.3.6) is the saturated potential hydraulic conductivity (L/t) from Equation 2; and \( i \) is the hydraulic gradient from Equation 3a or 3b.

The reduction factor of 2.0 is to account for the fine layering and relatively shallow depth to groundwater in local soils.

8. **Adjust the preliminary design infiltration rate or infiltration stage-discharge relationship obtained in Step 7:**

Adjustments of the initial infiltration rate estimate should have been made in Step 5. (As explained in Section 3.3.6).

This step adjusts the preliminary design infiltration rate for the effect of pond aspect ratio by multiplying the infiltration rate determined in Step 7 by the aspect ratio correction factor \( CF_{\text{aspect}} \) as shown in the following equation:

\[ CF_{\text{aspect}} = 0.02A_r + 0.98 \]

Where, \( A_r \) is the aspect ratio for the pond (length/width of the bottom area). In no case shall \( CF_{\text{aspect}} \) be greater than 1.4. **Note: The aspect ratio correction is not applied to trench configurations.**

The final design (long-term) infiltration rate will therefore be as follows:

\[ f_{\text{design}} = K_{\text{sat design}} \cdot i \cdot CF_{\text{aspect}} \]

9. **Size the facility:**

Size the facility to ensure that the maximum pond depth is between 2 to 6 feet with one-foot minimum required freeboard.

Where the infiltration facility is being used to meet treatment requirements, check that the Water Quality Design Storm Volume (indicated by WWHM or MGS Flood) can infiltrate through the infiltration basin surface within 48 hours. This can be calculated by multiplying a horizontal projection of the infiltration basin mid-depth dimensions by the estimated design infiltration rate; and multiplying the result by 48 hours (See SSC-4 in Section 3.3.7)

10. **Ground Water Mounding Analysis:**

Ground water Mounding Analysis: On projects where an infiltration facility has a drainage area exceeding 1 acre and has less than fifteen feet depth to seasonal high ground water (as measured from the bottom of the infiltration basin or trench) or other low permeability stratum, determine the final design infiltration rate using an analytical ground water model to investigate the effects of the local hydrologic conditions on facility performance. These larger projects can use the design infiltration rate determined above as input to an approved continuous runoff model (WWHM, MGS Flood, KCRTS) to do an initial sizing. Then complete the ground water modeling (mounding analysis) of the proposed
infiltration facility. Use MODRET or an equivalent model unless the local government approves an alternative analytic technique.

Export the full output hydrograph of the developed condition and use it as input to MODRET. Note that an iterative process may be required beginning with an estimated design rate, WWHM sizing, then ground water model testing. See Figure 3.3.3.

11. Construct the facility & Conduct Performance Testing:

Test and monitor the constructed facility per the verification testing guidelines in Section 3.3.7 to demonstrate that the facility performs as designed. Perform the testing after stabilizing the construction site. Submit the results and comparisons to the pre-project measured (initial) and design rates to the City of Olympia. If the rates are lower than the design saturated hydraulic conductivity, the applicant shall implement measures to improve infiltration capability within the footprint of the constructed facility and re-test. If less intensive measures prove unsuccessful, methods identified in the contingency plan for underperformance (Section 3.3.7) shall be implemented.

3.3.9 General Design, Maintenance, and Construction Criteria for Infiltration Facilities

Design Criteria – Sizing Facilities

The size of the infiltration facility can be determined by routing the influent runoff file generated by the continuous runoff model through it. To prevent the onset of anaerobic conditions, an infiltration facility designed for treatment purposes must be designed to drain the Water Quality Design volume within 48 hours (see explanation under SSC-4 in Section 3.3.7). In general, an infiltration facility would have 2 discharge modes. The primary mode of discharge from an infiltration facility is infiltration into the ground. However, when the infiltration capacity of the facility is reached, additional runoff to the facility will cause the facility to overflow. Overflows from an infiltration facility must comply with Core Requirement #7 for flow control in Volume I. Infiltration facilities used for runoff treatment must not overflow more than 9% of the influent runoff file. Infiltration facilities can also be used to demonstrate compliance with the LID Performance Standard of Core Requirement #5.

In order to determine compliance with the flow control requirements, use the Western Washington Hydrology Model, with the most current Thurston County precipitation data, or an appropriately calibrated continuous simulation model based on HSPF. When using WWHM for simulating flow through an infiltrating facility, represent the facility by using a Pond Element and entering the pre-determined infiltration rates. Below are the procedures for sizing a pond (A) to completely infiltrate 100% of runoff; (B) to treat 91% of runoff to meet the water quality treatment requirements, and (C) to partially infiltrate runoff to meet flow duration standard.

(A) For 100% infiltration

1. Input dimensions of your infiltration pond,
2. Input determined design infiltration rate (in Section 3.3.8).

3. Input a riser height and diameter (any flow through the riser indicates that you have less than 100% infiltration and must increase your infiltration pond dimensions).

4. Run only HSPF for Developed Mitigated Scenario (if that is where you put the infiltration pond). Do not need to run duration.

5. Go back to your infiltration pond and look at the Percentage Infiltrated at the bottom right. If less than 100% infiltrated, increase pond dimension until you get 100%.

(B) For 91% infiltration (water quality treatment volume)

The procedure is the same as above, except that your target is 91%.

Infiltration facilities for treatment can be located upstream or downstream of detention and can be off-line or on-line.

**On-line** treatment facilities placed **upstream or downstream** of a detention facility must be sized to infiltrate 91% of the runoff file volume directed to it.

*This section covers design, construction and maintenance criteria that apply to infiltration basins and trenches.*

**Off-line** treatment facilities placed **upstream** of a detention facility must have a flow splitter designed to send all flows at or below the 15-minute water quality flow rate, as predicted by WWHM (or other approved continuous runoff model), to the treatment facility. Within WWHM, the flow splitter element is placed ahead of the pond element which represents the infiltration basin. Size the treatment facility to infiltrate all the runoff sent to it (no overflows from the treatment facility are allowed).

**Off-line** treatment facilities 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 WWHM (or other approved continuous runoff model), to the treatment facility. Within WWHM, the flow splitter element is placed ahead of the pond element which represents the infiltration basin. Size the treatment facility to infiltrate all the runoff sent to it (no overflows from the treatment facility are allowed).

*See Section 4.5 of Volume V for flow splitter design details.*

(C) To meet flow duration standard with infiltration ponds

This design will allow something less than 100% infiltration as long as any overflows will meet the flow duration standard. Use a discharge structure with orifices and risers similar to a detention facility and include infiltration occurring from the pond.
**Additional Design Criteria**

- Slope of the base of the infiltration facility should be <3 percent.
- Spillways/overflow structures – Construct a non-erodible outlet or spillway with a firmly established elevation to discharge overflow. Calculate ponding depth, drawdown time, and storage volume from that reference point. Overflow Structure—Refer to Chapter 3 for design details.
- For infiltration treatment facilities, 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, line the side-walls with at least 18 inches of treatment soil to prevent seepage of untreated flows through the side walls.

**Design Criteria – Pretreatment**

A facility to remove a portion of the influent suspended solids should precede the infiltration facility. Use either an option under the basic treatment facility menu (See Chapter 2 of Volume V), or a pretreatment option from Chapter 6 of Volume V. The lower the influent suspended solids loading to the infiltration facility, the longer the infiltration facility can infiltrate the desired amount of water or more, and the longer interval between maintenance activity.

In facilities such as infiltration trenches where a reduction in infiltration capability can have significant maintenance or replacement costs, selection of a reliable treatment device with high solids removal capability is preferred. In facilities that allow easier access for maintenance and less costly maintenance activity (e.g., infiltration basins with gentle side slopes), there is a trade-off between using a treatment device with a higher solids removal capability and a device with a lower capability.

Generally, treatment options on the basic treatment menu are more capable at solids removal than pretreatment devices listed in Chapter 6 of Volume V. Though basic treatment options may be higher in initial cost and space demands, the infiltration facility should have lower maintenance costs.

**Construction Criteria**

- Conduct initial basin excavation 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 upgradient project drainage area have been permanently stabilized. The final phase of excavation should remove all accumulation of silt in the infiltration facility before putting it in service. After construction completion, prevent sediment from entering the infiltration facility by first conveying the runoff water through an appropriate pretreatment system such as a pre-settling basin, wet pond, or sand filter.
- Generally, do not use infiltration facilities as temporary sediment traps during construction. If an infiltration facility will be used as a sediment trap, do not excavated to final grade until after the stabilizing the upgradient drainage area. Remove any accumulation of silt in the basin putting it in service.
- Traffic Control – Relatively light-tracked equipment is recommended for this operation to avoid compaction of the basin floor. Consider the use of draglines and trackhoes for constructing infiltration basins. Flag or mark the infiltration area to keep heavy equipment away.

**Maintenance Criteria**

Make provisions for regular and perpetual maintenance of the infiltration basin/trench, including replacement and/or reconstruction of the any media relied upon for treatment purposes. Conduct maintenance when water remains in the basin
or trench for more than 24 hours after the end of runoff, or when overflows occur more frequently than planned. For example, off-line infiltration facilities should not have any overflows. Infiltration facilities designed to completely infiltrate all flows to meet flow control standards should not overflow. A Stormwater Facility Maintenance Program, approved by the City of Olympia, should ensure maintaining the desired infiltration rate.

Include adequate access for operation and maintenance in the design of infiltration basins and trenches.

Conduct removal of accumulated debris/sediment in the basin/trench every 6 months or as needed to prevent clogging. Indications that the facility is not infiltrating adequately include:

- The Water Quality Design Storm Volume does not infiltrate within 48 hours.
- Water remains in the pond for greater than 24 hours after the end of most moderate rainfall events.

For more detailed information on maintenance, see Volume IV, Chapter 4.

**Verification of Performance**

During the first 1-2 years of operation verification testing (specified in SSC-10) is required prior to the release of the completion bond (or other financial guarantee). A maintenance program that results in achieving expected performance levels is also required. Operating and maintaining ground water monitoring wells (specified in Section 3.3.7 - Site Suitability Criteria) is also strongly encouraged.

### 3.3.10 Infiltration Basins

This section covers design and maintenance criteria specific for infiltration basins.

**Description**

Infiltration basins are earthen impoundments used for the collection, temporary storage and infiltration of incoming stormwater runoff.

**Design Criteria Specific for Basins**

- Provide access for vehicles to easily maintain the forebay (presperttling basin) area and not disturb vegetation, or re-suspend sediment any more than absolutely necessary.
- The slope of the basin bottom should not exceed 3% in any direction.
- Size the basin for a maximum ponding depth of between 2 and 6 feet.
- A minimum of one foot of freeboard is recommended 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.
- 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. Provide erosion protection of inflow points to the basin (e.g., riprap, flow spreaders, energy dissipators). Select suitable vegetative materials to stabilize the basin floor and side slopes. Refer to detention pond guidance earlier in this chapter for recommended vegetation.
- Lining material – Basins can be open or covered 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. Select a nonwoven geotextile
that will function sufficiently without plugging (see geotextile specifications in Appendix V-C of Volume V). Replace or clean the filter layer when/if it becomes clogged.

- Vegetation – Stabilize the embankment, emergency spillways, spoil and borrow areas, and other disturbed areas and plant, preferably with grass, in accordance with the Drainage Control Plan (See Core Requirement #1 of Volume I). Without healthy vegetation, the surface soil pores will quickly plug.

- Maintain basin floor and side slopes to promote dense turf with extensive root growth. This enhances infiltration, prevents erosion and consequent sedimentation of the basin floor, and prevents invasive weed growth. Immediately stabilize and revegetate bare spots.

- Do not allow vegetation growth to exceed 18 inches in height. Mow the slopes periodically and check for clogging, and erosion.

- Use the same seed mixtures as those recommended in Table 3.2.2. The use of slow-growing, stoloniferous grasses will permit long intervals between mowing. Mowing twice a year is generally satisfactory. Apply fertilizers only as necessary and in limited amounts to avoid contributing to ground water pollution.

- Consult the local agricultural or gardening resources such as Washington State University Extension for appropriate fertilizer type, including slow release fertilizers, and application rates.

### 3.3.11 Infiltration Trenches

This section covers design, construction, and maintenance criteria specific for infiltration trenches.

**Description:**

Infiltration trenches are generally at least 24 inches wide, and are backfilled with a coarse stone aggregate, allowing for temporary storage of stormwater runoff in the voids of the aggregate material. Stored runoff then gradually infiltrates into the surrounding soil. The surface of the trench can be covered with grating and/or consist of stone, gabion, sand, or a grassed covered area with a surface inlet. Perforated rigid pipe of at least 8-inch diameter can also be used to distribute the stormwater in a stone trench.

See Figures 3.3.4, 3.3.5, 3.3.6, 3.3.7, and 3.3.8 for examples of trench designs.
Figure 3.3.4 - Schematic of an Infiltration Trench
Figure 3.3.5 - Parking Lot Perimeter Trench Design

Source: Schueler (reproduced with permission)

ACCMP – Asphalt Coated Corrugated Metal Pipe

Figure 3.3.6 - Median Strip Trench Design

Source: Schueler (reproduced with permission)

ACCMP – Asphalt Coated Corrugated Metal Pipe
Figure 3.3.7 - Oversized Pipe Trench Design

Source: Schueler (reproduced with permission)

Figure 3.3.8 - Swale/Trench Design

Source: Schueler (reproduced with permission)
**Design Criteria**

- Due to accessibility and maintenance limitations, carefully design and construct infiltration trenches.
- Consider including an access port or open or grated top for accessibility to conduct inspections and maintenance.
- Backfill Material - The aggregate material for the infiltration trench should consist of a clean aggregate with a maximum diameter of 3 inches and a minimum diameter of 1.5 inches. Void space for these aggregates should be in the range of 30 to 40 percent.
- Geotextile fabric liner – Completely encase the aggregate fill material in an engineering geotextile material. Geotextile should surround all of the aggregate fill material except for the top one-foot, which is placed over the geotextile. Carefully select geotextile fabric with acceptable properties to avoid plugging (see Appendix V-C of Volume V).
- The bottom sand or geotextile fabric as shown in Figure 3.3.4 is optional.

• Overflow Channel - Because an infiltration trench is generally used for small drainage areas, an emergency spillway is not necessary. However, provide a non-erosive overflow channel leading to a stabilized watercourse.

• Surface Cover - A stone filled trench can be placed under a porous or impervious surface cover to conserve space.

• Observation Well - Install an observation well at the lower end of the infiltration trench to check water levels, drawdown time, sediment accumulation, and conduct water quality monitoring. Figure 3.3.10 illustrates observation well details. It should consist of a perforated PVC pipe which is 4 to 6 inches in diameter and it should be constructed flush with the ground elevation. For larger trenches a 12-36 inch diameter well can be installed to facilitate maintenance operations such as pumping out the sediment. Cap the top of the well to discourage vandalism and tampering.

Construction Criteria

• Trench Preparation - Place excavated materials away from the trench sides to enhance trench wall stability. Take care to keep this material away from slopes, neighboring property, sidewalks and streets. It is recommended that this material be covered with plastic. (See Volume II, BMP C123 – Plastic Covering).

• Stone Aggregate Placement and Compaction - Place stone aggregate in lifts and compact using plate compactors. In general, 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. Remove all contaminated stone aggregate and replaced with uncontaminated stone aggregate.

• Overlapping and Covering-Following the stone aggregate placement, fold the geotextile over the stone aggregate to form a 12 inch minimum longitudinal overlap. When overlaps are required between rolls, the upstream roll should 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. Place natural soils in these voids at the most convenient time during construction to ensure geotextile conformity to the excavation sides. This remedial process will avoid soil piping, geotextile clogging, and possible surface subsidence.

• 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.

Maintenance Criteria

Monitor sediment buildup in the top foot of stone aggregate or the surface inlet on the same schedule as the observation well.
3.4 Stormwater-related Site Procedures and Design Guidance for Bioretention and Permeable Pavement

3.4.1 Purpose
The purpose is to locate and estimate the effectiveness of these distributed LID facilities in helping to meet the treatment, flow control, and LID requirements.

3.4.2 Description
The site procedures and design guidelines described in this section are meant to be implemented after a preliminary project layout has been developed. The preliminary project layout should be developed considering the procedures of Chapter 3 in Volume I. The designer must perform sufficient infiltration tests to confirm the feasibility of proposed bioretention and permeable pavement sites, and to provide a basis for estimating their contribution to meeting the treatment and flow reduction requirements. The same infiltration test sites may suffice for bioretention and permeable pavement as long as the soil receptor is the same. Testing should occur between December 1 and April 1.

The certified soils professional or engineer can exercise discretion concerning the need for and extent of infiltration rate (saturated hydraulic conductivity, Ksat) testing. The professional can consider a reduction in the extent of infiltration (Ksat) testing if, in their judgment, information exists confirming that the site is unconsolidated outwash material with high infiltration rates, and there is adequate separation from groundwater:

- 1 foot separation from the bottom of a rain garden (per BMP T5.14A)
- 1 foot or 3 foot minimum separation from the bottom of a bioretention installation depending upon drainage area size (per BMP T7.30 Infeasibility Criteria).
- 1 foot below the bottom of the base course for a permeable pavement (per BMP T5.15).
**Bioretention and Rain Gardens:**

**Field Testing Requirements Based upon Project Size:**

**Projects subject to Core Requirements #1 - #5:**

In accordance with Section 2.5.5 Core Requirement #5 in Volume 1, projects subject only to Core Requirements #1 - #5 have to evaluate the feasibility of rain gardens unless a higher priority LID BMP is feasible or the applicant is meeting the LID performance standard through other BMPs. Perform a Small-Scale Pilot Infiltration Test (see Section 3.3.6) – or an alternative small scale test specified by the City of Olympia – to determine if the minimum measured infiltration rate of 0.3 in/hr is exceeded at the proposed rain garden location. Also determine whether the site has at least one foot minimum clearance to the seasonal high ground water or other hydraulic restriction layer.

Please refer to BMP T5.14A in Chapter 5 of Volume V for further design guidance for rain gardens.

**Projects subject to Core Requirements #1 - #9:**

Also in accordance with Section 2.5.5 Core Requirement #5 in Volume I, projects subject to Core Requirements #1 - #9 have to evaluate the feasibility of bioretention facilities unless a higher priority LID BMP is feasible or the applicant is meeting the LID performance standard through other BMPs. Infeasibility criteria and design criteria for bioretention are found in Chapter 7 of Volume V.

On a single, smaller commercial property, one bioretention facility will likely be appropriate. In that case, a test pit/boring and grain size analysis – or an alternative small scale test approved by the City of Olympia - must be performed at the proposed bioretention location. Tests at more than one site could reveal the advantages of one location over another.

On larger commercial sites, a test pit/boring and grain size analysis every 5,000 sq. ft. is advisable. If soil characteristics across the site are consistent, a geotechnical professional may recommend a reduction in the number of tests.

On multi-lot residential developments, multiple bioretention facilities, or a facility stretching over multiple properties are appropriate. In most cases, it is necessary to perform grain size analyses on multiple test pit/borings or other small-scale tests if approved by the City of Olympia. A test is advisable at each potential bioretention site. Long, narrow bioretention facilities, such as one following the road right-of-way, should have a test location at least every 200 lineal feet, and within each length of road with significant differences in subsurface characteristics.

However, if the site subsurface characterization, including soil borings across the development site, indicate consistent soil characteristics and depths to seasonal high ground water conditions or a hydraulic restriction layer, the number of test locations may be reduced to a frequency recommended by a geotechnical professional.

If PIT tests are performed, infiltration sites should be over-excavated 3 feet below the projected infiltration facility's bottom elevation unless minimum clearances to seasonal high ground water have or will be determined by another method. This over excavation is to determine if there are restrictive layers or ground water. Observations through a wet season can identify a seasonal groundwater restriction.

If a single bioretention facility serves a drainage area exceeding 1 acre, a ground
water mounding analysis may be necessary in accordance with Section 3.3.8.

**Assignment of Appropriate Correction Factors to the Sub-grade Soil:** *(Applicable to projects subject to Core Requirements #1 - #9; and to projects that must or choose to demonstrate compliance with the LID Performance Standard of Core Requirement #5).*

If deemed necessary by a qualified professional engineer, a correction factor may be applied to the measured $K_{sat}$ of the subgrade soils to estimate its design (long term) infiltration rate. *(Note: This is separate design issue from the assignment of a correction factor to the overlying, designed bioretention soil mix. See Chapter 7 of Volume V for that design issue).*

The overlying bioretention soil mix provides excellent protection for the underlying native soil from sedimentation. Accordingly, the correction factor for the sub-grade soil does not have to take into consideration the extent of influent control and clogging over time. The correction factor to be applied to in-situ, small-scale infiltration test results is determined by the number of tests in relation to the number of bioretention areas and site variability. See Table 3.3.1 in Section 3.3.6.

Correction factors range from 0.33 to 1 (no correction) and are determined by a licensed geotechnical engineer or licensed engineering geologist.

Tests should be located and be at an adequate frequency capable of producing a soil profile characterization that fully represents the infiltration capability where the bioretention areas are to be located. The correction factor depends on the level of uncertainty that variable subsurface conditions justify. If a pilot infiltration test is conducted for all bioretention areas or the range of uncertainty is low (for example, conditions are known to be uniform through previous exploration and site geological factors) one pilot infiltration test may be adequate to justify a correction factor of one. If the level of uncertainty is high, a correction factor near the low end of the range may be appropriate. Two example scenarios where low correction factors may apply include:

- Site conditions are highly variable due to a deposit of ancient landslide debris, or buried stream channels. In these cases, even with many explorations and several pilot infiltration tests, the level of uncertainty may still be high.
- Conditions are variable, but few explorations and only one pilot infiltration test is conducted. That is, the number of explorations and tests conducted do not match the degree of site variability anticipated.

**Project Submission Requirements:**

Submit the results of infiltration ($K_{sat}$) testing and ground water elevation testing (or other documentation and justification for the rates and hydraulic restriction layer clearances) with the Drainage Design Report and Drainage Control Plan as justification for the feasibility decision regarding bioretention and as justification for assumptions made in the runoff modeling.

**Modeling:**

For projects that have to demonstrate compliance with Core Requirements #6 and/or #7, it is preferable to enter each bioretention device and its drainage area into the approved computer models for estimating their performance.
However, where site layouts involve multiple bioretention facilities, the modeling schematic can become extremely complicated or not accommodated by the available schematic grid.

In those cases, multiple bioretention facilities with similar designs (i.e., soil depth, ponding depth, freeboard height, and drainage area to ponding area ratio), and infiltration rates (Ecology suggests within a factor of 2) may have their drainage areas and ponded areas be combined, and represented in the runoff model as one drainage area and one bioretention device. In this case, use a weighted average of the design infiltration rates at each location. The averages are weighted by the size of their drainage areas.

Each design infiltration rate is the measured infiltration rate ($K_{sat}$) multiplied by the appropriate correction (reduction) factors. For these native soils below bioretention soils, a site variability correction factor, $CF_v$, should be considered.

For bioretention with side slopes of 3H:1V or flatter, infiltration through the side slope areas can be significant. Where side slopes are 3H:1V or flatter, bioretention can be modeled allowing infiltration through the side slope areas to the native soil. In WWHM, modeling of infiltration through the side slope areas is accomplished by switching the default setting for “Use Wetted Surface Area (sidewalls): from “NO” to “YES.”

Additional guidance concerning LID modeling is available during training sessions on WWHM 2012.

**Legal Documentation to Track Rain Garden and Bioretention Obligations:**

Where drainage plan submittals include assumptions with regard to size and location of rain garden or bioretention facilities, approval of the plat, short-plat, or building permit should identify the rain garden or bioretention obligation of each lot; and the appropriate lots should have deed requirements for construction and maintenance of those facilities. Rain gardens and bioretention BMPs sited on individual private lots should also be included in the Stormwater Site Management Plan for the project.

**Permeable Pavement:**

**Field Testing Requirements based upon Project Size:**

*Projects subject to Core Requirements #1 - #5:*

In accordance with Section 2.5.5 Core Requirement #5 in Volume 1, projects subject only to Core Requirements #1 - #5 have to evaluate the feasibility of permeable pavement for a development site unless a higher priority BMP is feasible or the applicant is choosing to meet the LID performance standard using other BMPs. A small-scale Pilot Infiltration Tests (PIT) – or other small-scale tests as allowed by the City - should be performed for every 5,000 sq. ft. of permeable pavement, but not less than 1 test per site. Procedures to test for high ground water and infiltration rate (aka, saturated hydraulic conductivity, $K_{sat}$) are referenced in Chapter 3 of Volume III. Detailed procedures for the Small-Scale Pilot Infiltration Test are in Section 3.3.6 of this volume. Submit results as part of the Drainage Design Report to establish a basis for a feasibility decision.
Projects subject to Core Requirements #1 - #9:

Projects subject to Core Requirements #1 - #9 will likely have to evaluate a site for permeable pavement feasibility. Projects subject to Core Requirement #7 shall perform a soils investigation (one test pit or boring per 5,000 sq. ft. of permeable pavement (not less than 1 test per site)) and grain size analysis to determine design infiltration rates in accordance with Section 3.3.6. On commercial property that cannot use full dispersion, permeable pavement should be the first choice for parking lots and walkways, unless infeasible or the applicant demonstrates compliance with the LID performance standard through other BMPs.

On residential developments not using full dispersion (BMP T5.30), permeable pavements should be the first choice for residential access roads and walks, and for private walks and driveways on residential lots unless infeasible or the applicant demonstrates compliance with the LID performance standard through other BMPs. Small-scale infiltration tests should be performed at every proposed lot and at least every 200 feet of sidewalk. However, if the site subsurface characterization - including soil borings across the development site - indicate consistent soil characteristics and depths to seasonal high ground water conditions, the number of test locations may be reduced to a frequency recommended by a geotechnical professional.

Unless seasonal high ground water elevations across the site have already been determined, upon conclusion of the infiltration testing, infiltration sites should be over-excavated 1 foot to see any restrictive layers or ground water. Observations through a wet season can identify a seasonal ground water restriction.

Perform infiltration testing in the soil profile at the estimated bottom elevation of base materials for the permeable pavement. If no base materials, (e.g., a pervious concrete sidewalk), perform the testing at the estimated bottom elevation of the pavement.

Assignment of Appropriate Correction Factors:

(Applicable to projects subject to Core Requirements #1 - #9; and to projects that must or choose to demonstrate compliance with the LID Performance Standard of Core Requirement #5).

The correction factor for in-situ, small-scale pilot infiltration test (Core Requirement #5 only) is determined by the number of tests in relation to the size of the permeable pavement installation, site variability and the quality of the aggregate base material. Correction factors range from 0.33 to 1 (no correction). Tests should be located and be at adequate frequency capable of producing a soil profile characterization that fully represents the infiltration capability where the permeable pavement is located. If used, the correction factor depends on the level of uncertainty that variable subsurface conditions justify. If enough pilot infiltration tests (Core requirement #5 only) are conducted across the permeable pavement subgrade to provide an accurate characterization, or the range of uncertainty is low (for example, conditions are known to be uniform through previous exploration and site geological factors), then a correction factor of one for site variability may be justified. Additionally, a correction factor of 1 for the quality of pavement aggregate base material may be necessary if the aggregate base is clean washed material with
1% or less fines passing the 200 sieve. See Table 3.4.1 - Correction factors for in-situ Saturated Hydraulic Conductivity ($K_{sat}$) Measurements - to estimate design (long-term) infiltration rates.

If the level of uncertainty is high, a correction factor near the low end of the range may be appropriate. Two example scenarios where low correction factors may apply include:

- Site conditions are highly variable due to a deposit of ancient landslide debris, or buried stream channels. In these cases, even with many explorations and several pilot infiltration tests, the level of uncertainty may still be high.
- Conditions are variable, but few explorations and only one pilot infiltration test is conducted. That is, the number of explorations and tests conducted do not match the degree of site variability anticipated.

<table>
<thead>
<tr>
<th>Site Analysis Issue</th>
<th>Correction Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>Site variability and number of locations tested</td>
<td>$C_{Fv} = 0.33$ to 1</td>
</tr>
<tr>
<td>Quality of pavement aggregate base material</td>
<td>$C_{Fm} = 0.9$ to 1</td>
</tr>
</tbody>
</table>

Total correction factor ($C_{Ft}$) = $C_{Fv} \times C_{Fm}$

### Soil Suitability Criteria Confirmation:

Where permeable pavements are used for pollution-generating hard surfaces (primarily roads, driveways, and parking lots), there must be a determination whether the soil suitability criteria of Section 3.3.6 are met. The applicable criteria are:

- Cation Exchange Capacity > 5%
- Organic Content > 1%
- Measured (initial) saturated hydraulic conductivity < 12 in./hr.
- One foot depth of soil with above characteristics

Sites not meeting these criteria should be considered infeasible for permeable pavements for pollution-generating hard surfaces.

The information to make this determination may be obtained from various sources: historic site information, estimated qualities of a general soil type, laboratory analysis of field samples. The City of Olympia may identify regional areas as infeasible for permeable pavements for pollution-generating hard surfaces based upon knowledge of the region’s soil characteristics in regard to the criteria listed above.

### Project Submission Requirements:

Submit results of infiltration ($K_{sat}$) testing, ground water elevation testing (or other documentation and justification for the rates and hydraulic restriction layer clearances) with the Drainage Design Report and/or Abbreviated Drainage Plan as justification for the feasibility decision regarding permeable pavement, and as
justification for assumptions made in the runoff modeling. If necessary, also submit
documentation of meeting the soil suitability criteria.

Modeling:

In the runoff modeling, similar designs throughout a development can be summed
and represented as one large facility. For instance, walkways can be summed into
one facility. Driveways with similar designs (and enforced through deed restrictions)
can be summed into one facility. In these instances, a weighted average of the
design infiltration rates (where within a factor of two) for each location may be used.
The averages are weighted by the size of their drainage area. The design infiltration
rate for each site is the measured $K_{sat}$ multiplied by the appropriate correction
factors.

As an alternative, simply enter walks, patios, and driveways with little storage
capacity in the gravel bedding beneath them as lawn/landscape areas in the
continuous runoff model. Roads and parking lots that have storage in a base course
below the wearing surface should use the permeable pavement element in the
continuous runoff model.

Legal Documentation to Track Permeable Pavement Obligations:

Where drainage plan submittals include assumptions in regard to size and location of
permeable pavement, approval of the plat or short-plat should identify the
permeable pavement obligation of each lot; and the appropriate lots should have
deed requirements for construction and maintenance of those facilities. Permeable
pavements sited on individual private lots should also be included in the Stormwater
Site Management Plan for the project.
Appendix III-A  Isopluvial Maps for Design Storms

Included in this appendix are the 2, 10 and 100-year, 24-hour design storm and mean annual precipitation isopluvial maps for Western Washington. These have been taken from NOAA Atlas 2 “Precipitation - Frequency Atlas of the Western United States, Volume IX, Washington and are available on link at the following web address:

Western Washington Isopluvial 2-year, 24 hour
Western Washington Isoplivial 100-year, 24 hour

100-YEAR 24-HOUR PRECIPITATION
- Isopluvials of 100-year 24-hour precipitation in tenths of an inch annual

USDA-SCS NATIONAL CARTOGRAPHIC CENTER, FT. WORTH, TX, 1984
This appendix describes some of the information and assumptions used in the Western Washington Hydrology Model (WWHM). However, since the first version of WWHM was developed and released to public in 2001, WWHM program has gone through several upgrades incorporating new features and capabilities. It is anticipated that the next upgrade to WWHM will add low impact development (LID) modeling capability. WWHM users should periodically check Ecology’s WWHM web site for the latest releases of WWHM, user manual, and any supplemental instructions. The web address for WWHM is: [http://www.ecy.wa.gov/programs/wq/stormwater/wwhmtraining/index.html](http://www.ecy.wa.gov/programs/wq/stormwater/wwhmtraining/index.html)

**WWHM Limitations**

WWHM has been created for the specific purpose of sizing stormwater control facilities for new development and redevelopment projects in Western Washington. WWHM can be used for a range of conditions and developments; however, certain limitations are inherent in this software. These limitations are described below.

The WWHM uses the EPA HSPF software program to do all of the rainfall-runoff and routing computations. Therefore, HSPF limitations are included in the WWHM. For example, HSPF does not explicitly model backwater or tailwater control situations. This is also true in the WWHM.

**WWHM Information and Assumptions**

1. **Precipitation data.**
   
   **Length of record.**
   
   The WWHM uses long-term (50 - 70 years) precipitation data to simulate the potential impacts of land use development in western Washington. A minimum period of 20 years is sufficient to simulate enough peak flow events to produce accurate flow frequency results. A 40 to 50-year record is preferred. The actual length of record of each precipitation station varies, but all exceed 50 years.

   **Rainfall distribution.**
   
   The precipitation data are representative of the different rainfall regimes found in western Washington. More than 17 precipitation stations are used. These stations represent rainfall at elevations below 1500 feet. WWHM does not include snowfall and melt.

   The primary source for precipitation data is National Weather Service stations. The secondary source is precipitation data collected by local jurisdictions. During development of WWHM, county engineers at 19 western Washington counties were contacted to obtain local precipitation data.
Earlier versions of WWHM used hourly data from the precipitation stations in the table below to generate precipitation time series for use in WWHM. For WWHM2012, more recent precipitation data have been used to generate precipitation time series in 15-min time steps.

<table>
<thead>
<tr>
<th>Precipitation Station</th>
<th>Years of Data</th>
<th>County Coverage</th>
</tr>
</thead>
<tbody>
<tr>
<td>Astoria, OR</td>
<td>1955-1998 = 43</td>
<td>Wahkiakum</td>
</tr>
<tr>
<td>Blaine</td>
<td>1948-1998 = 50</td>
<td>Whatcom, San Juan</td>
</tr>
<tr>
<td>Burlington</td>
<td>1948-1998 = 50</td>
<td>Skagit, Island</td>
</tr>
<tr>
<td>Clearwater</td>
<td>1948-1998 = 50</td>
<td>Jefferson (west)</td>
</tr>
<tr>
<td>Darrington</td>
<td>1948-1996 = 48</td>
<td>Snohomish (northeast)</td>
</tr>
<tr>
<td>Everett</td>
<td>1948-1996 = 48</td>
<td>Snohomish (excluding northeast)</td>
</tr>
<tr>
<td>Frances</td>
<td>1948-1998 = 50</td>
<td>Pacific</td>
</tr>
<tr>
<td>Landsburg</td>
<td>1948-1997 = 49</td>
<td>King (east)</td>
</tr>
<tr>
<td>Longview</td>
<td>1955-1998 = 43</td>
<td>Cowlitz, Lewis (south)</td>
</tr>
<tr>
<td>McMillian</td>
<td>1948-1998 = 50</td>
<td>Pierce</td>
</tr>
<tr>
<td>Montesano</td>
<td>1955-1998 = 43</td>
<td>Grays Harbor</td>
</tr>
<tr>
<td>Olympia</td>
<td>1955-1998 = 43</td>
<td>Thurston, Mason (south), Lewis (north)</td>
</tr>
<tr>
<td>Port Angeles</td>
<td>1948-1998 = 50</td>
<td>Clallam (east)</td>
</tr>
<tr>
<td>Portland, OR</td>
<td>1948-1998 = 50</td>
<td>Clark, Skamania</td>
</tr>
<tr>
<td>Quilcene</td>
<td>1948-1998 = 50</td>
<td>Jefferson (east), Mason (north), Kitsap</td>
</tr>
<tr>
<td>Sappho</td>
<td>1948-1998 = 50</td>
<td>Clallam (west)</td>
</tr>
<tr>
<td>SeaTac</td>
<td>1948-1997 = 49</td>
<td>King (west)</td>
</tr>
</tbody>
</table>

The records were reviewed for length, quality, and completeness of record. Annual totals were checked along with hourly maximum totals. Using these checks, data gaps and errors were corrected, where possible. A "Quality of Record" summary was produced for each precipitation record reviewed.

The reviewed and corrected data were placed in multiple WDM (Watershed Data Management) files. One WDM file was created per county and contains all of the precipitation data to be used by the WWHM for that particular county. A local government that believes that it has a more accurate precipitation record to use with the WWHM should petition Ecology to allow use of that record, and to possibly incorporate that record into the WWHM. This may be more easily done in the future if the WWHM is upgraded to allow use of custom precipitation time series.

Computational time step.

The computational time step used in the earlier versions of WWHM has been one hour. The one-hour time step was selected to better represent the temporal variability of actual precipitation than daily data. WWHM2012 incorporates 15-minute precipitation time series.
2. **Precipitation multiplication factors.**

Precipitation multiplication factors increase or decrease recorded precipitation data to better represent local rainfall conditions. This is particularly important when the precipitation gage is located some distance from the study area.

Precipitation multiplication factors were developed for western Washington. The factors are based on the ratio of the 24-hour, 25-year rainfall intensities for the representative precipitation gage and the surrounding area represented by that gage’s record. The 24-hour, 25-year rainfall intensities were determined from the NOAA Atlas 2 (Precipitation-Frequency Atlas of the Western United States, Volume IX – Washington, 1973).

These multiplication factors were created for the Puget Sound lowlands plus all western Washington valleys and hillside slopes below 1500 feet elevation. The factors were placed in the WWHM database and linked to each county’s map. They are transparent to the general user and the default range is set to 0.8 – 2. The advanced user will have the ability to change the precipitation multiplication factor for a specific site. However, such changes will be recorded in the WWHM output.

3. **Pan evaporation data.**

Pan evaporation data are used to determine the potential evapotranspiration (PET) of a study area. Actual evapotranspiration (AET) is computed by the WWHM based on PET and available moisture supply. AET accounts for the precipitation that returns to the atmosphere without becoming runoff. Soil moisture conditions and runoff are directly influenced by PET and AET.

Evaporation is not highly variable like rainfall. Puyallup pan evaporation data are used for all of the 19 western Washington counties.

Pan evaporation data were assembled and checked for the same time period as the precipitation data and placed in the appropriate county WDM files.

Pan evaporation data are collected in the field, but PET is used by the WWHM. PET is equal to pan evaporation times a pan evaporation coefficient. Depending on climate, pan evaporation coefficients for western Washington range from 0.72 to 0.82.

NOAA Technical Report NWS 33, *Evaporation Atlas for the Contiguous 48 United States*, was used as the source for the pan evaporation coefficients. Pan evaporation coefficient values are shown on Map 4 of that publication.

As with the precipitation multiplication factors, the pan evaporation coefficients have been placed in the WWHM database and linked to each county’s map. They will be transparent to the general user. The advanced user will have the ability to change the coefficient for a specific site. However, such changes will be recorded in the WWHM output.

4. **Soil data.**

Soil type, along with vegetation type, greatly influences the rate and timing of the transformation of rainfall to runoff. Sandy soils with high infiltration rates produce little or no surface runoff; almost all runoff is from ground water. Soils with a compressed till layer slowly infiltrate water and produce larger amounts of surface runoff during storm events.

WWHM uses three predominant soil type to represent the soils of western Washington: till, outwash, and saturated
Till soils have been compacted by glacial action. Under a layer of newly formed soil lies a compressed soil layer commonly called "hardpan". This hardpan has very poor infiltration capacity. As a result, till soils produce a relatively large amount of surface runoff and interflow. A typical example of a till soil is an Alderwood soil (SCS class C). Where field infiltration tests indicate a measured (initial) infiltration rate less than 0.30 in/hr, the user may model the site as a class C soil.

Outwash soils have a high infiltration capacity due to their sand and gravel composition. Outwash soils have little or no surface runoff or interflow. Instead, almost all of their runoff is in the form of ground water. An Everett soil (SCS class A) is a typical outwash soil.

Outwash soils over high ground water or an impervious soil layer have low infiltration rates and act like till soils. Where ground water or an impervious soil layer is within 5 feet from the surface, outwash soils may be modeled as till soils in the WWHM.

Saturated soils are usually found in wetlands. They have a low infiltration rate and a high ground water table. When dry, saturated soils have a high storage capacity and produce very little runoff. However, once they become saturated they produce surface runoff, interflow, and ground water in large quantities. Mukilteo muck (SCS class D) is a typical saturated/wetland soil.

The user will be required to investigate actual local soil conditions for the specific development planned. The user will then input the number of acres of outwash (A/B), till (C/D), and saturated/wetland soils for the site conditions.

Alluvial soils are found in valley bottoms. These are generally fine-grained and often have a high seasonal water table. There has been relatively little experience in calibrating the HSPF model to runoff from these soils, so in the absence of better information, these soils may be modeled as till soils.

Additional soils will be included in the WWHM if appropriate HSPF parameter values are found to represent other major soil groups.

The three predominant soil types are represented in the WWHM by specific HSPF parameter values that represent the hydrologic characteristics of these soils. More information on these parameter values is presented below.

5. Vegetation data.

As with soil type, vegetation types greatly influence the rate and timing of the transformation of rainfall to runoff. Vegetation intercepts precipitation, increases its ability to percolate through the soil, and evaporates and transpires large volumes of water that would otherwise become runoff.

WWHM represents the vegetation of western Washington with three predominant vegetation categories: forest, pasture, and lawn (also known as grass).

Forest vegetation represents the typical second growth Douglas fir found in the Puget Sound lowlands. Forest has a large interception storage capacity. This means that a large amount of precipitation is caught in the forest canopy before reaching the ground and becoming available for runoff. Precipitation intercepted in this way is later evaporated back into the atmosphere. Forest also has the ability to transpire moisture from the soil via its root system. This leaves less water available for runoff.

Pasture vegetation is typically found in rural areas where the forest has been cleared and replaced with shrub or grass lots. Some pasture areas may be used to graze livestock. The interception storage and soil evapotranspiration capacity of pasture are less than forest. Soils may have also been compressed by mechanized equipment during clearing activities. Livestock can also compact soil. Pasture areas typically produce more runoff (particularly surface runoff and interflow) than forest areas.
Lawn vegetation is representative of the suburban vegetation found in typical residential developments. Soils have been compacted by earth moving equipment, often with a layer of topsoil removed. Sod and ornamental bushes replace native vegetation. The interception storage and evapotranspiration of lawn vegetation is less than pasture, more runoff results.

Predevelopment default land conditions are forest, although the user has the option of specifying pasture if there is documented evidence that pasture vegetation was native to the predevelopment site. If this option is used, the change will be recorded in the WWHM output.

Forest vegetation is represented by specific HSPF parameter values that represent the forest hydrologic characteristics. As described above, the existing regional HSPF parameter values for forest are based on undisturbed second-growth Douglas fir forest found today in western Washington lowland watersheds.

Postdevelopment vegetation will reflect the new vegetation planned for the site. The user has the choice of forest, pasture, and landscaped vegetation. Forest and pasture are only appropriate for postdevelopment vegetation in parcels separate from standard residential or non-standard residential/commercial developments. Development areas must only be designated as forest or pasture where legal restrictions can be documented that protect these areas from future disturbances. WWHM assumes the pervious land portion of developed areas is covered with lawn vegetation, as described above.

6. Development land use data.

The WWHM user must enter land use information for the pre-developed condition and the proposed development condition into the model. WWHM users must select the appropriate land use category and slope, where slope of 0-5% is flat, 5-15% is moderate, and greater than 15% is steep. The land use categories include: Impervious areas such as Roads, Roof, Driveways, Sidewalks, Parking, Ponds; and Pervious areas such as Lawn (this includes lawn, garden, areas with ornamental plants, and any natural areas not legally protected from future disturbance), Forest, and Pasture. The soils types available are A/B (outwash), C (Till), and Saturated (wetland).

Forest and pasture vegetation areas are only appropriate for separate undeveloped parcels dedicated as open space, wetland buffer, or park within the total area of the standard residential development. Development areas must only be designated as forest or pasture where legal restrictions can be documented that protect these areas from future disturbances.

Impervious, as the name implies, allows no infiltration of water into the pervious soil. All runoff is surface runoff. Impervious land typically consists of paved roads, sidewalks, driveways, and parking lots. Roofs are also impervious.

For the purposes of hydrologic modeling, only effective impervious area is categorized as impervious. Effective impervious area (EIA) is the area where there is no opportunity for surface runoff from an impervious site to infiltrate into the soil before it reaches a conveyance system (pipe, ditch, stream, etc.). An example of an EIA is a shopping center parking lot where the water runs off the pavement and directly goes into a catch basin where it then flows into a pipe and eventually to a stream. In contrast, some homes with impervious roofs collect the roof runoff into roof gutters and send the water down downspouts. When the water reaches the base of the downspout it can be directed into an infiltration system. If roof runoff is infiltrated according to the requirements of BMP T5.10A, the roof area can be considered ineffective impervious area. The roof area may be discounted from the project area entered into WWHM.
The non-effective impervious area uses the adjacent or underlying soil and vegetation properties. Vegetation often varies by the type of land use. The assumption is made in the WWHM that the EIA equals the TIA (total impervious area). This is consistent with King County’s determination of EIA acres for new developments. Where appropriate, the TIA can be reduced through the use of runoff credits (more on that below).

Earlier versions of WWHM (WWHM1 and WWHM2) provided the 2 optional features below for modeling of Standard Residential development and obtaining flow credits for incorporating low impact development (LID) techniques. Later upgrades to WWHM have provided for direct input of the standard residential development details by the WWHM users. WWHM2012 allows direct modeling of some LID techniques through use of new LID Elements. Other LID techniques will continue to be modeled in accordance with Appendix C of the Stormwater Management Manual for Western Washington.

**Standard Residential:** For housing developments where lot-specific details (e.g., size of roof and driveway) are not yet determined, the earlier versions of WWHM provided a set of default assumptions about the amount of impervious area per lot and its division between driveways and rooftops under the “Standard Residential” development land use type. Later versions of WWHM (e.g., WWHM3 or WWHM2012) do not have this option programmed in the model but the land use assumptions for the “Standard Residential” development are given below.

Ecology has selected a standard impervious area of 4200 square feet per residential lot, with 1000 square feet of that as driveway, walkways, and patio area, and the remainder as rooftop area. The rest of the lot acres will be assumed to be landscaped area (including lawn). The user inputs the number of residential lots and the total acreage of the residential lots (public right-of-way acreages and non-residential lot acreages excluded). The number of residential lots and the associated number of acres will be used to compute the average number of residential lots per acre. This value together with the number of residential lots and the impervious area in the public right-of-way will be used by the model to calculate the TIA for the proposed development. The areas covered by streets, parking areas, and sidewalk areas are input separately by the user.

**Runoff Credits:** Please note that the modeling of runoff credits using some of the low impact development techniques described in Appendix C have been updated. WWHM 2012 can now provide LID modeling capabilities in accordance with this manual. The following LID credit modeling is based on modeling in earlier versions of WWHM (WWHM2 and WWHM3).

Runoff credits can be obtained using any or all of the low impact development methods listed below. The WWHM has an automated procedure for taking credits for infiltrating or dispersing roof runoff - methods #1 and #2 below. Credits for using methods 3,4,8, and 9 must be taken by following the guidance in Appendix C. Methods 5, 6, and 10 also have guidance in Appendix C for taking credits. However, the new LID elements in WWHM2012 would allow direct modeling of methods 4, 5, 6, and 10 which would be a better representation of how they function to reduce surface runoff. Roof areas using method #7 - rainwater harvesting systems designed in accordance with the guidance in Appendix C need not be entered into the model. Also, if using method 11 – Full dispersion – the runoff model need not be used for the area that meets the criteria in Appendix C.

1. Infiltrate roof runoff
2. Disperse roof runoff
3. Disperse driveway and other hard surface runoff
4. Porous pavement for driveways and walks
5. Porous pavement for roads and parking lots
6. Vegetated Roofs
7. Rainwater Harvesting
8. Reverse slope sidewalks
9. Low impact foundations
10. Bioretention Areas
11. Full dispersion

1. Infiltrate Roof Runoff
Credit is given for disconnecting the roof runoff from the development’s stormwater conveyance system and infiltrating on the individual residential lots. The WWHM assumes that this infiltrated roof runoff does not contribute to the runoff flowing to the stormwater detention pond site. It disappears from the system and does not have to be mitigated. See Section 3.1.1 of Volume III for design requirements for downspout infiltration systems.

2. Disperse Roof Runoff
Credit is also given for disconnecting the roof runoff from the development’s stormwater conveyance system and dispersing it on the lawn/landscaped surface of individual lots. If the runoff is dispersed using a dispersion trench designed according to the requirements of Section 3.1.2 of Volume III, on single-family lots greater than 22,000 square feet, and the vegetative flow path of the runoff is 50 feet or longer through undisturbed native or compost-amended soils, the roof area can be entered into the model as landscaped area rather than impervious surface.

3. Disperse driveway and other hard surface runoff:
If runoff is dispersed in accordance with the guidance in BMP T5.11 or BMP T5.12, the driveway or other hard surface may be modeled as landscaped area.

4 & 5. Permeable pavement
The third option for runoff credit is the use of permeable pavement for private driveways, sidewalks, streets, and parking areas. The LID credit guidance in Appendix C was developed before WWHM2012, with the capability to directly model permeable pavements, became available. The LID credit guidance in Appendix C will direct you to enter a certain percentage of the pervious pavement area into the landscaped area category rather than the street/sidewalk/parking lot category. Even though WWHM2012 has other methods for calculating the impacts of permeable pavement, the methods described in Part 1 of Appendix C are still appropriate to use where the pervious pavement does not have a significant depth of base course for storage.

Follow similar procedures for vegetated roofs, reverse slope sidewalks, and low impact foundations. The LID credit guidance of Appendix C directs how these surfaces should be entered into the model. If you do not know the specific quantities of the different land cover types for your development (e.g., the individual lots will be sold to builders who will determine layout and size of home), you should start with the assumption of 4200 sq. ft. of impervious area per lot – including 1,000 sq. ft. for driveways, and begin making adjustments in those totals as allowed in the LID guidance of Appendix C.
Other Development Options and Model Features

WWHM allows the flexibility of bypassing a portion of the development area around a flow control facility and/or having off-site inflow that is entering the development area pass through the flow control facility.

Bypass occurs when a portion of the development does not drain to a stormwater detention facility. On-site runoff from a proposed development project may bypass the flow control facility provided that all of the following conditions are met.

1. Runoff from both the bypass area and the flow control facility converges within a quarter-mile downstream of the project site discharge point.
2. The flow control facility is designed to compensate for the uncontrolled bypass area such that the net effect at the point of convergence downstream is the same with or without bypass.
3. The 100-year peak discharge from the bypass area will not exceed 0.4 cfs.
4. Runoff from the bypass area will not create a significant adverse impact to downstream drainage systems or properties.
5. Water quality requirements applicable to the bypass area are met.

Off-site Inflow occurs when an upslope area outside the development drains to the flow control facility in the development. If the existing 100-year peak flow rate from any upstream off-site area is greater than 50% of the 100-year developed peak flow rate (undetained) for the project site, then the runoff from the off-site area must not flow to the on-site flow control facility. The bypass of off-site runoff must be designed so as to achieve both of the following:

1. Any existing contribution of flows to an on-site wetland must be maintained.
2. Off-site flows that are naturally attenuated by the project site under predeveloped conditions must remain attenuated, either by natural means or by providing additional on-site detention so that peak flows do not increase.

Application of WWHM in Re-developments Projects

WWHM allows only forest or pasture as the predevelopment land condition in the Design Basin screen. This screen does not allow other types of land uses such as impervious and landscaped areas to be entered for existing condition. However, WWHM can be used for redevelopment projects by modeling the existing developed areas that are not subject to the flow control requirements of Volume I as off-site areas. For the purposes of predicting runoff from such an existing developed area, enter the existing area in the Off-site Inflow screen. This screen is designed to predict runoff from impervious and landscaped areas in addition to the forest and pasture areas. If the existing 100-year peak flow rate from the existing developed areas that are not subject to flow control is greater than 50% of the 100-year developed peak flow rate (undetained but subject to the flow control requirements of Volume I), then the runoff from the off-site area must not be allowed to flow to the on-site flow control facility.

7. PERLND and IMPLND parameter values.

In WWHM (and HSPF) pervious land categories are represented by PERLNDs; impervious land categories (EIA) by IMPLNDs. An example of a PERLND is a till soil covered with forest vegetation. This PERLND has a unique set of HSPF parameter values. For each PERLND there are 16 parameters that describe various hydrologic factors that influence runoff. These range from interception storage to infiltration to active ground water evapotranspiration. Only four parameters are required to represent IMPLND.
The PERLND and IMPLND parameter values to be used in the WWHM are listed below. These values are based on regional parameter values developed by the U.S. Geological Survey for watersheds in western Washington (Dinicola, 1990) plus additional HSPF modeling work conducted by AQUA TERRA Consultants.

**PERLND Parameters**

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</table>

**PERLND types:**

- **OP = Outwash Pasture**
- **OL = Outwash Lawn**
- **SF = Saturated Forest**
- **SP = Saturated Pasture**
- **OF = Outwash Forest**
- **SL = Saturated Lawn**

**PERLND parameters:**

- **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 = ground water exponent variable (inch\(^{-1}\))**
- **AGWRC = active ground water recession constant (day\(^{-1}\))**
- **INFEXP = infiltration exponent**
- **INFILD = ratio of maximum to mean infiltration**
- **BASETP = base flow evapotranspiration (fraction)**
- **AGWETP = active ground water 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 more complete description of these PERLND parameters is found in the HSPF User Manual (Bicknell et al, 1997).

PERLND parameter values for other additional soil/vegetation categories will be investigated and added to the WWHM, as appropriate.

**IMPLND Parameters**

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</table>

IMPLND parameters:

- **LSUR** = length of surface overland flow plane (feet)
- **SLSUR** = slope of surface overland flow plane (feet/feet)
- **NSUR** = roughness of surface overland flow plane (Manning’s n)
- **RETSCE** = retention storage (inches)

A more complete description of these IMPLND parameters is found in the HSPF User Manual (Bicknell et al, 1997).

The PERLND and IMPLND parameter values will be transparent to the general user. The advanced user will have the ability to change the value of a particular parameter for that specific site. However, the only PERLND and IMPLND parameters that are authorized to be adjusted by the user are LSUR, SLSUR, and NSUR. These are parameters whose values are observable at an undeveloped site, and whose values can be reasonably estimated for the proposed development site. Any such changes will be recorded in the WWHM output. The user should submit justifications for changes with their project submittal to the reviewing jurisdiction. Ecology will issue guidance within the WWHM Users Manual on the range of and methods for estimating acceptable parameter changes.

Earlier versions of WWHM (WWHM1 and WWHM2) provided only one category of moderate land slope (typically 5-15% slopes). In more recent versions of WWHM (WWHM3 and WWHM2012), two additional land categories have been added to account for the flat (0-5%) and steep (15-25%) land slopes.

Surface runoff and interflow will be computed based on the PERLND and IMPLND parameter values. Ground water flow can also be computed and added to the total runoff from a development if there is a reason to believe that ground water would be surfacing (such as where there is a cut in a slope). However, the default condition in WWHM assumes that no ground water flow from small catchments reaches the surface to become runoff. This is consistent with King County procedures (King County, 1998).

**8. Guidance for flow-related standards.**

Use flow-related standards to determine whether or not a proposed stormwater facility will provide a sufficient level of mitigation for the additional runoff from land development. Guidance is provided on the standards that must be met to comply with the Ecology Stormwater Management Manual.
There are three flow-related standards stated in Volume I: Core Requirement #5 – On-site Stormwater Management; Core Requirement #7 - Flow Control and Core Requirement #8 - Wetlands Protection.

Core Requirement #5 allows the user to demonstrate compliance with the LID Performance Standard of matching developed discharge durations to pre-developed durations for the range of pre-developed discharge rates from 8% of the 2-year peak flow to 50% of the 2-year peak flow. If the post-development flow duration values exceed any of the predevelopment flow levels between 8% and 50% of the 2-year predevelopment peak flow values, then the LID performance standard not been met.

Core Requirement #7 specifies that stormwater discharges to streams shall match developed discharge durations to predeveloped durations for the range of predeveloped discharge rates from 50% of the 2-year peak flow up to the full 50-year peak flow. In general, matching discharge durations between 50% of the 2-year and 50-year will result in matching the peak discharge rates in this range.

WWHM uses the predevelopment peak flow value for each water year to compute the predevelopment 2- through 100-year flow frequency values. The postdevelopment runoff 2- through 100-year flow frequency values are computed from the outlet of the proposed stormwater facility. The user must enter the stage-surface area-storage-discharge table (HSPF FTABLE) for the stormwater facility. The model then routes the postdevelopment runoff through the stormwater facility. As with the predevelopment peak flow values, the model will select the maximum developed flow value for each water year to compute the developed 2- through 100-year flow frequency.

The actual flow frequency calculations are made using the federal standard Log Pearson Type III distribution described in Bulletin 17B (United States Water Resources Council, 1981). This standard flow frequency distribution is provided in U.S. Geological Survey program J407, version 3.9A-P, revised 8/9/89. The Bulletin 17B algorithms in program J407 are included in the WWHM calculations.

Core Requirement #7 is based on flow duration. WWHM will use the entire predevelopment and postdevelopment runoff record to compute flow duration. The standard requires that post-development runoff flows must not exceed the flow duration values of the predevelopment runoff between the predevelopment flow values of 50 percent of the 2-year flow and 100 percent of the 50-year flow.

Flow duration is computed by counting the number of flow values that exceed a specified flow level. The specified flow levels used by WWHM in the flow duration analysis are listed below.

1. 50% of the 2-year predevelopment peak flow.
2. 100% of the 2-year predevelopment peak flow.
3. 100% of the 50-year predevelopment peak flow.

In addition, flow durations are computed for 97 other incremental flow values between 50 percent of the 2-year predevelopment peak flow and 100 percent of the 50-year predevelopment peak flow.

There are three criteria by which flow duration values are compared:

1. If the postdevelopment flow duration values exceed any of the predevelopment flow levels between 50% and 100% of the 2-year predevelopment peak flow values (100 Percent Threshold) then the flow duration requirement has not been met.
2. If the postdevelopment flow duration values exceed any of the predevelopment flow levels between 100% of the 2-year and 100% of the 50-year predevelopment peak flow values more than 10 percent of the time (110 Percent Threshold) then the flow duration requirement has not been met.
3. If more than 50 percent of the flow duration levels exceed the 100 percent threshold then the flow duration requirement has not been met.
The results are provided in the WWHM report.

Core Requirement #8 specifies that total discharges to wetlands must not deviate by more than 20% on daily basis, and must not deviate by more than 15% on monthly basis. Flow components feeding the wetland under both Pre-and Post-development scenarios are assumed to be the sum of the surface, interflow, and ground water flows from the project site. The WWHM is being revised to more easily allow this comparison.

**References for Western Washington Hydrology Model**

- King County. 1998. Surface Water Design Manual. Department of Natural Resources. Seattle, WA.

Note – The modeling guidance in this section was developed for use with an earlier version of WWHM, WWHM3. Since then, WWHM has been updated to incorporate direct modeling of some LID techniques in WWHM2012 to better represent how they would function to reduce surface runoff. The new LID elements include Permeable Pavement, Green Roof, and Bio-retention discussed in Part 2 of this Appendix.

The Washington State Department of Ecology (Ecology) requires the use of the Western Washington Hydrology Model (WWHM) and other approved runoff models (currently approved alternative models are the King County Runoff Time Series and MGS Flood) for estimating surface runoff and sizing stormwater control and treatment facilities. Part 1 of this appendix explains how to represent various LID techniques within WWHM 3 so that their benefit in reducing surface runoff can be estimated. The lower runoff estimates should translate into smaller stormwater treatment and flow control facilities. In certain cases, use of various techniques can result in the elimination of those facilities.

As Puget Sound gains more experience with and knowledge of LID techniques, the design criteria will evolve. Also, our ability to model their performance will change as our modeling techniques improve. Therefore, we anticipate this guidance will be updated periodically to reflect the new knowledge and modeling approaches.

One such update should be available later this year (2012). The updated guidance will explain modeling techniques to be used with the latest publicly available version of the WWHM (tentative name: WWHM 2012). A summary of the modeling techniques planned for WWHM 2012 is included as Part 2 in this appendix. Because WWHM 2012 and the updated LID modeling guidance won’t be released until later this year, municipal stormwater permittees are not obligated to require its use during the 2013-2018 permit term. However, because WWHM 2012 will make modeling LID developments easier and more technically accurate; and because it will include a number of other updates and improvements (e.g., updated rainfall files), Ecology will encourage its use. We anticipate that most local governments will choose to require its use or an equivalent program (e.g., an updated MGS Flood) once they are readily available. Ecology intends to make sure that sufficient training opportunities are available on WWHM 2012, so that municipal staff and designers have adequate opportunity to become familiar with it prior to the deadlines in the municipal permits for adopting and applying updated stormwater requirements.

In previous editions of the manual, Appendix III-C included a summary of design criteria for each LID BMP. The reader is now directed to Volume V for those design criteria.
Part 1: Guidance for Use with WWHM 3

C.1 Permeable Pavements

C.1.1 Porous Asphalt or Concrete

Description

1. Base material laid above surrounding grade:
   a) Without underlying perforated drain pipes to collect stormwater
   b) With underlying perforated drain pipes for stormwater collection:
      at or below bottom of base layer
      elevated within the base course

2. Base material laid partially or completely below surrounding grade:
   a) Without underlying perforated drain pipes underlying soil type
      Option 1: Grass over
      Option 2: Impervious surface routed to a Gravel Trench/Bed
   b) With underlying perforated drain pipes:
      at or below bottom of base layer
      elevated within the base course

C.1.2 Grid/lattice systems (non-concrete) and Paving Blocks

Description

1. Base material laid above surrounding grade
   a) Without underlying perforated drain pipes
   b) With underlying perforated drain pipes

2. Base material laid partially or completely below surrounding grade

---

1 See section C.11 for detailed instructions concerning how to represent the base material below grade as a gravel trench/bed in the Western Washington Hydrology Model.

2 If the perforated pipes function is to distribute runoff directly below the wearing surface, and the pipes are above the surrounding grade, follow the directions for 2a above.
a) Without underlying perforated drain pipes

Option 1:
Grid/lattice as grass on underlying soil.
Paving blocks as 50% grass; 50% impervious.

Option 2:
Impervious surface routed to a Gravel Trench/Bed.¹

b) With underlying perforated drain pipes

at or below bottom of base layer
Impervious surface

elevated within the base course²
Model as impervious surface routed to a Gravel Trench/Bed.¹

C.2 Dispersion

C.2.1 Full Dispersion for the Entire Development Site

Residential Developments that implement BMP T5.30 do not have to use approved runoff models to demonstrate compliance. They are assumed to fully meet the treatment and flow control requirements.

C.2.2 Full Dispersion for Part of the Development Site

Those portions of residential developments that implement BMP T5.30 do not have to use approved runoff models to demonstrate compliance. They are assumed to fully meet the treatment and flow control requirements.

C.2.3 Partial Dispersion on residential lots and commercial buildings

If roof runoff is dispersed on single-family lots or commercial lots according to the design criteria and guidelines in BMP T5.10B of Volume III, through undisturbed native landscape or lawn/landscape area that meets the guidelines in BMP T5.13, the user has two options.

Option 1: The roof area may be modeled as landscaped area if the vegetated flow path is 50 feet or more. In WWHM this can be done on the Mitigated Scenario screen by entering the roof area into one of the entry options for dispersal of impervious area runoff. Alternatively, in WWHM, this can be done by entering the roof area as landscaped area with the appropriate landscaped slope. Where the flow path is between 25 and 50 feet and a dispersion trench is used, the roof area may be modeled as 50% landscape/50% impervious. Do this in WWHM on the Mitigated Scenario screen by entering 50% of the roof area as impervious and the other 50% as landscaped area.

Option #2: Use the lateral flow basin elements in WWHM for dispersing runoff from the roof area on the landscaped area. In this option, the “Impervious Lateral Basin” element/icon is used to represent the roof area(s). That element/icon is then connected to a “Pervious Lateral Basin” icon that represents the pervious area into which the roof is being dispersed. The user must direct Surface Flow from the Impervious Lateral Basin (roof area) to the “Surface” Flow of the Pervious Lateral Basin (landscaped area). Then, the user should direct surface runoff and interflow from the Pervious Lateral Basin to a treatment system, retention/detention basin, or directly to a point of compliance.

Whether option #1 or #2 is used, the vegetated flow path is measured from the downspout or dispersion system discharge point to the downgradient edge of the vegetated area. That flow path must be at least 50 feet unless a dispersion trench per BMP T5.10B is used with a vegetated flow path of 25 to 50 feet.
Where BMP T5.11 (concentrated flow dispersion) or BMP T5.12 (sheet flow dispersion) of Volume V – Chapter 5 is used to disperse runoff from impervious areas other than roofs into a native vegetation area or an area that meets the guidelines in BMP T5.13 of Volume V – Chapter 5, the same two options as described above are available. The user may model the impervious area as landscaped area (50 feet or more of vegetated flow path), 50% landscape/50% impervious (25 to 50 feet of vegetated flow path), or the lateral flow element/icons may be used. As above, the vegetated flow path from the dispersal point to the downgradient edge of the vegetated area must be at least 50 feet, unless a dispersion trench (see BMP 5.10B) is used with a vegetated flow path of 25 to 50 feet.

C.3 Downspout Full Infiltration
Roof areas served by downspouts that drain to infiltration dry wells or infiltration trenches that are sized in accordance with the guidance in BMP T5.10A do not have to be entered into the runoff model. They are assumed to fully infiltrate the roof runoff.

C.4 Vegetated Roofs
C.4.1 Option 1 Design Criteria
• 3 inches to 8 inches of soil/growing media
Runoff Model Representation
• 50% till landscaped area; 50% impervious area
C.4.2 Option 2 Design Criteria
• > 8 inches of soil/media
Runoff Model Representation
• 50% till pasture; 50% impervious area

C.5 Rainwater Harvesting
Do not enter drainage area into the runoff model.

Note: This applies only to drainage areas for which a monthly water balance indicates no overflow of the storage capacity.

C.6 Reverse Slope Sidewalks
• Enter sidewalk area as landscaped area over the underlying soil type.
• Alternatively, use the “lateral flow” icons. Use the “Lateral Flow Impervious Area” icon for the sidewalk, and use the “Lateral Flow Basin” icon for the downgradient vegetated area.

C.7 Minimal Excavation Foundations
• Where residential roof runoff is dispersed on the upgradient side of a structure in accordance with the design criteria and guidelines in BMP T5.10B of Volume III – Chapter 3, the tributary roof area may be modeled as pasture on the native soil.
• In “step forming,” the building area is terraced in cuts of limited depth. This results in a series of level plateaus on which to erect the form boards. Where “step forming” is used on a slope, the square footage of roof that can be modeled as pasture must be reduced to account for lost soils. The following equation (suggested by Rick Gagliano of Pin Foundations, Inc.) can be used to reduce the roof area that can be modeled as pasture.

\[ A_1 - dC(.5) \times A_1 = A_2 \]
\[ \text{dP} \]

\[ \text{A}_1 = \text{roof area draining to up gradient side of structure} \]

\[ \text{dC} = \text{depth of cuts into the soil profile} \]

\[ \text{dP} = \text{permeable depth of soil} \ (\text{The A horizon plus an additional few inches of the B horizon where roots permeate into ample pore space of soil}). \]

\[ \text{A}_2 = \text{roof area that can be modeled as pasture on the native soil. The rest of the roof is modeled as impervious surface unless it is dispersed in accordance with the next bullet.} \]

- If roof runoff is dispersed downgradient of the structure in accordance with the design criteria and guidelines in BMP T5.10B of Volume III – Chapter 3, AND there is at least 50 feet of vegetated flow path through native material or lawn/landscape area that meets the guidelines in BMP T5.13 of Volume V – Chapter 5, the tributary roof areas may be modeled as landscaped area. Alternatively, use the lateral flow elements to send roof runoff onto the lawn/landscape area that will be used for dispersion.

C.8 Tree Retention and Planting

C.8.1 Tree Retention Flow Control Credit

Flow control credits for retained trees are provided in Table C.1 by tree type. These credits can be applied to reduce impervious or other hard 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.

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<td>Evergreen</td>
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<tr>
<td>Deciduous</td>
<td>10% of canopy area (minimum of 50 sq. ft./tree)</td>
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Impervious Area Mitigated = \( \Sigma \) Canopy Area \( \times \) Credit (sq. ft.).

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

C.8.2 Newly Planted Tree Flow Control Credits

Flow control credits for newly planted trees are provided in Table C.2 by tree type. These credits can be applied to reduce the impervious or other hard surface area requiring flow control. Credits range from 20 to 50 square feet per tree.
### Table C.2.
Flow Control Credits for Newly Planted Trees

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<tr>
<td>Deciduous</td>
<td>20 sq. ft. per tree</td>
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**Impervious Area Mitigated** = Σ Number of Trees x Credit (sq. ft.).

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

### C.9 Soil Quality and Depth

All areas that meet the soil quality and depth requirement may be entered into the model as pasture rather than lawn/landscaping.

### C.10 Bioretention

#### C.10.1 Runoff Model Representation

**Pothole design (bioretention cells).**

Bioretention is represented by using the “Gravel trench/bed” icon with a steady-state infiltration rate. Proper infiltration rate selection is described below. The user inputs the dimensions of the gravel trench. Layer 1 on the input screen is the bioretention soil layer. Enter the soil depth and a porosity of 40%. Layer 2 is the free standing water above the bioretention soil. Enter the maximum depth of free standing water (i.e., up to the invert of an overflow pipe or a spillway, whatever engages first for surface release of water), and 100% for porosity. Bioretention with underlying perforated drain pipes that discharge to the surface can also be modeled as gravel trenches/beds with steady-state infiltration rates. However, the only volume available for storage (and modeled as storage as explained herein) is the void space within the imported material (usually sand or gravel) below the bioretention soil and below the invert of the drain pipe.

Using one of the procedures explained in Volume III - Chapter 3 of this manual, estimate the initial measured (a.k.a., short-term) infiltration rate of the native soils beneath the bioretention soil and any base materials. Because these soils are protected from fouling, no correction factor will be applied.

**Facilities without an underdrain:**

If using the default bioretention soil mix from Chapter 7 of Volume V, 12 inches per hour is the initial infiltration rate. The long-term rate is either 3 inches per hour or 6 inches per hour depending upon the size of the drainage area, and the use of a pretreatment device for solids removal prior to the bioretention facility. See Chapter 7 of Volume V. If using a custom imported soil mix other than the default, its saturated hydraulic conductivity (used as the infiltration rate) must be determined using the procedures described in Chapter 7 of Volume V. The long-term infiltration rate is one-fourth or one-half of that rate depending upon the size of the drainage area and the use of a pretreatment device for solids removal. See Chapter 7 of Volume V.
**Facilities with an elevated underdrain:**

Note that only the estimated void space of the aggregate bedding layer that is below the invert of the underdrain pipe provides storage volume that provides a flow control benefit. Assume a 40% void volume for the Type 26 mineral aggregate specified in Chapter 7 of Volume V.

**Linear Design: (bioretention swale or slopes)**

**Swales**

Where a swale design has a roadside slope and a back slope between which water can pond due to an elevated, and an overflow/drainage pipe at the lower end of the swale, the swale may be modeled as a gravel trench/bed with a steady state infiltration rate. This method does not apply to swales that are underlain by a drainage pipe.

If the long-term infiltration rate through the imported bioretention soil is lower than the infiltration rate of the underlying soil, the surface dimensions and slopes of the swale should be entered into the WWHM as the trench dimensions and slopes. The effective depth is the distance from the soil surface at the bottom of the swale to the invert of the overflow/drainage pipe. If the infiltration rate through the underlying soil is lower than the estimated long-term infiltration rate through the imported bioretention soil, the trench/bed dimensions entered into the WWHM should be adjusted to account for the storage volume in the void space of the bioretention soil. Use 40 percent porosity for bioretention planting mix soils recommended above for Layer 1 in WWHM.

This procedure to estimate storage space should only be used on bioretention swales with a 1% slope or less. Swales with higher slopes should more accurately compute the storage volume in the swale below the drainage pipe invert.

For a swale design with an underdrain, the directions above under Pothole design apply.

**C.10.2 WWHM Routing and Runoff File Evaluation**

In WWHM3, all infiltrating facilities must have an overflow riser to model overflows that occur should the available storage be exceeded. So in the Riser/Weir screen, for the Riser head enter a value slightly smaller than the effective depth of the trench (say 0.1 ft below the Effective Depth); and for the Riser diameter enter a large number (say 10,000 inches) to ensure that there is ample capacity for overflows. The overflow should be routed to the point of compliance or a downstream facility. If the facility is underdrained, the underdrain must be similarly routed.

Within the model, route the runoff into the gravel trench by grabbing the gravel trench icon and placing it below the tributary “basin” area. Be sure to include the surface area of the bioretention area in the tributary “basin” area. Run the model to produce the effluent runoff file from the theoretical gravel trench. For projects subject to the flow control standard, compare the flow duration graph of that runoff file to the target pre-developed runoff file for compliance with the flow duration standard. If the standard is not achieved a downstream retention or detention facility must be sized (using the WWHM standard procedures) and located in the field. A conveyance system should be designed to route all overflows from the bioretention areas to centralized treatment facilities, and to flow control facilities if flow control applies to the project.

**C.10.3 Modeling of Multiple Bioretention facilities**

Where multiple bioretention facilities are scattered throughout a development, it may be possible to cumulatively represent a group of them that have similar characteristics as one large bioretention facility serving the cumulative area tributary to those facilities. For this to be a reasonable representation, the design of each bioretention facility in the group should be similar (e.g., same depth of soil, same depth of surface ponded water, roughly the same ratio of impervious area to bioretention...
volume). In addition, the group should have similar (0.5x to 1.5x the average) controlling infiltration rates (i.e., either the long-term rate of the bioretention soil, or the initial rate of the underlying soil) that can be averaged as a single rate.

**C.11 WWHM Instructions for Estimating Runoff Losses in Road Base Material Volumes that are Below Surrounding Grade**

**Introduction**

This section applies to roads or parking lots that have been constructed with a permeable pavement and whose underlying base materials extend below the surrounding grade of land. The over-excavated volume can temporarily store water before it infiltrates or overflows to the surrounding ground surface. This section describes design criteria and modeling approaches for such designs.

**Pre-requisite**

Before using this guidance to estimate infiltration losses, the designer should have sufficient information to know whether adequate depth to a seasonal high ground water table, or other infiltration barrier (such as bedrock) is available. Where the seasonal high groundwater or an underlying impermeable/low permeability layer would create saturated conditions within one foot of the bottom of the lowest gravel base course, permeable pavement is considered infeasible.

**C.11.1 Instructions for Roads on Zero to 2% Grade**

For road projects whose base materials extend below the surrounding grade, the below grade volume of base materials may be modeled in WWHM as a Gravel trench/bed with a set infiltration rate. The pervious pavement area is entered as a basin with an equivalent amount of impervious area that is routed to the gravel trench/bed. If an underdrain is installed at the bottom of the base materials, the pavement is modeled as impervious surface without a gravel trench.

First, place a “basin” icon in the “Schematic” grid. Enter the appropriate pre-developed and post-developed descriptions of your project site (or threshold discharge area of the project site). Assume that your pervious pavement surfaces are impervious surfaces. By placing a Gravel trench/bed icon below the basin icon in the Schematic grid, we are routing the runoff from the road and any other tributary area into the below grade volume that is represented by the Gravel trench/bed.

Enter the dimensions of the Gravel trench/bed: the length of the base materials that are below grade (parallel to the road); the width of the below grade material volume; and the depth. The available storage is the void volume in the gravel base layer below the pervious pavement. Enter the void ratio for the gravel base in the Layer 1 field. For example, for a project with a gravel base of 32% porosity, enter 0.32 for the Layer 1 porosity. If the below grade base course has perforated drainage pipes elevated above the bottom of the base course, but below the elevation of the surrounding ground surface, the “Layer 1 Thickness” is the distance from the invert of the lowest pipe to the bottom of the base course.

Also in WWHM3, the Gravel trench/bed facilities must have an overflow riser to model overflows that occur should the available storage get exceeded. So for the “Riser Height”, enter a value slightly smaller than the effective depth of the base materials (say 0.1 ft below the Effective Total Depth); and for the “Riser Diameter” enter a large value (say 10,000 inches) to ensure that there is ample capacity should overflows from the trench occur.

For all infiltration facilities, WWHM3 has a button that asks, “Use Wetted Surface Area?” The answer should remain “NO.”

Using one of the procedures explained in Chapter 3, estimate the initial measured (a.k.a., short-term) infiltration rate of the native soils beneath the base materials. Enter that into the “measured infiltration rate” field. For the Infiltration Reduction Factor, enter 0.5.
Run the model to produce the overflow runoff file from the gravel trench. Compare the flow duration graph of that runoff file to the target pre-developed runoff file for compliance with the flow duration standard. If the standard is not achieved a downstream retention or detention facility must be sized (using the WWHM standard procedures) and located in the field. Design the road base materials to direct any water that does not infiltrate into a conveyance system that leads to the retention or detention facility.

C.11.2 Instructions for Roads on Grades above 2%

Road base material volumes that are below the surrounding grade and that are on a slope can be modeled as a gravel trench with an infiltration rate and a nominal depth. Represent the below grade volume as the gravel trench. Grab the gravel trench icon and place it below the “basin” icon so that the computer model routes all of the runoff into the gravel trench.

The dimensions of the gravel trench are: the length (parallel to and beneath the road) of the base materials that are below grade; the width of the below grade base materials; and an Effective Total Depth of 1 inch. In WWHM3, all infiltrating facilities must have an overflow riser to model overflows that occur should the available storage get exceeded. So, enter 0.04 ft (½ inch) for the “Riser Height” and a large Riser Diameter (say 1000 inches) to ensure that there is no head build up.

Note: If a drainage pipe is embedded and elevated in the below grade base materials, the pipe should only have perforations on the lower half (below the spring line) or near the invert. Pipe volume and trench volume above the pipe invert cannot be assumed as available storage space. If a drainage pipe is placed at the bottom of the base material, the pavement is modeled as an impervious surface without any gravel trench.

Estimate the infiltration rate of the native soils beneath the base materials. See the previous section (Instructions for Roads on Zero to 2% Grade) for estimating options and for how to enter infiltration rates and infiltration reduction factors for the gravel trench. In the “Material Layers” field, enter ½ inch for Layer 1 Thickness and its appropriate porosity. For all infiltration facilities, WWHM3 has a button that asks, “Use Wetted Surface Area?” The answer should remain “NO.”

Run the model to produce the effluent runoff file from the gravel trench (base materials). Compare the flow duration graph of that runoff file to the target pre-developed runoff file for compliance with the flow duration standard. If the standard is not achieved a downstream retention or detention facility must be sized (using the WWHM standard procedures) and located in the field. The road base materials should be designed to direct any water that does not infiltrate into a conveyance system that leads to the retention or detention facility.

C.11.3 Instructions for Roads on a Slope with Internal Dams within the Base Materials that are Below Grade

In this option, a series of infiltration basins is created by placing relatively impermeable barriers across the below grade base materials at intervals downslope. The barriers inhibit the free flow of water down the grade of the base materials. The barriers must not extend to the elevation of the surrounding ground. Provide a space sufficient to pass water from upgradient to lower gradient basins without causing flows to surface out the sides of the base materials that are above grade.

Each stretch of trench (cell) that is separated by barriers can be modeled as a gravel trench. This is done by placing the “Gravel trench/bed” icons in series in WWHM. For each cell, determine the average depth of water within the cell (Average Cell Depth) at which the barrier at the lower end will be overtopped.

Specify the dimensions of each cell of the below-grade base materials using the “Gravel trench/bed” dimension fields for: the “Trench Length” (length of the cell parallel to the road); the “Trench Bottom
Width” (width of the bottom of the base material); and the Effective Total Depth (the Average Cell Depth as determined above).

Also in WWHM3, all infiltrating facilities must have an overflow riser to model overflows that occur should the available storage get exceeded. For each trench cell, the available storage is the void space within the Average Cell Depth. WWHM calculates the storage/void volume of the trench cell using the porosity values entered in the “Layer porosity” fields. The value for the “Riser Height” should be slightly below the “Effective Total Depth” (say by about 1/8” to ¼”). For the Riser diameter, enter a large number (say 10,000 inches) to ensure that there is ample capacity should overflows from the below-grade trench occur.

Each cell should have its own tributary drainage area that includes the road above it, any project site pervious areas whose runoff drains onto and through the road, and any off-site areas. Each drainage area is represented with a “basin” icon.

Below is the computer graphic representation of a series of Gravel trench/beds and the Basins that flow into them.

![Image of Gravel Trench Beds and Basins](image)

It is possible to represent a series of cells as one infiltration basin (using a single gravel trench icon) if the cells all have similar length and width dimensions, slope, and Average Cell Depth. A single “basin” icon is also used to represent all of the drainage area into the series of cells.

On the Gravel Trench screen under “Infiltration”, there is a field that asks the following “Use Wetted Surface Area?” By default, it is set to “NO”. It should stay “NO” if the below-grade base material trench has sidewalls steeper than 2 horizontal to 1 vertical.
Using the procedures explained above for roads on zero grade, estimate the infiltration rate of the native soils beneath the trench. Also as explained above, enter the appropriate values into the “Measured Infiltration Rate” and “Infiltration Reduction Factor” boxes.

Run the model to produce the effluent runoff file from the below grade trench of base materials. Compare the flow duration graph of that runoff file to the target pre-developed runoff file for compliance with the flow duration standard. If the standard is not achieved size a downstream retention or detention facility (using the WWHM standard procedures) and locate it in the field. Design the road base materials to direct any water that does not infiltrate into a conveyance system that leads to the retention or detention facility.

**Part 2: Summary of WWHM 2012 Representation of LID BMPs**

**Downspout Dispersion – BMP T5.10B**

Where BMP T5.10B – Downspout Dispersion - is used to disperse runoff into an undisturbed native landscape area or an area that meets BMP T5.13 – Soil Quality and Depth, and the vegetated flow path is at least 50 feet, the connected roof area should be modeled as a lateral flow impervious area. Do this in WWHM on the Mitigated Scenario screen by connecting the dispersed impervious area to the lawn/landscape lateral flow soil basin element representing the area that will be used for dispersion.

Ecology may develop guidance for representing multiple downspout dispersions in a project site. If such guidance is not forthcoming, in situations where multiple downspout dispersions will occur, Ecology may allow the roof area to be modeled as a landscaped area (where the 50 foot flowpath requirement is met), or as 50% landscape/50% lawn (where a gravel trench is used to disperse into a vegetated area with a 25 to 50 foot flowpath) so that the project schematic in WWHM becomes manageable.

**Concentrated Flow Dispersion – BMP T5.11**

Where BMP T5.11 - Concentrated Flow Dispersion - is used to disperse impervious area runoff into an undisturbed native landscape area or an area that meets BMP T5.13 – Soil Quality and Depth, and the vegetated flow path is at least 50 feet, the impervious area should be modeled as a lateral flow impervious area. Do this in WWHM on the Mitigated Scenario screen by connecting the dispersed impervious area to the lawn/landscape lateral flow soil basin element representing the area that will be used for dispersion.

Ecology may develop guidance for representing multiple concentrated flow dispersions in a project site. If such guidance is not forthcoming, in situations where multiple concentrated flow dispersions will occur, Ecology may allow the impervious area to be modeled as a landscaped area so that the project schematic in WWHM becomes manageable.

**Sheet Flow Dispersion – BMP T5.12**

Where BMP T5.12 – Sheet Flow Dispersion - is used to disperse impervious area runoff into an undisturbed native landscape area or an area that meets BMP T5.13 – Soil Quality and Depth, the impervious area should be modeled as a lateral flow impervious area. Do this in WWHM on the Mitigated Scenario screen by connecting the dispersed impervious area to the lawn/landscape lateral flow soil basin element representing the area that will be used for dispersion.

Ecology may develop guidance for representing multiple sheet flow dispersions in a project site. If such guidance is not forthcoming, in situations where multiple sheet flow dispersions will occur, Ecology may allow the impervious area to be modeled as a landscaped area so that the project schematic in WWHM becomes manageable.
Post-Construction Soil Quality and Depth – BMP T5.13

Enter area as pasture

Bioretention – BMP T7.30

Use new bioretention element for each type: cell, swale, or planter box.

The equations used by the elements are intended to simulate the wetting and drying of soil as well as how the soils function once they are saturated. This group of LID elements uses the modified Green Ampt equation to compute the surface infiltration into the amended soil. The water then moves through the top amended soil layer at the computed rate, determined by Darcy’s and Van Genuchten’s equations. As the soil approaches field capacity (i.e., gravity head is greater than matric head), the model determines when water will begin to infiltrate into the second soil layer (lower layer). This occurs when the matric head is less than the gravity head in the first layer (top layer). The second layer is intended to prevent loss of the amended soil layer. As the second layer approaches field capacity, the water begins to move into the third layer – the gravel underlayer. For each layer, the user inputs the depth of the layer and the type of soil.

For the Ecology-recommended soil specifications for each layer in the design criteria for bioretention, the model will automatically assign pre-determined appropriate values for parameters that determine water movement through that soil. These include: wilting point, minimum hydraulic conductivity, maximum saturated hydraulic conductivity, and Van Genuchten number.

If a user opts to use soils that deviate from the recommended specifications, the default parameter values do not apply. The user will have to use the Gravel Trench element to represent the bioretention facility and follow the procedures identified for WWHM3.

For Bioretention with underlying perforated drain pipes that discharge to the surface, the only volume available for storage (and modeled as storage as explained herein) is the void space within the aggregate bedding layer below the invert of the drain pipe. Use 40% void space for the Type 26 mineral aggregate specified in Chapter 7 of Volume V.

Using one of the procedures explained in Volume III - Chapter 3 of this manual, estimate the initial measured (a.k.a., short-term) infiltration rate of the native soils beneath the bioretention soil and any base materials. Because these soils are protected from fouling, no correction factor will be applied.

Permeable Pavements – BMP T5.15

Use new porous pavement element.

User specifies pavement thickness & porosity, aggregate base material thickness & porosity, maximum allowed ponding depth & infiltration rate into native soil. For grades greater than 2%, see additional guidance under the WWHM3 section.

Vegetated Roofs – BMP T5.17

Use new green roof element

User specifies media thickness, vegetation type, roof slope, and length of drainage.

Impervious Reverse Slope Sidewalks – BMP T5.18

Use the lateral flow elements to send the impervious area runoff onto the lawn/landscape area that will be used for dispersion.

Ecology may develop guidance for representing multiple impervious reverse slope sidewalks in a project site. If such guidance is not forthcoming, in situations where multiple impervious reverse slope sidewalks
will occur, Ecology may allow the impervious area to be modeled as a landscaped area so that the project schematic in WWHM becomes manageable.

**Minimal Excavation Foundations – BMP T5.19**

- Where residential roof runoff is dispersed on the up gradient side of a structure in accordance with the design criteria and guidelines in BMP T5.10B, the tributary roof area may be modeled as pasture on the native soil.

- In “step forming,” the building area is terraced in cuts of limited depth. This results in a series of level plateaus on which to erect the form boards. Where “step forming” is used on a slope, the square footage of roof that can be modeled as pasture must be reduced to account for lost soils. The following equation (suggested by Rick Gagliano of Pin Foundations, Inc.) can be used to reduce the roof area that can be modeled as pasture.

  \[
  A_1 - \frac{dC}{dP} \cdot A_1 = A_2
  \]

  \(A_1 = \text{roof area draining to up gradient side of structure}\)

  \(dC = \text{depth of cuts into the soil profile}\)

  \(dP = \text{permeable depth of soil (The A horizon plus an additional few inches of the B horizon where roots permeate into ample pore space of soil)}\).

  \(A_2 = \text{roof area that can be modeled as pasture on the native soil. The rest of the roof is modeled as impervious surface unless it is dispersed in accordance with the next bullet.}\)

- If roof runoff is dispersed down gradient of the structure in accordance with the design criteria and guidelines in BMP T5.10B, AND there is at least 50 feet of vegetated flow path through native material or lawn/landscape area that meets the guidelines in BMP T5.13, the tributary roof areas should be modeled as a lateral flow impervious area. This is done in WWHM on the Mitigated Scenario screen by connecting the dispersed impervious area to the lawn/landscape lateral flow soil basin element representing the area that will be used for dispersion.

  Ecology may develop guidance for representing multiple downspout dispersions in a project site. If such guidance is not forthcoming, in situations where multiple downspout (down gradient) dispersions will occur, Ecology may allow the roof area to be modeled as a landscaped area so that the project schematic in WWHM becomes manageable.

**Full dispersion – BMP T5.30**

**Full downspout infiltration – BMP T5.10A**

**Rainwater Harvesting – BMP T5.20**

If BMP design criteria are followed, the area draining to the three BMPs listed immediately above is not entered into the runoff model.

**Newly planted trees – BMP T5.16**

**Retained trees – BMP T5.16**

If BMP design criteria are followed, the total impervious/hard surface areas entered into the runoff model may be reduced by an amount indicated in the criteria for the tree BMPs listed immediately above.
Perforated Stub-out Connection – BMP T5.10C

Any flow reduction is variable and unpredictable. No computer modeling techniques are allowed that would predict any reduction in flow rates and volumes from the connected area.
Volume III References

- King County Runoff Time Series (KCRTS), King County Department of Natural Resources, Personal Communication, 1999.
Resource Materials (not specifically referenced in text)

- Caraco, D., Claytor, R., Stormwater BMP Design Supplement for Cold Climates USEPA, December 1997
- King County, Washington, Surface Water Design Manual, September 1, 1998.


Woodward-Clyde, BMP Design Recommendations, November 1995