Thurston County Drainage Design and Erosion Control Manual

Volume III
Hydrologic Analysis and Stormwater Conveyance

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Chapter 1 - Introduction to Volume III

1.1 What is the Purpose of this Volume?

This volume of the *Drainage Design and Erosion Control Manual* describes hydrologic analysis techniques and general design criteria for flow control and water quality Best Management Practices (BMPs). Design details and requirements for specific flow control and water quality BMPs are provided in Volume V. This volume also includes hydrologic analysis techniques, design criteria and specifications for stormwater conveyance systems including pipes, open channels, outfalls and other stormwater conveyance structures.

This volume is intended to prescribe approved methods and requirements for calculating infiltration rates, runoff flow volumes and rates to be used in sizing water quality treatment and flow control BMPs to minimize or eliminate impacts on downstream properties and natural resources. The County recognizes that it is not always possible to fully prevent any downstream impacts; in these cases, the County may require the project to provide off-site mitigation.

These regulations and criteria are based on fundamental principles of drainage, hydraulics, and hydrology, environmental considerations, and publications, manuals, and texts accepted by the professional engineering community. The project design engineer is responsible for being knowledgeable of and proficient with necessary design methodologies identified in this manual. The following is a partial list of publications which may be used as reference documents:

- The Washington State Department of Ecology *Stormwater Management Manual For Western Washington*
- Any Washington State Department of Ecology Approved Stormwater Management Manual, such as one produced by an NPDES Phase I community
- The Low Impact Development Technical Guidance Manual for Puget Sound (Washington State University Extension and the Puget Sound Partnership)
- Washington State Department of Transportation *Highway Runoff Manual.*
- *Applied Handbook of Hydrology*, by V.T. Chow
- *Handbook of Hydraulics*, by E.G. Brater and H.W. King
- Washington State Department of Transportation *Hydraulics Manual*
- *Soil Survey of Thurston County, Washington*, published by the Natural Resource Conservation Service, U.S. Department of Agriculture

- Washington State Department of Transportation *Standard Plans for Road, Bridge and Municipal Construction*

- *Thurston County Road Standards*, or the latest amendment

The most current edition of all publications shall be used.

1.2 How This Volume is Organized

Volume III is organized into three chapters and three appendices:

- **Chapter 1**: Introduction.

- **Chapter 2**: Hydrologic design standards and acceptable analysis methods, including the use of hydrograph methods for BMP design, an overview of computerized modeling methods, analysis of closed depressions, and evaluation of the feasibility and sizing of infiltration facilities.

- **Chapter 3**: Natural and constructed conveyance systems and acceptable analysis methods. This chapter also discusses hydraulic structures linking conveyance systems to runoff treatment and flow control facilities.

- **Appendix A**: Infiltration testing procedures. This appendix also includes the USDA soil textural triangle, used for alternative methods of determining infiltration rates.

- **Appendix B**: SBUH/SCS computer models and charts and tables useful in designing conveyance systems with event-based hydrologic models. This includes: design storm rainfall totals, isopluvial maps for western Washington, common Thurston County Soil types, and hydrologic groupings, SCS curve numbers, and hydraulic roughness coefficients.

- **Appendix C**: Nomographs useful for culvert sizing.

- **Appendix D:** Summarizes the feasibility criteria that can be used to determine if various on-site stormwater management BMPs in the List #1 or List #2 option of Core Requirement #5 can or cannot be used on the site. This information is also presented under the description of each BMP, but is summarized in Appendix D as a quick reference point.

1.3 How Do I Get Started?

First, consult Chapter 2 of Volume I to determine which Core Requirements apply to your project and to select BMPs. After determining the Core Requirements for your project and selecting BMPs, use Volume III (this volume) to determine the methods of estimating design volume or flow rates for those BMPs. Design guidelines for stormwater BMPs are included in Volume V. These facilities can then be included in any required stormwater submittals (see Volume I, Chapter 3). Chapter 3 of this volume also includes information on the design of stormwater conveyance systems.

Chapter 2 - Hydrologic Analysis and Design Standards

Hydrology is the study of the source, properties, distribution, and laws of water as it moves through its closed cycle (the hydrologic cycle). In this manual, however, the term “hydrologic analysis” addresses and quantifies only a small portion of this cycle, the relatively short-term movement of water over land resulting from precipitation, called surface water or stormwater runoff. Localized and long-term groundwater movement is also a concern for successful stormwater management, but only as this relates to the movement of water on or near the surface, such as stream base flow or shallow groundwater effects on stormwater infiltration systems.

This chapter defines the minimum computational standards for conducting hydrologic analysis and how to apply these standards. It also explains the hydrologic design process, including flow routing through on-site stormwater management facilities.

Due to the relationship between stormwater runoff quantity (both flow and volume) and quality, it is critical to consider runoff treatment when designing for flow control and vice versa. Runoff treatment and flow control goals can often be accomplished in one facility. For example, wet ponds can be designed to provide both runoff treatment and flow control by providing for live storage volume above the permanent pool.

Site planning and layout play an important role in the amount of stormwater runoff generated by a project site. Reductions in impervious areas result in smaller runoff treatment and flow control facilities, thereby reducing stormwater management costs. Low Impact Development (LID) directly addresses this idea by limiting runoff and creating more aesthetically appealing sites. LID is discussed in Chapter 2 of Volume V.

Some of the things that must be considered during site planning and layout include: minimizing creating hard and impervious surfaces, clustering buildings and preserving larger areas of open space, minimizing directly connected hard and impervious areas (try to separate impervious surfaces with areas of turf, or other vegetation or gravel), incorporation of low maintenance landscaping that doesn't need frequent applications of fertilizers, herbicides and pesticides and minimizing the impact area and soil compaction during construction.

2.1 Minimum Computational Standards

The Ecology approved methods available to compute stormwater infiltration and runoff, which is then used to size Runoff Treatment and Flow Control BMPs depends on the type of information required and the size of the drainage area to be analyzed, as follows:

- For the purpose of designing flow-based Runoff Treatment BMPs, an Ecology approved 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 rate.

- For the purpose of designing volume-based Runoff Treatment BMPs (i.e. Wet pool BMPs), there are two acceptable methods to calculate the water quality design storm volume:
  - An Ecology approved continuous simulation hydrologic model based on the EPA's HSPF program, or an approved equivalent model. (See Continuous Simulation Models)
  - The single event hydrograph method, using precipitation depth from the 6-month 24-hour storm and NRCS curve number equations. (See Single Event Hydrograph Method)
- For conveyance system design, the designer may use a single event hydrologic model, a continuous simulation model, or the Rational Method to determine peak flow rate. For conveyance facilities that are also designed as water quality or flow control BMPs a continuous simulation runoff model shall be used to design the facility to meet the water quality or flow control requirements and the methodologies of this chapter shall be used to design the same facility for conveyance of stormwater. A single event hydrologic model may be used to determine the peak flow rate. The peak flow rate from a continuous runoff model will vary depending on the time step used in the model. Therefore, the length of the time step must be sufficiently short relative to the time of concentration of the watershed to provide for reasonable conveyance system design flows. For most situations in Thurston County, a 15-minute (maximum) time step will be sufficient for conveyance system design. If the project is in a predominantly urbanized watershed with a time of concentration less than about 15 minutes (roughly 10 acres in size), the conveyance design must either use a 5-minute time step (if available), or use an event-based model for conveyance sizing. Conveyance design is discussed in detail in Chapter 3 of this Volume.
- For the purpose of designing flow control BMPs, an Ecology approved continuous simulation hydrologic model, based on the U.S. EPA's HSPF program, or an approved equivalent model, must be used. Flow Control BMP criteria are discussed in Volume 1, Chapter 4. Circumstances where different methodologies apply are summarized in Table III - 2.1 Summary of Applicable Hydrologic Design Methodologies for Design of Stormwater Best Management Practices in Thurston County

Table III - 2.1 Summary of Applicable Hydrologic Design Methodologies for Design of Stormwater Best Management Practices in Thurston County

Method	Runoff Treatment	Flow Control	Conveyance
Continuous Runoff Models: (WWHM2012 or MGSFlood)	Method applies to all BMPs	Method applies to all BMPs	Method applies with appropriate time step based on time of concentration
SCSUH/SBUH (Soil Conservation Service Unit Hydrograph/Santa Barbara Urban Hydrograph)	Not Applicable	Not Applicable	Method applies
Rational Method	Not Applicable	Not Applicable	Method applies for some conveyance design

[a] can be used for biofiltration BMPs (BF.01 – BF.05)

- By default, the Department of Ecology's WWHM2012 uses rainfall/runoff relationships originally developed for specific basins in the Puget Sound region for all parts of western Washington. These default parameters may be replaced with basin-specific rainfall/runoff data established by extensive field monitoring approved by the County where such data will improve the model's accuracy.

  Free WWHM2012 software and documentation can be found at the Department of Ecology website: http://www.ecy.wa.gov/programs/wq/stormwater/wwhmtraining/index.html.

  A professional version of WWHM2012 with expanded capabilities can be purchased from Clear Creek Solutions, Inc. at http://www.clearcreeksolutions.com/.

- Use of continuous simulation runoff models other than WWHM2012 or MGSFlood must be approved by the County before being used as a computational standard.

- If a basin plan is being prepared, then a hydrologic analysis shall be performed using a continuous simulation runoff model such as the U.S. EPA's HSPF model, the U.S. EPA's Stormwater Management Model (SWMM), or an equivalent model as approved by Thurston County.

  For large, master-planned developments, the County may require a basin-specific calibration of HSPF program, rather than the use of the default parameters from Ecology approved continuous simulation hydrologic models based on the EPA's HSPF program. Basin-specific calibrations may be required for projects that encompass more than 320 acres.

Continuous Simulation Modeling Vs. Single Event Hydrograph Method

A continuous simulation runoff 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.

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 and the shape of the hydrograph such that the hydrograph would better reflect actual pre-developed conditions.

Another reason for the overestimation of the runoff is the curve numbers (CN) in Ecology's 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. The CN values are typically higher than the standard CN values published in Technical Release 55 (USDA et al., 1986). In 1995, the NRCS recalled the use of the western Washington CNs for floodplain management and found that the stand CNs better describe the hydrologic conditions for rainfall events in western Washington. However, based on runoff comparisons with the King County Runoff Time Series (KCRTS), better estimates of runoff are obtained when using the western Washington CNs for the developed areas such a parks, lawns, and other landscaped areas. Accordingly, the CNs in this manual 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 single event hydrograph method.

Another major weakness of the 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 fluctuation are necessary for discharges to wetlands and for tracking influent water elevations and bypass quantities to properly size stormwater BMPs.

2.2 Continuous Simulation Models

Continuous Simulation Model Approval

As of July 1, 2019, Ecology reviewed the following continuous simulation models for use to comply with 2019 – 2024 Phase I and Western Washington Phase II Municipal Stormwater Permit requirements.

- Western Washington Hydrology Model (WWHM2012) Version 4.2.16 (or later), released October 10, 2018 (**approved**)
- MGSFlood Version 4.49, released May 9, 2019 (**limited approval – see below**)
- King County Runoff Time Series (KCRTS) (**not approved**)

(At this time, MGSFLood is not approved for use in modeling BMP LID.08: Bioretention. MSGFlood Version 4.49 is approved for other modeling scenarios, using either the gage data or the 158 year synthetic precipitation time series.)

The approval status for the programs is provided in the “Additional Resources” folder in the interactive online SWMMWW. The approval status is specific to whether the program may be used to gain compliance with the 2019 – 2024 Municipal Stormwater General Permit requirements.

Note that the approval status may change. Check the “Additional Resources” folder in the interactive online SWMMWW.

2.3 Western Washington Hydrology Model

This section summarizes the assumptions made in creating the WWHM and discusses limitations of the model. Note that the WWHM is being updated regularly and much of the following information is for background and overview only. 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 LID modeling capability. For example, WWHM2012 now includes modeling elements for stormwater LID BMPs. WWHM users should periodically check Ecology’s WWHM web site for the latest releases of WWHM, user manuals, and any supplemental instructions. The web address for WWHM is:

https://ecology.wa.gov/Regulations-Permits/Guidance-technical-assistance/Stormwater-permittee-guidance-resources/Stormwater-manuals/Western-Washington-Hydrology-Model

Using WWHM to Model Flow-Related Standards

Flow related standards are used to determine whether or not a proposed Flow Control BMP will provide a sufficient level of mitigation for the additional runoff form land development. There are three flow-related standards described in this Manual: The LID performance standard, the Flow Control performance standard, and the wetlands protection standards.

- Core Requirement #5: On-site Stormwater Management 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 percent of the 2-year peak flow to 50 percent of the 2-year peak flow. If the post development duration values exceed any of the predevelopment flow levels between 8 percent and 50 percent of the 2-year predevelopment peak flow values, then the LID performance standard has not been met.

- Core Requirement #7: Flow Control specifies that stormwater discharges shall match developed discharge durations to pre-developed durations for the range of pre-developed discharge rates from 50 percent of the 2-year peak flow up to the full 50-year peak flow. This is the Flow Control Performance Standard.

  WWHM computes the pre-development and post-development runoff for the 2- through 100-year flow frequency values from the outlet of the proposed stormwater facility as follows:

  - WWHM uses the pre-development peak flow value for each water year to compute the pre-development 2- through 100-year flow frequency values. The post-development runoff 2- through 100-year flow frequency values are computed form the outlet of the proposed Flow Control BMP. The user must enter the stage-surface area-storage-discharge table (HSFP FTABLE) for the Flow Control BMP. The model then routes the post-development runoff through the Flow Control BMP. As with the pre-development 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 *Guidelines for Determining Flood Flow Frequency* (Interagency Advisory

Committee on Water Data, 1982). This standard flow frequency distribution is provided in U.S. Geologic Survey program J407, version 3.9A-P, revised 8/9/89. The Guidelines for Determining Flood Flow Frequency (Interagency Advisory Committee on Water Data, 1982) algorithms in program J407 are included in the WWHM calculations.

The Flow Control Performance Standard is based on flow duration. WWHM uses the entire pre-development and post-development runoff record, and computes flow durations 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:

- 50% of the 2-year pre-development peak flow.
- 100% of the 2-year pre-development peak flow.
- 100% of the 50-year pre-development peak flow.

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

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

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

The results are provided in the WWHM report.

- Core Requirement #8: Wetlands Protection refers to Appendix I-C: Wetlands Protection Guidelines in Ecology's 2019 SWMMWW, which includes measures to protect the hydroperiod of the wetland. Flow components feeding the wetland under both pre- and post-development scenarios are assumed to be the sum of the surface, interflow, and groundwater flows form the project. Site. WWHM has the capability to model flows to wetlands and analyze the criteria described in Appendix I-C: Wetlands Protection Guidelines in Ecology's 2019 SWMMWW.

As of the publication date of Ecology's 2019 SWMMWW (July 2019), the algorithms needed to perform the analysis associated with the hydroperiod protection guidelines described in I-C.4 Wetland Hydroperiod Protection (2019 SWMMWW) are not available in WWHM. However, WWHM can be used to provide model simulation of flows to wetlands under both existing condition and post-development condition. The analysis and comparisons of those flows (under existing and post-development conditions) must be conducted outside WWHM; for example, by using a spreadsheet.

Limitations to WWHM

Ecology created WWHM for the specific purpose of sizing stormwater control facilities for new development and redevelopment project 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 the WWHM. For example, backwater or tailwater control situations are not explicitly modeled by HSPF. This is also true in the 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.

Assumptions Made in Creating WWHM

Precipitation Data

- *Length of record:* 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 the ones used in WWHM exceed 50 years.

- *Computational time step:* The computational time step used in earlier versions of WWHM was one hour. The one-hour time step was selected to better represent the temporal variability of actual precipitation than daily data. WWHM no incorporates 15-minute time steps.

The 15-minute time step was selected to better represent the temporal variability of actual precipitation. These data are used in WWHM computations to generate runoff hydrographs. The computations include generating the water quality design flow rates and volumes for sizing Runoff Treatment BMPs.

- *Rainfall Distribution:* WWHM uses over 17 precipitation stations, representing the different rainfall regimes found in western Washington. These stations represent rainfall at elevations below 1500 feet. WWHM does not include snowfall and melt. As previously noted, these default parameters may be replaced with basin-specific rainfall data established by extensive field monitoring approved by the County where such data will improve the model's accuracy.

  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 listed in Ecology's 2019 SWMMWW to generate precipitation timeseries for use in WWHM. WWHM now uses more recent precipitation data to generate precipitation timeseries in 15-minute time steps.

  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 WWHM for that particular county.

Precipitation Multiplication Factors

- WWHM uses precipitation multiplication factors to 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.

- The multiplication factors were created for the Puget Sound lowlands plus all western Washington valleys and hillside slopes below 1500 feet elevation.

- 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 *NOAA ATLAS 2, Precipitation-Frequency Atlas of the Western United States, Volume IX – Washington* (Miller, et al., 1973).

- The factors have been placed in the WWHM database and linked to each county's map. They are transparent to the general user. However, the advanced user has the ability to change the precipitation multiplication factor for a specific site where justified and approved by the County. Changes made by the user are recorded in the WWHM output. By default, WWHM does not allow the precipitation multiplication factor to be below 0.8 or above 2.

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. WWHM's default setting uses Puyallup pan evaporation data 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* (Farnsworth et al., 1982) 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 int eh WWHM database and linked to each county's map. They are transparent to the general user. However, the advanced use has the ability to change the pan evaporation coefficient for a specific site where justified and approved by the County. Changes made by the user are recorded in the WWHM output.

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 groundwater. Soils with a compressed till layer slowly infiltrate water and produce larger amounts of surface runoff during storm events.

- WWHM uses three predominant soil types 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 lines 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 groundwater. An Everett soil (SCS class A) is a typical outwash soil.

    Outwash soils over high groundwater or an impervious soil layer have low infiltration rates and act like till soils. Where groundwater 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 groundwater 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 groundwater in large quantities.
- 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 GSPF to model runoff form 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.

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 form the soil via its root system. This leaves less water available for runoff.

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

    - *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.

- The pre-development land conditions are generally assumed as forest (the default condition), however, the user has the ability to specify pasture or the existing land cover, when appropriate. See Core Requirement #7: Flow

Control in Volume I, Chapter 2 for guidance on when Ecology allows the designer to use pasture or the existing land cover as the pre-developed land condition.

- Post-development 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 post-development vegetation in parcels separate from standard residential or nonstandard residential/commercial developments. WWHM assumes the pervious land portion of developed areas is coved with lawn vegetation, as described above.

  Post-development vegetative areas must only be designed as forest or pasture where legal restriction can be documented that protect these areas from future disturbances; unless, these are amended in accordance with BMP LID.02: Post-Construction Soil Quality and Depth. Where lawn/landscaped areas use BMP LID.02: Post-Construction Soil Quality and Depth, they may be entered into approved runoff models as “Pasture” rather than “Lawn/Landscaping”.

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 BMP.

- 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:

  - A 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.

    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 down-spouts. 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 LID.04: Downspout Infiltration Systems, 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 WWHM that the EIA equals the TIA (total impervious area). This is consistent with King County's determination of EIA acres for new developments.

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 (except as specified in LID modeling, such as BMP LID.02: Post-Construction Soil Quality and Depth) must only be modeled as forest or pasture where legal restrictions can be documented that protect these areas from future disturbances**.

  - The soils types available are A/B (outwash), C (Till), and Saturated (wetland).

- 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 4,200 square feet per residential lot, with 1'000 square feet of that as driveway, walkways, and patio area, and the remainder as rooftop area.

  WWHM no longer includes the standard residential development category. Designers can use the above land use assumptions when 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.

- Previous guidance for modeling LID BMPs in WWHM directed users to apply runoff credits for BMPs that WWHM was unable to model (such as dispersion and permeable pavements). WWHM now allows direct modeling of some LID BMPs through use of LID Elements. If a LID BMP does not have a modeling element in WWHM, guidance is provided within the BMP in Volume V for how to model the BMP.

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.

- 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 over 20 parameters that describe various hydrologic factors that influence runoff. These range from interception storage to infiltration to active groundwater evapotranspiration. Only four parameters are required to represent IMPLND.

- The PERLND and IMPLND parameter 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. Groundwater flow can also be computed and added to the total runoff from a development if there is a reason to believe that groundwater would be surfacing (such as where there is a cut in a slope). However, the default condition in WWHM assumes that no groundwater flow from small catchments reaches the surface to become runoff.

- The PERLND and IMPLND parameter values are transparent to the general user. However, the advanced user has the ability to change the value of a particular parameter for that specific site. 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 are recorded in the WWHM output. The user should submit justifications for changes with their project submittal to Thurston County. Ecology will issue guidance within the WWHM User’s Manual on the range of and methods for estimating acceptable parameter changes.

- The 16 PERLND and four IMPLND parameter values originally used when creating WWHM are listed in Table III-2.3: Original WWHM PERLND Parameters. A more complete description of these PERLND parameters is found in the HSPF User Manual (Bicknell et al., 1997). Since the original creation of WWHM, new PERLND parameters for other soil/vegetation categories have been added.

- The four IMPLND parameter values originally used when creating WWHM are listed in Table III-2.4: Original WWHM IMPLND Parameters. A more complete description of these IMPLND parameters is found in the HSPF User Manual (Bicknell et al., 1997). No new IMPLND para- meters have been added since the original creation of WWHM.

Table III - 2.2 Original WWHM PERLND Parameters

PERLND Parameters	Land Types								
	Till Soils			Outwash Soils			Saturated Soils		
	Forest	Pasture	Lawn	Forest	Pasture	Lawn	Forest	Pasture	Lawn
	TF	TP	TL	OF	OP	OL	SF	SP	SL
LZSN Lower Zone Storage Nominal (inches)	4.5	4.5	4.5	5.0	5.0	5.0	4.0	4.0	4.0
INFILT Infiltration Capacity (inches/hour)	0.08	0.06	0.03	2.0	1.6	0.80	2.0	1.8	1.0
LSUR Length of Surface OverlandFlow Plane (feet)	400	400	400	400	400	400	100	100	100
SLSUR Slope of Surface Overland Flow Plane (feet/feet)	0.10	0.10	0.10	0.10	0.10	0.10	0.001	0.001	0.001
KVARY Ground Water Exponent Variable (inch^{-1})	0.5	0.5	0.5	0.3	0.3	0.3	0.5	0.5	0.5
AGWRC Active Ground Water Recession Constant (day^{-1})	0.996	0.996	0.996	0.996	0.996	0.996	0.996	0.996	0.996
INFEXP Infiltration Exponent	2.0	2.0	2.0	2.0	2.0	2.0	10.0	10.0	10.0

INFILD Ratio of Maximum to Mean Infiltration	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0
BASETP Base Flow Evapotranspiration (fraction)	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
AGWETP Active Ground Water Evapotranspiration (fraction)	0.0	0.0	0.0	0.0	0.0	0.0	0.7	0.7	0.7
CEPSC Interception Storage(inches)	0.20	0.15	0.10	0.20	0.15	0.10	0.18	0.15	0.10
UZSN Upper Zone Storage Nominal (inches)	0.5	0.4	0.25	0.5	0.5	0.5	3.0	3.0	3.0
NSUR Roughness of Surface OverlandFlow Plane (Manning's n)	0.35	0.30	0.25	0.35	0.30	0.25	0.50	0.50	0.50
INTFW Interflow Index	6.0	6.0	6.0	0.0	0.0	0.0	1.0	1.0	1.0
IRC Interflow Recession Constant (day^{-1})	0.5	0.5	0.5	0.7	0.7	0.7	0.7	0.7	0.7
LZETP Lower Zone Evapotranspiration (fraction)	0.7	0.4	0.25	0.7	0.4	0.25	0.8	0.8	0.8

Table III - 2.3 Original WWHM IMPLND Parameters

IMPLND Parameters	Land Type = Impervious
LSUR Length of Surface Overland Flow Plane (feet)	400
SLSUR Slope of Surface Overland Flow Plane (feet/feet)	0.01
NSUR Roughness of Surface Overland Flow Plane (Manning's n)	0.10
RETSC Retention Storage (inches)	0.10

Hydrologic Analysis of LID and Flow Control BMPs

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.

The LID performance and flow control standards (Core Requirements #5 and #7) must be met using an approved continuous runoff model. The compliance options for the project depend on the amount of improvement proposed, the location of the project, the size of the parcel the project is on, and whether or not the project is flow control exempt. See Volume I, Sections 2.4.6 Core Requirement # 5: Onsite Stormwater Management and 2.4.8 Core Requirement #7: Flow Control, and 2.4.9 Core Requirement #8: Wetlands Protection for determining LID and flow control requirements.[1]

Hydrologic Analysis of Runoff Treatment BMPs

Sizing Runoff Treatment BMPs

Size Runoff Treatment BMPs for the entire area that drains to them, even if some of those areas are not pollution-generating.

Runoff Treatment BMPs are sized by using either a volume (the Water Quality Design Volume) or a flow rate (the Water Quality Design Flow Rate), depending on the Runoff Treatment BMP selected. Refer to the selected Runoff Treatment BMP to determine whether the BMP is sized based on a volume or a flow rate. See below for details about the Water Quality Design Volume and the Water Quality Design Flow Rate used to size Runoff Treatment BMPs.

Water Quality Design Volume

The Water Quality Design Volume may be calculated by either of the following methods:

- *Continuous Simulation Method*: Using an approved continuous runoff model, the Water Quality Design Volume shall be the simulated daily volume that represents the upper limit of the range of daily volumes that accounts for 91% of the entire runoff volume over a multi-decade period of record.

- *Single Event Hydrograph Method:* The Water Quality Design Volume shall be the volume of runoff predicted by the Natural Resources Conservation Service (NRCS) curve number equations in 2.4 Single Event Storms – Hydrograph. The precipitation depth used in the equations shall be as predicted from a 24-hour storm with a 6-month return frequency (a.k.a., 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 III-C: Rainfall Amounts and Statistics of

Ecology's 2019 SWMMWW. For other areas, interpolating between isopluvials for the 2-year, 24-hour precipitation and multiplying by 72% yields the appropriate storm size. Isopluvials for 2-year, 24-hour amounts for Western Washington are reprinted in Appendix III-B: Isopluvial Maps for Design Storms.

Water Quality Design Flow Rate

The Water Quality Design Flow Rate is dependent on the location of the Runoff Treatment BMP relative to Detention BMP(s):

- *Downstream of detention facilities*: The Water Quality Design Flow Rate shall be the full 2-year release rate from the Detention BMP .

- *Upstream of Detention BMPs or when there are no Detention BMPs*: The Water Quality Design Flow Rate at or below which 91 percent of the runoff volume, as estimated by an approved continuous runoff model, will be treated

  Ecology has assigned design criteria for Runoff Treatment BMPs to achieve the BMP's Runoff Treatment Performance Goal (e.g., Basic Treatment Performance Goal, Enhanced Treatment Performance Goal, etc.) at the Water Quality Design Flow Rate. At a minimum, 91% of the total runoff volume, as estimated by an approved continuous run- off model, must pass through Runoff Treatment BMP(s) at or below the approved hydraulic loading rate for the BMP(s).

The Water Quality Design Storm Volume and Water Quality Design Flow Rate are intended to capture and effectively treat about 90-95% of the annual runoff volume in western Washington.

Water Quality Design Flow Rate for On-Line and Off-line Runoff Treatment BMPs

Approved continuous runoff models will calculate both an "on-line" and "off-line" Water QualityDesign Flow Rate.

Off-Line Runoff Treatment BMPs

Off-line Runoff Treatment BMPs make use of a flow splitter directly upstream of the Runoff Treatment BMP to regulate the amount of flow entering the Runoff Treatment BMP. Design the flow splitter to direct flows up to and including the "off-line" Water Quality Design Flow Rate (as determined by an approved continuous runoff model) to the Runoff Treatment BMP. The Runoff Treatment BMP must be sized to treat the "off-line" Water Quality Design Flow Rate, per the individual BMP's design guidance.

If the off-line Runoff treatment BMP is preceded by an equalization basin (that is, a basin that helps attenuate flow fluctuations to the BMP), the designer may identify a lower "off-line" Water Quality Design Flow Rate. If you choose this option, you must provide a hydraulic analysis with your design documentation showing that the "off-line" Water

Quality Design Flow Rate identified will provide treatment for 91 percent of the runoff volume as estimated by an approved continuous runoff model

Ecology allows off-line designs in which the flow splitter directs flows higher than the "off-line" Water Quality Design Flow Rate to the Runoff Treatment BMP. Ecology assumes that these designs will act similarly to an "on-line" Runoff Treatment BMP, where flows higher than the "off-line" Water Quality Design Flow Rate will not achieve the full performance goal but will achieve some level of pollutant removal. If you choose this design option, you must document that the higher flows will not damage the BMP, and you may need to consider an increased maintenance frequency to accommodate the increase in pollutant accumulation within the BMP.

On-Line Runoff Treatment BMPs

On-line Runoff Treatment BMPs do not make use of a flow splitter, and receive all of the stormwater runoff from the contributing basin. On-line Runoff Treatment BMPs must be designed using the "on- line" Water Quality Design Flow Rate (as determined by an approved continuous runoff model). On-line Runoff Treatment BMPs treat flows up to the "on-line" Water Quality Design Flow Rate to meet the performance goal, and flows higher than the "on-line" Water Quality Design Flow Rate pass through the BMP at a lower percent removal. Ecology does not give Runoff Treatment credit for the higher flows that pass through the BMP at a lower percent removal.

When designing on-line Runoff Treatment BMPs, you must ensure that the higher flows will not dam-age the BMPs. If higher flows will damage the proposed Runoff Treatment BMP, you should con- sider attenuating the flows to the BMP or using an off-line Runoff Treatment BMP

Minimize Runoff Treatment BMP Size

The Core Requirement #6: Runoff Treatment requirement is to treat at least 91% of the post-development runoff, as predicted by an approved continuous runoff model. If a BMP sized to meet Core Requirement #5: Onsite Stormwater Management also qualifies as a Runoff Treatment BMP (i.e., bioretention, permeable pavement with a sand sublayer or native soils that meet the soil suitability requirement), the total amount of runoff that passes through the BMP sized to meet Core Requirement #5: Onsite Stormwater Management counts towards meeting the 91% Core Requirement #6: Runoff Treatment requirement.

When BMPs that are sized to meet Core Requirement #5: Onsite Stormwater Management (that provide Runoff Treatment) do not quite achieve the 91% Core Requirement #6: Runoff Treatment requirement, they can be upsized to meet the requirement (e.g., a larger bioretention BMP, or a deeper gravel sub- base below permeable pavement to achieve more infiltration), or an additional Runoff Treatment BMP can be located to treat additional surface runoff. However, Ecology advises against using an additional Runoff Treatment BMP that is very small.

For volume-based Runoff Treatment BMPs, the minimum recommended size is 0.0093 ac-ft. For flow-rate based Runoff Treatment BMPs, the minimum recommended design flow rate is 0.0081 cubic feet per second (cfs). Rather than construct a Runoff Treatment BMP for a volume or flow rate below these minima, Ecology recommends expanding the size of the BMP sized to meet Core Requirement #5: Onsite Stormwater Management. A second option is to build the Runoff Treatment BMP using the minimum volume or flow rate cited above.

Hydrologic Analysis of Conveyance Systems

For design of storm drainage conveyance systems, several design storms may have to be used to adequately assess the project and any downstream impact. The design of conveyance systems can be performed using the flow rates generated by an approved continuous simulation model per Section 2.1 or by one of two other methods, either the single event hydrograph method (SCS, SBUH) or the Rational Method (for small projects).

2.4 Single Event Storms – Hydrograph

Hydrograph analysis uses a plot of runoff flow versus time for a given single design storm event, allowing the key runoff characteristics like peak discharge, volume, and timing to be considered in drainage facility design. All storm event hydrograph methods require parameters that describe physical drainage basin characteristics. These parameters provide the basis of development of the runoff hydrograph. Because single event methods are only used in this manual to size conveyance systems and flow-through treatment facilities (biofiltration swales), discussion of design storms, curve numbers and peak runoff calculation is limited (see Appendix III-B).

For conveyance design, the preferred single event method is the Santa Barbara Urban Hydrograph Method or, if unavailable, the SCS Unit Hydrograph Method.

Water Quality Design Storm

As stated above (Sizing Runoff Treatment BMPs), a single event design storm may be used for determining the Water Quality Design Storm Volume as an alternative to using an approved continuous simulation model. This design storm 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 III-C: Rainfall Amounts and Statistics of Ecology’s 2019 SWMMWW. For other areas, interpolating between isopluvials for the 2-year, 24-hour precipitation and multiplying by 72% yields the appropriate storm size. Isopluvials for 2-year, 24-hour amounts for Western Washington are reprinted in Appendix III-B.

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 Appendix III-B, and the NOAA ATLAS 2, Precipitation - Frequency Atlas of the Western United States, Volume IX - Washington (Miller et al., 1973). 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).

Curve Numbers

All single event hydrograph methods require input of parameters that describe the 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 volume from the water quality design storm.

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 SCS National Engineering Handbook Section 4: Hydrology* (Rallison et al., 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 *Technical Release No. 55: Urban Hydrology for Small Watersheds* (USDA et al., 1986). 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 III - B.5: Major Soil Groups in Thurston County shows the hydrologic soil group of most soils in the county and provides a brief description of the four groups. For details on other soil types refer to *Technical Release No. 55: Urban Hydrology for Small Watersheds* (USDA et al., 1986).

Table III - B.4: Runoff Curve Numbers for Selected Agricultural, Suburban, and Urban Areas 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 Qd (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 III - B-4 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.

Calculating the Water Quality Design Storm Volume Using the NRCS Curve Number Equations

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:

$$Qd = (P - 0.2S)2/ (P + 0.8S), \text{ for } P \geq 0.2S$$

and

$$Qd = 0, \text{ for } P < 0.2S$$

Where:

Qd = runoff depth in inches over the area,

P = precipitation depth in inches over the area. For calculating the water quality design storm volume, this number is the 6-month 24-hour storm (in inches), as described in Chapter 2,

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:

$$S = (1000 /CN) - 10$$

The combination of the above equations allows for estimation of the total runoff volume by computing total runoff depth, Qd, 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:

$$Qd = [2.0 - 0.2\ (4.29)]2\ /\ [2.0 + 0.8\ (4.29)] = 0.24 \text{ inches}$$

This computed runoff represents inches over the tributary area.

Therefore, the total volume of runoff is found by multiplying Qd by the tributary area (with necessary conversions):

$$\text{Total Runoff Volume (cu. ft.)} = 3{,}630 \text{ (cu. ft./ac. in.)} \times Qd \text{ (in)} \times A \text{ (ac)}$$

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

$$3{,}630 \text{ (cu. ft./ac. in.)} \times 0.24 \text{ (in.)} \times 10 \text{ (ac.)} = 8{,}712 \text{ cu. ft.}$$

This is the Water Quality Design Storm Volume used to size volume based Runoff Treatment BMPs.

Rational Method

The rational method is a simple method used to estimate peak flows, and may be used for conveyance sizing on sites 25 acres or less in size, and having a time of concentration of less than 100 minutes. See Appendix III-B for details on the method.

2.5 Flow Bypass and Additional Area inflow

Bypassing Areas that Require Flow Control

This guidance applies to Flow Control BMPs that are not receiving flow from the entire amount of area that must be mitigated.

A portion of an area that requires a Flow Control BMP to meet Volume I, Sections 2.4.6 Core Requirement #5: Onsite Stormwater Management, 2.4.8 Core Requirement #7: Flow Control, and/or 2.4.9 Core Requirement #8: Wetlands Protection may bypass the Flow Control BMP, provided that all of the following conditions are met:

1. Runoff from both the bypass area and the Flow Control BMP converges within a quarter-mile downstream of the project site discharge location.

2. The Flow Control BMP 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 drain- age systems or properties.

5. Runoff Treatment requirements applicable to the bypass area are met.

Inflow From Areas that Don't Require Flow Control

This guidance applies to Flow Control BMPs that are receiving flow from areas in addition to the areas that must be mitigated.

Depending on site layout and topography, Flow Control BMPs may need to be positioned on a site such that runoff from areas that do not need to be mitigated are directed to the Flow Control BMP. In previous versions of the SWMMWW, this was referred to as "off-site inflow", however, these additional areas may come from on-site or off-site.

For example, a redevelopment project may need to provide Flow Control for the new hard surfaces (and not for the replaced hard surfaces), but the proposed Flow Control BMP is placed such that flow from the new AND replaced hard surfaces is directed to it. The flow from the replaced hard sur- faces would be considered additional flow to the Flow Control BMP.

Runoff from these additional areas must be modeled using the acreages associated with the existing land use areas. For the purposes of modeling in an Ecology approved continuous simulation model, these additional areas are entered under both the "Predeveloped" and "Mitigated" scenarios.
The performance of Flow Control BMPs can be compromised if the additional area, beyond the area that needs to be mitigated, is too large. Therefore, if the existing 100-year peak flow rate from the additional area is greater than 50% of the 100-year developed peak flow rate (undetained) from the area requiring mitigation, then the runoff from the additional area must not flow to the Flow Control BMP. The bypass of the additional area must be designed to achieve both of the following:

1. Any existing contribution of flows to an on-site wetland must be maintained.

2. Flows from the additional areas that are naturally attenuated by the project site under pre- developed conditions must remain attenuated, either by natural means or by providing additional on-site Flow Control BMP(s) so that peak flows do not increase.

2.6 Closed Depression Analysis

Closed depressions (potholes, kettles) represent a "dead end" for surface water flows and generally facilitate infiltration of runoff. If a closed depression is classified as a wetland or the discharge path flows through a wetland, then Core Requirement #8 for wetlands applies. If there is an outflow from this depression to a surface water (such as a creek), then the flow must also meet Core Requirement #7 for flow control.

A calibrated continuous simulation runoff 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 continuous runoff model.

Analysis and Design Criteria

The infiltration rates used in the analysis of closed depressions must be determined according to the procedures of Section 2.3. For closed depressions containing standing water, soil texture tests must be performed on dry land adjacent to, and on opposite sides of the standing water (as practicable). The elevation of the testing surface at the bottom of the test pit must be one foot above the standing water elevation. A minimum of four tests must be performed to estimate an average surface infiltration rate.

The criteria which must be met for discharge to a closed depression depend upon the location, whether the proponent has control of, or a right to discharge to the closed depression and the results of a hydrologic analysis of the closed depression.

Closed Depression Located On-Site or with a Legal Right to Discharge to Closed Depression

For a closed depression entirely on the subject property, or a closed depression to which the Proponent has acquired a legal right to discharge, analyze the closed depression using hydrologic methods described in Section 2.1. Infiltration must be addressed where appropriate. In assessing the impacts of the proposed project on the performance of the closed depression, there are two cases that dictate different approaches to meeting Core Requirement #7 – *Flow Control.*

Case 1

The 100-year recurrence interval storm runoff from an approved continuous simulation program, flowing from the TDA to the closed depression, is routed into the closed depression using only infiltration as outflow. If predevelopment runoff does not overflow the closed depression, then no runoff may leave the closed depression at the 100-year recurrence interval storm runoff following development of the proposed project. This may be accomplished by excavating additional storage volume in the closed depression, subject to all applicable requirements (for example, providing a defined overflow system).

Case 2

The 100-year recurrence interval storm runoff from an approved continuous simulation program, from the TDA to the closed depression, is routed into the closed depression using only infiltration as outflow, and overflow occurs in both the existing and the proposed conditions. The closed depression must then be analyzed as a detention/infiltration pond. The required performance, therefore, is to meet the runoff duration standard specified in Core Requirement 7 – *Flow Control*, using an adequately calibrated continuous simulation model. This will require a control structure, emergency overflow spillway, access road, and other design criteria and may require excavating additional storage volume in the closed depression. Also depending on who will maintain the system, it will require placing the closed depression in a tract dedicated to the responsible party.

Closed Depression Located Off-Site

For a closed depression shared with, or entirely on other properties, absent a legal agreement to the contrary, the peak water elevation for the 100-year recurrence interval storm runoff from an approved continuous simulation program, from the Threshold Discharge Area to the closed depression shall not cause an increase in water levels exceeding:

- 0.1 feet above the base, if available information indicates that the base is to be dry at all times, or
- 0.1 feet above the current peak water elevation, if this elevation can be clearly demonstrated.

In all cases, discharge to a closed depression shall be allowed only if the Project Engineer can satisfactorily demonstrate that no significant public health, safety, welfare, or property damage issues are present.

2.7 Site Suitability and Hydrologic Analysis of Infiltration Facilities

Infiltration is the percolation of surface water into the ground, and is an effective way to meet the flow control requirements of Core Requirement #7. While other flow control facilities, such as detention ponds, just reduce peak flow rates associated with developed areas, infiltration facilities reduce the total volume of surface runoff as well as peak flow rates. When properly sited and designed, infiltration facilities can help recharge groundwater and protect downstream receiving waters. In some cases, infiltration facilities can also be used to meet the runoff treatment requirements of Core Requirement #6.

Site Suitability and Analysis Procedures

The following procedures must be followed when considering and designing an infiltration facility. Each step is outlined in more detail in the subsequent sections. Figure III **-** 2.1 illustrates the process of analyzing and sizing infiltration facilities.

Step 1 – Conduct general site reconnaissance, and review survey and other information to identify existing drinking water wells or aquifers, designated well head protection areas for public water systems, existing and proposed buildings, steep slopes, and septic systems in the vicinity of the proposed facility.

Step 2 – Evaluate the Site Suitability Criteria (SSC) for infiltration facilities to determine whether infiltration is feasible for the site.

Step 3 – Infiltration Receptor Characterization. Estimate depth to groundwater from the bottom of proposed infiltration facility. If estimated depth to groundwater is less than 50 feet, installation of groundwater monitoring wells and characterization of the infiltration receptor will be required. If less than 6 feet to groundwater, then a mounding analysis will be required.

Step 4 – Determine whether the simplified or detailed approach will be used to establish a design infiltration rate. Consultation with Thurston County is required at this stage to obtain acceptance of the proposed method of analysis (simplified or detailed).

Step 5 – Complete simple analysis or detailed analysis, as determined in Step 4 and described in more detail below. Prepare geotechnical report.

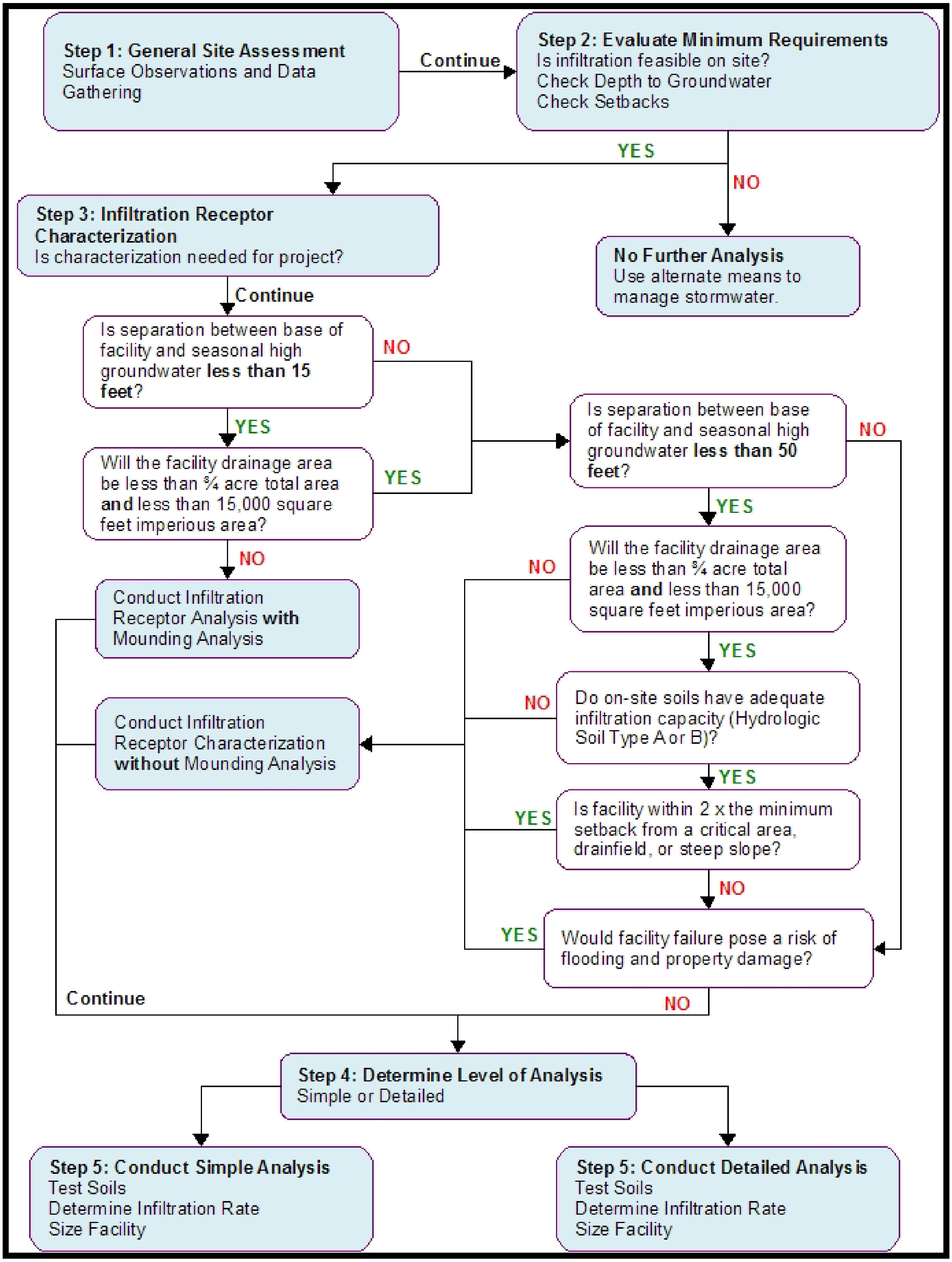

Figure III - 2.1 Infiltration Analysis and Sizing Flow Chart

Details of these five steps are provided in the sections below.

Step 1: General Site Characterization

One of the first steps in siting and designing infiltration BMPs is to conduct a characterization study that includes surface and subsurface features characterization, as described below.

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

Surface Features Characterization

The characterization study should document the following surface features:

1. Topography within 500 feet of the proposed infiltration BMP.
2. Anticipated site use (street/highway, residential, commercial, high-use site).
3. Location of water supply wells within 500 feet of proposed infiltration BMP.
4. Location of project relative to any designated well head protection areas for public water systems and/or 1-, 5-, and 10-year time of travel zones for municipal well protection areas (if available)
5. Location of steep slopes (>15%) or landslide hazard areas
6. Location of septic systems in the vicinity of the proposed facility
7. Location of areas known to have contaminated soils.
8. A description of local site geology, including soil or rock units likely to be encountered, the groundwater regime, and geologic history of the site.
9. Analysis of site borings and soil testing and review of any available existing soils information for the site or adjacent sites.
10. Analyze any existing runoff flowing into and out of the site. Speculate on possible flows generated by greater than the 100-year event. Check the proximity of other stormwater facilities on adjacent properties.
11. Location of any high groundwater hazard areas or wetlands per the Thurston County Critical Areas Ordinance, TCC Title 17 and Title 24.

Subsurface Characterization

The characterization study should document the following subsurface data:

1. Subsurface explorations (test holes or test pits) to a depth below the base of the infiltration BMP of at least 5 times the maximum design depth of ponded water proposed for the infiltration BMP, but not less than 10 feet below the base of the BMP. However, at sites with shallow groundwater (less than 15 feet from the estimated base of the infiltration BMP), if a groundwater 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 BMP of 2.5 times the maximum design ponded water depth, but not less than 10 feet. For large infiltration BMPs 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). These samples provide information on the treatment capabilities of the soils.

   The depth and number of test holes or test pits, and samples should be increased, if in the judgment of a licensed engineer in the state of Washington with geotechnical expertise (P.E.), a licensed geologist, engineering geologist, hydrogeologist, or other licensed professional acceptable to the local jurisdiction, the conditions are highly variable and such increases are necessary to accurately estimate the performance of the infiltration BMP.

2. If proposing to estimate the infiltration rate using the soil grain size analysis method (see Appendix III-A), obtain samples adequate for the purposes of that gradation/classification testing.

   - For BMP IN.01: Infiltration Basins, at least one test pit or test hole per 5,000 ft^2 of BMP infiltrating surface (in no case lower than two per BMP).

   - For BMP IN.02: Infiltration Trenches, at least one test pit or test hole per 200 feet of trench length (in no case less than two per trench).

   The depth and number of test holes or test pits, and samples should be increased, if in the judgment of a licensed engineer in the state of Washington with geotechnical expertise (P.E.), a licensed geologist, engineering geologist, hydrogeologist, or other licensed professional acceptable to the local jurisdiction, the conditions are highly variable and such increases are necessary to accurately estimate the performance of the infiltration BMP.

   The exploration program may be decreased if, in the opinion of the licensed engineer in the state of Washington or other professional, the conditions are relatively uniform, and the borings/test pits omitted will not influence the design or successful operation of the BMP.

In high water table sites, the subsurface exploration sampling need not be conducted lower than two (2) feet below the groundwater table.

3. 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.

   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 BMP.

4. Provide groundwater monitoring wells (or driven well points if there is shallow depth to groundwater) to locate the groundwater table and establish its gradient, direction of flow, and seasonal variations, considering both confined and unconfined aquifers. For infiltration BMPs with a contributing basin that is less than an acre, establish that the depth to groundwater or other hydraulic restriction layer will be at least 10 feet below the base of the BMP. Use sub-surface explorations or information from nearby wells.

   In general, a minimum of three wells per infiltration BMP, or three hydraulically connected surface or groundwater 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 groundwater levels may also be considered. If the groundwater in the area is known to be greater than 50 feet below the proposed infiltration BMP, detailed investigation of the groundwater regime is not necessary.

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

5. If using the soil Grain Size Analysis Method for estimating infiltration rates: Complete laboratory testing as necessary to establish the soil gradation characteristics and other properties, to complete the infiltration facility design. At a minimum, conduct one-grain size analysis per soil stratum in each test hole within 2.5 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 BMP, requiring soil gradation/classification testing for layers deeper than indicated above.

Soil Testing Data

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 (including type of clay, if known)
- Color/mottling
- Variations and nature of stratification

If the infiltration BMP will provide Runoff 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 BMP 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 Step 2 below)

This information, along with additional geotechnical information necessary to design the facility, shall be summarized in the geotechnical report prepared in Step 5.

Step 2: Evaluate Site Suitability Criteria (SSC) for Infiltration Facilities

Criteria that must be considered for siting infiltration BMPs is provided below. When a site investigation reveals that any of the applicable site suitability criteria cannot be met, appropriate mitigation measures must be implemented so that the infiltration BMP 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 in the state of Washington with geotechnical and hydrogeologic experience, or a licensed geologist, hydrogeologist, or engineering geologist. The designer may utilize a team of certified or registered professionals in soil science, hydrogeology, geology, and other related fields.

Setbacks

Infiltration basins may not be constructed within a floodplain area or high groundwater flood hazard area as defined in Thurston County Code, Title 17 and Title 24. Additional setbacks are summarized in Appendix V-E.

Groundwater Protection Areas

A site is not suitable for an infiltration BMP if the infiltration BMP will cause a violation of the Water Quality Standards for Groundwaters of the State of Washington (Chapter

173-200 WAC). See High Vehicle Traffic Areas through Soil Physical and Chemical Suitability for Treatment, and Cold Climate and Impact of Roadway Deicers for measures to protect groundwater quality. Thurston County staff and ordinances should be consulted for applicable pretreatment requirements if the project site is located in an aquifer sensitive area, sole source aquifer, wellhead protection area, or critical aquifer recharge area

High Vehicle Traffic Areas

An infiltration BMP may be considered for runoff from areas that require an oil control BMP per Volume I, 4.2 Step-by-Step Runoff Treatment BMP Selection. For such applications, provide the oil control BMP upstream of the infiltration BMP to ensure that groundwater quality standards will not be violated and that the infiltration BMP is not adversely affected

Soil Infiltration Rate/Drawdown Time

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

For infiltration BMPs used for Runoff Treatment purposes, the measured (initial) soil infiltration rate should be 9 in/hr or less (For BMP LID.09: Permeable Paving, this rate can be 12 in/hr 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 Soil Physical and Chemical Suitability for Treatment to adequately control the target pollutants. Project sites with infiltration rates lower than those identified in the infeasibility criteria may be used for infiltration of stormwater with prior approval from the County.

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

Drawdown Time

For infiltration BMPs designed strictly for Flow Control purposes, there isn’t a maximum drawdown time.

For infiltration BMPs designed to provide Runoff Treatment, document that the Water Quality Design Volume (as described in 2.3 Sizing Your Runoff Treatment BMPs) can infiltrate through the infiltration BMP surface within 48 hours. This can be calculated by multiplying the horizontal projection of the infiltration BMP 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 infiltration BMP size. If the design fails the check procedure, redesign the infiltration BMP.

Depth to Bedrock, Water Table, or Impermeable Layer

The base of BMP IN.01: Infiltration Basins and BMP IN.02: Infiltration Trenches shall be ≥ 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 groundwater 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 other site suitability criteria specified in this section.

Soil Physical and Chemical Suitability for Treatment

This SSC applies to infiltration BMPs that intend to use the native soil to provide Runoff Treatment. If the native soils do not meet the criteria below, Runoff Treatment must be provided prior to infiltration either by a layer within the infiltration BMP (such as is the case for BMP LID.08: Bioretention), a Runoff Treatment BMP upstream of the infiltration BMP, or by a layer of engineered soil that meets the criteria below. Refer to Chapter 3 – Infiltration BMPs for guidance to determine the appropriate level of Runoff Treatment, based on land use and project type, that is necessary to precede the infiltration BMP.

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 ≥ 5 milliequivalents CEC/100 g dry soil (USEPA, 1986). 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 Runoff Treatment must be a minimum of 18 inches. Depth of soil used for infiltration Runoff Treatment below BMP LID 09: Permeable Paving that is a pollution-generating hard surface 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 these design criteria. Field performance evaluation(s), using protocols cited in this manual, would be needed to determine feasibility and acceptability by the County.

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.

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 groundwater quality standards.

Step 3: Infiltration Receptor Characterization

An Infiltration receptor characterization consists of monitoring and analysis of groundwater, and (in some cases) a mounding analysis. This characterization must be conducted if any of the following conditions are present:

- Proposed facility would pose a risk of flooding or property damage if failure were to occur.
- Separation between base of facility and seasonal high groundwater is less than 50 feet AND tributary drainage area contains more than 15,000 square feet impervious surface or ¾ acre total area.
- Separation between base of facility and seasonal high groundwater is less than 50 feet AND on-site soils may not have adequate infiltration capacity (Hydrologic Soil Group C or D [till soils]).
- Separation between base of facility and seasonal high groundwater is less than 50 feet AND there is less than 2 times the minimum setback to a critical area, drainfield, or steep slope (>15%).

In addition, mounding analysis must be conducted if BOTH of the following conditions are present:

- Separation between base of facility and seasonal high groundwater is less than 15 feet, AND
- Tributary drainage area is greater than 3/4 acre or there is greater than 15,000 square feet of impervious surface contributing to the facility.

A mounding analysis may also be required by the Administrator for conditions other than those listed above if any of the following conditions are present:

- Hydrologic Soil Group C or D soils with an estimated infiltration rate of less than 0.5 inches/hour.
- The potential impact to downstream properties and/or critical areas is high as a result of a facility failure.
- Urban environment (> 4 units per acre).
- Facility is within 100-feet of a steep slope (>15%) with soils having less than a 1 inch/hour infiltration rate.
- When soils work indicates there may be a perched low permeability layer above the water table.

An exemption from the mounding analysis may be granted if the geotechnical professional can demonstrate to the satisfaction of the Administrator that it is not necessary. This demonstration shall be based on site specific information that in the judgment of the geotechnical professional mitigates against the requirement to conduct a mounding analysis. Examples of circumstances that the Administrator will consider in granting an exemption include:

- Soils are classified as outwash with an estimated design infiltration rate of greater than 5 in/hr.
- Soils are uniform and easily characterized as outwash. Risk of low permeability lenses is low.
- Site topography, etc. indicates no substantial risk to slopes, wetlands, structures etc. in the event groundwater breaches the surface.
- Other studies of groundwater mounding for the same or adjacent sites indicate that mounding would not be a concern.

If it is determined that an Infiltration Receptor Characterization is not required for a project, continue to Step 4.

Monitor Groundwater Levels

A minimum of three groundwater monitoring wells shall be installed per infiltration facility that will establish a three-dimensional relationship for the groundwater table. Seasonal groundwater levels must be monitored at the site through at least one wet season (December 1 through April 30). Where longer term groundwater monitoring information is available, normalize the single wet season observations to historic groundwater records in the region.

Monitoring wells shall be installed and monitored in accordance with the following requirements:

- Well shall be screened across the water table.
- Maximum screen and sand pack length of 15 feet.
- Weekly water level monitoring resulting in a minimum of 16 measurements over 4 months.

Document Characterization

A geotechnical report will be developed in Step 5. This report shall include the following information to characterize the infiltration receptor (unsaturated and saturated soil receiving the stormwater):

- The information obtained from groundwater monitoring in #4 of the Subsurface Characterization above.
- Depth to groundwater and to bedrock/impermeable layers.
- Seasonal variation of groundwater table based on well water levels and observed mottling of soils. Provide an estimated seasonal high groundwater level and an estimated maximum high groundwater level taking into account historical and seasonal groundwater table fluctuations.
- Existing groundwater flow direction and gradient.
- An estimate of the volumetric water holding capacity of the infiltration receptor soils. The volumetric water holding capacity is the storage volume in the soil layer directly below the infiltration facility and above the seasonal high groundwater mark, bedrock, hardpan, or other low permeability layer. Conduct this analysis at a conservatively high infiltration rate based on vadose zone porosity, and the Water Quality Design Volume to be infiltrated. This, along with an analysis of groundwater movement, will be useful in determining if there are volumetric limitations that would adversely affect drawdown, and if a groundwater mounding analysis should be conducted.
- Consider the potential for both unconfined and confined aquifers, or confining units, at the site that may influence the proposed infiltration facility as well as the groundwater gradient.
- An assessment of the ambient groundwater quality, if that is a concern.
- Horizontal hydraulic conductivity of the saturated zone to assess the aquifer’s ability to laterally transport the infiltrated water.

- Approximation of the lateral extent of infiltration receptor.
- Impact of the infiltration rate and proposed added volume from the project site on local groundwater mounding, flow direction, and water table; and the discharge point or area of the infiltrating water determined by hydrogeologic methods.
- Location of the project within the Salmon Creek Basin requires specific groundwater characterization elements be met and reference to the Salmon Creek Basin Plan and Interim Site Development Standards for New Development in Salmon Creek Basin should be referred to for specific requirements.
- State whether location is suitable for infiltration and recommend a method for estimating the design infiltration rate (simple or detailed, in-situ or gradation based).

Mounding Analysis

If a mounding analysis is required, the geotechnical professional shall develop an approach and obtain its acceptance from Thurston County prior to initiating the study. Simple, conservative methods of estimating groundwater mounding are available and may be acceptable with the use of conservative parameters to demonstrate that risks from groundwater mounding are acceptable. The methodology, approach, software program, input data, calibration requirements and output format for the mounding analysis shall be proposed by the geotechnical professional in the geotechnical report for acceptance by Thurston County.

The purpose of the mounding analysis is to identify the impact of groundwater mounding on the estimated design infiltration rate, the seasonal high groundwater elevation at the property boundary and at any on-site or off-site structures, critical areas, or other site features that might be impacted by groundwater mounding.

The results of the mounding analysis will be reported by the geotechnical professional as part of the Infiltration Receptor Characterization and shall include the following determinations:

- A minimum separation of at least 3-feet to seasonal high groundwater will be maintained from the bottom of the facility with mounding.
- There will be no breakout of groundwater to the surface in the vicinity of the project as a result of mounding.
- That a minimum separation to groundwater from the estimated lowest elevation of any basement, building foundation, road, or other structure will be at least 3-feet.

- That there will be no intrusion of the groundwater mound into any existing or proposed drainfield or reserve area and that there will be no greater than a 6-inch increase in groundwater elevation beneath any septic drainfield or reserve area as a result of groundwater mounding.

- That the increase in groundwater elevation at the property boundaries of the project will not result in impacts to adjacent property owners. Generally demonstrating that the increase in groundwater level at the property boundary is less than 1-foot due to mounding would meet this criterion unless there are special circumstances.

Step 4: Determine Method of Analysis

Thurston County requires consideration of infiltration facilities for sites where conditions are appropriate. Some sites may not be appropriate for infiltration due to soil characteristics, groundwater levels, steep slopes, or other constraints.

The design infiltration rate for a proposed infiltration BMP shall be calculated based on either the Simple Method or Detailed Method as described in this section.

Simplified Approach

The simplified approach was derived from high groundwater 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 BMP and for small BMPs serving short plats or commercial developments with less than one acre of contributing area. Designs of infiltration BMPs for larger projects should use the detailed approach (as described below) and may have to incorporate the results of a groundwater mounding analysis as described above. Note: A groundwater mounding analysis is advisable for BMPs 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.

Detailed Approach

The detailed approach of analysis is more suitable when it is unclear if a site is well-suited to infiltration and in cases where failure of an infiltration facility would create a high risk of flooding and/or property damage. The detailed method of analysis, described below, includes more intensive field testing and soils investigation and analyses than the simplified approach and takes into account the depth to groundwater. Sites that have **ANY** of the following conditions should be considered for use of the detailed method:

- Low infiltration capacity soils (NRCS [SCS] soil types C or D)

- History of unsuccessful infiltration facility performance, or no history of successful infiltration performance at nearby locations

- A large contributing drainage area (greater than 1-acre)
- Shallow groundwater levels (Less than 50 feet to seasonal high groundwater)
- High risk of flooding and property damage in the event of clogging or other failure.

The County may allow the simplified approach in circumstances that might warrant the detailed approach if it is demonstrated that the infiltration facility could be converted to a detention facility of adequate size if the infiltration facility were to fail.

Step 5: Conduct Simple or Detailed Analysis

Based on the results of Step 3 and 4, conduct a simple analysis or a detailed analysis as described below .

Determine Design Infiltration Rate

The Simplified Approach to Calculating the Design Infiltration Rate of the Native Soils

Using the simplified approach, estimate the design (long-term) infiltration rate as follows:

- Use any of the three options described in Appendix III-A to estimate the initial $K_{sat.}$
- Assume that the K_{sat} is the measured (initial infiltration rate for the native soils.
- Determine the design infiltration rate by adjusting the initial infiltration rate using the appropriate correction factors, as detailed below.

  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 III - 4.1: Correction Factors to be Used With In-Situ Saturated Hydraulic Conductivity Measurements to Estimate Design Rates 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 in the state of Washington 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.

  - **Site variability and number of locations tested (CF_v)** – The number of locations tested must be capable of producing a picture of the subsurface conditions that fully represents the conditions throughout the proposed location of the infiltration BMP. 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 (CF_t)** accounts for uncertainties in the testing methods. For the full scale PIT method, CFt = 0.75; for the small-scale PIT method, CFt = 0.50; for smaller-scale infiltration tests such as the double-ring infiltrometer test, CFt = 0.40; for grain size analysis, CFt = 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 BMP 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 BMP is infiltrating at only 90% of its design capacity. Therefore, a correction factor, CF_m, of 0.9 is called for.

Table III - 4.1 Correction Factors to be Used With In-Situ Saturated Hydraulic Conductivity Measurements to Estimate Design Rates (source: Ecology)

Issue	Partial Correction Factor
Site variability and number of locations tested	CF_V = 0.33 to 1.0
Test Method	
• Large-scale PIT	• CF_t = 0.75

• Small-scale PIT • Other small-scale (e.g. Double ring, falling head) • Grain Size Method	• = 0.50 • = 0.40 • = 0.40
Degree of influent control to prevent siltation and bio-buildup	CF_m = 0.9

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

- The design infiltration rate (K_{sat}design) is calculated by multiplying the initial Ksat by the total correction factor:

$$K_{sat}\text{ design} = K_{sat}\text{ initial X } CF_T$$

The Detailed Approach to Calculating the Design Infiltration Rate of the Native Soils

This detailed approach was obtained from Massmann (2003).

Using the detailed approach, estimate the design (long-term) infiltration rate as follows:

1. Use any of the options listed in Appendix III-A to estimate the initial K_{sat}.
2. Calculate the steady state hydraulic gradient as follows:

$$\text{Gradient} = i = \frac{D_{wt} + D_{pond}}{138.62(K^{0.1})} \times CF_{size}$$

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

Where:

D_{wt} is the depth from the base of the infiltration facility to the water table in feet

K is the saturated hydraulic conductivity in feet/day

D_{pond} is the depth of water in the facility in feet (see Massmann et al. 2003, for the development of this equation)

CF_{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. For small ponds (ponds with area less than or equal to 2/3 acre), the correction factor is equal to 1.0. For large ponds (ponds with area greater than or equal to 6 acres), the correction factor is 0.2, as shown below

$$CF_{size} = 0.73(A_{pond})^{-0.76}$$

Where:

A_{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 groundwater depths (or to a low permeability layer) below the BMP, and conservatively accounts for the development of a groundwater mound. A more detailed groundwater 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 the equation above. 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 groundwater 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, no Ecology approved continuous runoff models 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 ¼ of the maximum ponded depth – as measured from the basin floor to the overflow.

3. Calculate the preliminary design infiltration rate using Darcy's law as follows:

$$f = K\left(\frac{dh}{dz}\right) = Ki$$

Where:

f is the specific discharge or infiltration rate of water through a unit cross-section of the infiltration facility (L/t)

K is the hydraulic conductivity (L/t)

dh/dz (= "*i*") is the hydraulic gradient (L/L)

4. Adjust the preliminary design infiltration rate to determine the design (long term) infiltration rate:

This step adjusts the preliminary design infiltration rate (as determined in Step 3 above) for the effect of pond aspect ratio by multiplying the preliminary design infiltration rate by the aspect ratio correction factor Faspect as shown in the following equation:

CFaspect = 0.02Ar + 0.98

Where:

Ar is the aspect ratio for the pond (length/width of the bottom area). In no case shall CFaspect be greater than 1.4.

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

final design (long-term) infiltration rate = Ksat x i x CFaspect

General Design Criteria for Infiltration BMPs

Design Criteria – Sizing Infiltration BMPs

- The size of the infiltration BMP can be determined using a continuous runoff model by routing the inflow runoff file through the proposed infiltration BMP.

To prevent the onset of anaerobic conditions, an infiltration BMP designed for Runoff Treatment purposes (either by a layer within the infiltration BMP, as in BMP LID.08: Bioretention, or by treatment through native soils that meet the criteria for Runoff Treatment per the Site Suitability Criteria (SSC)) must be designed to drain the Water Quality Design Volume within 48 hours (see explanation under Soil Infiltration Rate/Drawdown Time).

In general, an infiltration facility would have two 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 BMP must comply with the performance standard they are designed to meet - typically either the LID Performance Standard within Volume I, Core Requirement #5: On-Site Stormwater Management and/or the Flow Control Performance Standard within Core Requirement #7: Flow Control. Infiltration BMPs used for Runoff Treatment must not overflow more than 9% of the influent runoff file.

In order to determine compliance with the LID Performance Standard and/or the Flow Control Performance Standard, use an approved continuous runoff model. When using the continuous runoff model for simulating flow through an infiltrating BMP, represent the BMP by using the appropriate element within the software (pond, trench, permeable pavement, or bioretention), and entering the pre-determined infiltration rates. Below are the procedures for sizing an infiltration BMP to:

- Completely infiltrate 100% of the runoff,
- Treat 90% of runoff to meet the Runoff Treatment requirements, and
- Partially infiltrate runoff to meet the LID Performance Standard and/or the Flow Control Performance Standard.

Sizing an Infiltration BMP For 100 Percent Infiltration

1. Input dimensions of your infiltration pond.
2. Input infiltration rate and safety (rate reduction) factor.
    - When the native soil infiltration rate was calculated using the Simplified Approach (as described above), you may enter the measured (initial) saturated hydraulic conductivity (K_{sat}) as the infiltration rate and the Total Correction Factor as the safety factor, OR,

      Enter the estimated final design infiltration rate after application of the Total Correction Factor, and a safety factor of 1.
    - When the native soil infiltration rate was calculated using the Detailed Approach (above) you should enter the aspect ratio for the pond, as calculated in #4, as the safety factor in the model input.
3. Input a riser height and diameter (any flow through the riser indicates that you have less than 100 percent infiltration and must increase your infiltration pond dimensions).
4. Run the model only for Developed Mitigated Scenario (if that is where you put the infiltration BMP).
5. After running the model, go back to your infiltration BMP and look at the Percentage Infiltrated (this is at the bottom right if using WWHM). If less than 100 percent infiltrated, increase the BMP dimension until you get 100 percent infiltrated.

Sizing an Infiltration BMP to Infiltration 91% of the Runoff (The Water Quality Design Volume)

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

Infiltration BMPs for Runoff Treatment can be located upstream or downstream of detention, and can be off-line or on-line.

Refer to 2.3 Hydrologic Analysis of Runoff Treatment BMPs for more information about the flows that must be treated for on-line and off-line Runoff Treatment BMPs. For infiltration BMPs serving as Runoff Treatment BMPs, the designer must use continuous runoff modeling software to show that all of the applicable flow is treated by passing through the infiltration BMP.

Sizing an Infiltration BMP to Meet LID and/or Flow Control Performance Standards

This design will allow something less than 100% infiltration as long as any overflows will meet the applicable performance standard. Use a discharge structure with orifices and risers similar to a detention BMP, and include infiltration occurring from the infiltration BMP.

Treatment Prior to Infiltration BMPs

Pretreatment Prior to Infiltration BMPs

A pretreatment BMP to remove a portion of the influent suspended solids should precede all infiltration BMPs. This is to reduce potential plugging of the soils and prolong the life of the infiltration BMP. Use either a basic treatment BMP, as described in Volume I, or a pretreatment BMP as described by BMP WP.05: Presettling Basins & Pretreatment. The lower the influent suspended solids loading to the infiltration BMP, the longer the infiltration BMP can infiltrate the desired amount of water, and the longer interval between maintenance activity.

In BMPs such as BMP IN.02: Infiltration Trenches where a reduction in infiltration capability can have significant maintenance or replacement costs, selection of a reliable pretreatment or basic treatment BMP prior to the infiltration BMP with high solids removal capability is preferred. For infiltration BMPs that allow easier access for maintenance and less costly maintenance activity (e.g., BMP IN.01: Infiltration Basins with gentle side slopes), there is a trade-off between using a pretreatment or basic treatment BMP with a higher solids removal capability and a device with a lower capability. Generally, basic treatment BMPs are more capable at solids removal than pretreatment BMPs. Though basic treatment BMPs may be higher in initial cost and space demands, the infiltration BMP should have lower maintenance costs.

Runoff Treatment Prior to Infiltration BMPs

In an effort protect groundwater, projects must apply the appropriate level of Runoff Treatment whenever infiltration is proposed. The appropriate level of Runoff Treatment varies by land use and project type, and is determined by one of the following methods:

- If the project is required to meet Core Requirement #6: Runoff Treatment, use the guidance in Volume I, 4.2 Step-by-Step Runoff Treatment BMP Selection Process to determine the appropriate level of Runoff Treatment prior to infiltration.

- If the project is installing a UIC well, use the guidance in I-4 UIC Program from Ecology's 2019 SWMMM to determine the appropriate level of Runoff Treatment prior to infiltration.

- If the conditions below the infiltration BMP meet the criteria for Runoff Treatment per the Site Suitability Criteria (SSC), this will satisfy the Runoff Treatment requirements for both the Core Requirement #6: Runoff Treatment and Ecology's I-4 UIC Program.

- If the project is proposing infiltration, but is not required to meet Core Requirement #6: Runoff Treatment or follow the guidance in Ecology's I-4 UIC Program, the designer has the following options to determine the appropriate level of Runoff Treatment:
    - Follow the guidance in Volume I, 4.2 Step-by-Step Runoff Treatment BMP Selection Process
    - Follow the guidance in Ecology's I-4 UIC Program
    - Provide another protective measure consistent with all applicable regulations. See Volume IV, Chapter 7 – Regulations and Requirements for some of the regulations and standards that may apply to the project.
- Infiltration or dispersion BMPs that are only used to meet the List Approach in Core Requirement #5: Onsite Stormwater Management do not require additional Runoff Treatment prior to infiltration.

Chapter 3 - Conveyance Systems and Hydraulic Structures

3.1 Overview

This chapter presents acceptable methods for analysis and design of conveyance systems. It also discusses hydraulic structures linking the conveyance system to runoff treatment and flow control facilities. The chapter is organized as follows:

- Design and analysis methods (Sections 3.2 through 3.6)
- Pipe systems (Section 3.7)
- Outfalls (Section 3.8)
- Flow spreaders (Section 3.9)
- Culverts (Section 3.10)
- Open conveyances (Section 3.11)
- Private Drainage Systems (Section 3.12)
- Floodplains/floodways (covered in TCC 17.15 and TCC 24).

Where space and topography permit, open conveyances are the preferred means of collecting and conveying stormwater.

3.2 Design Event Storm Frequency

Ideally, every conveyance system and hydraulic structure would be designed for the largest possible amount of flow. Since this would require unusually large structures and be too costly, hydraulic structure designs are analyzed using a specific storm frequency. When selecting a storm frequency, consideration is given to potential adjacent property damage, potential hazard and inconvenience to the public, the number of users, and initial construction cost of the conveyance system or hydraulic structure.

The design event recurrence interval is related to the probability that such an event will occur in any one-year period. For example, a peak flow having a 25-year recurrence interval has a 4 percent probability of being equaled or exceeded in any future year (100/25 = 4). A peak flow having a 2-year recurrence interval has a 50 percent probability of being equaled or exceeded in any future year (100/2 = 50). The greater the recurrence interval, the lower the probability that the event will occur in any given year.

Conveyance systems shall be designed to convey the peak flows from the following storm events:

- The project's internal piped conveyance system shall be designed for a 25-year, 24-hour storm event. In areas where the County determines there is a high risk of damage or vital service interruption, a backwater analysis of the peak flows from the 100-year, 24-hour storm events shall be conducted.
- All open channel conveyance systems shall be designed for the 100-year, 24-hour storm event.
- Piped conveyance under public roads and arterials shall convey a 25-year, 24-hour storm event under fully developed basin conditions. Additional criteria:
    - In the urban area inside of the long-term urban growth management boundary (boundary is depicted on current zoning maps available at the County) the outside driving lane of public roads and streets must not have water over more than 50 percent of the lane for a design event of a 25-year, 24-hour storm.
    - In the area outside of the long-term urban growth management boundary, the design event shall be the 100-year, 24-hour storm.
    - In areas where the County determines there is a high risk of damage or vital service interruption (e.g., more than 6 inches of standing water in the streets), the Administrator or designee may specify up to the 100-year, 24-hour event as the design event.
- Natural channel bridges and culverts shall be designed to convey at least the 100-year, 24-hour storm event under fully developed drainage basin conditions based on the tributary area zoning. Culvert and bridge designs must also meet applicable fish passage and scour criteria.

3.3 Determination of Design Flows

All existing and proposed conveyance systems shall be analyzed and designed using peak flows from hydrographs developed through single event storm hydrologic analyses described in Section 2.1.3 or from a continuous simulation hydrologic model using 15 minute time steps. See Chapter 2 and Appendix III-B for more information.

EXCEPTION: For drainage subbasins 25 acres or less, and having a time of concentration of less than 100 minutes, peak flows for analyzing the capacity of conveyance elements may be determined using the Rational Method (see Chapter 2 and Appendix III-B).

3.4 Open Channel Flow – Hydraulic Analysis

Two hydraulic analysis methods are used to analyze and design conveyance systems:

- The Uniform Flow Analysis Method (Section 3.4.1 below), commonly referred to as the Manning's equation, is used for the design of open conveyances (Section 3.10) and new pipe systems (Section 3.7), as well as for analysis of existing pipe systems. Manning’s equation is only valid for pipe flow when the pipe is flowing less than full. If the pipe is surcharged, the backwater method must be used.
- The Backwater Analysis Method (Section 3.4.2 below), is used to analyze the capacity of both proposed and existing pipe systems when a pipe is surcharged. If the County determines that, as a result of the project, runoff for any event up to and including the 100-year, 24-hour event would exceed the pipes’ un-surcharged capacity, a backwater (pressure sewer) analysis shall be required. Results shall be submitted in tabular and graphic format showing hydraulic and energy gradient.

Uniform Flow Analysis - Manning’s Equation

Manning’s equation can be used for open channel flow or for a pipe that is flowing less than full. Manning’s equation is expressed as:

$$V = \frac{1.486}{n} \times R^{0.67} \times S^{0.5}$$

Where:

V = velocity (feet per second),

n = Manning’s roughness factor (-)

R = hydraulic radius (area/wetted perimeter; feet), and

S = Channel slope (feet/foot)

Manning’s equation can also be expressed in terms of discharge (Q):

$$Q = \frac{1.486}{n} \times A \times R^{0.67} \times S^{0.5}$$

Where *A* = cross-sectional area of flow (square feet).

Manning's roughness factors (n) for open channels are shown in Table III - 3.1, and for piped conveyances in Table III - 3.2. A more extensive table of Manning's roughness factors can be found in Table III - B.3 in Appendix III-B.

Table III - 3.1 Manning's Roughness Factors for Open Channel Conveyances

Channel Lining	Manning's Roughness Factor (n)
Concrete	0.012
Short grass	0.030
Stony bottom and weedy grass	0.035
Cobble bottom and grass banks	0.040
Dense weeds as high as flow	0.080
Dense woody brush as high as flow	0.120
Biofiltration swale	see Volume V

Table III - 3.2 Manning's Roughness Factors for Pipe Conveyances

	Analysis Method	
Type of Pipe Material	**Backwater Flow**	**Manning's Equation Flow** [a]
A. Concrete pipe	0.013	0.015
B. Annular Corrugated Metal Pipe or Pipe Arch:		
1. 2-2/3" x 1/2" corrugation (riveted)	0.024	0.028
2. 3" x 1" corrugation	0.027	0.031
3. 6" x 2" corrugation (field bolted)	0.030	0.035
C. Helical 2-2/3" x 1/2" corrugation	0.024	0.028
D. Spiral rib metal pipe	0.016	0.018
E. Ductile iron pipe cement lined	0.013	0.015
F. Plastic	0.010	0.012

[a] The roughness values for this method are 15 percent higher in order to account for entrance, exit, junction, and bend head losses

Backwater Analysis

When a backwater calculation is required for a pipe conveyance, the design engineer shall analyze for the 100-year, 24-hour design storm event against the following criteria:

- For the 100-year event, overtopping of the pipe conveyance system may occur; however, the additional flow shall not extend beyond half the lane width of the outside lane of the traveled way and shall not exceed 4 inches in depth at its deepest point.

- Off-channel storage on private property is allowed with recording of the proper easements (see Section 3.6). The additional flow shall be analyzed by open channel flow methods.

A backwater profile analysis computer program such as the King County Backwater (KCBW) computer program prepared by the King County Department of Natural Resources and Parks, Water and Land Resources Division is recommended over manual calculations. The BPIPE subroutine of KCBW may be used for quick computation of backwater profiles, given a range of flows through the existing or proposed pipe system. This program is available free of charge from King County.

3.5 Conveyance System Route Design and Off-Site Drainage

All pipe shall be located under the pavement flow line or lie outside of the pavement. Perpendicular crossings and cul-de-sacs are exempted from this requirement. New conveyance system alignments that are not in dedicated tracts or right-of-way shall be located in drainage easements that are adjacent and parallel to property lines. The width of the permanent easement will be completely within a single parcel or tract. Topography and existing conditions are the only conditions under which a drainage easement that is not adjacent and parallel to a property line may be placed. Requirements for conveyance system tracts and easements are discussed in Section 3.6.

EXCEPTION: Streams and natural drainage channels cannot be relocated to meet this routing requirement.

Development projects are required to handle off-site drainage in the same manner as exists in the predeveloped condition. In other words, after development of the subject site, off-site flows shall be infiltrated within or passed through the project site in the same proportion as occurred prior to development. The area and existing use of the off-site land area should be included in any modeling performed to design new facilities. If the adjacent site is undeveloped, model the off-site land area as if it were developed with a detention facility discharging per the Core Requirements of this manual and factor the future flow into the design of the facilities. To avoid this analysis, it would be preferable to collect and bypass off-site drainage around the site or infiltrate it prior to the flow being combined with on-site drainage. If the off-site drainage is to be infiltrated on site, the infiltration facilities shall be sized to accommodate the correct proportion of off-site flows.

Off-site pass-through flows shall be routed separately across the development site. They shall not be routed through the project’s conveyance, runoff treatment, or flow control systems. Storage and treatment of off-site pass-through flows is not required.

However, if the Project Engineer and the Administrator or designee agree that separate handling of off-site flows is impracticable, then off-site flows may be routed through the project’s stormwater management systems. Those systems affected by the off-site flows shall be sized as if the off-site flows were generated within the development project’s boundaries.

3.6 Easements, Access, and Dedicated Tracts

All man-made drainage facilities and conveyances, and all natural channels (on the project site) used for conveyance of altered flows due to development shall be located within easements or dedicated tracts as required by the County. Easements shall contain the natural features and facilities and shall allow County access for purposes of inspection, maintenance, repair or replacement, flood control, water quality monitoring, and other activities permitted by law.

The easement shall include easement boundary markers which shall be fiberglass utility markers with a reflective easement tag, located at each corner of the easement, at angle points and at least every 100-ft along the length of the easement. Contact Thurston County Water Resources Division for additional information on easement marker requirements.

Maintenance Access to Stormwater Facilities

All drainage facilities such as detention or wet ponds or infiltration systems whether privately maintained or maintained by the County shall be located in separate tracts. Conveyance systems and dedicated stormwater dispersion areas can be in easements with County acceptance.

The dedicated tract for a stormwater facility shall include a minimum 20-foot wide access from a public street or right-of-way. If the development is served by private roads or is gated, then the Proponent shall provide for County access through the gate or private roads to access stormwater facilities. This may include providing a pass code to the Administrator or other means acceptable to the County.

An easement shall be granted through the tract for access to the stormwater facility and shall not be included as part of any individual lots within a subdivision. Access easements across individual lots for access to a stormwater facility are discouraged and shall only be allowed with specific acceptance of Thurston County (including the Administrator or designee) and only upon demonstration that measures are in place to ensure that the easement will not be encroached upon by the lot owner.

The access shall be surfaced with a minimum 12-foot width of crushed rock or other approved surface to allow year-round equipment access to the facility and delineated by a gate, fencing or some other measure to indicate to adjacent property owners that an easement exists. See individual BMP descriptions in Volume V for additional stormwater facility access requirements.

Drainage facilities that are designed to function as multi-use recreational facilities shall be located in separate tracts or in designated open space and shall be privately maintained and owned, unless accepted by and dedicated to the County.

Maintenance vehicle access, i.e., vactor truck, must be provided for all manholes, catch basins, vaults, or other underground drainage facilities. Maintenance shall be through an access easement (see requirements above) or dedicated tract. Drainage structures for conveyance, other than open channels, must have vehicular access.

Access to Conveyance Systems

All publicly and privately maintained conveyance systems shall be located in dedicated tracts, drainage easements, or public rights-of-way in accordance with this manual. Exception: Roof downspout, minor yard, and footing drains unless they serve other adjacent properties.

Conveyance systems to be maintained and operated by Thurston County must be located in a dedicated tract or drainage easement granted to the County. Any new conveyance system on private property conveying drainage from other private properties must be located in a dedicated tract or private drainage easement granted to the stormwater contributors.

Any easement for access to a conveyance system shall include measures to ensure that the easement will not be encroached upon by adjacent lot owners such as delineation by a gate, fencing, signage or some other measure to indicate to adjacent property owners that an easement exists.

All drainage tracts and easements must have a minimum width of 20 feet. All pipes and channels must be located within the easement in accordance with Table III - 3.3. If circumstances require the location of the pipe or channel within the easement to differ from the requirements of Table III - 3.3, then, at a minimum each pipe face or top channel edge shall be no closer than 5 feet from its adjacent easement boundary. Easements or Tract widths shown in Table III - 3.3 are minimums for drainage facilities and may be increased depending on pipe/channel size, depth or other factors.

Table III - 3.3 Minimum Easement Widths for Conveyance Systems for Access, Inspection and Maintenance

Conveyance Width	Easement/Tract Width
Channels ≤ 30 feet wide	Channel Width + 20 feet from top, one side
Channels > 30 feet wide	Channel Width + 20 feet from top, both sides
Pipes/Outfalls ≤ 36 inches	20 feet centered on pipe
Pipes/Outfalls ≤ 60 inches	20 feet centered on pipe*
Pipes/Outfalls > 60 inches	30 feet centered on pipe*

* May be greater, depending on depth and number of pipes in easement.

Discharge to Private Property

When the proposed project site discharges to an adjacent property where no public drainage facility or no defined drainage course exists (e.g., a natural channel such as a Department of Natural Resources (DNR) Type “Ns” rated stream), the Proponent shall obtain an easement from the adjacent property owner(s) to establish a drainage way to connect to a defined drainage system. In the absence of such an easement, the discharge from stormwater management facilities shall be distributed along the property line in approximately the same flow pattern as before development. A quantitative downstream analysis shall be conducted to determine any potential impacts of the distributed flow to downstream property.

The Administrator or designee may, under highly unusual circumstances, excuse the Proponent from requirements of this section (e.g., adjacent property is a wetland and is not a closed basin, and discharge to the wetland would not significantly alter the hydrology, degrade wetland functions and values, or reduce the value of the property).

3.7 Pipe System Design Criteria

Pipe systems are networks of storm drain pipes, catch basins, manholes, and inlets designed and constructed to convey storm and surface water. The hydraulic design of new storm drain pipes is limited to gravity flow; however, in analyzing existing systems, it may be necessary to address pressurized conditions.

Analysis Methods

Two methods of hydraulic analysis (using Manning's Equation) are used for pipe system analysis (see Section 3.4):

- Uniform Flow Analysis Method (Section 3.4.1), commonly referred to as the Manning's Equation.
- Backwater Analysis Method (Section 3.4.2).

When using the Manning's Equation for design, each pipe within the system shall be sized and sloped so that its barrel capacity at normal full flow is equal or greater than the required conveyance capacity as identified in Section 3.2. Pipes should not be designed to surcharge.

Nomographs may also be used for sizing the pipes. For pipes flowing partially full, the actual velocity may be estimated from engineering nomographs by calculating Q_{full} and V_{full} and using the ratio of Q_{design}/Q_{full} to find V and d (depth of flow). Appendix III-C includes several nomographs that may be useful for culvert sizing.

Acceptable Pipe Sizes

Storm drainage pipe are subject to the following minimum diameters:

- Private drainage system ≥8 inches for pipes other than French drains, foundation drains and downspout drains. See the Uniform Plumbing Code for minimum sizes and cleanout locations for other pipes such as French drains and downspout pipes,
- Public right-of-way = 12 inches

The Administrator or designee may waive these minimums in cases where topography and existing drainage systems make it impractical to meet the standard. For culverts, see Section 3.10.

Pipe Materials

All storm drainage pipe, except as otherwise provided for in these standards, shall be as per current WSDOT Standard Specifications 9-05. When extreme slope conditions or other unusual topographic conditions exist, pipe materials and methods such as, but not limited to, PVC, HDPE, or ductile iron pipe should be used. See the WSDOT Hydraulics Manual for minimum and maximum depth of cover criteria.

Pipe Slope and Velocity

Minimum velocity is 2 feet per second at design flow. The County may waive these minimums when topography and existing drainage systems make it impractical.

Maximum slopes, velocities, and anchor spacings are shown in Table III - 3.4. If velocities exceed 15 feet per second for the conveyance system design event, provide anchors at bends and junctions.

Table III - 3.4 Maximum Pipe Slopes and Velocities

Pipe Material	Pipe Slope Above Which Pipe Anchors Required	Max. Slope Allowed	Max. Velocity @ Full Flow
PVC(1), CPEP-single wall(1) Corrugated Metal Pipe(1)	20% (1 anchor per 100 LF of pipe)	30% (3)	30 fps
Concrete(1) or CPEP-smooth interior(1)	10% (1 anchor per 50 LF of pipe)	20% (3)	30 fps
Ductile Iron (4)	40% (1 anchor per pipe section)	None	None
HDPE (2)	50% (1 anchor per 100 LF of pipe – cross slope installations may be allowed with additional anchoring and analysis)	None	None

NOTES:

(1) Not allowed in landslide hazard areas.

(2) Butt-fused pipe joints required. Above ground installation is required on slopes greater than 40% to minimize disturbance to steep slopes.

(3) Maximum slope of 20% allowed for these pipe materials with no joints (one section) if structures are provided at each end and the pipes are properly grouted or otherwise restrained to the structures.

(4) Restrained joints required on slopes greater than 25%. Above-ground installation is required on slopes greater than 40% to minimize disturbance to steep slopes:

KEY:

PVC = Polyvinyl chloride pipe
HDPE = High density polyethylene
fps = Feet per second

Downsizing of pipes is only allowed under special conditions (i.e. no hydraulic jump can occur; downstream pipe slope is significantly greater than the upstream slope; velocities remain in the 3 to 8 feet per second range, etc.).

Downsizing of downstream culverts within a closed system with culverts 18 inches in diameter or smaller will not be permitted.

Pipes on Steep Slopes

Steep slopes (greater than 30 percent) shall require all drainage to be piped from the top to the bottom in HDPE pipe (butt fused) or ductile iron pipe welded or mechanically restrained. Pipes may be installed in trenches with standard bedding on slopes up to 20 percent. In order to minimize disturbance to slopes greater than 20 percent, it is recommended that pipes be placed at grade with proper pipe anchorage and support. If slopes exceed 40 percent, then pipe shall be installed above ground and anchored (see Table III - 3.4). Additional anchoring design may be required for these pipes.

Pipe System Layout Criteria

Pipes must be laid true to line and grade with no curves, bends, or deflections in any direction (except for HDPE and ductile iron with flanged restrained mechanical joint bends, not greater than 30°, on steep slopes).

A break in grade or alignment or changes in pipe material shall occur only at catch basins or manholes.

Connections to a pipe system shall be made only at catch basins or manholes. No wyes or tees are allowed except on private roof/footing/yard drain systems on pipes 8 inches in diameter, or less, with clean-outs upstream of each wye or tee.

Provide 6 inches minimum vertical and 3 feet minimum horizontal clearance (outside surfaces) between storm drain pipes and other utility pipes and conduits. Development Standards for Water and Sewer Systems, Thurston County will apply for crossings of or parallel runs with Thurston County sewer lines and for crossings of water lines. Additional requirements for crossings of septic transport lines or water supply lines may apply. Contact the Thurston County Environmental Health Division or the local water purveyor for these requirements. Contact the Environmental Health Division of the Thurston County Department of Public Health and Social Services at 360-867-2673 for more information.

Suitable pipe cover over storm pipes in road rights-of-way shall be calculated for HS-20 loading by the Project Engineer. Pipe cover is measured from the finished grade elevation to the top of the outside surface of the pipe. Pipe manufacturer recommendations are acceptable, if verified by the Project Engineer.

Except as indicated above, pipes or conveyances that traverse the marine intertidal zone and connect to outfalls should be buried at a depth sufficient to avoid exposure of the line during storm events or future changes in beach elevation. If non-native material is used to bed the pipe, such material should be covered with at least 3 feet of native bed material or equivalent

PVC SDR 35 minimum cover shall be 3 feet in areas subject to vehicular traffic; maximum cover shall be 30 feet or per the manufacturer's recommendations and as verified with calculations from the Project Engineer.

Pipe cover in areas not subject to vehicular loads, such as landscape planters and yards, may be reduced to a 1 foot minimum.

Access barriers are required on all pipes 18 inches and larger exiting a closed pipe system. Debris barriers (trash racks) are required on all pipes entering a pipe system.

Where a minimal fall is necessary between inlet and outlet pipes in a structure, pipes must be aligned vertically by one of the following in order of preference:

- Match pipe crowns
- Match 80 percent diameters of pipes
- Match pipe inverts

Where inlet pipes are higher than outlet pipes, drop manhole connections may be required or increased durability in the structure floor may be required.

High Density Polyethylene (HDPE) pipe systems longer than 100 feet must be anchored at the upstream end if the slope exceeds 25 percent and the downstream end placed in a minimum 4 foot long section of the next larger pipe size. This sliding sleeve connection allows for the high thermal expansion/contraction coefficient of the pipe material. These sleeve connections should be located as close to the discharge end of the outfall system as is practical.

Note that all new storm drain pipelines 8-inches in diameter and greater shall be closed-circuit television (CCTV) inspected and air pressure tested (APT) by the developer, contractor, or applicant prior to final project acceptance. See Appendix I-H: Closed-Circuit Television Inspection and Air Pressure Test in Volume I of this Manual for specific requirements.

Pipe Structure Criteria

Catch Basins and Manholes

All catch basins and manholes shall meet current WSDOT Standard Specifications and Plans. The following criteria shall be used when designing a conveyance system which uses catch basins or manholes.

Unless otherwise required by the County, Type 1 catch basins shall be used at the following locations or for the following situations:

- When overall structure height does not exceed 8 feet, or when invert does not exceed 5 feet.
- When pipe sizes do not exceed 18 inches and connect at right angles to the long side of the structure; or 12 inches connecting to the short side.
- When all pipes tying into the structure connect at or very near to right angles.

Unless otherwise required by the County, Type 1L catch basins must be used at the following locations or for the following situations:

- When overall structure height does not exceed 8 feet or when invert does not exceed 5 feet.
- When any pipes tying into the structure exceed 18 inches connecting to the long side, or 15 inches connecting to the short side at or very near to right angles.

Unless otherwise required by the County, Type 2 (48-inch minimum diameter) catch basins shall be used at the following locations or for the following situations:

- When overall structure height does not exceed 15 feet.
- When all pipes tying into the structure do not exceed the limits set forth by the manufacturers. Type 2 catch basins over 4 feet in height shall have standard ladders. Ladders shall not cover inlet or outlet pipes.

Where an approved connection of a private storm drainage system into a County system occurs, a minimum of a Type 1 catch basin shall be used in Thurston County.

Maximum spacing on main storm sewers between access structures, whether catch basins or manholes, shall be 300 feet (Table III - 3.5).

Table III - 3.5 Maximum Surface Runs Between Inlet Structures on the Paved Roadway Surface in Thurston County

Roadway Slope (%)	Thurston County Max. Spacing (ft)
0.5 to 1.0	150
1.0 to 3.0	200
>3.0	300

Catch basin (or manhole) diameter shall be determined by pipe diameter and orientation at the junction structure. A plan view of the junction structure, drawn to scale, is required when more than four pipes enter the structure on the same plane, or if angles of approach and clearance between pipes is of concern. The plan view (and sections if necessary) must insure a minimum distance (of solid concrete wall) between pipe openings of 8 inches for 48-inch and 54-inch diameter catch basins and 12 inches for 72-inch and 96-inch diameter catch basins.

Catch basin evaluation of structural integrity for H-20 loading will be required for multiple junction catch basins and other structures which exceed the recommendations of the manufacturers.

The WSDOT Hydraulics Manual can be used to determine inlet grate capacity when capacity is of concern. When verifying capacity, assume grate areas on slopes are 80 percent free of debris, and “vaned” grates are 95 percent free. In sags or low spots, assume grates are 50 percent free of debris, and “vaned” grates are 75 percent free.

The maximum slope of the ground surface shall be 3:1 for a radius of 5 feet around a catch basin grate.

Catch basin and manhole frames installed in the curb shall not exceed 2 percent.

Concrete collars shall be installed around cleanouts and manholes in paved areas, or areas to be paved.

When connecting PVC pipe to a manhole or catch basin with knockouts, a coupling (sand collar) shall be used.

Catch basins shall be provided within 50 feet of the entrance to a pipe system to provide for silt and debris removal.

Maximum spacing of structures for storm drainage conveyance lines running within an easement area shall be 300 feet for pipe grades greater than 0.3 percent and 200 feet for grades less than 0.3 percent. Structures not acting as points of entry for stormwater shall have locking lids and have solid covers.

Locking lids shall be installed on all drainage structures not located within a traveled roadway or sidewalk, and structures containing restrictor or flow control devices. Locking lids shall use WSDOT Standard Plan B-30.70-01 with the lettering of “STORM” or other county pre-approved design.

A metal frame and grate for catch basin and inlet, WSDOT Standard Plan B-30.10 and B-30.30-01or pre-approved county standard grate that is deemed bicycle safe, shall be used for all structures collecting drainage from the paved roadway surface.

When the road profile equals or exceeds 6 percent between structures, install combination inlet frame, hood, and directional grate.

Table III - 3.6 presents the allowable structures and pipe sizes allowed by size of structure. All catch basins, inlets, etc., shall be marked as shown in Volume IV, Figure IV - 4.24.

Table III - 3.6 Allowable Structure and Pipe Sizes

	Maximum Pipe Diameter	
Catch Basin Type (1)	Spiral Rib CPEP, HDPE, PVC (2) (Inches)	Concrete and Ductile Iron (Inches)
Inlet (4)	12	12
Type 1 (3)	15	15
Type IL (3)	18	18
Type 2-48-inch dia.	30	24
Type 2-54-inch dia.	36	30
Type 2-72-inch dia.	54	48
Type 2-96-inch dia.	72	72

(1) Catch basins, including manhole steps, ladder, and handholds shall conform to the WSDOT Standard Plans or an approved equal based upon submittal for approval.

(2) Maintain the minimum side wall thickness per WSDOT standards.

(3) Maximum 5 vertical feet allowed between grate and invert elevation.

(4) Normally allowed only for use in privately maintained drainage systems and must discharge to a catch basin immediately downstream.

NOTE: The applicant shall check with the County to determine the allowable pipe materials.

Flow Splitter Designs

Many runoff treatment facilities can be designed as flow-through or on-line systems with flows above the water quality design flow or volume simply passing through the facility at a lower pollutant removal efficiency. However, it is sometimes desirable to restrict flows to runoff treatment facilities and bypass the remaining higher flows around them through off-line facilities. This can be accomplished by splitting flows in excess of the water quality design flow upstream of the facility and diverting higher flows to a bypass pipe or channel. The bypass typically enters a detention pond or the downstream receiving drainage system, depending on flow control requirements. In most cases, it is a designer's choice whether runoff treatment facilities are designed as on-line or off-line; an exception is oil/water separators, which must be designed off-line.

A crucial factor in designing flow splitters is to ensure that low flows are delivered to the treatment facility up to the water quality design flow rate. Above this rate, additional flows are diverted to the bypass system with minimal increase in head at the flow splitter structure to avoid surcharging the runoff treatment facility under high flow conditions. Flow splitters may be used for purposes other than diverting flows to runoff treatment facilities. However, the following discussion is generally focused on using flow splitters in association with runoff treatment facilities.

Flow splitters are typically manholes or vaults with concrete baffles. In place of baffles, the splitter mechanism may be a half tee section with a solid top and an orifice in the bottom of the tee section. A full tee option may also be used as described below in the "General Design Criteria." Two possible design options for flow splitters are shown in Figure III - 3.1 and Figure III - 3.2. Other equivalent designs that achieve the result of splitting low flows and diverting higher flows around the facility are also acceptable.

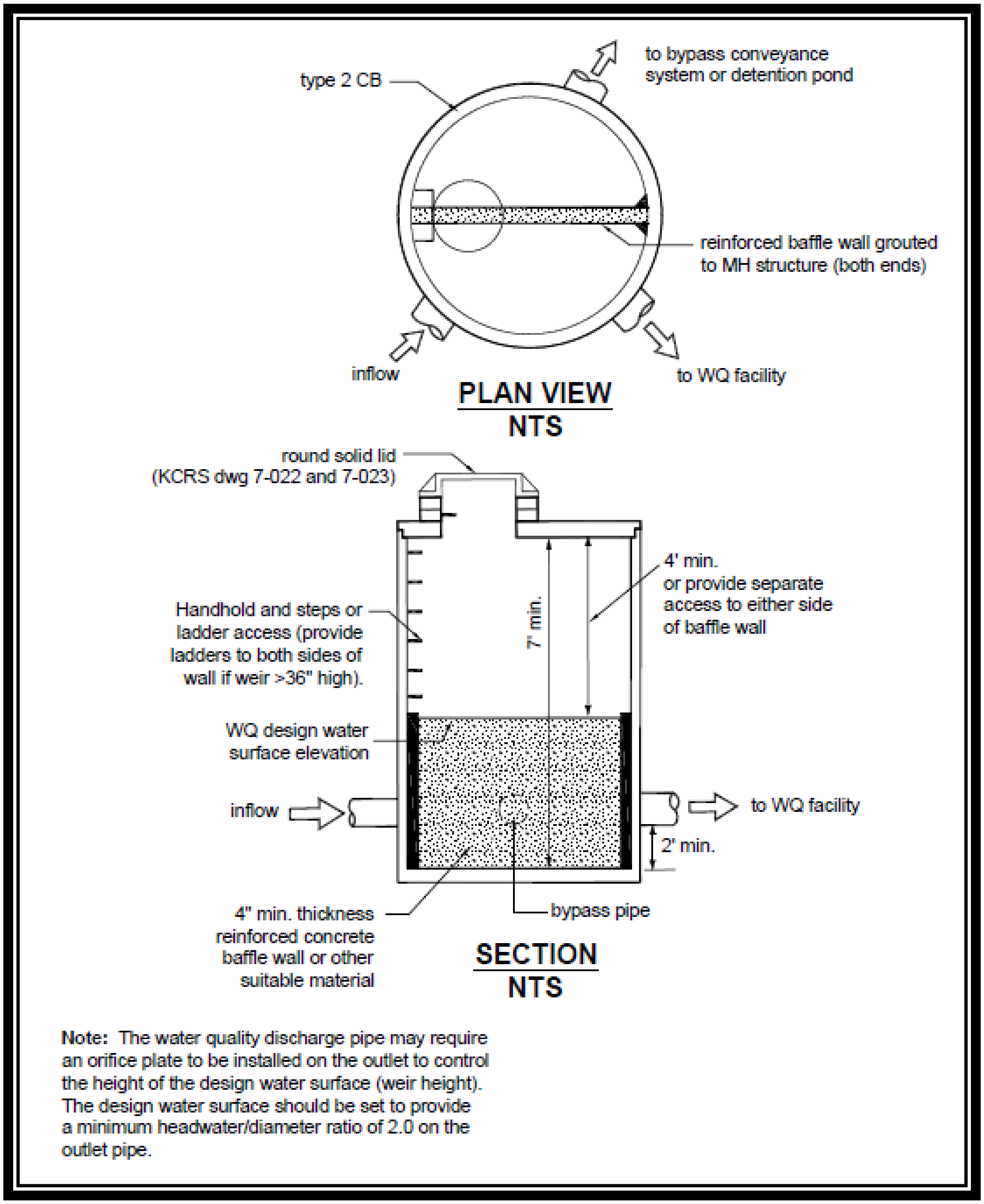

Figure III - 3.1 Flow Splitter, Option A. (Source, King County Surface Water Design Manual)

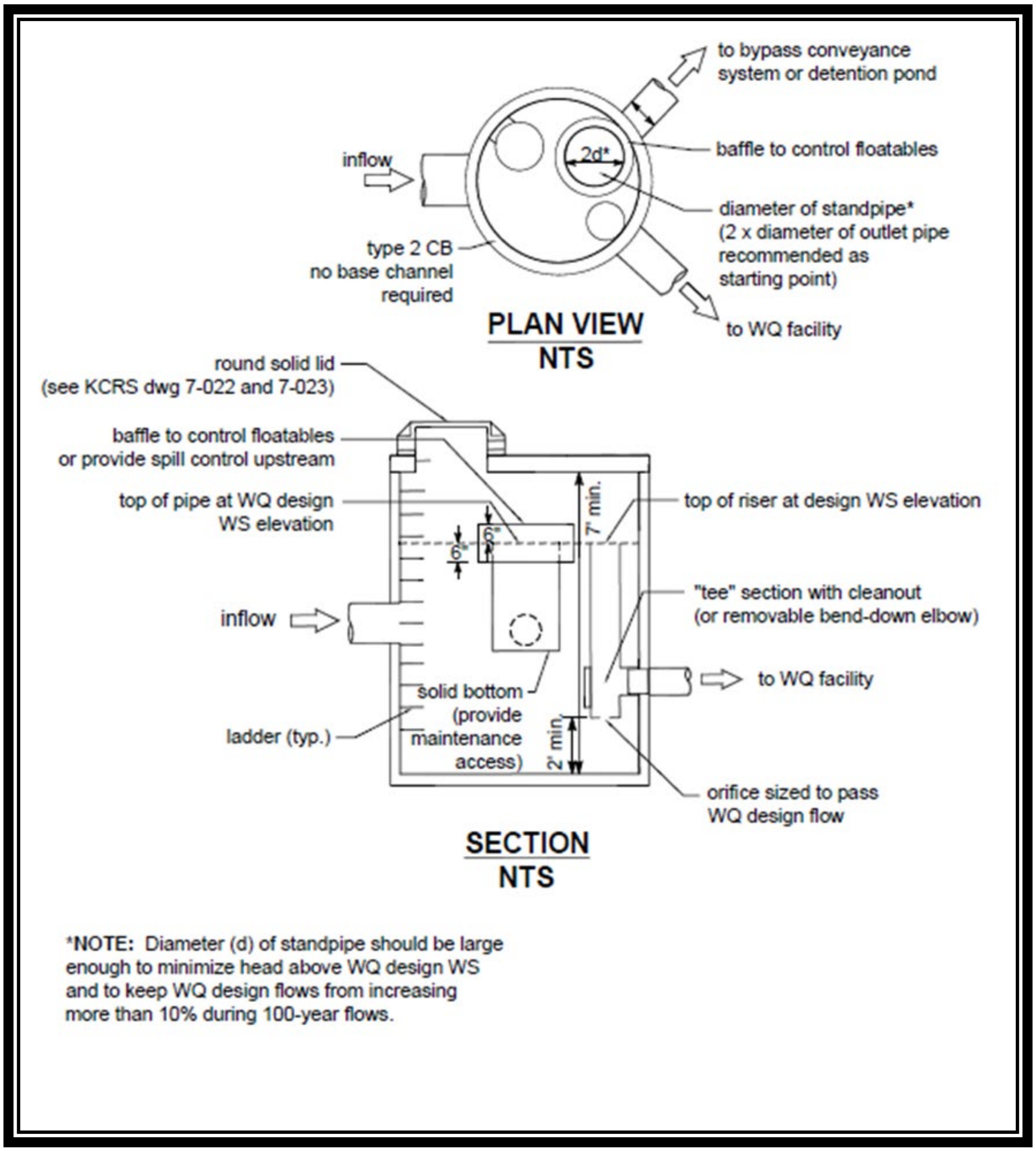

Figure III - 3.2 Flow Splitter, Option B. (Source, King County Surface Water Design Manual)

General Design Recommendations

- Unless otherwise specified, a flow splitter should be designed to deliver the water quality design flow rate specified to the runoff treatment facility. Flows modeled using a continuous simulation runoff model shall use 15-minute time steps.

- The top of the weir should be located at the water surface for the design flow. Remaining flows enter the bypass line.

- The maximum head should be minimized for flow in excess of the water quality design flow. Specifically, flow to the runoff treatment facility at the 100-year water surface should not increase the water quality design flow by more than 10 percent.

- Either design shown in Figure III - 3.1 and Figure III - 3.2 or an equivalent design may be used.

- As an alternative to using a solid top plate in Figure III - 3.2, a full tee section may be used with the top of the tee at the 100-year water surface. This alternative would route emergency overflows (if the overflow pipe were plugged) through the runoff treatment facility rather than back up from the manhole.

- Special applications, such as roads, may require the use of a modified flow splitter. The baffle wall may be fitted with a notch and adjustable weir plate to proportion runoff volumes other than high flows.

- For ponding facilities, back water effects must be included in designing the height of the standpipe in the manhole.

- Ladder or step and handhold access must be provided. If the weir wall is higher than 36 inches, two ladders, one to either side of the wall, should be used.

Materials

- The splitter baffle may be installed in a Type 2 manhole or vault.

- The baffle wall should be made of reinforced concrete or another suitable material resistant to corrosion, and have a minimum 4-inch thickness. The minimum clearance between the top of the baffle wall and the bottom of the manhole cover should be 4 feet; otherwise, dual access points shall be provided.

- All metal parts must be corrosion resistant. Examples of preferred materials include aluminum, stainless steel, and plastic. Zinc and galvanized materials are discouraged because of aquatic toxicity. Painted metal parts should not be used because of poor longevity.

3.8 Outfalls

All piped discharges to streams, rivers, ponds, lakes, or other open bodies of water are designated outfalls and shall provide for energy dissipation to prevent erosion at or near the point of discharge. Properly designed outfalls are critical to reducing the risk of adverse impacts of concentrated discharges from on-site and downstream pipe systems and culverts. Outfall systems include rock splash pads, flow dispersal trenches, gabion or other energy dissipaters, and tightline systems. A tightline system is typically a continuous length of pipe used to convey flows down a steep or sensitive slope with appropriate energy dissipation at the discharge end.

Outfalls to streams, wetlands, or other waters of the State may be subject to review through the SEPA process, Shorelines Management Act, Thurston County Critical Areas Ordinance requirements and other applicable regulations, as well as subject to state or federal requirements including hydraulic and permitting requirements of the Washington State Department of Fish and Wildlife, Army Corps of Engineers or Washington State Department of Natural Resources. The requirements of these other reviews and permitting processes shall take precedence where more restrictive than those stated herein.

General Design Criteria for Outfall Features

Outfalls shall be designed to pass the peak flow from the design event for conveyances (Section 3.2) and to suffer no structural damage or undercutting during the 100-year, 24-hour storm event. The Project Engineer shall present calculations showing the velocity, discharge, and flow path of the 100-year, 24-hour event. For outfalls downstream of a flow control BMP, the unmitigated 100-year, 24-hour event flow shall be used.

The standard for outfall design is as shown in Figure III - 3.3. This design is limited to slopes of 2:1 or flatter where native vegetation is well established or where slope armoring is engineered to the Administrator or designee's satisfaction. For sites where the Project Engineer determines, and the Administrator or designee agrees, that the standard is impractical because of lack of space, danger of erosion, etc., alternate outfall designs shown in Figures III - 3.6 and 3.7 may be used. Other outfall designs will be allowed upon acceptance of the Administrator or designee.

See Table III - 3.8 for a summary of the rock protection requirements at outfalls.

Table III - 3.8. Rock Protection at Outfalls

Discharge Velocity at Design Flow in feet per second (fps)	Required Protection				
	Minimum Dimensions				
	Type	Thickness	Width	Length	Height
0 – 5	Rock lining (1)	1 foot	Diameter + 6 feet	8 feet *or* 4 x diameter, whichever is greater	Crown + 1 foot
5^+ - 10	Riprap (2)	2 feet	Diameter + 6 feet *or* 3 x diameter, whichever is greater	12 feet *or* 4 x diameter, whichever is greater	Crown + 1 foot
10^+ - 20	Gabion	As required	As required	As required	Crown + 1 foot
20^+	Engineered energy dissipater required				

Footnotes:

(1) **Rock lining** shall be quarry spalls with gradation as follows:

Passing 8-inch square sieve: 100%
Passing 3-inch square sieve: 40 to 60% maximum
Passing ¾-inch square sieve: 0 to 10% maximum

(2) **Riprap** shall be reasonably well graded with gradation as follows:

Maximum stone size: 24 inches (nominal diameter)
Median stone size: 16 inches
Minimum stone size: 4 inches

Note: Riprap sizing governed by side slopes on outlet channel is assumed to be approximately 3:1.

Outfalls with flow velocity under 12 feet per second and discharge under 2 cfs for the conveyance system design event (Section 3.2) are to be provided (at minimum) with a splash pad (e.g., rock, gabions, concrete).

Outfalls where flow is 2 cfs or greater or velocity is 20 feet per second or greater for the conveyance system design event (Section 3.2), an engineered energy dissipater is required. Examples are stilling basins, drop pools, hydraulic jump pools, baffled aprons, bubble up structures, etc.

Outfalls must be protected against undercutting. Also consider scour, sedimentation, anchor damage, etc. Pipe and fittings materials shall be corrosion resistant such as aluminum, plastic, fiberglass, high density polyethylene, etc. Galvanized or coated steel will not be acceptable.

Outfalls on Steep Slopes

Outfall pipes on steep slopes (refer to Table III - 3.4) must be anchored and must be fused or butt-welded or mechanically restrained. They may not be gasketed, slip fit, or banded.

On steep slopes, High Density Polyethylene (HDP) pipe may be laid on the surface or in a shallow trench, anchored, protected against sluicing, and hand compacted.

HDP outfall systems must be designed to address the material limitations as specified by the manufacturer, in particular thermal expansion and contraction. The coefficient of thermal expansion and contraction for HDP is on the order of 0.001-inch per foot per Fahrenheit degree. Sliding connections to address this thermal expansion and contraction must be located as close to the discharge end of the outfall system as is practical.

HDP systems longer than 100 feet must be secured at the upstream end and the downstream end placed in a four-foot section of the next larger pipe size. This sliding sleeve connection allows for high thermal expansion/contraction.

HDP shall comply with the requirements of Type III C5P34 as tabulated in ASTM D1248 and have the PPI recommended designation of PE3408 and have an ASTM D3350 cell classification of 345434C or 345534C. The pipe shall have a manufacturer's recommended hydrostatic design stress rating of 800 psi based on a material with a 1,600 psi design basis determined in accordance with ASTM D2837-69. The pipe shall have a suggested design working pressure of 50 psi at 73.4 degrees F and SDR of 32.5.

Outfall Pipe Energy Dissipation

Outfall pipes that discharge directly into a channel or water body shall be provided at a minimum with a rock splash pad (Figure III - 3.3). See Table III - 3.8 for minimum rock protection at outfalls.

Due to HDP pipe's ability to transmit flows of very high energy, special consideration for energy dissipation must be made. A sample gabion mattress energy dissipater for this purpose has been provided as Figure III - 3.6. This mechanism may not be adequate to address flows of very high energy: therefore, a more engineered energy dissipater structure as described above, may be warranted.

Mechanisms which reduce velocity prior to discharge from an outfall are encouraged. Examples are drop manholes and rapid expansion into pipes of much larger diameter.

The following sections provide general design criteria for various types of Outfall Features.

General Design Criteria to Protect Aquatic Species and Habitat

Outfall structures should be located where they minimize impacts to fish, shellfish, and their habitats. However, new pipe outfalls are also opportunities for low-cost fish habitat improvements. For example, an alcove of low-velocity water can be created by constructing the pipe outfall and energy dissipater back from the stream edge and digging a channel, over-widened to the upstream side, from the outfall to the stream (as shown in Figure III - 3.8). Overwintering juvenile and migrating adult salmonids may use the alcove as shelter during high flows. Potential habitat improvements should be discussed with the Washington Department of Fish and Wildlife area habitat biologist prior to inclusion in design.

Bank stabilization, bioengineering, and habitat features may be required for disturbed areas. Outfalls that discharge to the Puget Sound or a major waterbody may require tide gates. For more information see the Thurston County Critical Areas Ordinance at http://www.co.thurston.wa.us/planning/critical_areas/criticalareas_home.htm and the Shoreline Master Program at http://www.co.thurston.wa.us/planning/shoreline/shoreline_qa.htm. For design guidance see the Washington Department of Fish and Wildlife Marine Shoreline Design Guidelines at http://wdfw.wa.gov/publications/01583/ or the Integrated Streambank Protection Guidelines at http://wdfw.wa.gov/publications/00046/.

Flow Dispersal Trench

The flow dispersal trenches shown in Figure III - 3.4 and Figure III - 3.5 should only be used when an outfall is necessary to disperse concentrated flows across uplands where no conveyance system exists, and the natural (existing) discharge is unconcentrated. The 100-year peak discharge rate per dispersal trench shall be less than or equal to 0.5 cfs. Other flow dispersal BMPs are described in Volume V.

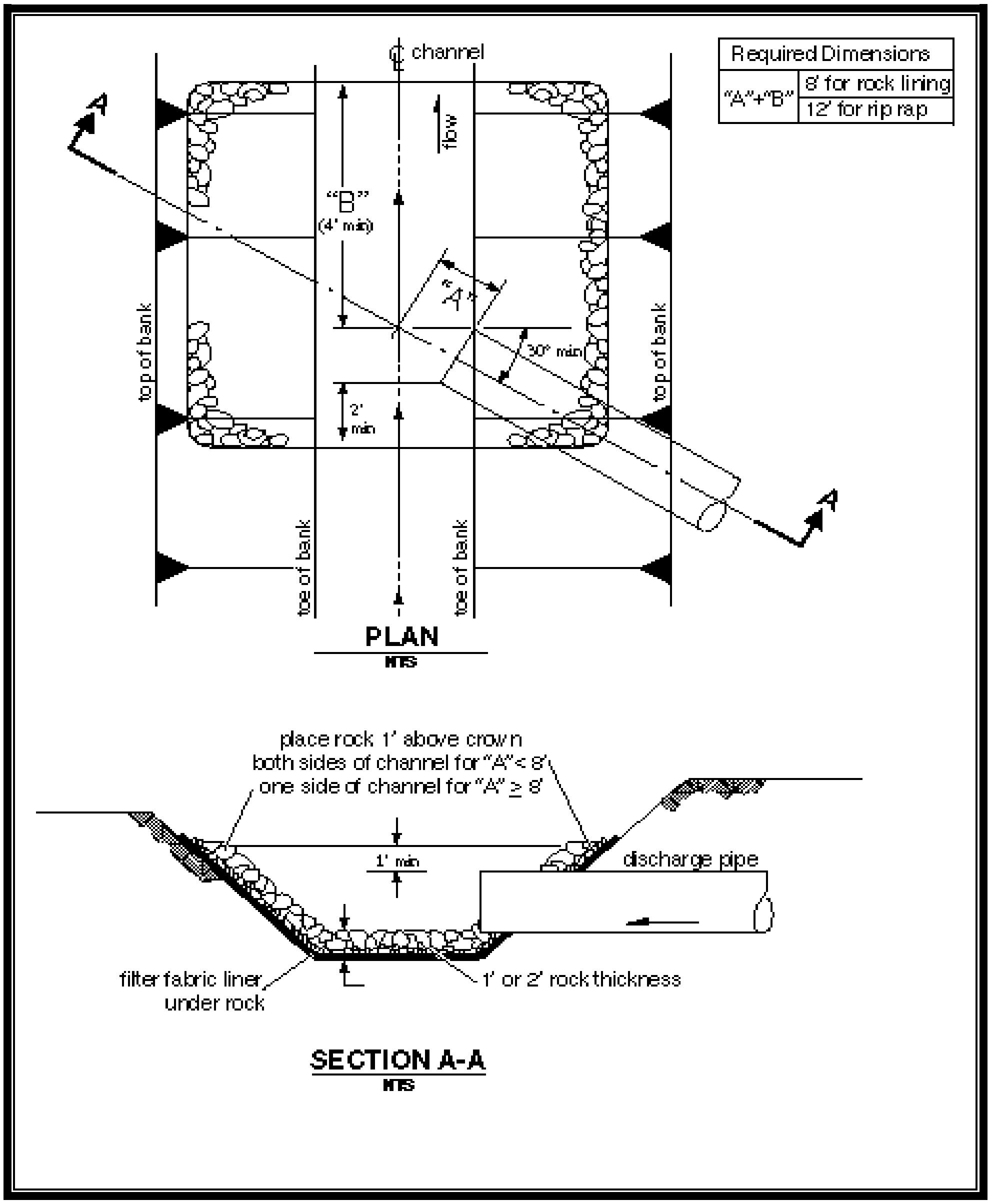

Figure III - 3.3 Pipe/Culvert Outfall Discharge Protection

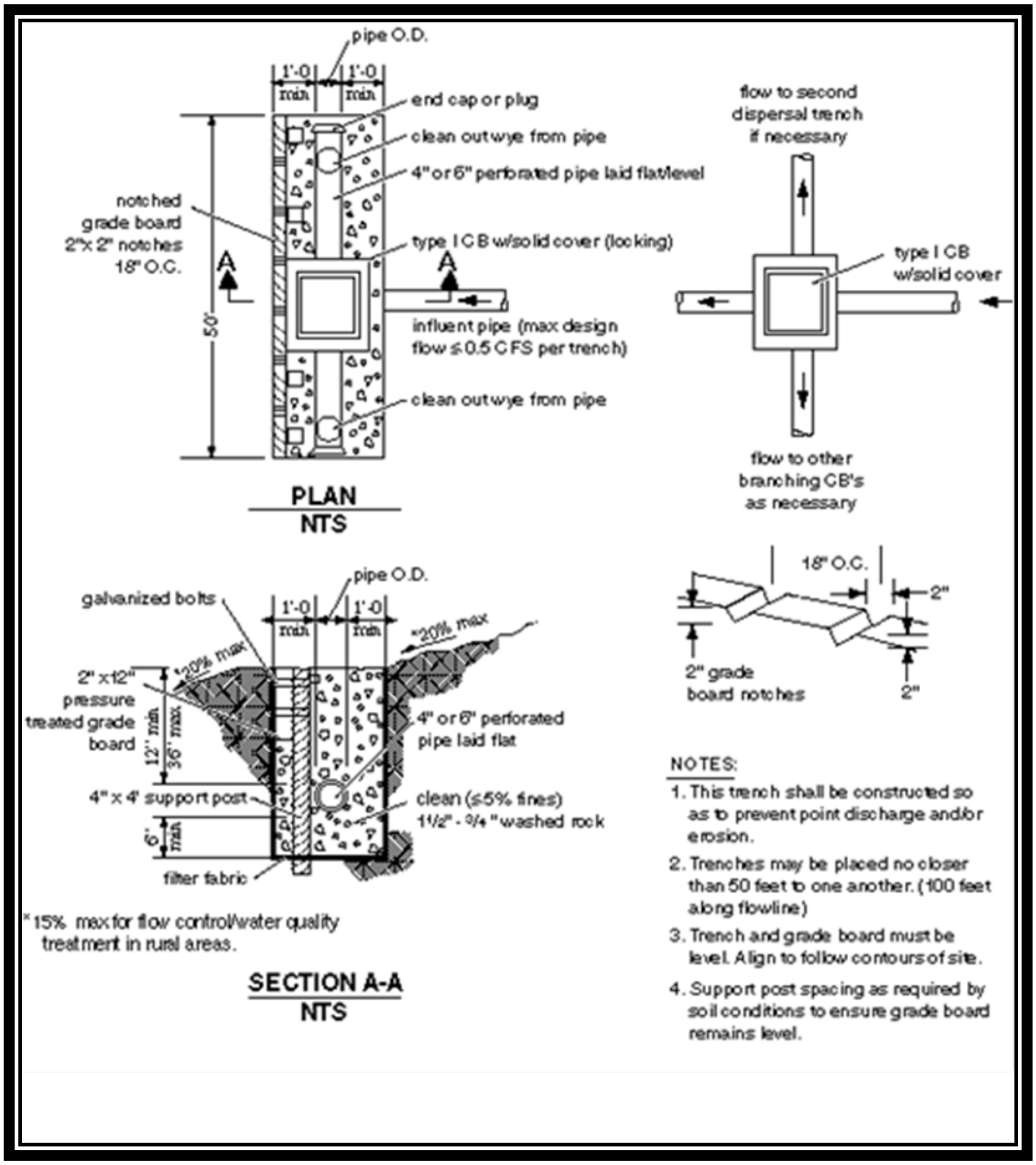

Figure III - 3.4 Flow Dispersal Trench

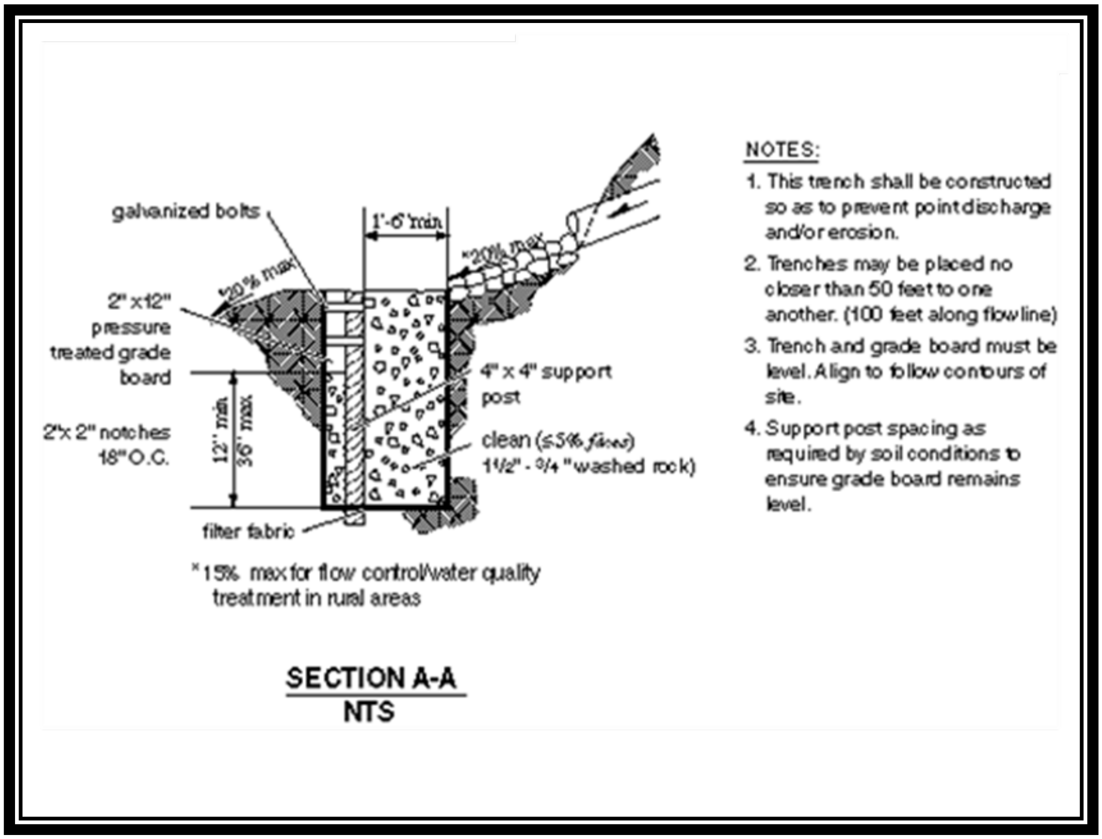

Figure III - 3.5 Alternative Flow Dispersal Trench

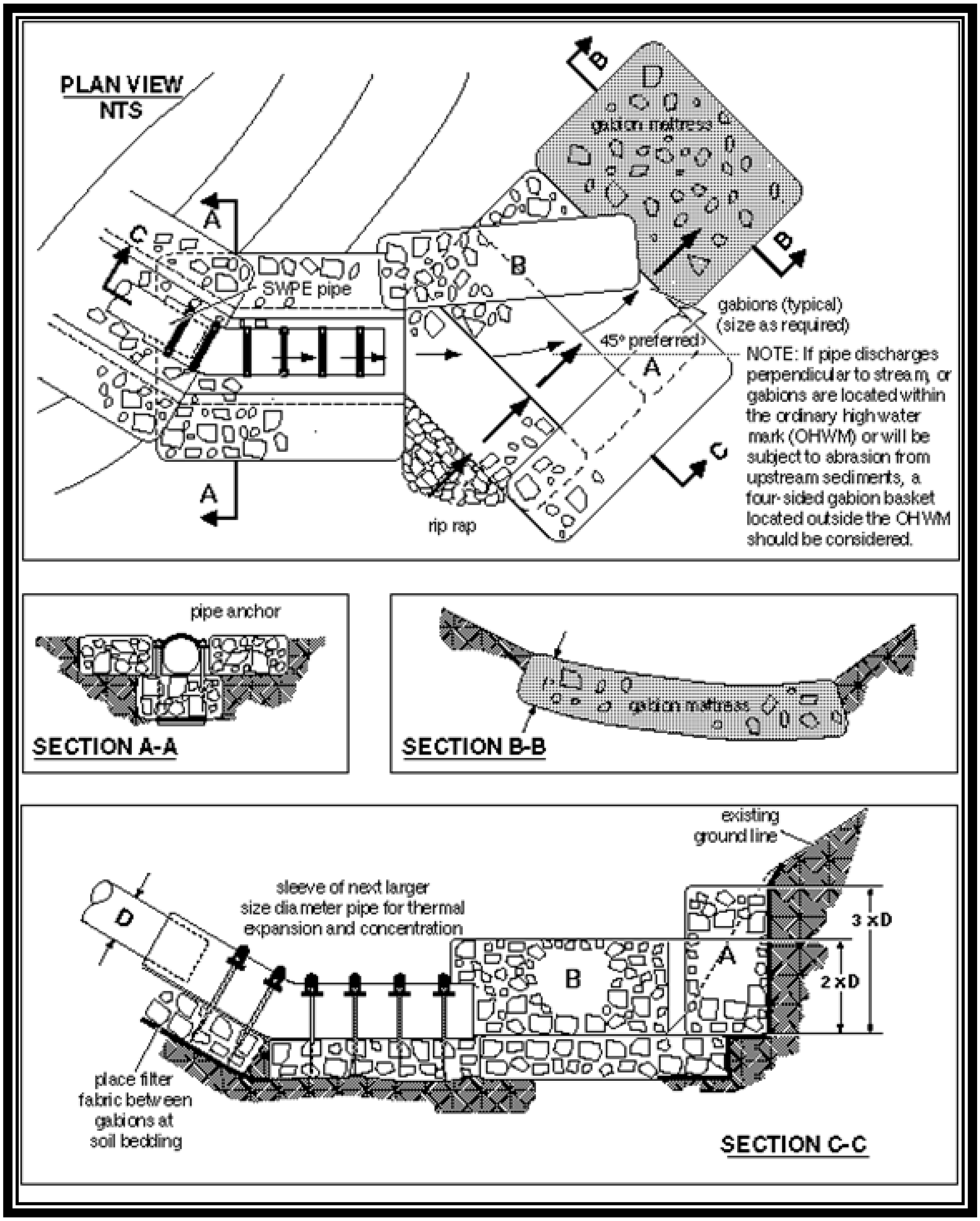

Figure III - 3.6 Gabion Outfall Detail

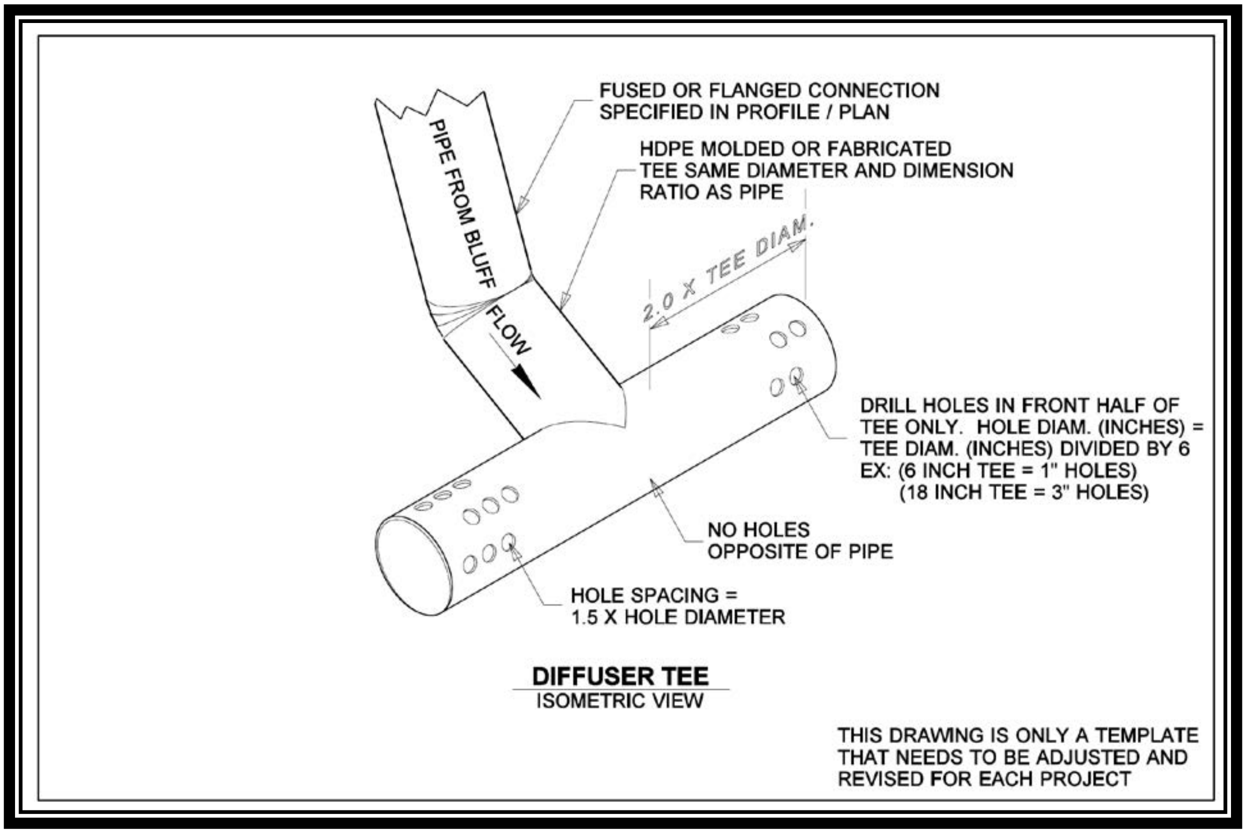

Figure III - 3.7. Diffuser TEE (an example of energy dissipating end feature) (Source: WSDOT Highway Runoff Manual)

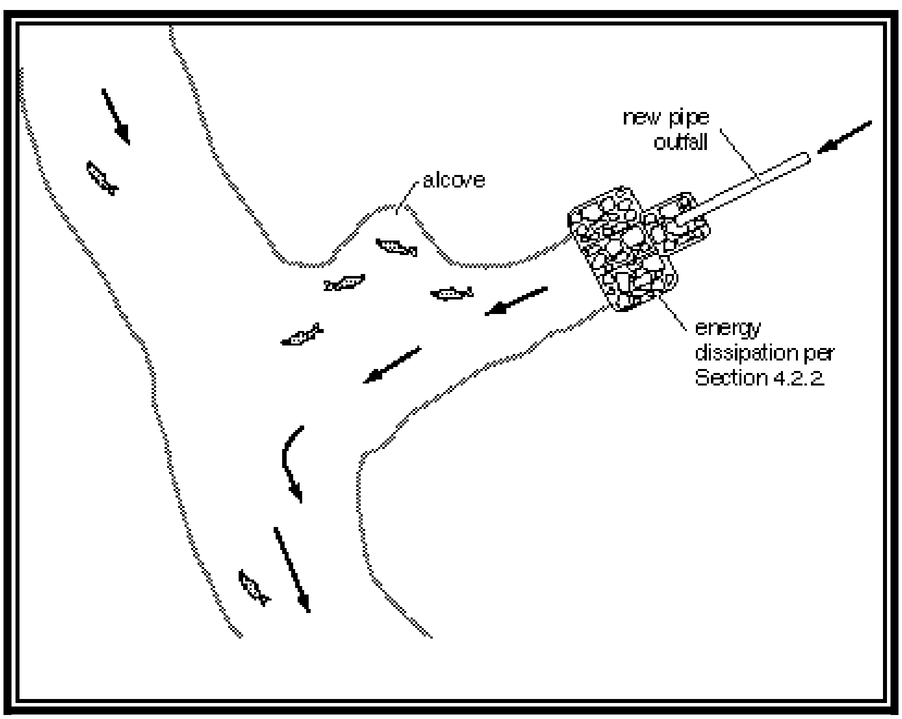

Figure III - 3.8 Fish Habitat Improvement at New Outfalls

3.9 Flow Spreading Options

Flow spreaders function to uniformly spread flows across the inflow portion of several types of stormwater management facilities (e.g., sand filters, biofiltration swales, filter strips, bioretention areas). There are five flow spreader options presented in this section:

- Option A – Anchored plate
- Option B – Concrete sump box
- Option C – Notched curb spreader
- Option D – Through-curb ports
- Option E – Interrupted curb.

Options A through C can be used for spreading flows that are concentrated. Any one of these options can be used when spreading is required by the facility design criteria. Options A through C can also be used for unconcentrated flows, and in some cases must be used, such as to correct for moderate grade changes along a filter strip.

Options D and E are only for flows that are already unconcentrated and enter a filter strip, bioretention area or continuous inflow biofiltration swale. Other flow spreader options are possible with approval from the Administrator or designee.

General Design Criteria

- Where flow enters the flow spreader through a pipe, it is recommended that the pipe be submerged to the extent practical to dissipate energy as much as possible.
- For higher inflows (velocities greater than 5 feet per second for the 100-year recurrence interval storm), a Type 1 catch basin should be positioned in the spreader and the inflow pipe should enter the catch basin with flows exiting through the top grate. The top of the grate should be lower than the level spreader plate, or if a notched spreader is used, lower than the bottom of the V-notches.

Option A – Anchored Plate (Figure III - 3.9)

- An anchored plate flow spreader should be preceded by a sump having a minimum depth of 8 inches and minimum width of 24 inches. If not otherwise stabilized, the sump area should be lined to reduce erosion and to provide energy dissipation.

- The top surface of the flow spreader plate should be level, projecting a minimum of 2 inches above the ground surface of the water quality facility, or V-notched with notches 6 to 10 inches on center and 1 to 6 inches deep (use shallower notches with closer spacing). Alternative designs may also be used.
- A flow spreader plate should extend horizontally beyond the bottom width of the facility to prevent water from eroding the side slope. The horizontal extent should be such that the bank is protected for all flows up to the 100-year recurrence interval flow or the maximum flow that will enter the water quality facility.
- Flow spreader plates should be securely fixed in place.
- Flow spreader plates may be made of either wood, metal, fiberglass reinforced plastic, or other durable material. If wood, pressure treated 4- by 10-inch lumber or landscape timbers are acceptable.
- Anchor posts should be 4-inch square concrete, tubular stainless steel, or other material resistant to decay.

Option B – Concrete Sump Box (Figure III - 3.10)

- The wall of the downstream side of a rectangular concrete sump box should extend a minimum of 2 inches above the treatment bed. This serves as a weir to spread the flows uniformly across the bed.
- The downstream wall of a sump box should have “wing walls” at both ends. Side walls and returns should be slightly higher than the weir so that erosion of the side slope is minimized.
- Concrete for a sump box can be either cast-in-place or precast, but the bottom of the sump should be reinforced with wire mesh for cast-in-place sumps.
- Sump boxes should be placed over bases that consists of 4 inches of crushed rock, five-eighths-inch minus to help assure the sump remains level.

Option C – Notched Curb Spreader (Figure III - 3.11)

Notched curb spreader sections should be made of extruded concrete laid side-by-side and level. Typically five “teeth” per 4-foot section provide good spacing. The space between adjacent “teeth” forms a V-notch.

Option D –Through-Curb Ports (Figure III - 3.12)

Unconcentrated flows from paved areas entering filter strips, bioretention areas, or continuous inflow biofiltration swales can use curb ports or interrupted curbs (Option E) to allow flows to enter the strip or swale. Curb ports use fabricated openings that allow concrete curbing to be poured or extruded while still providing an opening through the curb to admit water to the water quality facility.

Openings in the curb should be at regular intervals but at least every 6 feet (minimum). The width of each curb port opening should be a minimum of 11 inches. Approximately 15 percent or more of the curb section length should be in open ports, and no port should discharge more than about 10 percent of the flow.

Option E – Interrupted Curb (No Figure)

Interrupted curbs are sections of curb placed to have gaps spaced at regular intervals along the total width (or length, depending on facility) of the treatment area. At a minimum, gaps should be every 6 feet to allow distribution of flows into the treatment facility before they become too concentrated. The opening should be a minimum of 12 inches. As a general rule, no opening should discharge more than 10 percent of the overall flow entering the facility.

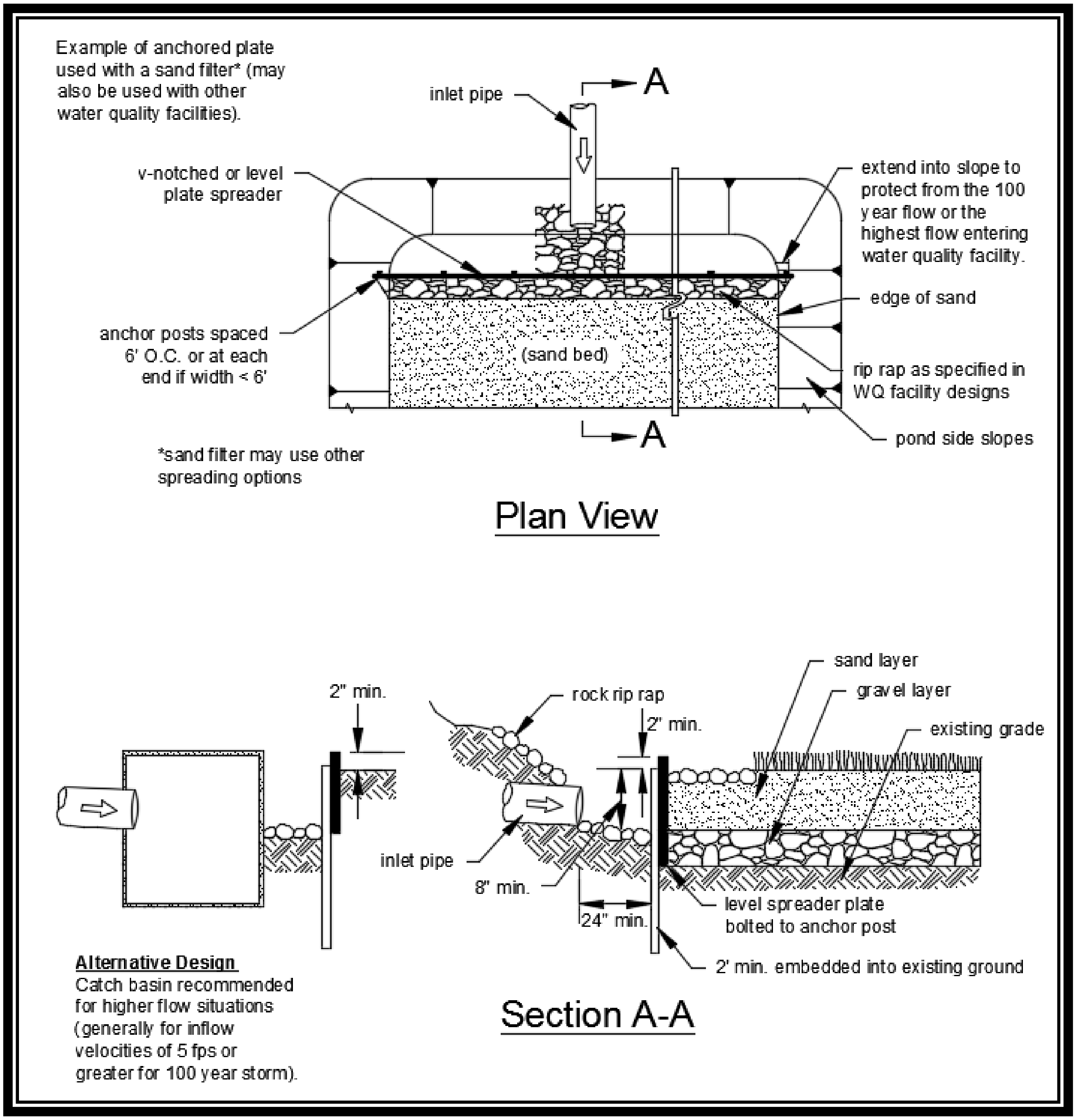

Figure III - 3.9 Flow Spreader Option A: Anchored Plate. (Source: Stormwater Management Manual for Western Washington)

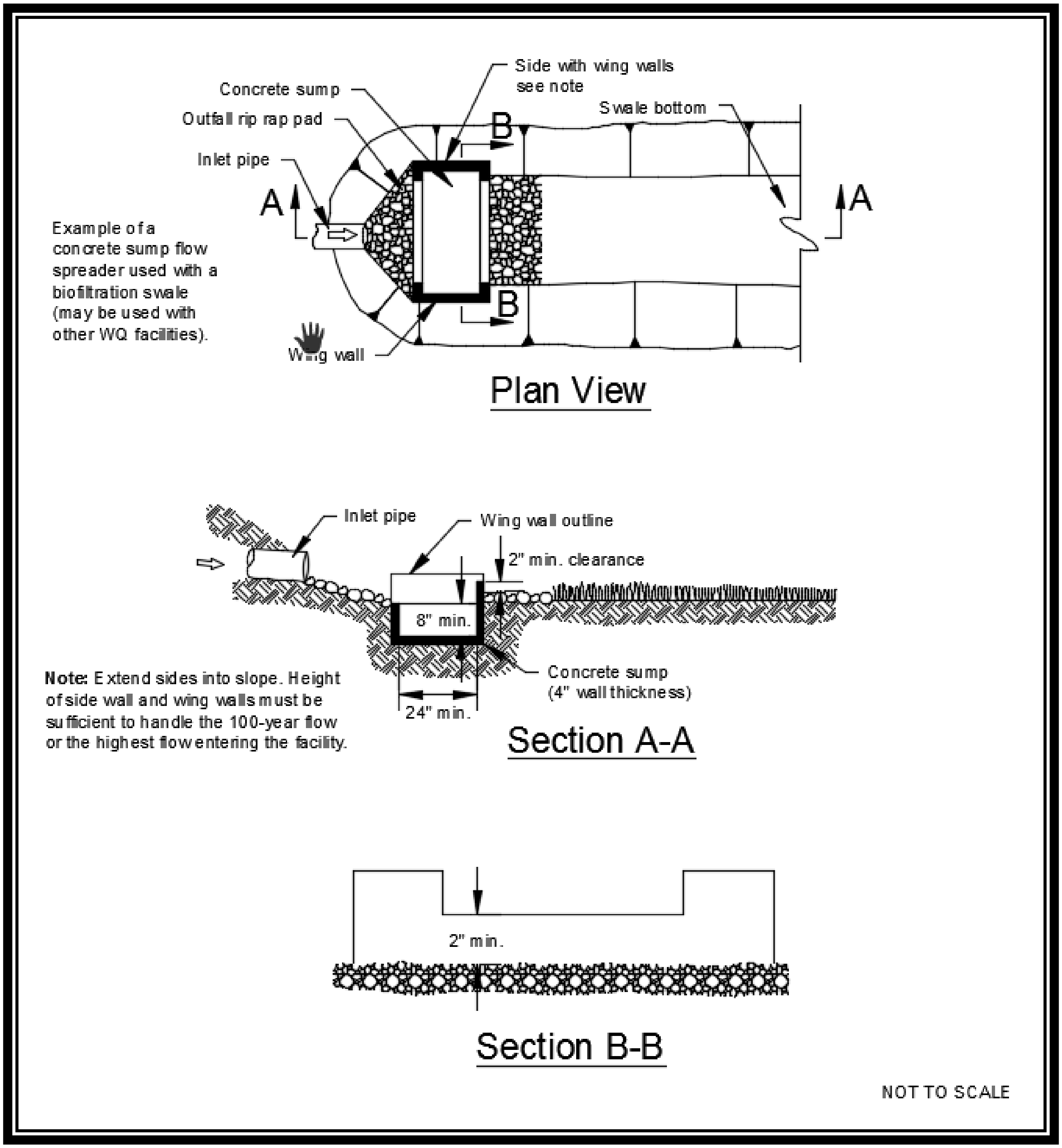

Figure III - 3.10 Flow Spreader Option B: Concrete Sump Box (Source: Stormwater Management Manual for Western Washington)

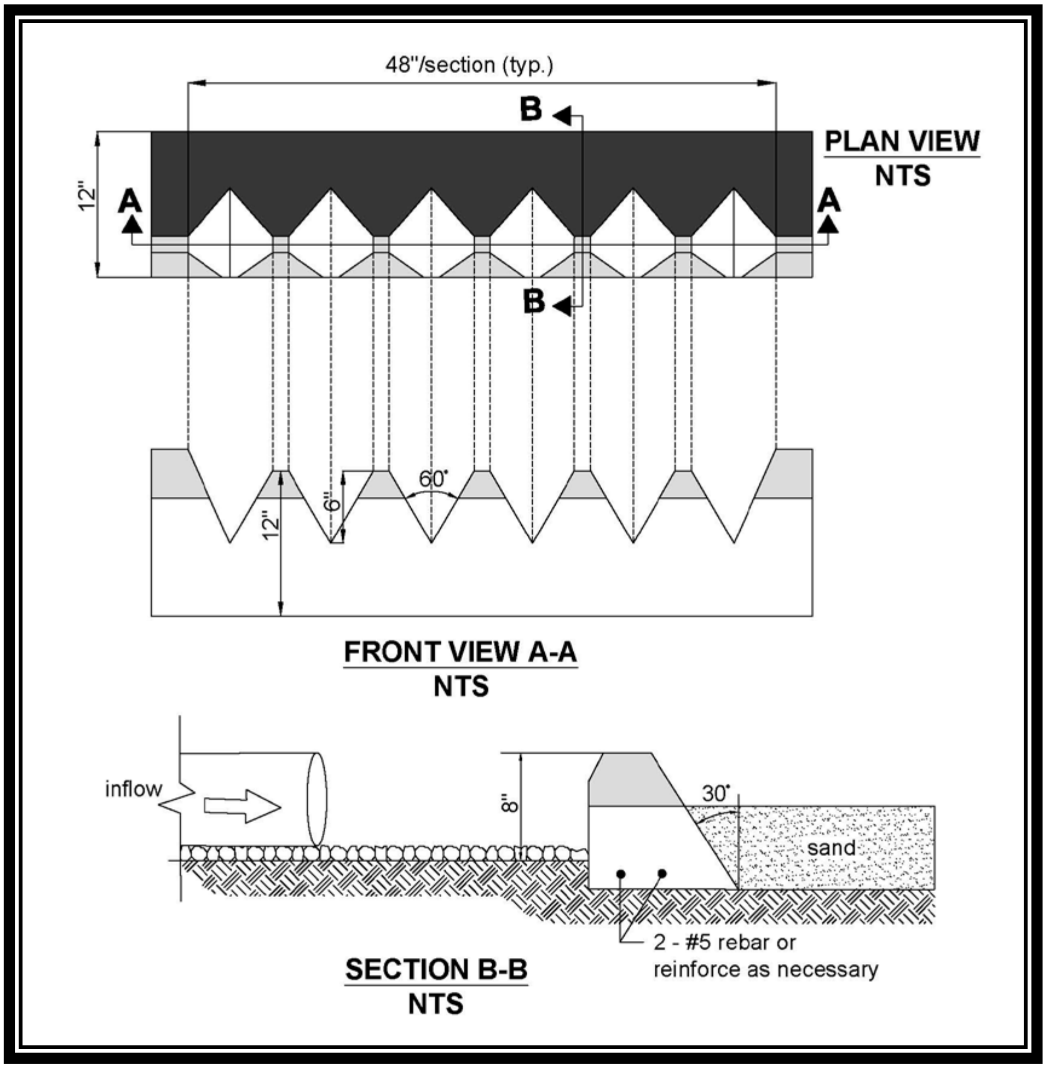

Figure III - 3.11. Flow Spreader Option C: Notched Curb Spreader. (Source Pierce County Stormwater and Site Development Manual)

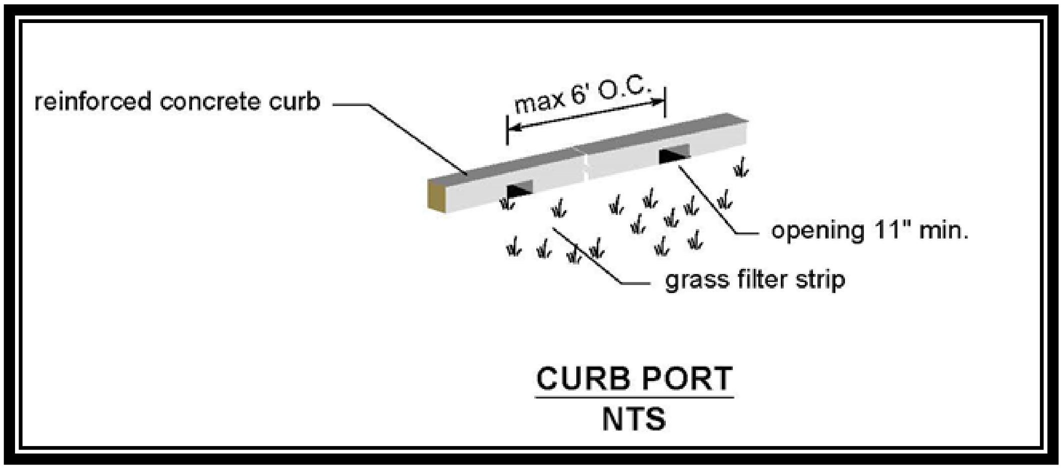

Figure III - 3.12. Flow Spreader Option D: Through Curb Port. (Source Pierce County Stormwater and Site Development Manual)

3.10 Culvert Criteria

Culverts are single runs of pipe that are open at both ends and have no structures, such as manholes or catch basins.

Approved pipe materials are detailed in Section 3.7. Galvanized or aluminized pipe is not permitted in marine environments or where contact with salt water may occur, even infrequently through backwater events.

Culvert Design Criteria

Flow capacity shall be determined by analyzing inlet and outlet control for headwater depth. Nomographs used for culvert design shall be included in the submitted Drainage Report. Appendix III-C also includes several nomographs useful for culvert sizing.

All culverts shall be designed to convey the flows per Section 3.2. The maximum design headwater depth shall be 1.5 times the diameter of the culvert, with no saturation of roadbeds. Minimum culvert diameters are as follows:

- For cross culverts under public roadways – minimum 18 inches, 12 inches if grade and cover do not allow for 18 inches, with County acceptance.
- For roadside culverts, including driveway culverts – minimum 12 inches.
- For culverts on private property – minimum 8 inches.

Inlets and outlets shall be protected from erosion by rock lining, riprap, or bio-stabilization as detailed in Table III - 3.7, Channel Protection.

Debris and access barriers are required on inlet and outlet ends of all culverts equal to or greater than 18 inches in diameter. Culverts equal to or greater than 36 inches in diameter or within stream corridors are exempt.

Minimum culvert velocity shall be 2 feet per second and maximum culvert velocity shall be 15 feet per second. Thirty (30) feet per second may be used with an engineered outlet protection design. There is no maximum velocity for ductile iron or HDPE pipe, but outlet protection shall be provided.

All CPEP and PVC culverts and pipe systems shall have concrete or rock headwalls at exposed pipe ends.

Bends are not permitted in culvert pipes.

The following minimum cover shall be provided over culverts:

- 2 feet under roads.
- 1 foot under roadside applications and on private property, exclusive of roads.
- If the minimum cover cannot be provided on a flat site, use ductile iron pipe and analyze for loadings.
- Maximum culvert length: 250 feet
- Minimum separation from other pipes:
  - 6 inches vertical (with bedding) (and in accord with the sewer or water purveyor design criteria).
  - 3 feet horizontal.

Culvert trench bedding, backfill and compaction shall be in accordance with the WSDOT standard specifications for the type of culvert pipe used in the application.

All driveway culverts shall be of sufficient length to provide a minimum 3:1 slope from the edge of the driveway to the bottom of the ditch. Culverts shall have beveled end section to match the side slope. Ductile pipe shall use PVC or CPEP for beveled end sections.

Fish Passage Criteria

Culverts in stream corridors must meet applicable fish passage requirements of the Washington Department of Fish and Wildlife.

3.11 Open Conveyances

Open conveyances can be roadside ditches, grass lined swales, or a combination thereof. Where space and topography permit, open conveyances are preferred for collecting and conveying stormwater as they better reflect LID design. Consideration must be given to public safety when designing open conveyances adjacent to traveled ways and when accessible to the public. A vegetated open channel BMP is the preferred conveyance method.

Open conveyances shall be designed by one of the following methods:

- Manning's Equation (for uniform flow depth, flow velocity, and constant channel cross-section; see Section 3.4.1).
- Backwater Method (utilizing the energy equation or a computer program; see Section 3.4.2).

Velocities must be low enough to prevent channel erosion based on the native soil characteristics or the compacted fill material. For velocities above 5 feet per second, channels shall have either rock-lined bottoms and side slopes to the roadway shoulder top with a minimum thickness of 8 inches, or shall be stabilized in a fashion acceptable to the County. Water quality shall not be degraded due to passage through an open conveyance. See Table III - 3.7.

Table III - 3.7 Channel Protection

Velocity at Design Flow (fps)		**REQUIRED PROTECTION**		
Greater than	**Less than or equal to**	**Type of Protection**	**Thickness**	**Minimum Height Above Design Water Surface**
0	5	Grass lining or bioengineered lining	N/A	0.5 foot
5	8	Rock lining (1) or bioengineered lining	1 foot	2 foot
8	12	Riprap (2)	2 feet	2 feet
12	20	Slope mattress gabion, etc.	Varies	2 feet

(1) Rock Lining shall be reasonably well graded as follows:

Maximum stone size: 12 inches

Median stone size: 8 inches

Minimum stone size: 2 inches

(2) Riprap shall be reasonably well graded as follows:

Maximum stone size: 24 inches
Median stone size: 16 inches
Minimum stone size: 4 inches
Note: Riprap sizing is governed by side slopes on channel, assumed to be approximately 3:1

Channels having a slope less than 6 percent and having peak velocities less than 5 feet per second shall be lined with vegetation.

Channel side slopes shall not exceed 2:1 for undisturbed ground (cuts) as well as for disturbed ground (embankments). All constructed channels shall be compacted to a minimum 95 percent compaction as verified by a Modified Proctor test. Channel side slopes adjacent to roads shall meet all AASHTO and county road standards.

Channels shall be designed with a minimum freeboard of 0.5 feet when the design flow is 10 cubic feet per second or less and 1 foot when the design flow is greater than 10 cubic feet per second.

Check dams for erosion and sedimentation control may be used for stepping down channels being used for biofiltration.

3.12 Private Drainage Systems

The engineering analysis for a private drainage system is the same as a County system.

Discharge Locations

Stormwater cannot discharge directly onto County roads or into a County system without prior County approval[2]. Discharges to a County system shall be into a structure such as an inlet, catch basin, manhole, through an approved sidewalk underdrain or curb drain, or into an existing or created County ditch. Concentrated drainage will not be allowed to discharge across sidewalks, curbs, or driveways.

All buildings are required to have roof downspouts and subsurface drains directed to either an infiltration system, dispersion system, or to the storm drainage system.

Drainage Stub-outs

If drainage outlets (stub outs) are to be provided for each individual lot, the stub outs shall conform to the requirements outlined below. Note that all applicable Core Requirements in Volume I, in particular Core Requirement #5, must also be addressed for the project site.

[2] A County connection authorization form must be completed and submitted for approval.

- Each outlet shall be suitably located at the lowest elevation on the lot, so as to service all future roof downspouts and footing drains, driveways, yard drains, and any other surface or subsurface drains necessary to render the lots suitable for their intended use. Each outlet shall have free-flowing, positive drainage to an approved storm water conveyance system or to an approved outfall location.

- Outlets on each lot shall be located with a 5-foot-high, 2" x 4" stake marked "storm" or "drain." For stub-outs to a surface drainage, the stub-out shall visibly extend above surface level and be secured to the stake.

- The developer and/or contractor is responsible for coordinating the locations of all stub-out conveyance lines with respect to the utilities (e.g., power, gas, telephone, television).

- All individual stub-outs shall be privately owned and maintained by the lot home owner including from the property line to the riser on the main line.

Use of Pump Stations, Mechanical Equipment and Other Related Appurtenances

The installation and use of privately owned and operated pump stations, mechanical equipment, and other related appurtenance for the purpose of conveying, directing or managing storm and surface water is not permitted. The installation and operation of a pump station, mechanical equipment, or other similar appurtenances may only be accepted upon written approval by the DDECM Administrator (or designee), provided that the pump station will be owned, operated and maintained by a municipality (local, state or federal) in perpetuity.

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Appendix III-A
Methods for Determining Design Infiltration Rates

A crucial element to infiltration BMP design is the long term (design) infiltration rate of the native soils. In order to determine the design infiltration rate, the designer must first determine the measured (initial) saturated hydraulic conductivity (K_{sat}) of the native soils.

This appendix provides details on each method for determining initial K_{sat}. A safety/correction factor is applied to the initial rate to determine the design infiltration rate. Note that the subgrade safety/correction factors in this appendix may not apply to bioretention, permeable pavement, and rain gardens. Refer to individual BMPs in Volume V for additional guidance.

- Method 1 – Field Testing Procedures (must incorporate safety factor)
    - U.S. EPA Falling Head Percolation Test Procedure (as Modified for Thurston County). This test applies to all infiltration facilities but may not be used to demonstrate infeasibility of bioretention, permeable pavement, or rain gardens in meeting Core Requirement #5.
    - Large-Scale Pilot Infiltration Test (PIT). This test applies to infiltration facilities with drainage areas greater than one acre and may be used to demonstrate infeasibility of bioretention, permeable pavement, or rain gardens in meeting Core Requirement #5.
    - Small-Scale (PIT). This test applies to infiltration facilities with drainage areas less than one acre and may be used to demonstrate infeasibility of bioretention, permeable pavement, or rain gardens in meeting Core Requirement #5.
- Method 2 – Soil Property Relationships (USDA Soil Textural Classification). This method only applies to project sites inside the County's municipal stormwater permit (NDPES) boundary that trigger Core Requirement #1 through #5 or any project outside the NPDES boundary, and that are underlain by hydrologic soil group A soils (as defined by the NRCS Web Soil Survey and field verified by a qualified professional). This method may not be used to demonstrate infeasibility of bioretention, permeable pavement, or rain gardens in meeting Core Requirement #5.
- Method 3 – Soil Grain Size Analysis . This method applies to project sites that are underlain by type A soils (as defined by the NRCS Web Soil Survey and filed verified by a qualified professional) and may not be used

to demonstrate infeasibility of bioretention, permeable pavement, or rain gardens in meeting Core Requirement #5.

Method 1 – Field Testing Procedures (In-Situ)

1. Excavate to the bottom elevation of the proposed infiltration facility. Measure the infiltration rate of the underlying soil using either the EPA falling head percolation test procedure as modified for Thurston County (described below), the double ring infiltrometer test (ASTM D3385, not described in this appendix), or the Department of Ecology large and small scale Pilot Infiltration Test (PIT) described below and presented in the 2019 Ecology *Stormwater Management Manual for Western Washington*.

2. Fill test hole or apparatus with water and maintain at depths above the test elevation for saturation periods specific to the appropriate test.

3. Following the saturation period, the infiltration rate shall be determined in accordance with the specified test procedures.

4. See individual BMP descriptions for requirements related to the number and location of tests required.

For all field testing procedures, apply safety factor to obtain design infiltration rate (see next section).

Safety Factor for Field Measurements

The following equation incorporates safety factors to account for uncertainties related to testing, depth to the water table or impervious strata, infiltration receptor geometry, and long-term reductions in permeability due to biological activity and accumulation of fine sediment. Note that the safety factors below may not apply to the infiltration testing conducted for bioretention, permeable pavement and/or rain gardens (see Volume V, Sections 2.2.5 and 2.2.6 for additional information). This equation estimates the maximum design infiltration rate, I_{design}. Depending on site conditions, additional reduction of the design infiltration rate may be appropriate. **In no case may the design infiltration rate exceed 30 inches/hour**.

$$I_{design} = I_{measured} \times F_{testing} \times F_{geometry} \times F_{plugging}$$

$F_{testing}$ accounts for uncertainties in the testing methods.

- For the full scale PIT method, Ftesting = 0.75;
- For the small-scale PIT method, Ftesting = 0.50;
- For smaller-scale infiltration tests such as the double-ring infiltrometer test, Ftesting = 0.40;

- For grain size analysis, Ftesting = 0.40;
- For the EPA method, the SDI (ASTM D3385) method, Ftesting = 0.50.

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. $F_{testing}$ accounts for uncertainties in the testing methods.

$F_{geometry}$ accounts for the influence of facility geometry and depth to the water table or impervious strata on the actual infiltration rate. A shallow water table or impervious layer reduces the effective infiltration rate of a large pond, but this would not be reflected in a small scale test. $F_{geometry}$ must be between 0.25 and 1.0 as determined by the following equation:

$F_{geometry} = 4\ D/W + 0.05$

Where: D = Depth from the bottom of the proposed facility to the maximum wet season water table or nearest impervious layer, whichever is less

W = Width of facility

If $F_{geometry}$ is calculated as greater than 1, use 1, if calculated value is less than 0.25, use 0.25.

$F_{plugging}$ accounts for reductions in infiltration rates over the long term due to plugging of soils. This factor is:

- 0.7 for loams and sandy loams
- 0.8 for fine sands and loamy sands
- 0.9 for medium sands
- 1.0 for coarse sands or cobbles, or any soil type in an infiltration facility preceded by a water quality facility (not including a pre-treatment unit or forebay for course sediment removal).

Falling Head Percolation Test Procedure (as Modified for Thurston County)[3]

Note: This test may not be used to demonstrate infeasibility of bioretention, permeable pavement, or rain gardens in meeting Core Requirement #5.

1. Location of Tests

Tests shall be spaced uniformly throughout the area. For larger facilities or if soil conditions are highly variable, more tests may be required .

2. **Preparation of Test Hole** (as modified for Thurston County)

The diameter of each test hole is 8 inches, dug or bored to the proposed bottom elevation of the infiltration facility or to the most limiting soil horizon. To expose a natural soil surface, the bottom of the hole is scratched with a sharp pointed instrument and the loose material is removed from the test hole. A 6-inch-inner-diameter, 4-foot long, PVC pipe is set into the hole and pressed 6 inches into the soil, then 2 inches of 1/2- to 3/4-inch rock are placed in the pipe to protect the bottom from scouring when water is added.

3. Soaking Period

The pipe is carefully filled with at least 12 inches of clear water. The depth of water must be maintained for at least 4 hours and preferably overnight if clay soils are present. A funnel with an attached hose or similar device may be used to prevent water from washing down the sides of the hole. Automatic siphons or float valves may be employed to automatically maintain the water level during the soaking period. It is extremely important that the soil be allowed to soak for a sufficiently long period of time to allow the soil to swell if accurate results are to be obtained.

In sandy soils with little or no clay, soaking is not necessary. If, after filling the pipe twice with 12 inches of water, the water seeps completely away in less than 10 minutes, the test can proceed immediately.

4. Percolation Rate Measurement

Except for sandy soils, percolation rate measurements are made at least 15 hours but no more than 30 hours after the soaking period began. The water level is adjusted to 6 inches above the gravel (or 8 inches above the bottom of the hole). At no time during the test is the water level allowed to rise more than 6 inches above the gravel. Immediately after adjustment, the water level is measured from a fixed reference point to the nearest 1/16th-inch, at 30 minute intervals. The test is continued until two successive water level drops do not vary by more than 1/16th-inch within a 90 minute period. At least three measurements are to be made.

[3] (Source: EPA, *On-site Wastewater Treatment and Disposal Systems,* 1980)

After each measurement, the water level is readjusted to the 6-inch level. The last water level drop is used to calculate the percolation rate.

In sandy soils or soils in which the first 6 inches of water added after the soaking period seeps away in less than 30 minutes, water level measurements are made at 10-minute intervals for a 1-hour period. The last water level drop is used to calculate the percolation rate.

5. **Percolation Rate Calculation**

The percolation rate is calculated for each test site by dividing the time interval used between measurements by the magnitude of the last water level drop. This calculation results in a percolation rate in minutes/inch. To calculate the percolation rate for the area, average the rates obtained from each hole. (If tests in the area vary by more than 20 minutes/inch, variations in soil type are indicated. Under these circumstances, percolation rates should not be averaged.) The percolation rate in minutes/inch should be converted to infiltration rate in inches/hour and then t**o compute the design infiltration rate (I_{design}), the final infiltration rates must then be adjusted by the appropriate correction factors outlined previously.**

Example: If the last measured drop in water level after 30 minutes is 5/8-inch, then:

percolation rate = (30 minutes)/(5/8 inch) = 48 minutes/inch. Convert this to inches per hour by inverting & multiplying by 60: infiltration rate – 1/48*60 = 1.25 inches/hour. (At a minimum, a safety factor “$F_{testing}$” of 0.5 is be applied to all field methods for determining infiltration rates.)

Washington Department of Ecology Infiltration PIT Method

The **Large-Scale Pilot Infiltration Test (PIT)** consists of a relatively large-scale infiltration test to better approximate infiltration rates for design of stormwater infiltration facilities. The PIT reduces some of the scale errors associated with relatively small-scale tests such as the Modified Falling Head Percolation Test, 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. It is the preferred method for estimating the measured (initial) saturated hydraulic conductivity (K_{sat}) of the soil profile beneath the proposed infiltration facility. Following is a step-by-step description of the testing procedure.

Infiltration Test

1. Testing should occur between December 1 and April 1.

2. The horizontal and vertical locations of the PIT shall be surveyed by a licensed land surveyor and accurately shown on the design drawings.

3. Excavate the test pit to the estimated elevation of the proposed infiltration into the native soil. Note that for some proposed BMP, such as

bioretention and permeable paving, this will be below the proposed finished grade. If the native soils will to meet a minimum subgrade compaction requirement, compact the native soil to that requirement prior to testing. Lay back the slopes sufficiently to avoid caving and erosion during the test. Alternatively, consider shoring the sides of the test pit.

4. 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.

5. Install a vertical measuring rod (minimum 5 feet long) marked in half-inch increments in the center of the pit bottom.

6. 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.

6. 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: 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. However, the depth must not exceed the proposed maximum depth of water expected in the completed facility.

7. Every 15 to 30 minutes, 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.

8. Add water to the pit until 1 hour after the flow rate into the pit has stabilized (constant flow rate; a goal of 5 percent variation or less variation in the total flow) while maintaining the same pond water level (usually 6 hours). The total of the pre-soak time plus one hour after the flow rate has stabilized should be no less than 6 hours.

9. After the flow rate has stabilized for at least 1 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.

Data Analysis

Calculate and record the infiltration rate in inches per hour in 30 minute or one-hour increments until 1 hour after the flow has stabilized.

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

To compute the design infiltration rate (I_{design}), apply appropriate correction factors outlined previously.

Example:

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

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. Divide the flow rate by the area of the test pit and convert to inches per hour to get an average of (9.8 + 12.3) / 2 = 11.1 inches per hour.

To compute the design infiltration rate (I_{design}), the infiltration rate must then be adjusted by the appropriate correction factors outlined previously.

Small-Scale Pilot Infiltration Test

A smaller-scale PIT can be used in any of the following instances:

The drainage area to the infiltration site is less than one acre.

The testing is for bioretention or permeable paving that either serve small drainage areas and/or are widely dispersed throughout a project site.

The site has conditions that make a large-scale PIT difficult, such as high infiltration rates (>4 in/hr)and the site geotechnical investigation suggests uniform subsurface characteristics.

Infiltration Test

Use the same procedures described above in Large Scale Pilot Infiltration Test (PIT), with the following changes:

1. 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.

2. The rigid pipe with a splash plate used to convey water to the pit may be 3-inch diameter pipe for pits on the smaller end of the recommended surface area, or a 4 inch pipe for pits on the larger end of the recommended surface area.

3. 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.

4. 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.

5. 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 – 12 inches) on the measuring rod. The specific depth should be the same as the maximum designed ponding depth (usually 6 – 12 inches).

6. 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.

7. A self-logging pressure sensor may also be used to determine water depth and drain-down.

8. 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 the 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.

Method 2 – Soil Property Relationships

USDA Soil Textural Classification

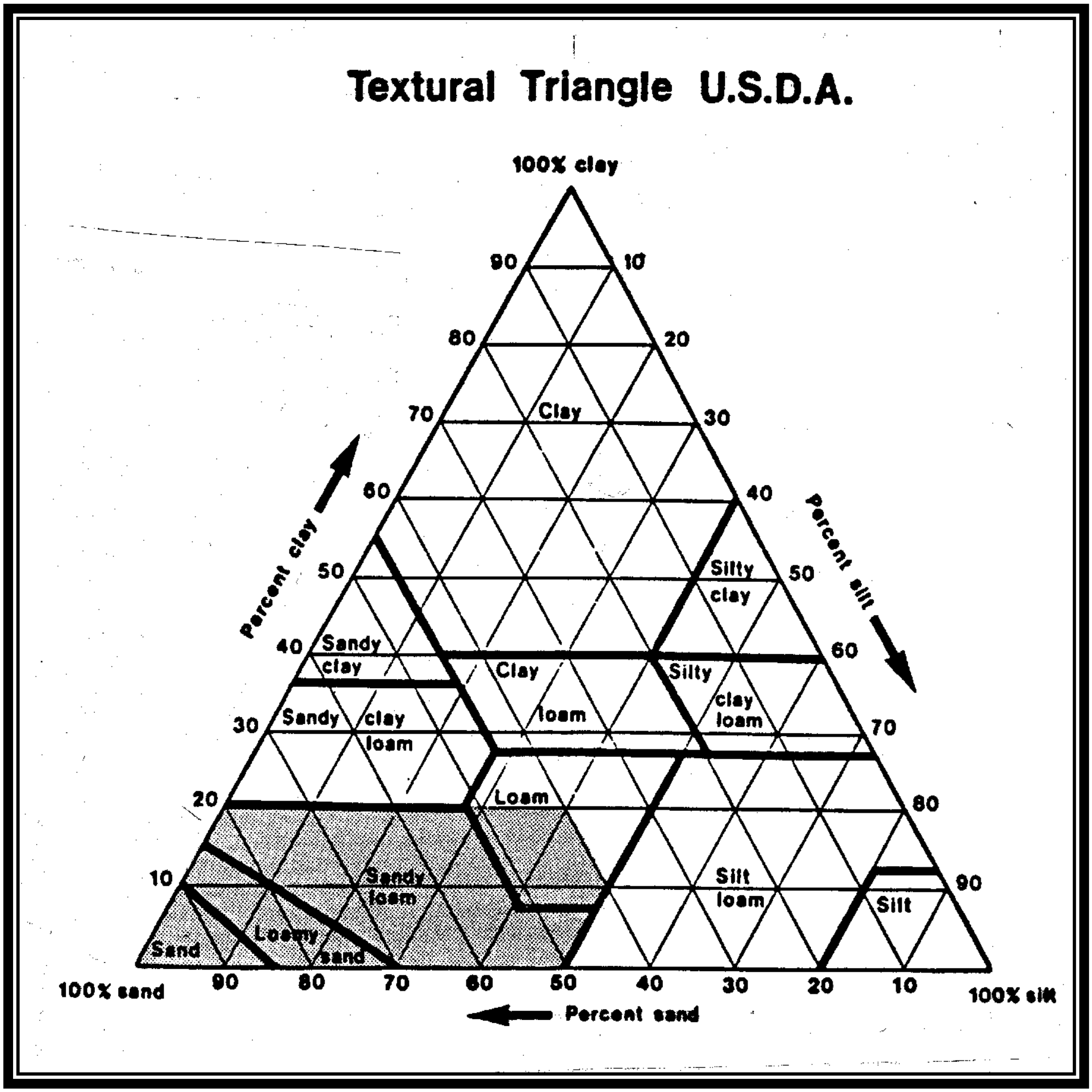
Infiltration rates may be estimated from soil grain size distribution (gradation) data using the United States Department of Agriculture (USDA) textural analysis approach. Conduct the grain size distribution test in accordance with the USDA test procedure (Soil Survey Manual, USDA, October 1993, page 136). This manual only considers soil passing the #10 sieve (2 mm) (US Standard) to determine percentages of sand, silt, and clay for use in Figure III - A.1. This method may only be applied to projects sites inside Thurston County's municipal stormwater permit (NPDES) boundary that trigger Core Requirement #1 through #5 or any project outside the NPDES boundary, and that are underlain by hydrologic soil group A soils (as defined by the NRCS Web Soil Survey and field verified by a qualified professional). A map of the County's municipal stormwater permit (NPDES) boundary may be found on the County's GeoData website at: https://www.geodata.org/.

Short-term (field) infiltration rates, required correction factors, and design (long-term) infiltration rates based on gradations from soil samples and textural analysis are summarized in Table III - A.1. With prior acceptance of Thurston County, the correction factors may be reduced (to a minimum of 2.0) if there is little soil variability, there will be a high degree of long-term facility maintenance, and there is adequate pre-treatment to reduce total suspended solids in influent stormwater.

Table III - A.1 Recommended Infiltration Rates based on USDA Soil Textural Classification

	*Short-Term Infiltration Rate (in./hr)	Correction Factor, CF	Estimated Design (Long-term) Infiltration Rate (in./hr)
Clean sandy gravels and gravelly sands (i.e., 90% of the total soil sample is retained in the #10 sieve)	20	2	10
Sand	8	4	2
Loamy Sand	2	4	0.5
Sandy Loam	1	4	0.25
Loam	0.5	4	0.13

Source: *Stormwater Management Manual for Western Washington* (Ecology 2005).
*From WEF/ASCE, 1998.



Shaded area is applicable for design of infiltration BMPs. Source: U.S. Department of Agriculture

Figure III – A.1 USDA Textural Triangle.

Method 3 - Soil Grain Size Analysis Method

The following grain size analysis may be used to determine initial infiltration rates if the site has soils unconsolidated by glacial advance. This 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. The detailed method described below is based on Massmann (2003).

Determine the Saturated Hydraulic Conductivity

For each defined layer below the pond to a depth below the pond bottom of 2.5 times the maximum depth of water in the pond, but not less than 6 feet, estimate the saturated hydraulic conductivity (K_{sat}) in centimeters per second (cm/s) using the following relationship (see Massmann 2003, and Massmann et al. 2003). For 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).

$$\log_{10}\left(K_{sat}\right) = -1.57 + 1.90D_{10} + 0.015D_{60} - 0.013D_{90} - 2.08f_{fines}$$

Where, D_{10}, D_{60} and D_{90} are the grain sizes in millimeters (mm) for which 10 percent, 60 percent and 90 percent of the sample is more fine and f_{fines} is the fraction of the soil (by weight) that passes the US #200 sieve. (K_{sat} is in cm/s)

For bioretention areas, 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 (reservoir) course, but not less than 3 feet (1 meter).

If the licensed 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. Note that only the layers near and above the water table or low permeability zone (e.g., a clay, dense glacial till, or rock layer) need to be considered, as the layers below the groundwater table or low permeability zone do not significantly influence the rate of infiltration. Also note that this equation for estimating hydraulic conductivity 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, or is heavily over consolidated due to its geologic history (e.g., overridden by continental glaciers), 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_{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_{sat} will be close to an order of magnitude. For soils that contain clay, the reduction in K_{sat} could be greater than an order of magnitude.

For critical designs (facilities that pose a high risk of flooding and property damage in the event of clogging or other failure), the in-situ saturated conductivity of a specific layer can be obtained through the use of a pilot infiltration test (PIT) as described above. Note that some field tests provide a direct estimate of infiltration rate, which is the product of hydraulic conductivity and hydraulic gradient (see Equation 5). In this case, the infiltration rate must be divided by the hydraulic gradient to calculate the hydraulic conductivity. This issue will need to be evaluated on a case-by-case basis when interpreting the results of field tests to ensure an accurate estimate of K_{sat}. It is important to recognize that the gradient in the test may not be the same as the gradient likely to occur in the full-scale infiltration facility in the long-term (i.e., when groundwater mounding is fully developed).

Once the saturated hydraulic conductivity for each layer has been identified, determine the effective average saturated hydraulic conductivity of the native soils. Hydraulic conductivity estimates from different layers can be combined the harmonic mean:

(equation 2):

$$K_{equiv} = \frac{d}{\sum \frac{d_i}{K_i}}$$

Where:

d is the total depth of the soil column

d_i is the thickness of layer “i” in the soil column

K_i is the saturated hydraulic conductivity of layer “i” in the soil column.

The depth of the soil column, *d*, typically would include all layers between the pond bottom and the water table. However, for sites with very deep water tables (>100 feet) where groundwater mounding to the base of the pond is not likely to occur, it is recommended that the total depth of the soil column in Equation 2 be limited to approximately 20 times the depth of pond, but not more than 50 feet. 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 pond bottom should not be included in Equation 2.

Equation 2 may over-estimate the effective hydraulic conductivity value at sites with low conductivity layers immediately beneath the infiltration BMP. For sites where the lowest conductivity layer is within five feet of the base of the BMP, it is suggested that this lowest hydraulic conductivity value be used as the equivalent hydraulic conductivity rather than the value from Equation 2. Using the layer with the lowest Ksat is advised for designing bioretention areas or permeable pavement surfaces.

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 groundwater mounding.

Appendix III-B
Design Aids

Single Event Model Guidance

The only approved use of a single event model is for the sizing of conveyance systems. Approved continuous simulation runoff models will be used for the design of water quality and quantity BMPs.

SBUH or SCS Methods

The applicant shall use the Western Washington SCS "curve numbers" included in Table III - B.4, not the SCS national curve numbers. Individual curve numbers for a drainage area may be averaged into a "composite" curve number for use with SCS or SBUH methods. 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 identified relationships between land use, soil type, vegetation cover, interception, infiltration, surface storage, and runoff. These relationships have been characterized by a single runoff coefficient called a "curve number." 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.

The curve numbers 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 III - B.5 shows the hydrologic soil group of most soils in Thurston County and provides a brief description of the four groups. For details on other soil types, the NRCS publication described above (TR-55, 1986).

Isopluvial Maps

National Oceanic and Atmospheric Administration (NOAA) isopluvial maps for western Washington are included below. The design engineer shall use the best engineering judgment in selecting the runoff totals for the project site.

Time of Concentration

Time of concentration (T_c) is the sum of travel times for sheet flow, shallow concentrated flow, and channel flow. For lakes and submerged wetlands, travel time can be determined with storage routing techniques if the stage-storage versus discharge relationship is known or may be assumed to be zero.

Sheet Flow

With sheet flow, the friction value (n_s) is used. This is a modified Manning's effective roughness coefficient that includes the effect of raindrop impact, drag over the plane surface, obstacles such as litter, crop ridges and rocks, and erosion and transportation of sediment. These n_s values are for very shallow flow depths of about 0.1 foot and are used only for travel lengths up to 300 feet. Table III - B.2 gives Manning's n_s values for sheet flow for various surface conditions.

For sheet flow of up to 300 feet, use Manning's kinematic solution to directly compute T_t.

$$T_t = \frac{0.42\,(n_s L)^{0.8}}{(P_2)^{0.527}(S_o)^{0.4}}$$

Where:

- T_t = Travel time (min),
- n_s = Sheet flow Manning's effective roughness coefficient (Table III - B.2),
- L = Flow length (ft),
- P_2 = 2-year, 24-hour rainfall (in), and
- s_o = Slope of hydraulic grade line (land slope, ft/ft)

The maximum allowable distance for sheet flow shall be 300 feet. The remaining overland flow distance shall be shallow concentrated flow until the water reaches a channel.

Shallow Concentrated Flow

After a maximum of 300 feet, sheet flow is assumed to become shallow concentrated flow. The average velocity for this flow can be calculated using the k_s values from Table III - B.2 in which average velocity is a function of watercourse slope and type of channel.

The average velocity of flow, once it has measurable depth, shall be computed using the following equation:

$$V = k\sqrt{s_o}$$

Where:

- V = Velocity (ft/s)
- k = Time of concentration velocity factor (ft/s)
- s_o = Slope of flow path (ft/ft)

"k" is computed for various land covers and channel characteristics with assumptions made for hydraulic radius using the following rearrangement of Manning's equation:

$$k = (1.49(R)^{0.667})/n$$

where: R = An assumed hydraulic radius

n = Manning's roughness coefficient for open channel flow (see Table III - B.3)

Open Channel Flow

Open channels are assumed to begin where surveyed cross section information has been obtained, where channels are visible on aerial photographs, or where lines indicating streams appear (in blue) on United States Geological Survey (USGS) quadrangle sheets. The k_c values from Table III - B.2 used in the Velocity Equation above or water surface profile information can be used to estimate average flow velocity.

Lakes or Wetlands

This travel time is normally very small and can be assumed as zero. Where significant attenuation may occur due to storage effects, the flows should be routed using a "level pool routing" technique.

Limitations

The following limitations apply in estimating travel time (T_t).

- Manning's kinematic solution should not be used for sheet flow longer than 300 feet.
- In watersheds with storm drains, carefully identify the appropriate hydraulic flow path to estimate Tc.
- Consult a standard hydraulics textbook to determine average velocity in pipes for either pressure or nonpressure flow.
- A culvert or bridge can act as a reservoir outlet if there is significant storage behind it. A hydrograph should be developed to this point and a level pool routing technique used to determine the outflow rating curve through the culvert or bridge.

Design Storm Hyetographs

The standard design hyetograph is the SCS Type 1A 24-hour rainfall distribution, resolved into 6-minute time intervals (see Table III - B.8). Various interpretations of the hyetograph are available and may differ slightly from distributions used in other unit hydrograph based computer simulations. Other distributions will be accepted with adequate justification and as long as they do not increase the allowable release rates.

For project sites with tributary drainage areas above elevation 1,000 feet MSL, an additional total precipitation must be added to the total depth of rainfall for the 25-, 50-, and 100-year design storm events to account for the potential average snow melt which occurs during major storm events.

The MSL "factor" is computed as follows:

$$M_s \text{ (in inches)} = 0.004\,(MB_{el} - 1000)$$

Where: M_s = Rainfall amount to be added to P_r

MB_{el} = The mean tributary basin elevation above sea level (in feet)

Sub-Basin Delineation

Within an overall drainage basin, it may be necessary to delineate separate sub-basins based on similar land uses and/or runoff characteristics or when hydraulically "self-contained" areas are found to exist. When this is necessary, separate hydrographs shall be generated, routed, and recombined, after travel time is considered, into a single hydrograph to represent runoff flows into the quantity or quality control facility.

Hydrograph Phasing Analysis

Where flows from multiple basins or subbasins having different runoff characteristics and/or travel times combine, the design engineer shall sum the hydrographs after shifting each hydrograph according to its travel time to the discharge point of interest. The resultant hydrograph shall be either routed downstream as required in the downstream analysis see (Volume 1 Chapter 3 [Drainage Report section 8]), or routed through the control facility.

Included in this appendix are the 2-, 10-, 25-,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. The Applicant shall use the NOAA Isopluvials for selection of the design storm precipitation.

Rational Method

The only approved use of the Rational Method is for the sizing of conveyance systems. This method is applicable to smaller drainage basins, 25 acres in size or less. This method provides an estimate of peak discharge (Q_p in cubic feet per second [cfs]) using the following formula:

$$Q_p = CIA$$

Where: C = runoff coefficient (unitless),

A = area of watershed (acres), and

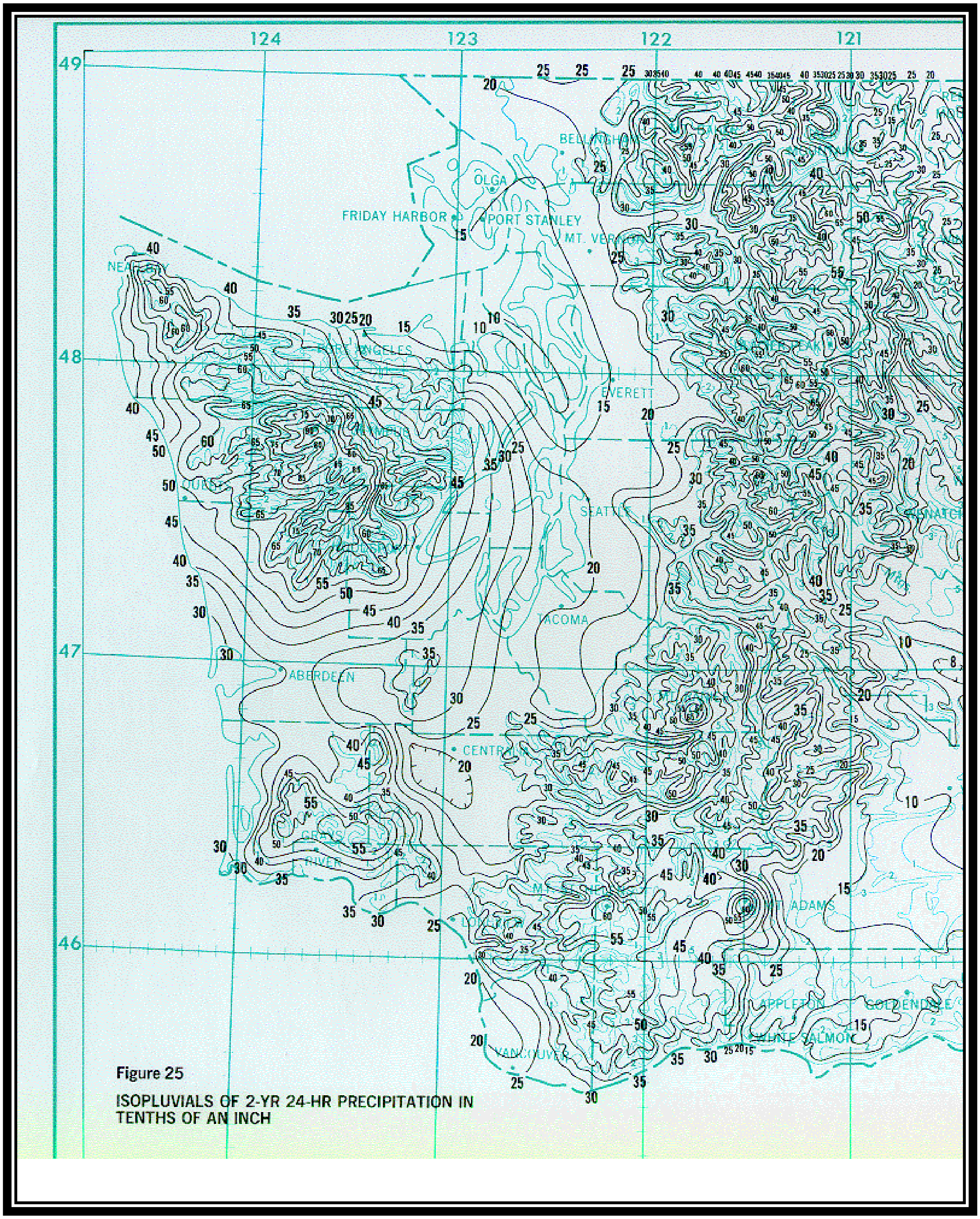
I = rainfall intensity (inches per hour) for a chosen frequency expressed as:

$$I = \frac{m}{(T_c)^n}$$

Where: m, n are regression coefficients (unitless), and

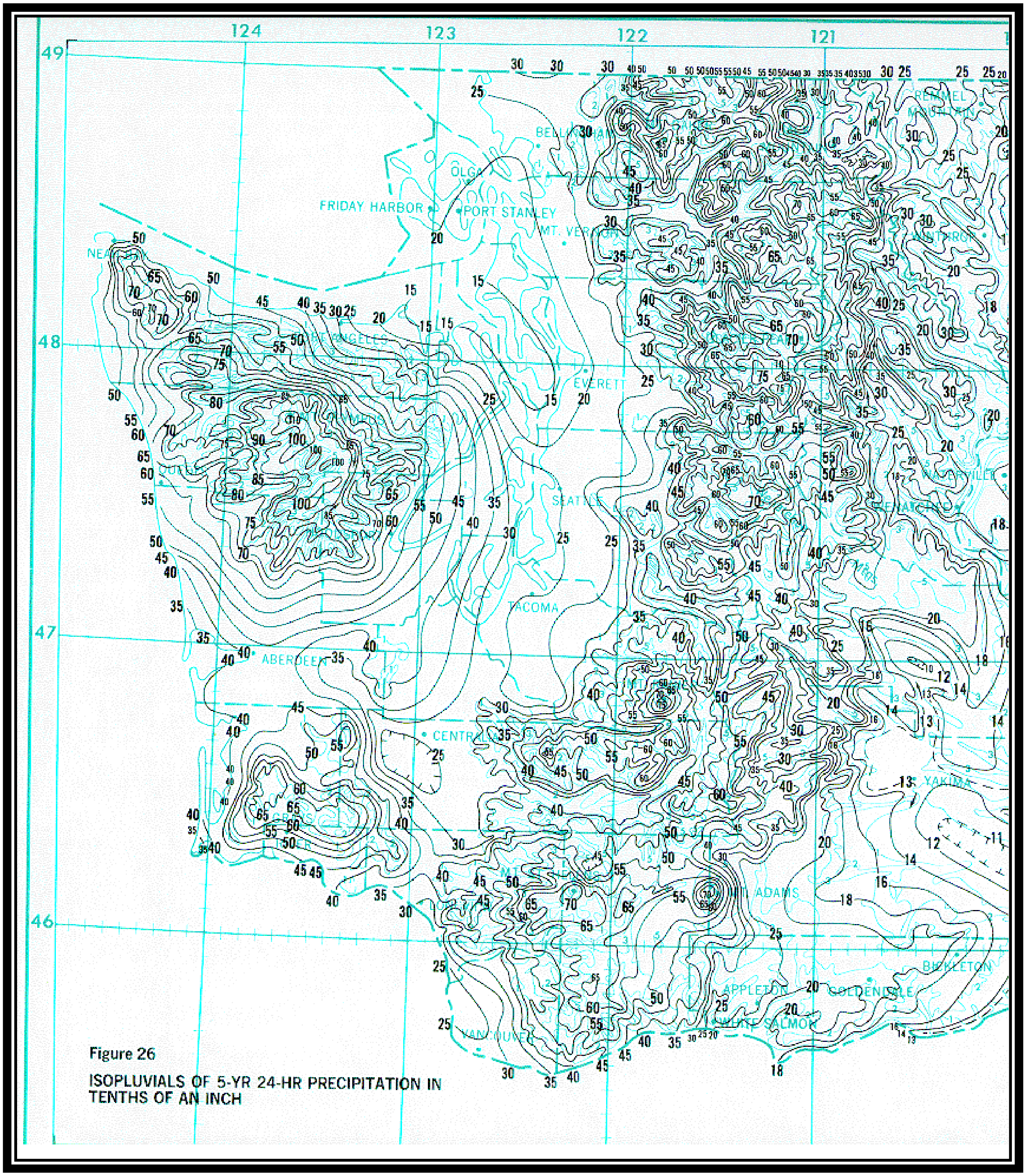
T_c = time of concentration (in hours).

Runoff coefficient (C) values are listed in Table III - B-6 for a range of land cover types. Regression coefficients (m, n) for determining rainfall intensity can be found in Table III - B.7. Time of concentration (T_c) is calculated as described in the Single Event Model Guidance section above.

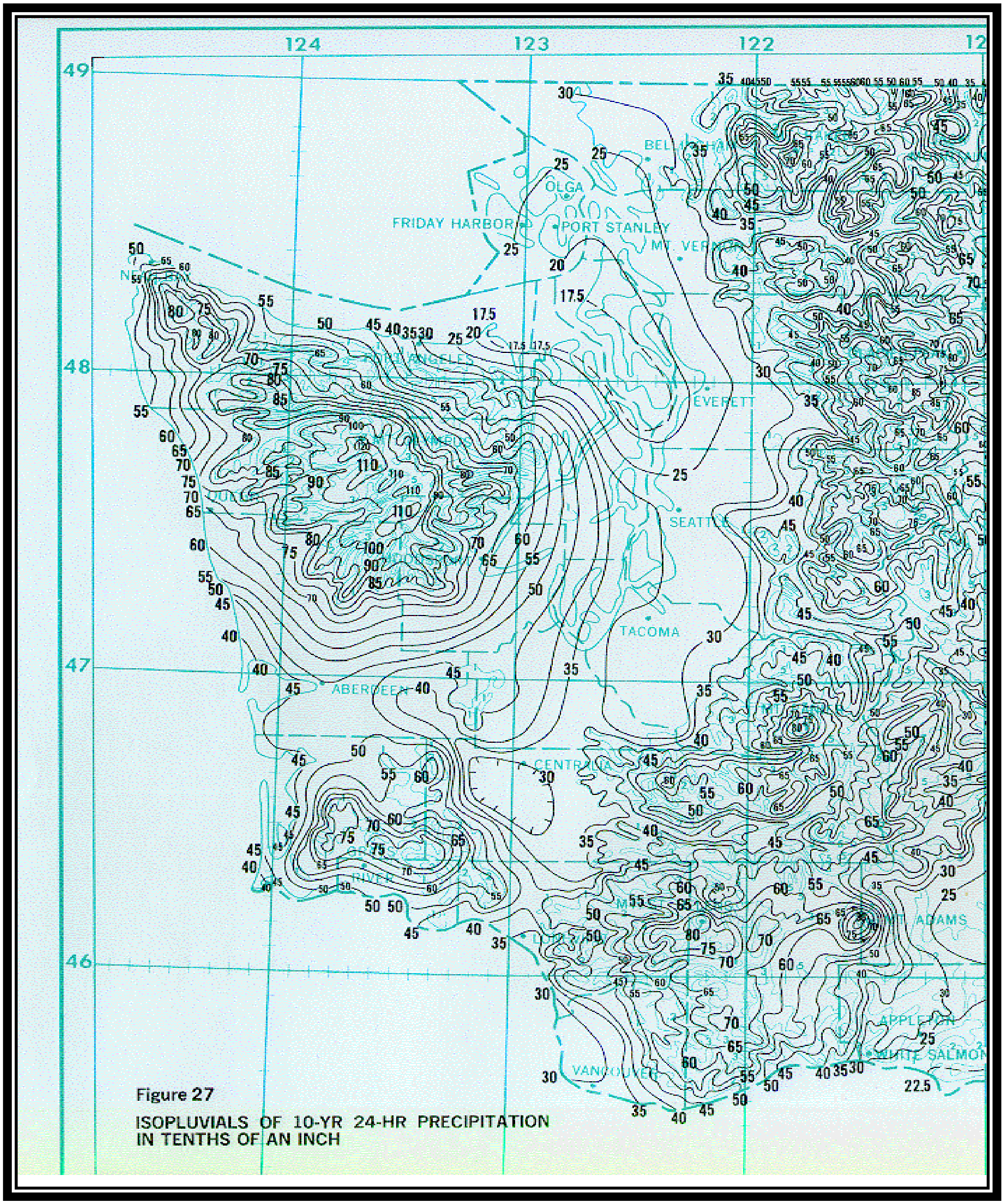


Source: NOAA

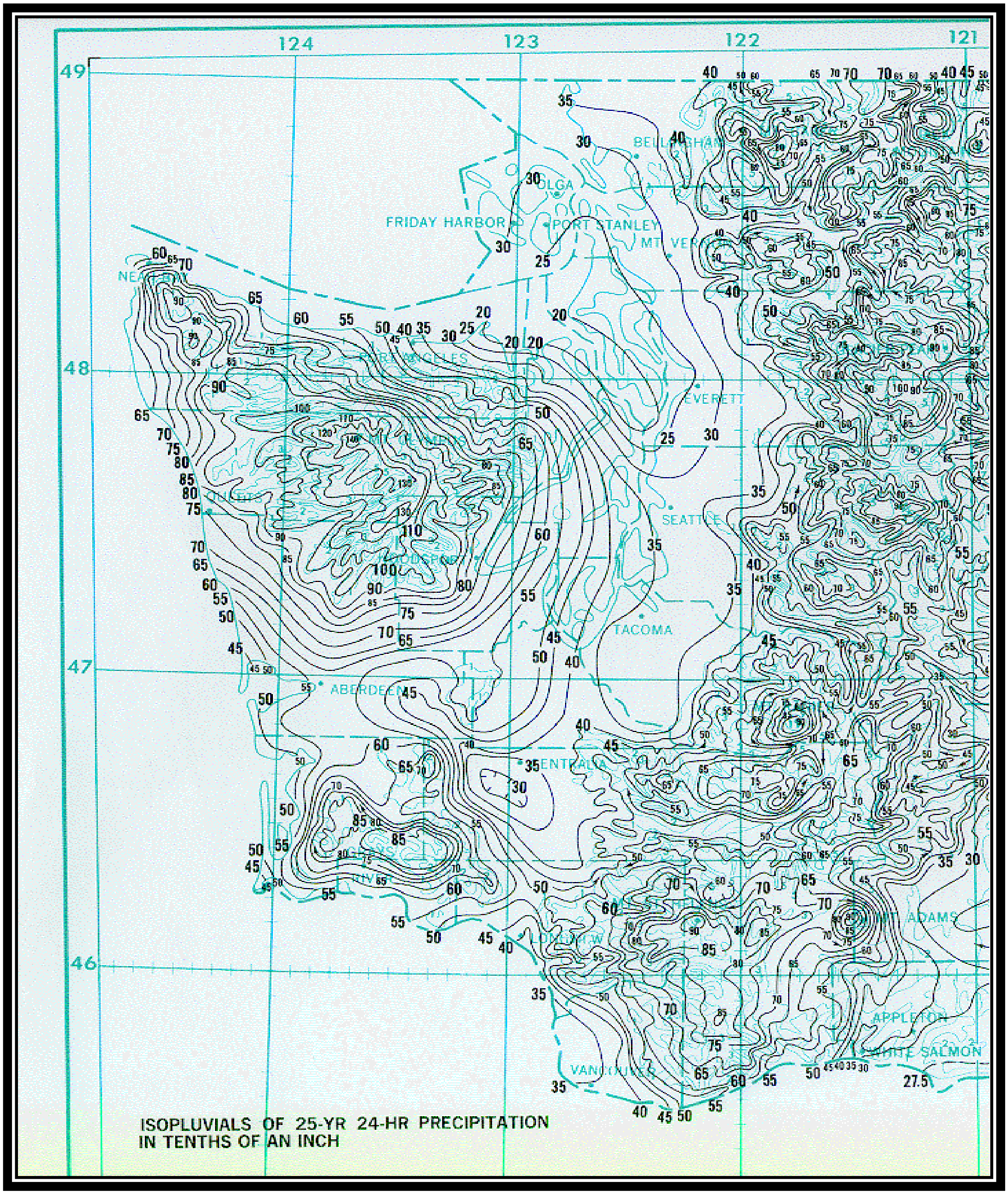
Western Washington Isopluvial 2-year, 24-hour



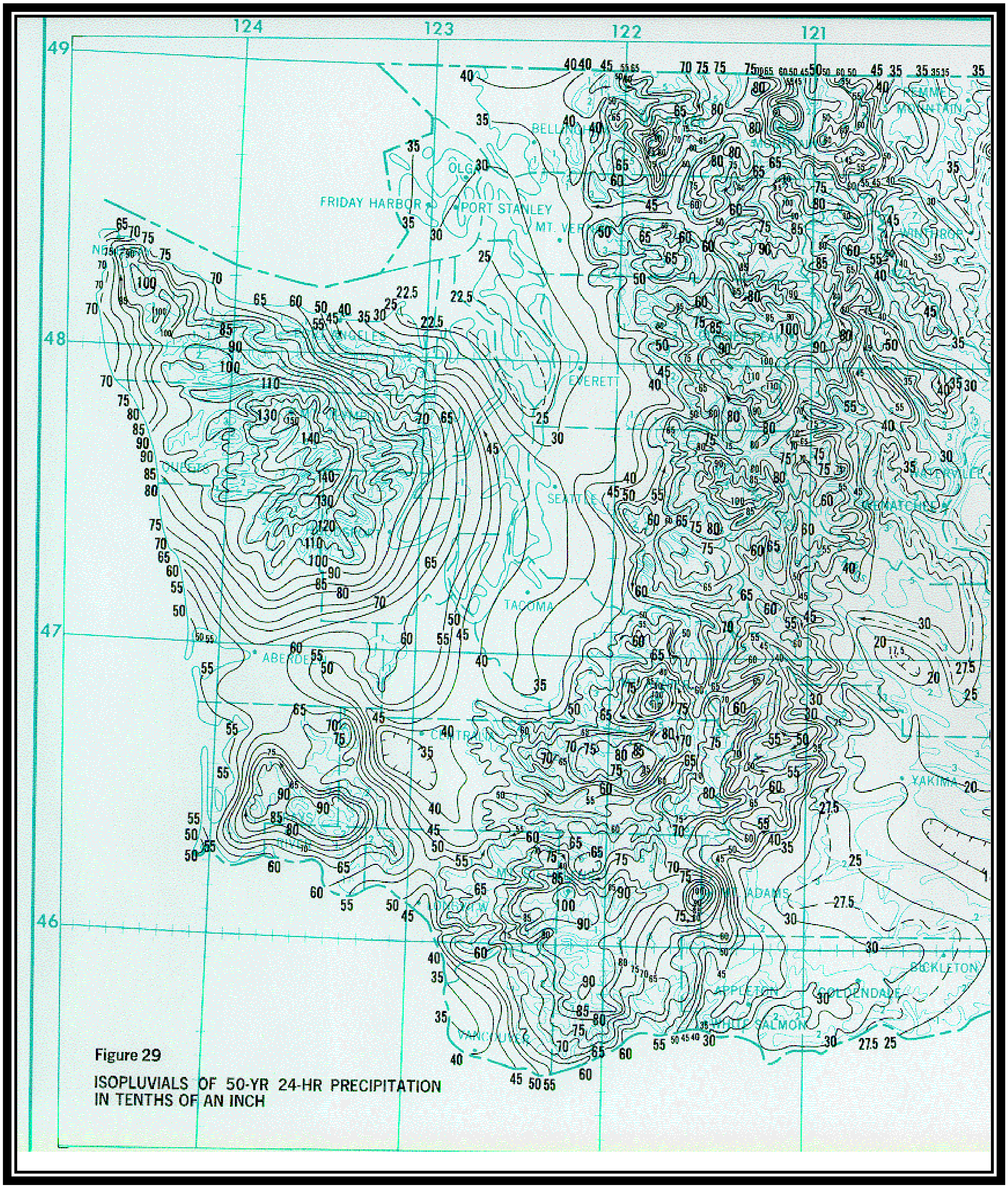
Western Washington Isopluvial 5-year, 24-hour



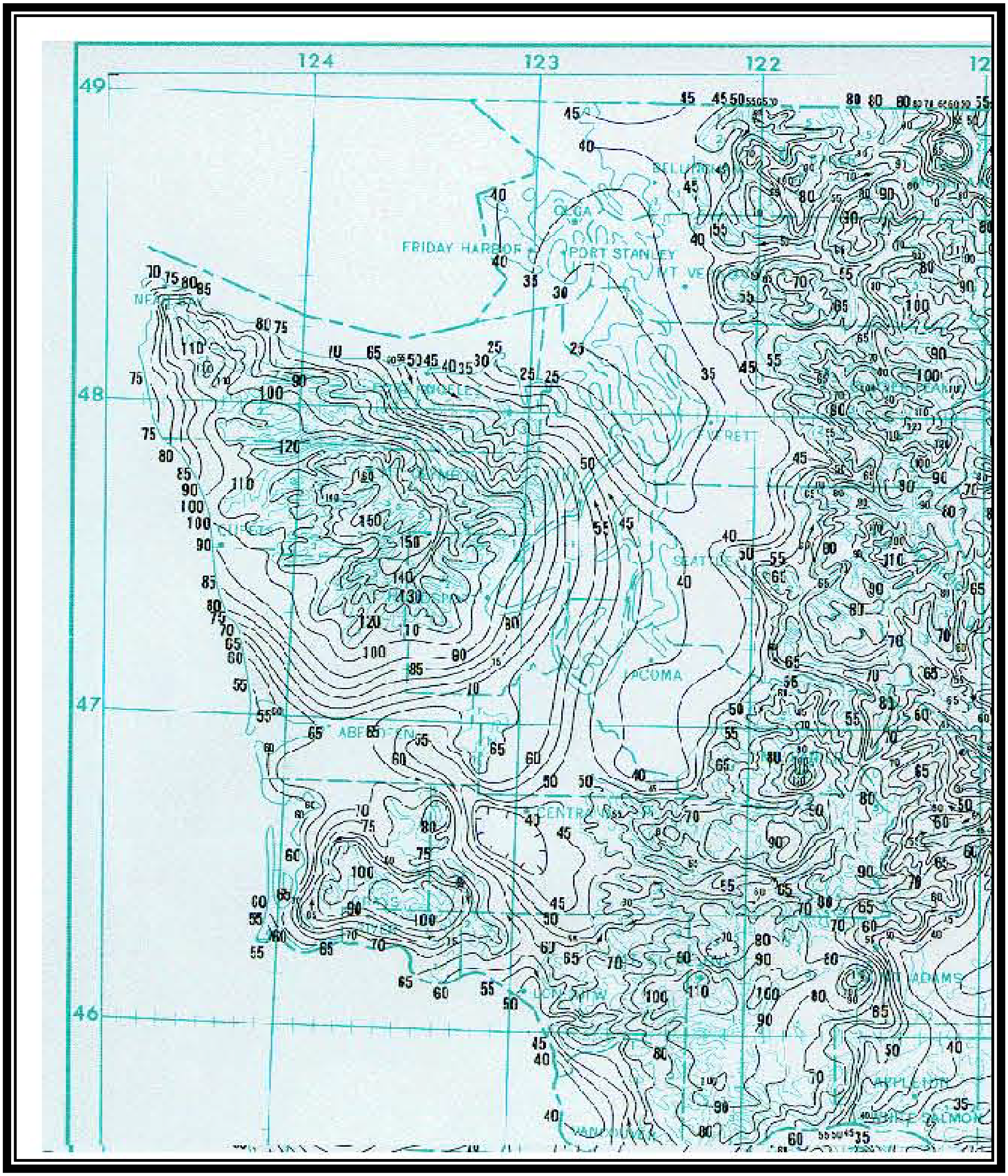
Western Washington Isopluvial 10-year, 24-hourSource: NOAA



Western Washington Isopluvial 25-year, 24-hour



Western Washington Isopluvial 50-year, 24-hour



Western Washington Isopluvial 100-year, 24-hour

Table III - B.2 "n" and "k" Values Used in Time Calculations for Hydrographs

"n_s" Sheet Flow Equation Manning's Values (for the initial 300 ft. of travel)	n_s *
Smooth surfaces (concrete, asphalt, gravel, or bare hand packed soil)	0.011
Fallow fields or loose soil surface (no residue)	0.05
Cultivated soil with residue cover ($s \leq 0.20$ ft/ft)	0.06
Cultivated soil with residue cover ($s > 0.20$ ft/ft)	0.17
Short prairie grass and lawns	0.15
Dense grasses	0.24
Bermuda grass	0.41
Range (natural)	0.13
Woods or forest with light underbrush	0.40
Woods or forest with dense underbrush	0.80

*Manning values for sheet flow only, from Overton and Meadows 1976 (See TR-55, 1986)

"k" Values Used in Travel Time/Time of Concentration Calculations

Shallow Concentrated Flow (After the initial 300 ft. of sheet flow, R = 0.1)	k_s
1. Forest with heavy ground litter and meadows (n = 0.10)	3
2. Brushy ground with some trees (n = 0.060)	5
3. Fallow or minimum tillage cultivation (n = 0.040)	8
4. High grass (n = 0.035)	9
5. Short grass, pasture and lawns (n = 0.030)	11
6. Nearly bare ground (n = 0.025)	13
7. Paved and gravel areas (n = 0.012)	27

Channel Flow (intermittent) (At the beginning of visible channels R = 0.2)	k_c
1. Forested swale with heavy ground litter (n = 0.10)	5
2. Forested drainage course/ravine with defined channel bed (n = 0.050)	10
3. Rock-lined waterway (n = 0.035)	15
4. Grassed waterway (n = 0.030)	17
5. Earth-lined waterway (n = 0.025)	20
6. CMP pipe (n = 0.024)	21
7. Concrete pipe (0.012)	42
8. Other waterways and pipe	0.508/n

Channel Flow (Continuous stream, R = 0.4)	k_c
9. Meandering stream with some pools (n = 0.040)	20
10. Rock-lined stream (n = 0.035)	23
11. Grass-lined stream (n = 0.030)	27
12. Other streams, man-made channels and pipe n** determined from Table III - B.3	0.807/n**

Ref: DOE Stormwater Management Manual for the Puget Sound Basin, February 1992.

Table III - B.3 Values of the Roughness Coefficient, "n"

Type of Channel and Description	Manning's "n"	Type of Channel and Description	Manning's "n"
A. Constructed Channels		6. Sluggish reaches, weedy deep pools	0.070
a. Earth, straight and uniform		7. Very weedy reaches, deep pools, or floodways with heavy stand of timber and underbrush	0.100
1. Clean, recently completed	0.018	b. Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stages	
2. Gravel, uniform section, clean	0.025	1. Bottom: gravel, cobbles, and few boulders	0.040
3. With short grass, few weeds	0.027	2. Bottom: cobbles with large boulders	0.050
b. Earth, winding and sluggish	0.025	B-2 Flood plains	
1. No vegetation	0.025	a. Pasture, no brush	
2. Grass, some weeds	0.030	1. Short grass	0.030
3. Dense weeds or aquatic plants in deep channels	0.035	2. High grass	0.035
4. Earth bottom and rubble sides	0.030	b. Cultivated areas	
5. Stony bottom and weedy banks	0.035	1. No crop	0.030
6. Cobble bottom and clean sides	0.040	2. Mature row crops	0.035
c. Rock lined		3. Mature field crops	0.040
1. Smooth and uniform	0.035	c. Brush	
2. Jagged and irregular	0.040	1. Scattered brush, heavy weeds	0.050
d. Channels not maintained, weeds and brush uncut		2. Light brush and trees	0.060
1. Dense weeds, high as flow depth	0.080	3. Medium to dense brush	0.070
2. Clean bottom, brush on sides	0.050	4. Heavy, dense brush	0.100
3. Same as above, highest stage of flow	0.070	d. Trees	
4. Dense brush, high stage	0.100	1. Dense willows, straight	0.150
B. Natural Streams		2. Cleared land with tree stumps, no sprouts	0.040
B-1 Minor streams (top width at flood stage < 100 ft.)		3. Same as above, but with heavy growth of sprouts	0.060
a. Streams on plain		4. Heavy stand of timber, a few down trees, little undergrowth, flood stage below branches	0.100
1. Clean, straight, full stage no rifts or deep pools	0.030	5. Same as above, but with flood stage reaching branches	0.120
2. Same as above, but more stones and weeds	0.035		
3. Clean, winding, some pools and shoals	0.040		
4. Same as above, but some weeds	0.040		
5. Same as 4, but more stones	0.050		

Ref: DOE Stormwater Management Manual for the Puget Sound Basin, February 1992.

Table III - B.4 Runoff Curve Numbers for Selected Agricultural, Suburban, and Urban Areas

(Source: Stormwater Management Manual for Western Washington, 2019.)				
	CNs for hydrologic soil group			
Cover type and hydrologic condition.	A	B	C	D
Curve Numbers for Pre-Development Conditions				
Pasture, grassland, or range-continuous forage for grazing:				
Fair condition (ground cover 50% to 75% and not heavily grazed).	49	69	79	84
Good condition (ground cover >75% and lightly or only occasionally grazed)	39	61	74	80
Woods:				
Fair (Woods are grazed but not burned, and some forest litter covers the soil).	36	60	73	79
Good (Woods are protected from grazing, and litter and brush adequately cover the soil).	30	55	70	77
Curve Numbers for Post-Development Conditions				
Open space (lawns, parks, golf courses, cemeteries, landscaping, etc.)[1]				
Fair condition (grass cover on 50% - 75% of the area).	77	85	90	92
Good condition (grass cover on >75% of the area)	68	80	86	90
Impervious areas:				
Open water bodies: lakes, wetlands, ponds etc.	100	100	100	100
Paved parking lots, roofs[2], driveways, etc. (excluding right-of-way)	98	98	98	98
Paved	98	98	98	98
Gravel (including right-of-way)	76	85	89	91
Dirt (including right-of-way)	72	82	87	89
Permeable Pavement (See Volume V to decide which condition below to use)				
Landscaped area	77	85	90	92
50% landscaped area/50% impervious	87	91	94	96
100% impervious area	98	98	98	98
Pasture, grassland, or range-continuous forage for grazing:				
Poor condition (ground cover <50% or heavily grazed with no mulch).	68	79	86	89
Fair condition (ground cover 50% to 75% and not heavily grazed).	49	69	79	84
Good condition (ground cover >75% and lightly or only occasionally grazed)	39	61	74	80
Woods:				
Poor (Forest litter, small trees, and brush are destroyed by heavy grazing or regular burning). 83		45	66	77
Fair (Woods are grazed but not burned, and some forest litter covers the soil).	36	60	73	79
Good (Woods are protected from grazing, and litter and brush adequately cover the soil).	30	55	70	77
Single family residential[3]**:** Dwelling Unit/Gross Acre	Should only be used for subdivisions > 50 acres	Average Percent impervious area[3,4]		
1.0 DU/GA		15	Separate curve number shall be selected for pervious & impervious portions of the site or basin	
1.5 DU/GA		20		
2.0 DU/GA		25		
2.5 DU/GA		30		
3.0 DU/GA		34		
3.5 DU/GA		38		
4.0 DU/GA		42		
4.5 DU/GA		46		
5.0 DU/GA		48		
5.5 DU/GA		50		
6.0 DU/GA		52		
6.5 DU/GA		54		
7.0 DU/GA		56		
7.5 DU/GA		58		
PUDs, condos, apartments, commercial business, industrial areas & subdivisions < 50 acres:				
% impervious must be computed	Separate curve numbers shall be selected for pervious and impervious portions of the site			
For a more detailed and complete description of land use curve numbers refer to chapter two (2) of the Soil Conservation Service's Technical Release No. 55 , (210-VI-TR-55, Second Ed., June 1986).				

[1] Composite CNs may be computed for other combinations of open space cover type.

[2]Where roof runoff and driveway runoff are infiltrated or dispersed according to the requirements in Volume V, the average percent impervious area may be adjusted in accordance with the procedure described in LID.04: Downspout Infiltration Systems, LID.05: Downspout Dispersion Systems, and LID.11: Full Dispersion" .
[3]Assumes roof and driveway runoff is directed into street/storm system.
[4]All the remaining pervious area (lawn) are considered to be in good condition for these curve numbers.

Table III - B.5 Major Soil Groups in Thurston County

Soil Type *	Hydrologic Soil Group	Soil Type *	Hydrologic Soil Group
ALDERWOOD	C	MUKILTEO	C/D
BALDHILL	B	NEWBERG	B
BAUMGARD	B	NISQUALLY	B
BELLINGHAM	C	NORMA	D
BOISTFORT	B	OLYMPIC	B
BUNKER	B	PHEENEY	C
CAGEY	C	PILCHUCK	C
CATHCART	B	PITS	*
CENTRALIA	B	PRATHER	C
CHEHALIS	B	PUGET	D
DELPHI	B	PUYALLUP	B
DUPONT	D	RAINIER	C
DYSTRIC XEROCHREPTS	C	ROCK OUTCROP	*
ELD	B	RAUGHT	B
EVERETT	A	RIVERWASH	D
EVERSON	D	SALKUM	B
GALVIN	D	SCAMMAN	D
GILES	B	SCHNEIDER	B
GODFREY	D	SEMIAHMOO	C
GROVE	A	SHALCAR	D
HOOGDAL	C	SHALCAR VARIANT	D
HYDRAQUENTS	D	SKIPOPA	D
INDIANOLA	A	SPANA	D
JONAS	B	SPANAWAY	B
KAPOWSIN	D	SULTON	C
KATULAS	C	TACOMA	D
LATES	C	TENINO	C
MAL	C	TISCH	D
MASHEL	B	VAILTON	B
MAYTOWN	C	WILKESON	B
MCKENNA	D	XERORTHENTS	C
MELBOURNE	B	YELM	C

**See the description of the map unit*
Soils Table Notes:
Hydrologic Soil Group Classifications, as Defined by the NRCS (formerly Soil Conservation Service):
Note: If there is a discrepancy between this table and the NRCS website, the classification on the NRCS website shall prevail.

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.).*

** = From NRCS Database for Thurston surveys, SCS, TR-55, Second Edition, June 1986, Exhibit A-1. Revisions made from SCS, Soil Interpretation Record, Form #5, September 1988 and various county soil surveys.*

Table III - B.6. Runoff Coefficients for Rational Method Calculations.

Type of Cover	Flat	Rolling (2%-10%)	Hilly Over 10%)
Pavement and Roofs	0.90	0.90	0.90
Earth Shoulders	0.50	0.50	0.50
Drives and Walks	0.75	0.80	0.85
Gravel Pavement	0.50	0.55	0.60
City Business Areas	0.80	0.85	0.85
Suburban Residential	0.25	0.35	0.40
Single Family Residential	0.30	0.40	0.50
Multi Units, Detached	0.40	0.50	0.60
Multi Units, Attached	0.60	0.65	0.70
Lawns, Very Sandy Soil	0.05	0.07	0.10
Lawns, Sandy Soil	0.10	0.15	0.20
Lawns, Heavy Soil	0.17	0.22	0.35
Grass Shoulders	0.25	0.25	0.25
Side Slopes, Earth	0.60	0.60	0.60
Side Slopes, Turf	0.30	0.30	0.30
Median Areas, Turf	0.25	0.30	0.30
Cultivated Land, Clay and Loam	0.50	0.55	0.60
Cultivated Land, Sand and Gravel	0.25	0.30	0.35
Industrial Areas, Light	0.50	0.70	0.80
Industrial Areas, Heavy	0.60	0.80	0.90
Parks and Cemeteries	0.10	0.15	0.25
Playgrounds	0.20	0.25	0.30
Woodland and Forests	0.10	0.15	0.20
Meadows and Pasture Land	0.25	0.30	0.35
Pasture with Frozen Ground	0.40	0.45	0.50
Unimproved Areas	0.10	0.20	0.30

Source: WSDOT Hydraulics Manual (2007)

Table III - B.7 Regression Coefficients for Rational Method Calculations.

	2-year MRI		**5-year MRI**		**10-year MRI**		**25-year MRI**		**50-year MRI**		**100-year MRI**	
Location	**m**	**n**	**m**	**n**	**m**	**n**	**m**	**n**	**m**	**n**	**m**	**n**
Olympia	3.82	0.466	4.86	0.472	5.62	0.474	6.63	0.477	7.40	0.478	8.17	0.480
Centralia and Chehalis	3.63	0.506	4.85	0.518	5.76	0.524	7.00	0.530	7.92	0.533	8.86	0.537
Tacoma	3.57	0.516	4.78	0.527	5.70	0.533	6.93	0.539	7.86	0.542	8.79	0.545

Source: WSDOT Hydraulics Manual (2007).
MRI: Mean Recurrence Interval (frequency).

Table III - B.8. SCS Type IA Storm Rainfall Distribution, 6-minute intervals.

Time (hours)	Incremental Rainfall	Cumulative Rainfall	Time (hours)	Incremental Rainfall	Cumulative Rainfall
0	0	0	3.8	0.004	0.109
0.1	0.002	0.002	3.9	0.003	0.112
0.2	0.002	0.004	4	0.004	0.116
0.3	0.002	0.006	4.1	0.004	0.12
0.4	0.002	0.008	4.2	0.003	0.123
0.5	0.002	0.01	4.3	0.004	0.127
0.6	0.002	0.012	4.4	0.004	0.131
0.7	0.002	0.014	4.5	0.004	0.135
0.8	0.002	0.016	4.6	0.004	0.139
0.9	0.002	0.018	4.7	0.004	0.143
1	0.002	0.02	4.8	0.004	0.147
1.1	0.003	0.023	4.9	0.005	0.152
1.2	0.003	0.026	5	0.004	0.156
1.3	0.003	0.029	5.1	0.005	0.161
1.4	0.003	0.032	5.2	0.004	0.165
1.5	0.003	0.035	5.3	0.005	0.17
1.6	0.003	0.038	5.4	0.005	0.175
1.7	0.003	0.041	5.5	0.005	0.18
1.8	0.003	0.044	5.6	0.005	0.185
1.9	0.003	0.047	5.7	0.005	0.19
2	0.003	0.05	5.8	0.005	0.195
2.1	0.003	0.053	5.9	0.005	0.2
2.2	0.003	0.056	6	0.006	0.206
2.3	0.004	0.06	6.1	0.006	0.212
2.4	0.003	0.063	6.2	0.006	0.218
2.5	0.003	0.066	6.3	0.006	0.224
2.6	0.003	0.069	6.4	0.007	0.231
2.7	0.003	0.072	6.5	0.006	0.237
2.8	0.004	0.076	6.6	0.006	0.243
2.9	0.003	0.079	6.7	0.006	0.249
3	0.003	0.082	6.8	0.006	0.255
3.1	0.003	0.085	6.9	0.006	0.261
3.2	0.003	0.088	7	0.007	0.268
3.3	0.003	0.091	7.1	0.007	0.275
3.4	0.004	0.095	7.2	0.008	0.283
3.5	0.003	0.098	7.3	0.008	0.291
3.6	0.003	0.101	7.4	0.009	0.3
3.7	0.004	0.105	7.5	0.01	0.31

Time	Incremental Rainfall	Cumulative Rainfall	Time	Incremental Rainfall	Cumulative Rainfall
(hours)			(hours)		
7.6	0.021	0.331	11.4	0.004	0.641
7.7	0.024	0.355	11.5	0.004	0.645
7.8	0.024	0.379	11.6	0.004	0.649
7.9	0.024	0.403	11.7	0.004	0.653
8	0.022	0.425	11.8	0.004	0.657
8.1	0.014	0.439	11.9	0.003	0.66
8.2	0.013	0.452	12	0.004	0.664
8.3	0.01	0.462	12.1	0.004	0.668
8.4	0.01	0.472	12.2	0.003	0.671
8.5	0.008	0.48	12.3	0.004	0.675
8.6	0.009	0.489	12.4	0.004	0.679
8.7	0.009	0.498	12.5	0.004	0.683
8.8	0.007	0.505	12.6	0.004	0.687
8.9	0.008	0.513	12.7	0.003	0.69
9	0.007	0.52	12.8	0.004	0.694
9.1	0.007	0.527	12.9	0.003	0.697
9.2	0.006	0.533	13	0.004	0.701
9.3	0.006	0.539	13.1	0.004	0.705
9.4	0.006	0.545	13.2	0.003	0.708
9.5	0.005	0.55	13.3	0.004	0.712
9.6	0.006	0.556	13.4	0.004	0.716
9.7	0.005	0.561	13.5	0.003	0.719
9.8	0.006	0.567	13.6	0.003	0.722
9.9	0.005	0.572	13.7	0.004	0.726
10	0.005	0.577	13.8	0.003	0.729
10.1	0.005	0.582	13.9	0.004	0.733
10.2	0.005	0.587	14	0.003	0.736
10.3	0.005	0.592	14.1	0.003	0.739
10.4	0.004	0.596	14.2	0.004	0.743
10.5	0.005	0.601	14.3	0.003	0.746
10.6	0.005	0.606	14.4	0.003	0.749
10.7	0.004	0.61	14.5	0.004	0.753
10.8	0.005	0.615	14.6	0.003	0.756
10.9	0.005	0.62	14.7	0.003	0.759
11	0.004	0.624	14.8	0.004	0.763
11.1	0.004	0.628	14.9	0.003	0.766
11.2	0.005	0.633	15	0.003	0.769
11.3	0.004	0.637	15.1	0.003	0.772

Time	Incremental Rainfall	Cumulative Rainfall	Time	Incremental Rainfall	Cumulative Rainfall
(hours)			(hours)		
15.2	0.004	0.776	19	0.003	0.887
15.3	0.003	0.779	19.1	0.003	0.89
15.4	0.003	0.782	19.2	0.002	0.892
15.5	0.003	0.785	19.3	0.003	0.895
15.6	0.003	0.788	19.4	0.002	0.897
15.7	0.004	0.792	19.5	0.003	0.9
15.8	0.003	0.795	19.6	0.003	0.903
15.9	0.003	0.798	19.7	0.002	0.905
16	0.003	0.801	19.8	0.003	0.908
16.1	0.003	0.804	19.9	0.002	0.91
16.2	0.003	0.807	20	0.003	0.913
16.3	0.003	0.81	20.1	0.002	0.915
16.4	0.003	0.813	20.2	0.003	0.918
16.5	0.003	0.816	20.3	0.002	0.92
16.6	0.003	0.819	20.4	0.002	0.922
16.7	0.003	0.822	20.5	0.003	0.925
16.8	0.003	0.825	20.6	0.002	0.927
16.9	0.003	0.828	20.7	0.003	0.93
17	0.003	0.831	20.8	0.002	0.932
17.1	0.003	0.834	20.9	0.002	0.934
17.2	0.003	0.837	21	0.003	0.937
17.3	0.003	0.84	21.1	0.002	0.939
17.4	0.003	0.843	21.2	0.002	0.941
17.5	0.003	0.846	21.3	0.003	0.944
17.6	0.003	0.849	21.4	0.002	0.946
17.7	0.002	0.851	21.5	0.002	0.948
17.8	0.003	0.854	21.6	0.003	0.951
17.9	0.003	0.857	21.7	0.002	0.953
18	0.003	0.86	21.8	0.002	0.955
18.1	0.003	0.863	21.9	0.002	0.957
18.2	0.002	0.865	22	0.002	0.959
18.3	0.003	0.868	22.1	0.003	0.962
18.4	0.003	0.871	22.2	0.002	0.964
18.5	0.003	0.874	22.3	0.002	0.966
18.6	0.002	0.876	22.4	0.002	0.968
18.7	0.003	0.879	22.5	0.002	0.97
18.8	0.003	0.882	22.6	0.002	0.972
18.9	0.002	0.884	22.7	0.002	0.974

Time	Incremental Rainfall	Cumulative Rainfall			
(hours)					
22.8	0.002	0.976			
22.9	0.002	0.978			
23	0.002	0.98			
23.1	0.002	0.982			
23.2	0.002	0.984			
23.3	0.002	0.986			
23.4	0.002	0.988			
23.5	0.002	0.99			
23.6	0.002	0.992			
23.7	0.002	0.994			
23.8	0.002	0.996			
23.9	0.002	0.998			
24	0.002	1			

Appendix III-C – Nomographs for Culvert Sizing Needs

Figure III - C.1. Headwater Depth for Smooth Interior Pipe Culverts with Inlet Control.

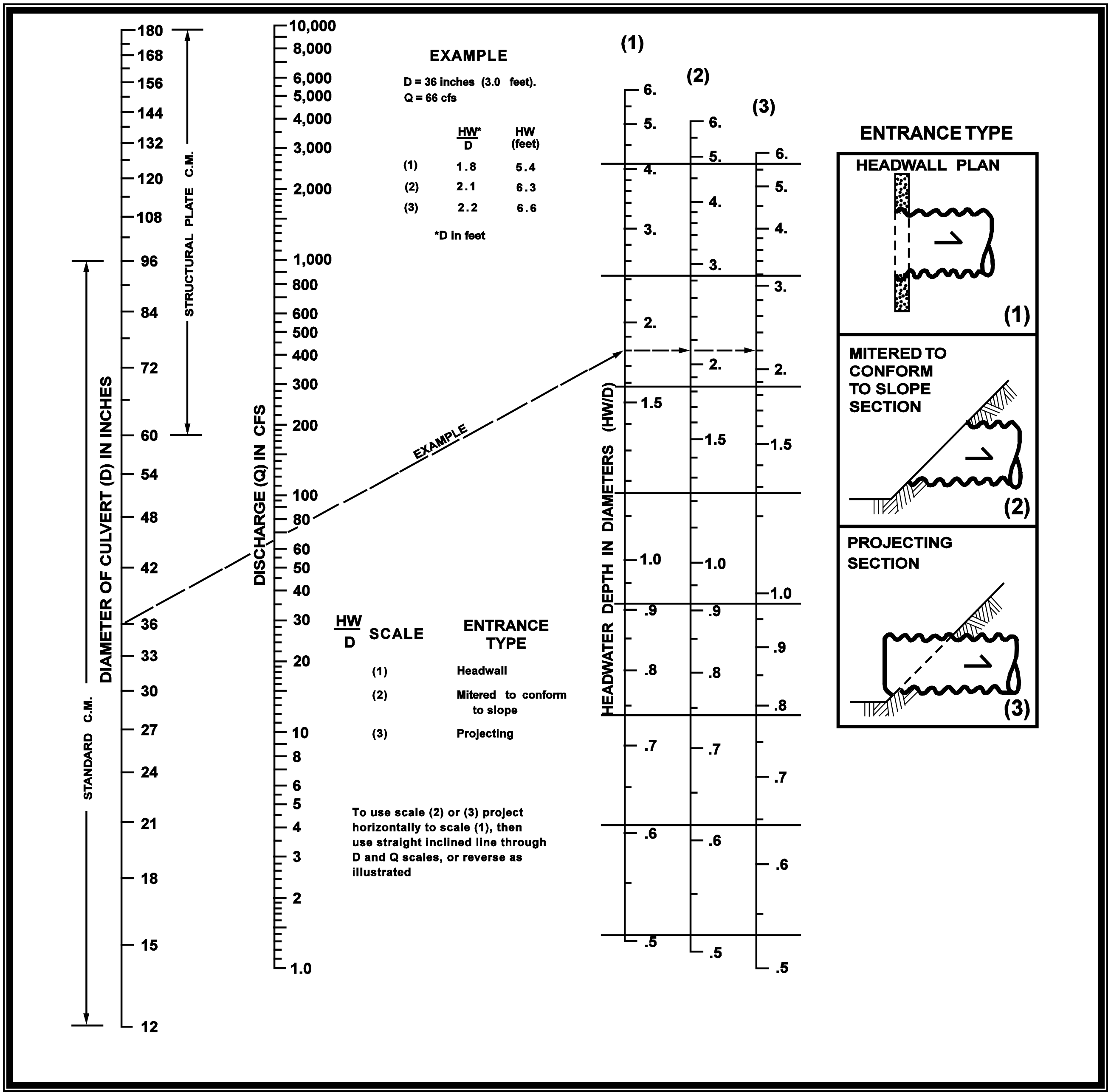


Figure III - C.2 Headwater Depth for Corrugated Pipe Culverts with Inlet Control.

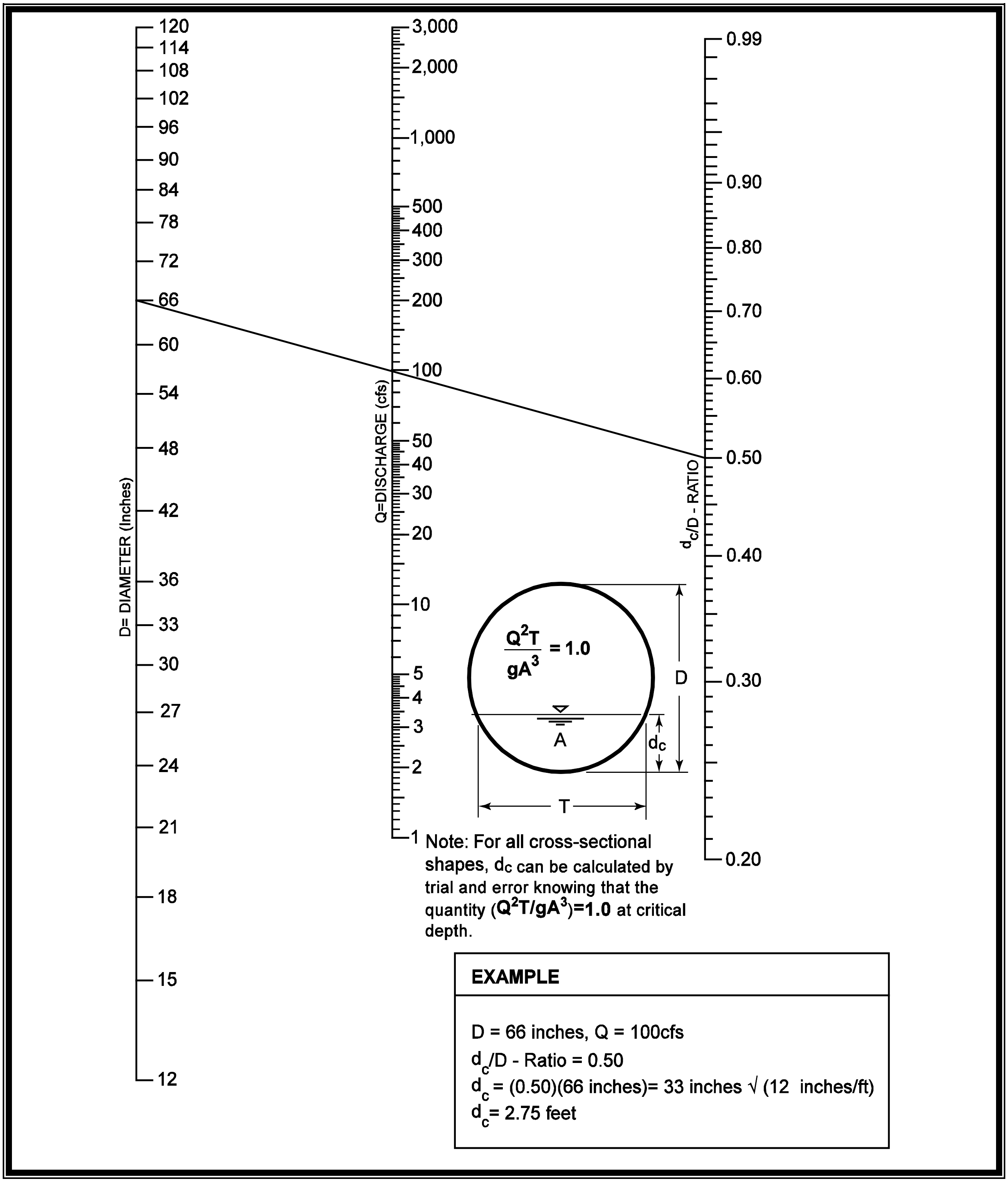


Figure III - C.3 Critical Depth of Flow for Circular Culverts.

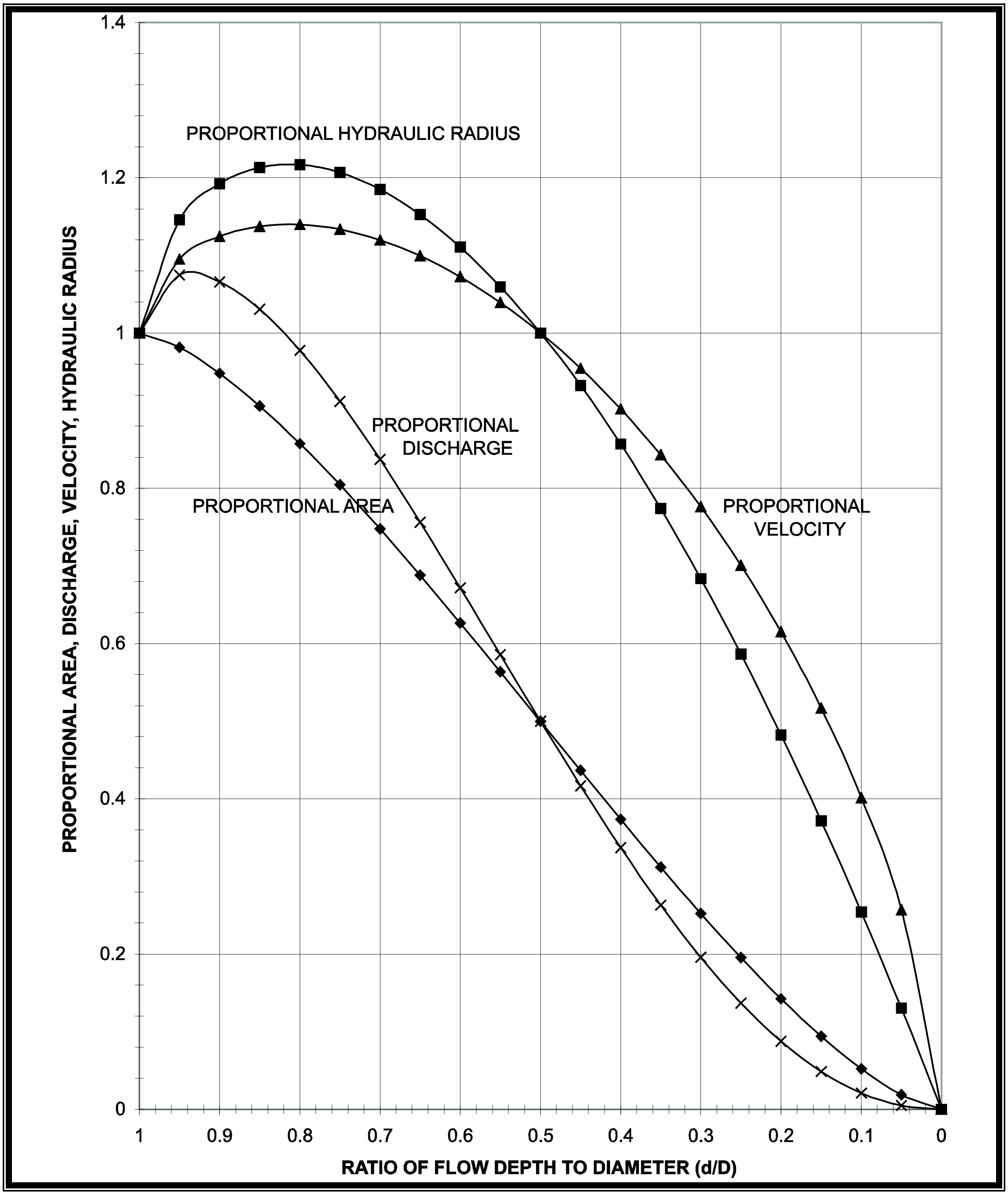


Figure III - C.4 Circular Channel Ratios.

Appendix III-D – On-site Stormwater Management BMP Infeasibility Criteria

The following tables present infeasibility criteria that can be used to justify not using various on-site stormwater management BMPs for consideration in the List #1, List #2, or List #3 option of Core Requirement #5. This information is also included under the detailed descriptions of each BMP, but is provided here in this appendix for additional clarity and efficiency. Where any inconsistencies or lack of clarity exists, the requirements in the main text of each volume shall be applied. If a project is limited by one or more of the infeasibility criteria specified below, but still wishes to use the given BMP, they may propose a functionally equivalent design to the County for review and approval.

Lawn and Landscaped Areas	
BMP	**Infeasibility Criteria**
Post-Construction Soil and Depth	• Structural and Engineered soils on slopes, cuts or fill areas where a geotechnical engineer has recommended alternative soil restoration methods. • Site setbacks and design criteria provided in Volume V, Appendix E cannot be achieved.
Roofs	
BMP	**Infeasibility Criteria**
Full Dispersion (See Downspout Dispersion Systems)	

Bioretention or Rain Gardens	• Note: criteria with setback distances are as measured from the bottom edge of the bioretention soil mix.
	• Site setbacks provided in Volume V, Appendix E cannot be achieved.
	• Citation of any of the following infeasibility criteria must be based on an evaluation of site-specific conditions and a written recommendation from an appropriate licensed professional (e.g., engineer, geologist, hydrogeologist):
	  ◦ Where professional geotechnical evaluation recommends infiltration not be used due to reasonable concerns about erosion, slope failure, or downgradient flooding.
	  ◦ Within 50 feet from the top of slopes that are greater than 20% and over 10 feet of vertical relief.

	• In accordance with TCC 24 limitations may exist and reports may be required when bioretention area is within a Landslide Hazard Area or a Marine Bluff Hazard Area. • Where the only area available for siting would threaten the safety or reliability of pre-existing underground utilities, pre-existing underground storage tanks, pre-existing structures, or pre-existing road or parking lot surfaces. • Where the only area available for siting does not allow for a safe overflow pathway to stormwater drainage system or private storm sewer system. • Where there is a lack of usable space for bioretention areas at re-development sites, or where there is insufficient space within the existing public right-of-way on public road projects. • Where infiltrating water would threaten existing below grade basements or building foundations. • Where infiltrating water would threaten shoreline structures such as bulkheads. The following criteria can be cited as reasons for infeasibility without further justification (though some require professional services to make the observation): • Where they are not compatible with surrounding drainage system as determined by the county (e.g., project drains to an existing stormwater collection system whose elevation or location precludes connection to a properly functioning bioretention area).
Bioretention or Rain Gardens (continued)	• Where land for bioretention is within a Geologic Hazard Area or associated buffer (as defined by TCC Title 17 or Title 24). • Within setbacks provided in Section 3.4.6. • Where the site cannot be reasonably designed to locate bioretention areas on slopes less than 8 percent. • For properties with known soil or groundwater contamination (typically federal Superfund sites or state cleanup sites under the Model Toxics Control Act (MTCA)): o Within 100 feet of an area known to have deep soil contamination. o Where groundwater modeling indicates infiltration will likely increase or change the direction of the migration of pollutants in the groundwater. o Wherever surface soils have been found to be contaminated unless those soils are removed within 10 horizontal feet from the infiltration area. o Any area where these facilities are prohibited by an approved

	cleanup plan under the state Model Toxics Control Act or Federal Superfund Law, or an environmental covenant under Chapter 64.70 RCW. • Within 100 feet of a closed or active landfill or a drinking water supply well. • Within 10 feet of small on-site sewage disposal drainfield, including reserve areas, and grey water reuse systems (per WAC 246-272A-0210). This requirement may be modified by the Thurston County Health Department 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. For setbacks from a "large on-site sewage disposal system", see Chapter 246-272B WAC. • Within 10 feet of an underground storage tank and connecting underground pipes when the capacity of the tank and pipe system is 1100 gallons or less. (As used in these criteria, an underground storage tank means any tank used to store petroleum products, chemicals, or liquid hazardous wastes of which 10 percent or more of the storage volume (including volume in the connecting piping system) is beneath the ground surface.
Bioretention or Rain Gardens (continued)	• Where field testing indicates potential bioretention/rain garden sites have a measured (a.k.a., initial) native soil saturated hydraulic conductivity less than 0.30 inches per hour. A small-scale or large-scale PIT in accordance with Appendix III-A shall be used to demonstrate infeasibility of bioretention areas. If the measured native soil infiltration rate is less than 0.30 in/hour, bioretention/rain garden BMPs are not required to be evaluated as an option in List #1 or List #2. In these slow draining soils, a bioretention area with an underdrain may be used to treat pollution-generating surfaces to help meet Core Requirement #6, Runoff Treatment. If the underdrain is elevated within a base course of gravel, it will also provide some modest flow reduction benefit that will help achieve Core Requirement #7. • Within 100 feet of an underground storage tank and connecting underground pipes when the capacity of the tank and pipe system is greater than 1,100 gallons.
Downspout Infiltration Systems	• Site setbacks and design criteria provided in Volume V, Appendix E cannot be achieved. • The lot(s) or site does not have outwash or loam soils. • There is not at least 3 feet or more of permeable soil from the proposed bottom (final grade) of the infiltration system to the seasonal high groundwater table. • There is not at least 1-foot of clearance from the expected bottom elevation of the infiltration trench or dry well to the seasonal high

	groundwater table. • Lot size of greater than 22,000 square feet where downspout dispersion is feasible. • Within 100-feet of a drinking water supply well.
Downspout Dispersion Systems	• Downspout Dispersion Systems Site setbacks and design criteria provided in Volume V; Appendix E cannot be achieved. • A vegetated flow path at least 50 feet in length from the downspout to the downstream property line, structure, slope over 20 percent, stream, wetland, or other impervious surface is not feasible. • A vegetated flow path of at least 25 feet in between the outlet of the trench and any property line, structure, stream, wetland, or impervious surface is not feasible.
Perforated Stub- Out Connections	• Site setbacks and design criteria provided in Volume III; Section 3.9.5 cannot be achieved. • There is not at least 12 inches or more of permeable soil from the proposed bottom (final grade) of the perforated stub-out connection trench to the highest estimated groundwater table. • The only location available for the perforated stub-out connection is under impervious or heavily compacted soils. • For sites with septic systems, the only location available for the perforated portion of the pipe is located upgradient 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. • The connecting pipe discharges to a stormwater facility designed to meet Core Requirement #7.
Other Hard Surfaces	
BMP	**Infeasibility Criteria**
Full Dispersion	• See Full Dispersion under “roofs” section above.
Permeable Pavement	• Setbacks and site constraints provided in Volume V, Section 2.2.6 cannot be achieved. Citation of any of the following infeasibility criteria must be based on an evaluation of site-specific conditions and a written recommendation from an appropriate licensed professional (e.g., engineer, geologist, hydrogeologist) o Wherever surface soils have been found to be contaminated unless those soils are removed within 10 horizontal feet from the infiltration area. o Any area where these facilities are prohibited by an approved cleanup plan under the state Model Toxics Control Act or Federal Superfund Law, or an environmental covenant under Chapter 64.70 RCW.

	• Within 100 feet of a closed or active landfill or drinking water supply well. • Within 10 feet of any underground storage tank and connecting underground pipes, regardless of tank size. As used in these criteria, an underground storage tank means any tank used to store petroleum products, chemicals, or liquid hazardous wastes of which 10 percent or more of the storage volume (including volume in the connecting piping system) is beneath the ground surface. • At multi-level parking garages, and over culverts and bridges. • Where the site design cannot avoid putting pavement in areas likely to have long-term excessive sediment deposition after construction (e.g., construction and landscaping material yards). • Where the site cannot reasonably be designed to have a porous asphalt surface at less than 5 percent slope, or a pervious concrete surface at less than 10 percent slope, or a permeable interlocking concrete pavement surface (where appropriate) at less than12 percent slope. Grid systems upper slope limit can range from 6 to 12 percent; check with manufacturer and local supplier. • Where professional geotechnical evaluation recommends infiltration not be used due to reasonable concerns about erosion, slope failure, or downgradient flooding. • In accordance with TCC Title 17 or Title 24 limitations may exist and reports may be required when permeable pavement is within 300 feet of a landslide hazard area or within 200 feet of an erosion hazard area. • Where infiltrating and ponded water below the new permeable pavement area would compromise adjacent impervious pavements. • Where infiltrating water below a new permeable pavement area would threaten existing below grade basements or building foundations. • Where infiltrating water would threaten shoreline structures such as bulkheads. • Down slope of steep, erosion prone areas that are likely to deliver sediment. • Where fill soils are used that can become unstable when saturated. • Excessively steep slopes where water within the aggregate base layer or at the subgrade surface cannot be controlled by detention structures and may cause erosion and structural failure, or where surface runoff velocities may preclude adequate infiltration at the pavement surface. • Where permeable pavements cannot provide sufficient strength to support heavy loads at industrial facilities such as ports.

	• Where installation of permeable pavement would threaten the safety or reliability of pre-existing underground utilities, pre-existing underground storage tanks, or pre-existing road subgrades. The following criteria can be cited as reasons for infeasibility without further justification (though some require professional services to make the observation): • Within setbacks provided that the length of sheet flow across the paved section is no more than twice the length of sheet flow across the porous pavement section.in Section 3.5.6. • For properties with known soil or groundwater contamination (typically federal Superfund sites or state cleanup sites under the Model Toxics Control Act (MTCA)): o Within 100 feet of an area known to have deep soil contamination. o Where groundwater modeling indicates infiltration will likely increase or change the direction of the migration of pollutants in the groundwater. o Wherever surface soils have been found to be contaminated unless those soils are removed within 10 horizontal feet from the infiltration area o Any area where these facilities are prohibited by an approved cleanup plan under the state Model Toxics Control Act or Federal Superfund Law, or an environmental covenant under Chapter 64.70 RCW.
Permeable Pavement (continued)	• Where the subgrade soils below a pollution-generating permeable pavement (e.g., road or parking lot) do not meet the soil suitability criteria for providing treatment. See soil suitability criteria for treatment in Chapter 6 of Volume V. Note: In these instances, the county may approve installation of a six-inch sand filter layer meeting county specifications for treatment as a condition of construction. • Where underlying soils are unsuitable for supporting traffic loads when saturated. Soils meeting a California Bearing Ratio of 5 percent are considered suitable for residential access roads. • Where underlying soils are unsuitable for supporting traffic loads when saturated. Soils meeting a California Bearing Ratio of 5 percent are considered suitable for residential access roads. • Where appropriate field testing indicates soils have a measured (a.k.a., initial) subgrade soil saturated hydraulic conductivity less than 0.3 inches per hour. Only small-scale PIT or large-scale PIT methods in accordance with Appendix III-A shall be used to evaluate infeasibility of permeable pavement areas. (Note: In these instances, unless other infeasibility restrictions apply, roads and parking lots may be built with an underdrain, preferably elevated within the base

	course, if flow control benefits are desired.) • Where the road type is classified as arterial or collector rather than access. See RCW 35.78.010, RCW 36.86.070, and RCW 47.05.021. Note: This infeasibility criterion does not extend to sidewalks and other non-traffic bearing surfaces associated with the collector or arterial. • Where replacing existing impervious surfaces unless the existing surface is a non-pollution generating surface over an outwash soil with a saturated hydraulic conductivity of four inches per hour or greater. • At sites defined as "high-use sites." For more information on high-use sites, refer to the Glossary in Volume I; and Volume V, Section 2.1, Step 3. • In areas with "industrial activity" as defined in the Glossary (located in Volume I). • Where the risk of concentrated pollutant spills is more likely such as gas stations, truck stops, and industrial chemical storage sites. • Where routine, heavy applications of sand occur in frequent snow zones to maintain traction during weeks of snow and ice accumulation.
Bioretention or Rain Gardens	• See Bioretention or Rain Gardens under "roofs" section above.
Sheet Flow Dispersion	• Site setbacks and design criteria provided in Volume V; Appendix E cannot be achieved. • Positive drainage for sheet flow runoff cannot be achieved. • Area to be dispersed (e.g., driveway, patio) cannot be graded to have less than a 15 percent slope. • At least a 10-foot wide vegetation buffer for dispersion of the adjacent 20 feet of impervious surface cannot be achieved. • Erosion or flooding of downstream properties may result.
Concentrated Flow Dispersion	• Site setbacks and design criteria provided in Volume V; Appendix E cannot be achieved. • A minimum 3 foot length of rock pad and 50-foot flow path for every 700 sf of impervious area followed with applicable setbacks cannot be achieved. • Erosion or flooding of downstream properties may result. • A vegetated flow path of at 25 feet between the discharge point and any property line, structure, steep slope, stream, wetland, lake, or other impervious surface cannot be maintained.