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 offsite 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
- 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
- "Hydraulics Manual", published by the Washington State Department of Transportation – WSDOT
- *Soil Survey of Thurston County, Washington*, published by the Natural Resource Conservation Service, U.S. Department of Agriculture

- *Standard Plans for Road, Bridge and Municipal Construction*, published by WSDOT
- *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.

1.3 How Do I Get Started?

First, consult Chapter 2 of Volume I to determine which minimum requirements apply to your project and to select BMPs. After determining the minimum 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 ground water 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 onsite 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.

2.1 Minimum Computational Standards

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

When designing runoff treatment and flow control BMPs, use a continuous simulation hydrologic model based on the EPA's Hydrologic Simulation Program-Fortran (HSPF) program (i.e., Ecology's Western Washington Hydrology Model (WWHM) or WSDOT's MGS Flood model) to calculate runoff and determine flow rates and volumes. Continuous models simulate rainfall and runoff over a long period of time, usually years, encompassing many storm events. Additional design standards applicable for selection and sizing of specific runoff treatment and flow control BMPs are found in Volume V.

For conveyance system design, the designer may use a single event hydrologic 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.

Conveyance design is discussed in detail in Chapter 3 of this Volume.

Circumstances where different methodologies apply are summarized in Table 2.1 below.

Table 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: (WWHM or MGS Flood)	Method applies to all BMPs	Method applies to all BMPs	Not Allowed
SCSUH/SBUH (Soil Conservation Service Unit Hydrograph/Santa Barbara Urban Hydrograph)	Not Applicable[a]	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 WWHM 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.

Thurston County has incorporated additional field monitoring stations into the WWHM and where applicable the Thurston County specific rainfall/runoff data shall be used for design. Thurston County has also developed a modification to the WWHM that provides site infiltration data for use in meeting the minimum infiltration requirement for projects in Thurston County (see Volume 1). These Thurston County modifications will be incorporated into the versions WWHM available for download at Ecology’s web-site.

Free WWHM software and documentation can be found at the Department of Ecology website:

http://www.ecy.wa.gov/programs/wq/stormwater/wwhmtraining/wwhm/wwhm_v3/index.html.

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

Use of continuous simulation runoff models other than WWHM or MGS Flood 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 EPA's HSPF model, Stormwater Management Model (SWMM), or a model approved by the County and the Department of Ecology. WWHM and MGS Flood do not have the capability to model and route flows in conveyance systems.

For large, master-planned developments, the County may require a basin-specific calibration of HSPF rather than default parameters. Basin-specific calibrations may be required for projects that encompass more than 320 acres.

2.1.1 Hydrologic Analysis of LID BMPs

The implementation of Low Impact Development (LID) BMPs may affect flow control and runoff treatment analysis for a project by reducing "Effective Impervious Area" which affects the thresholds associated with Minimum Requirements #6 (Runoff Treatment) and #7 (Flow Control). LID BMPs may also have associated flow credits for use in the WWHM that allow modification of how the impervious area is modeled – effectively reducing the size of downstream flow control and runoff treatment facilities. Newer versions of WWHM allow directly modeling LID BMPs and eliminate the need for the use of flow credits.

Hydrologic modeling for LID BMPs must conform to all applicable minimum requirements outlined in Volume I, specifically:

- Minimum Requirement #4 – Preservation of Natural Drainage System and Outfalls
- Minimum Requirement #5 – Onsite Stormwater Management
- Minimum Requirement #6 – Runoff Treatment
- Minimum Requirement #7 – Flow Control

- Minimum Requirement #8 – Wetland Protection (including applicable predeveloped land-cover assumptions used in hydrologic modeling).

Modeling requirements and applicable flow credits for use in modeling individual LID BMPs are described in Volume V. Simplified sizing methods can be applied for some LID BMPs, as described in Volume V.

2.1.2 Hydrologic Analysis of Flow Control BMPs

The flow control standard (Minimum Requirement #7) must be met using an approved continuous runoff model. The standard flow control requirement is summarized below:

- 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. The pre-developed condition to be matched shall be a forested land cover unless:
    - Reasonable, historic information is available that indicates the site was prairie prior to settlement (modeled as "pasture" in the Western Washington Hydrology Model); or
    - The drainage area of the immediate stream and all subsequent downstream basins have had at least 40 percent total impervious area since 1985. In this case, the predeveloped condition to be matched shall be the existing land cover condition. Where basin-specific studies determine a stream channel to be unstable, even though the above criterion is met, the pre-developed condition assumption shall be the "historic" land cover condition, or a land cover condition commensurate with achieving a target flow regime identified by an approved basin study.

See the documentation for WWHM (or alternate model) for instructions on how to use the model to meet this standard.

If off-site drainage combines with site runoff, these off-site flows must be included in the flow control BMP sizing analysis. See Chapter 3 for conveyance requirements for off-site drainage.

2.1.3 Hydrologic Analysis of Runoff Treatment BMPs

Water Quality Design Storm Volume

The 91st percentile, 24-hour runoff volume estimated by an approved continuous runoff model shall be used as the water quality design storm volume.

Water Quality Design Flow Rate

Downstream of detention facilities: The full 2-year recurrence interval release rate from a detention facility (using an approved continuous runoff model) designed to meet the flow duration standard shall be used as the design flow rate.

Preceding detention facilities or when detention facilities are not required: The flow rate at or below which 91 percent of the runoff volume, as estimated by an approved continuous runoff model, is routed through the treatment facility shall be used as the design flow rate. The 91 percent volume for treatment facilities is designed to achieve the applicable performance goal at the water quality design flow rate (e.g., 80 percent total suspended solids removal).

- *Offline facilities*: When runoff flow rates exceed the water quality design flow rate and treatment facilities are not preceded by an equalization or storage basin, the treatment facility should continue to receive and treat the water quality design flow rate to the applicable treatment performance goal. Only the portion of flow rates that exceed the water quality design flow may be bypassed around a treatment facility.

  Treatment facilities preceded by an equalization or storage basin may identify a lower water quality design flow rate provided that at least 91 percent of the estimated runoff volume in the time series of an approved continuous runoff model is treated to the applicable performance goals (e.g., 80 percent total suspended solids removal at the water quality design flow rate and 80 percent total suspended solids removal on an annual average basis).

- *Online facilities*: Runoff flow rates in excess of the water quality design flow rate can be routed through the facility provided a net pollutant reduction is maintained, and the applicable annual average performance goal is likely to be met. When on-line runoff treatment facilities experience flows greater than the water quality design flow rate, it is assumed that no pollutant removal is occurring. For this reason, water quality design flow rates for on-line facilities are higher than design flow rates for off-line facilities with the same drainage characteristics.

Treatment facilities that are located downstream of detention facilities shall only be designed as on-line facilities.

2.1.4 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 one of two methods, either the single event hydrograph method (SCS, SBUH) or the Rational Method (for small projects).

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.

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.

2.2 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 Minimum 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 Minimum 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.

2.2.1 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.

2.2.2 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 three cases that dictate different approaches to meeting Minimum 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 Minimum 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.

2.2.3 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.

2.3 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 Minimum 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 be used to meet the runoff treatment requirements of Minimum Requirement #6 also (see Section 2.3.1 below).

2.3.1 Infiltration Facilities for Runoff Treatment

Infiltration facilities can be designed for runoff treatment within Thurston County. The soil texture and design infiltration rates should be considered along with the physical and chemical characteristics specified below to determine if the soil is adequate for removing the target pollutants.

- Short-term soil infiltration rate should be 2.4 inches per hour, or less, to a depth of 2.5 times the maximum design pond water depth, or a minimum of 6 feet below the base of the infiltration facility. This infiltration rate is also typical for soil textures that possess sufficient physical and chemical properties for adequate treatment, particularly for soluble pollutant removal. It is comparable to the textures represented by Hydrologic Group B and C. Long-term infiltration rates up to 2.0 inches per 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 considered suitable for treatment (see description later in this section) to control target pollutants.

- Cation exchange capacity (CEC) of the treatment soil must be ≥5 milliequivalents CEC/100 g dry soil (USEPA Method 9081). Consider empirical testing of soil sorption capacity, if practicable. Ensure that soil CEC is sufficient for expected pollutant loadings, particularly heavy metals. CEC values of >5 meq/100g are expected in loamy sands, according to Rawls et al. Lower CEC content may be considered if it is based on a soil loading capacity determination for the target pollutants that is accepted by Thurston County.

- Depth of suitable treatment soil used for infiltration treatment must be a minimum of 18 inches. If native soils cannot meet the treatment criteria of this section, soils may be amended or an engineered soil (minimum depth of 18 inches) may be used. See BMP LID.08 Bioretention in Volume V for an acceptable engineered soil for runoff treatment.

- Organic content of the treatment soil (ASTM D 2974): Organic matter can increase the sorptive capacity of the soil for many pollutants. The site professional shall evaluate whether the organic matter content is sufficient for control of the target pollutant(s). Generally, a minimum organic content of 10% by weight is required to meet treatment requirements.

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

- Engineered soils may be used to meet infiltration BMP design criteria in Volume V and the performance goals in Minimum Requirement 6 (Runoff Treatment; Volume I). BMP LID.08 Bioretention provides an acceptable engineered soil specification for runoff treatment. Use of alternate engineered soils must be accepted by the County, and requires field performance evaluation(s), using acceptable protocols, to determine effectiveness, feasibility, and acceptability.

Also note that although infiltration is one of the preferred methods for disposing of excess stormwater, and may be required to meet Minimum Requirement #7 – *Flow Control*, infiltration may be regulated by the Department of Ecology and the Underground Injection Control (UIC) Program (WAC 173-218) if an injection device, such as a dry well or trench with distribution pipe is used. Additional information and requirements on UIC and how it applies to infiltration and stormwater management is included in Volume V, Section 3.1.3.

2.3.2 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 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 minimum requirements 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 15 feet to groundwater, then a mounding analysis will be required.

Step 4 – Determine whether the simple or detailed method of analysis 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 (simple or detailed).

Step 5 – Complete simple analysis or detailed analysis, as determined in Step 4 and described in Sections 2.3.3 and 2.3.5.

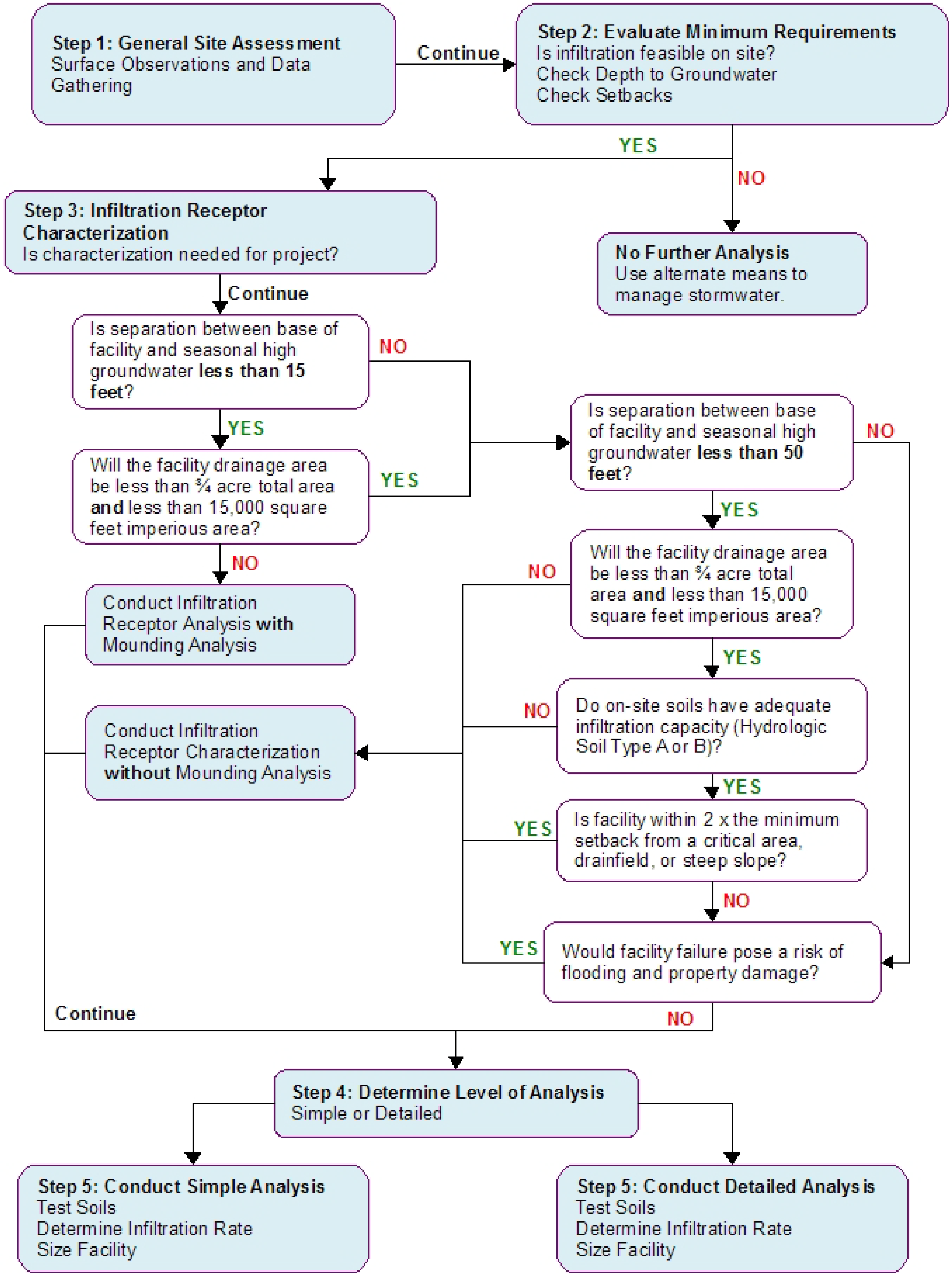

Figure 2.1. Infiltration Analysis and Sizing Flow Chart.

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

Step 1: General Surface Characterization

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

- Topography within 500 feet of the proposed facility
- Anticipated site use (street/highway, residential, commercial, high-use site)
- Location of water supply wells within 500 feet of proposed facility
- Location of project relative to any designated well head protection areas for public water systems. (enhanced treatment required prior to infiltration if located within a designated WHPA).
- Location of steep slopes (>15%) or landslide hazard areas
- Location of septic systems in the vicinity of the proposed facility
- A description of local site geology, including soil or rock units likely to be encountered, the groundwater regime, and geologic history of the site.
- Analysis of site borings and soil testing and review of any available existing soils information for the site or adjacent sites.
- 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.
- Location of any high groundwater hazard areas or wetlands per the Thurston County Critical Areas Ordinance, TCC Title 17.

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 Minimum Requirements for Infiltration Facilities

Infiltration is not permitted unless all of the Depth to Seasonal High Groundwater and Setbacks criteria below are met. Note: not all sites that meet the following criteria will be suitable for infiltration – these are **minimum** requirements only.

Depth to Seasonal High Groundwater

The base of all infiltration basins or trench systems shall be a minimum of 3 feet above seasonal high groundwater levels, bedrock (or hardpan), or any other low permeability layer. Small bioretention (BMP LID.08) facilities with less than 10,000 square feet of impervious area contributing to the facility may be designed with a reduced vertical separation of 1 foot minimum.

Seasonal high groundwater level is the upper level at which the groundwater table normally is located during the season of the year when such levels are at their highest (typically December 1 through April 30). This level is determined using a test pit (reviewed by a soil analyst for soil color patterns in the soil profile) or using groundwater monitoring data gathered through a minimum of one wet period (December through April). See Step 3 for additional criteria related to groundwater depth.

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. Additional setbacks are summarized in Appendix V-E.

Step 3: Infiltration Receptor Characterization

An Infiltration receptor characterization consists of monitoring and analysis of groundwater, and (in some cases) 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 condition 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 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 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).

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 at least 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):

- 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
- 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.
- 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.
- Determine ambient ground water 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.

- 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 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 facility shall be calculated based on either the Simple Method or Detailed Method as described in this section.

Simple Method

The Simple Method of calculating a design infiltration rates includes several alternative methods as follows:

- Field Testing by In-Situ Methods (must incorporate safety factors) including:
    - Split Double Ring Infiltrometer
    - Ecology Pilot Infiltration Test (PIT)
    - Single Ring Falling Head Infiltration Method (US EPA 1980) as Modified by Thurston County.

- USDA Soil Textural Classification

- ASTM Gradation Testing

The Simple Method was derived from high ground water and shallow pond sites in western Washington, and in general will produce conservative designs. The Simple Method (Section 2.3.3) should be considered a suitable method of calculating design infiltration rates in the following circumstances:

- When determining the trial geometry of the infiltration facility,
- For small or low impact facilities
- For facilities where a more conservative design is acceptable.
- For Type A/B soils

Where the combination of depth to ground water/low permeability layer and soil type results in the possibility of groundwater mounding effects the Simple Method should not be applied. The suitability of the Simple Method should be discussed in the geotechnical report.

Detailed Method

The detailed method 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 in Section 2.3.4, includes more intensive field testing and soils investigation and analyses than the Simple Method 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
- 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 Simple Method in circumstances that might warrant the detailed method 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, conduct a simple analysis (Section 2.3.3), or a detailed analysis (Section 2.3.4).

2.3.3 Simple Analysis Procedures

All proposed infiltration projects must evaluate soils, determine the design infiltration rate, prepare a geotechnical report, and estimate the volume of stormwater to be infiltrated.

Soil Testing

Test holes or test pits must be dug according to the following guidelines (see Table 2.3):

- Test hole or test pit explorations shall be conducted during mid to late in the wet season (with the wet season defined as December 1 through April 30).
- Collect representative samples from each soil type and/or unit to a depth of 6 feet below the proposed base of the infiltration facility or 2.5 times the estimated depth of the infiltration pond, whichever is greater. For infiltration ponds, there shall be one test pit or test hole per 5,000 square feet of pond infiltrating surface with a minimum of two per pond, regardless of pond size. For infiltration trenches, there shall be one test pit or test hole per 100 feet of trench length with a minimum of two required per trench, regardless of length.
- 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).
    - Textural class (USDA).
    - Percent clay content (include type of clay, if known).
    - Color/mottling.
    - Variations and nature of stratification.
    - Cation exchange capacity (CEC) and organic matter content (if facility may be considered to provide treatment as well as flow control).

- For small-scale infiltration facilities (contributing drainage area is less than 7,500 square feet), only one testing location is required.

- The required number of test pits/test holes may be modified by the Administrator or designee if provided adequate evidence of consistent subsurface conditions.

- 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 the depth, soil descriptions, depth to water, evidence of seasonal high groundwater elevation, existing ground surface elevation, proposed pond bottom elevation, and presence of stratification that may impact the infiltration design. Elevations shall be referenced to a vertical datum such as NGVD 29. Use the soil evaluation report forms in Appendix I-F.

Table 2.3. Required Number of Test Pits, Test Holes, and In-Situ Testing Locations for infiltration Facilities.

Contributing Drainage Area	BMP Type	Number of Test Pits/Test Holes per BMP	Number of In-Situ Infiltration Testing Locations per BMP (If Using In-Situ Testing Method of Simple Method)[a]
SFR or Commercial, less than 7,500 square feet	All Infiltration BMPs	1	1
Greater than 7,500 square feet or other land use type	Infiltration trench (BMP IN.02) or linear configuration of other Infiltration BMP	1 per 100 linear feet (2 minimum)	1 per 500 linear feet (2 minimum) [a,b]
Greater than 7,500 square feet or other land use type	Bioretention Area (BMP LID.08) or Infiltration Pond (BMP IN.01)	1 per 5,000 square feet (2 minimum)	1 per 10,000 square feet (2 minimum) [a,c]

BMP: best management practice
SFR: single family residential
[a] In-Situ testing only required if applicant intends to use In-Situ Method for Estimating Design Infiltration Rate. Test pits are still required to characterize subsurface. For small scale in-situ methods, a minimum of three tests are required at each location. Small scale in-situ testing includes ASTM D3385 Method (DRI) and Single-Ring Falling Head Infiltration method.
[b] Tests must be conducted at the test pits with the least permeable soils, as determined by observation of grain size gradation.
[c] Tests must be conducted at the test pits with the least permeable soils, as determined by observation of grain size gradation
Note: The required number of test pits/test holes may be modified by the Administrator or designee if provided adequate evidence of consistent subsurface conditions

2.3.4 Determine Design Infiltration Rate

There are two ways of estimating design infiltration rates: in-situ testing or using relationships between soil properties and infiltration rates.

Note: It should be recognized that there is a distinction between infiltration rate and hydraulic conductivity. These two parameters are related by Darcy's equation where:

f = Ki where f = infiltration rate, i = hydraulic gradient (head in ft/ft) and K = hydraulic conductivity.

In cases where water percolates under free draining conditions the hydraulic gradient is 1.0 and the infiltration rate equals the hydraulic conductivity. However, in circumstances where groundwater mounding or pond depth creates a hydraulic gradient, the infiltration rate and hydraulic conductivity would not be equal. In the simple methods, it is likely that the hydraulic gradient is close to 1.0 and therefore the infiltration rate and hydraulic conductivity are close to equal. The design professionals should keep these distinctions in mind and account for the differences as appropriate to the circumstances.

Prescriptive BMP sizing methods can be used in lieu of estimating an infiltration rate for downspout infiltration (BMP LID.04) when the following conditions apply:

- Contributing drainage area is less than 7,500 square feet.
- Property is a single family residential lot or commercial development.
- Soils are characterized as outwash by a soils professional (including a septic system designer).

These prescriptive methods are included in the BMP descriptions in Volume V.

The two following general methods of estimating the design infiltration rate can be used:

Method 1 – In-Situ Testing Methods

- Ecology Pilot Infiltration Test (PIT) is a large-scale test of infiltration. The PIT (described in Appendix III-A) is the preferred method of determining infiltration rate in Thurston County, and can be used for any infiltration BMP. The PIT method requires a substantial amount of water, which may not be available at some sites. If the test is not feasible for this reason, the two alternative methods described below can be used.
- Single-Ring Falling-Head Infiltration method (US EPA 1980), as modified in Appendix III-A or as modified by Clark County (2008)

is an acceptable in-situ method when the PIT method cannot be conducted due to site constraints, or the availability of sufficient water.

- Double-Ring Infiltrometer method (ASTM D3385) is an acceptable in-situ method when the PIT method cannot be conducted due to site constraints, or the availability of sufficient water.

Method 2 – Soil Property Relationships

- USDA Soil Textural Classification method (USDA 1993). This method is applicable to sites with soils classified as loam, sandy loam, loamy sand, sand, sandy gravel or gravelly sand, and is described in Appendix III-A.

- ASTM Gradation Testing method (ASTM D422). This method is applicable to sites with soils classified as sand or sandy gravel, and is described in Appendix III-A.

If conducting in-situ testing of infiltration rates, see Table 2.3 for guidelines on the frequency of in-situ infiltration tests.

Determine Infiltration Rate of Engineered Treatment Soils

If engineered soils are used for the treatment soils the following procedure will be used to determine the design infiltration rate for the facility and inputs for hydrologic modeling (WWHM).

1. The infiltration rate used for hydrologic modeling and facility sizing shall be the lower of the long-term infiltration rate of the engineered soils and the short term infiltration rate of the underlying soils.

2. The long term rate of the engineered soils will be based on ASTM 2434 Standard Test Method for Permeability of Granular Soils (Constant Head) with a compaction rate of 85 percent of maximum density using ASTM 1557 Test Method (Modified Proctor) with an infiltration reduction factor of 4 (multiply calculated infiltration rate by 0.25 to get long term infiltration rate).

3. The short term rate for the underlying soils will be based on the calculated rate as determined by the methods described above without application of the adjustment factor for clogging of the soils. This is based on the assumption that the treatment soil layer removes the silt and sediment that would have resulted in clogging of the underlying soils.

4. Use the lower infiltration rate of the two determined above in the hydrologic model and use an infiltration reduction factor of 1.

Prepare Geotechnical Report

A geotechnical report shall be prepared by or under the direct supervision of, and stamped by either a professional engineer with geotechnical expertise, or a licensed geologist, engineering geologist, or hydrogeologist. The report must summarize site characteristics and demonstrates that sufficient permeable soil for infiltration exists. In addition to the information required by Step 3 – *Infiltration Receptor Characterization* (as applicable), at a minimum, the report must contain the following:

- Figure showing the following:
    - Topography within 500 feet of the proposed facility
    - Locations of any water supply wells within 500 feet of the proposed facility
    - Location of groundwater protection areas, aquifer recharge areas, or 1-, 5-, and 10-year times of travel zones for designated wellhead protection areas.
    - Location of high groundwater hazard or flood plain areas in the project vicinity.
    - Locations of test pits or test holes.
- Results of soils tests, including detailed soil logs
- Description of local site geology, including soil or rock units likely to be encountered at soil sampling depths and the seasonal high groundwater elevation
- Detailed documentation of the design infiltration rate determination, as specified above
- State whether location is suitable for infiltration and recommend a design infiltration rate.

Estimate Volume of Stormwater

Use the Western Washington Hydrologic Model (WWHM), MGSFlood, or other approved continuous simulation runoff model to generate a runoff inflow file that will be used to size the infiltration facility. The facility must either:

- Infiltrate all of the flow volume as specified by the inflow file without any overflow, or
- Infiltrate a sufficient amount of the flow volume such that any overflow/bypass meets the flow duration standard in Minimum Requirement #7 – Flow Control, or
- Be designed as a combined infiltration/detention facility such that the minimum infiltration standard is met and any discharge to surface water from the facility meets the flow duration standards in Minimum Requirement #7 – Flow Control.

If the facility is designed to meet runoff treatment requirements of Minimum Requirement #6, it must infiltrate the 91st percentile, 24-hour runoff volume indicated by an approved continuous runoff model.

For downspout infiltration (BMP LID.04) a simplified sizing table can be used if the facility meets soils requirements and contributing drainage area thresholds. Simplified sizing methods are presented in the corresponding BMP description in Volume V.

2.3.5 Detailed Analysis Procedure

This detailed approach was obtained from Massmann (2003). Procedures for the detailed approach are as follows (see Figure 2.2):

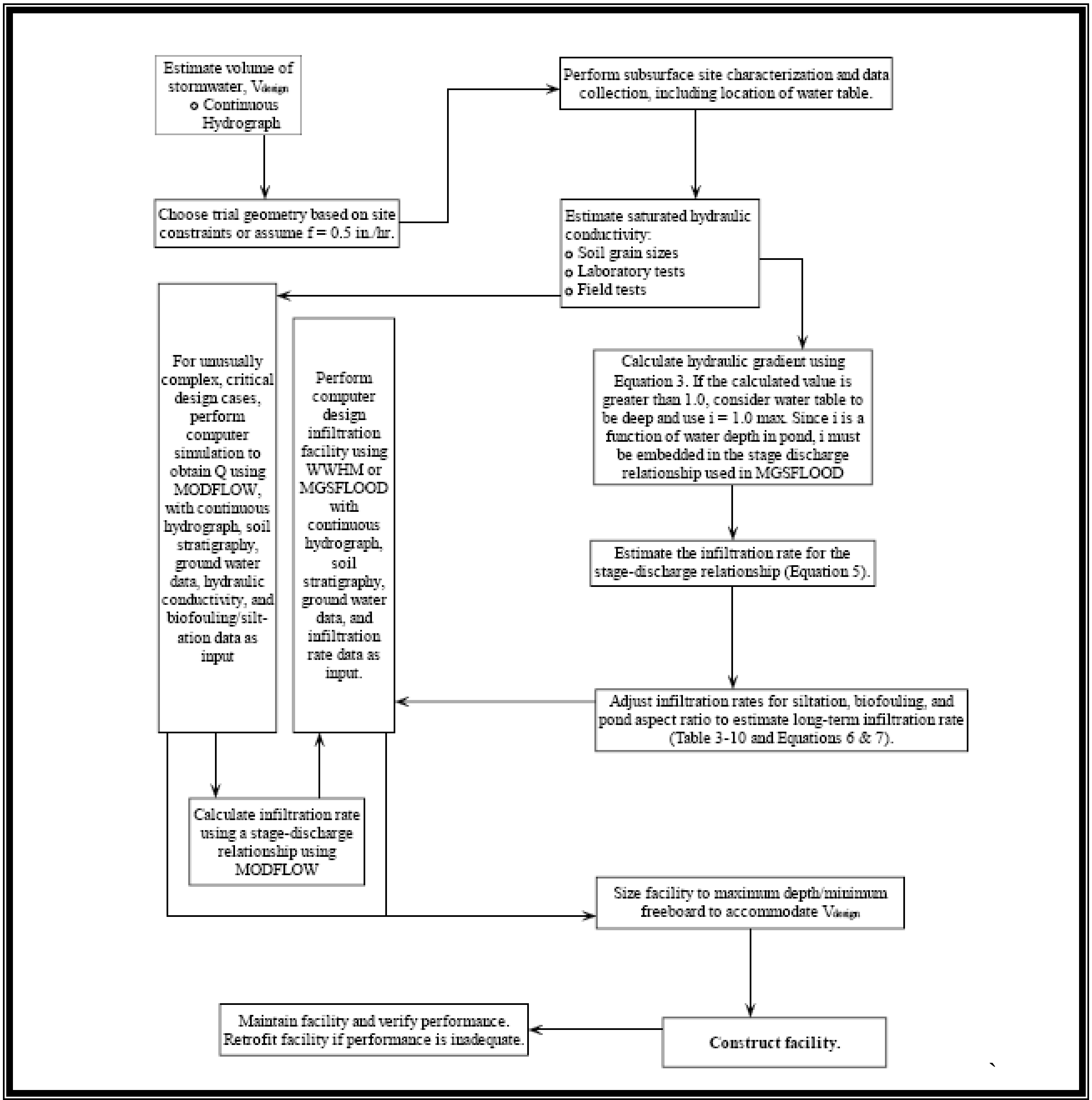

Figure 2.2. Engineering Design Steps for Final Design of Infiltration Facilities Using the Continuous Hydrograph Method (from Ecology [2005]).

Develop a Trial Infiltration Facility Geometry Based on Length, Width, and Depth

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

Conduct a Geotechnical Investigation

A geotechnical investigation must be conducted to evaluate the site's suitability for infiltration, to establish the infiltration rate for design, and to evaluate slope stability, foundation capacity, and other geotechnical design information needed to design and assess constructability of the facility. Geotechnical investigation requirements are provided below.

The depth, number of test holes or test pits, and sampling described below should be increased if a licensed engineer with geotechnical expertise (P.E.), or a licensed geologist or hydrogeologist judges that conditions are highly variable and make it necessary to increase the depth or the number of explorations to accurately estimate the infiltration system's performance. The exploration program described below may be decreased if the licensed professional judges that conditions are relatively uniform, or design parameters are known to be conservative based on site specific data or experience, and the borings/test pits omitted will not influence the design or successful operation of the facility.

- For infiltration basins (ponds), at least one test pit or test hole per 5,000 ft^2 of basin infiltrating surface (two minimum).

- For infiltration trenches, at least one test pit or test hole per 100 feet of trench length (two minimum).

- Subsurface explorations (test holes or test pits) to a depth below the base of the infiltration facility of at least 5 times the maximum design depth of water proposed for the infiltration facility, or at least 2 feet into the saturated zone (whichever is less).

- Continuous sampling to a depth below the base of the infiltration facility of 2.5 times the maximum design depth of water proposed for the infiltration facility, or at least 2 feet into the saturated zone, but not less than 6 feet. Samples obtained must be adequate for the purpose of soil gradation/classification testing.

- Conduct Infiltration Receptor Characterization as described in Step 3 if required.

- Laboratory testing as necessary to establish the soil gradation characteristics and other properties as necessary, to complete the infiltration facility design. At a minimum, one-grain size analysis per soil stratum in each test hole must be conducted within 2.5 times the maximum design water depth, but not less than 6 feet. When assessing the hydraulic conductivity characteristics of the site, soil layers at greater depths must be considered if the licensed professional conducting the investigation determines that deeper layers will influence the rate of infiltration for the facility, requiring soil gradation/classification testing for layers deeper than indicated above.

Prepare Geotechnical Report

A report must be prepared by or under the direction supervision of and stamped by either a professional engineer with geotechnical expertise, or a licensed geologist, engineering geologist, or hydrogeologist The report must summarize site characteristics and demonstrates that sufficient permeable soil for infiltration exists. In addition to information required in Step 3 – *Infiltration Receptor Characterization* (as applicable), at a minimum, the report must contain the following:

- Figure showing the following:
    - Topography within 500 feet of the proposed facility
    - Locations of any water supply wells within 500 feet of the proposed facility
    - Location of groundwater protection areas, aquifer recharge areas, or 1-, 5-, and 10-year times of travel zones for designated wellhead protection areas
    - Location of high groundwater hazard areas and flood plains in the vicinity of the project.
    - Locations of test pits or test holes.
- Results of soils tests, including detailed soil logs
- Description of local site geology, including soil or rock units likely to be encountered at soil sampling depths and the seasonal high groundwater elevation
- Detailed documentation of the design infiltration rate determination, as specified above
- State whether location is suitable for infiltration and recommend a design infiltration rate
- The stratification of the soil/rock below the infiltration facility, including the soil gradation (and plasticity, if any) characteristics of each stratum
- The depth to the ground water table and to any bedrock/ impermeable layers

- Seasonal variation of the ground water table
- The existing ground water flow direction and gradient
- The hydraulic conductivity or the infiltration rate for the soil/rock at the infiltration facility
- The porosity of the soil below the infiltration facility but above the water table
- The lateral extent of the infiltration receptor
- Impact of the infiltration rate and volume on flow direction and water table at the project site, and the potential discharge point or area of the infiltrating water.

Determine Design Infiltration Rate

Procedures for determining the design infiltration rate of the site soils are included in Appendix III-A.

As with the simple analysis described above, if engineered soils are used for the treatment soils, the lower of the long-term infiltration rate of the engineered soils and the short term infiltration rate of the underlying soils shall be used for facility sizing.

2.3.6 Sizing of Infiltration Facilities

Design Criteria – Sizing Facilities

- The size of the infiltration facility can be determined using a continuous runoff model by routing the inflow runoff file through the proposed infiltration facility. 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. If a project is subject to Minimum Requirement #7 (Flow Control), overflows from an infiltration facility must comply with the flow control standard. Infiltration facilities designed to meet Minimum Requirement #6 (Runoff Treatment) must not overflow more than 9 percent of the total volume of runoff in the inflow runoff file. However, if the facility is an infiltration basin (BMP IN.01) configured as an off-line facility, it must be sized as follows: ***Off-line, upstream of detention facility (or without detention facility):*** A flow splitter shall be designed to send all flows at or below the 15-minute water quality flow rate, as predicted by an approved continuous runoff model to the treatment facility. Within the

WWHM, the flow splitter icon is placed ahead of the pond icon which represents the infiltration basin. The treatment facility must be sized to infiltrate all the runoff sent to it (no overflows from the treatment facility are allowed).

- ***Off-line, downstream of detention facility:*** A flow splitter shall be designed to send all flows at or below the 2-year flow frequency from the detention pond, as predicted by an approved continuous runoff model, to the infiltration basin. Within the WWHM, the flow splitter icon is placed ahead of the pond icon which represents the infiltration basin. The treatment facility must be sized to infiltrate all the runoff sent to it (no overflows from the treatment facility are allowed).

For infiltration facilities designed for runoff treatment, document that the 91st percentile, 24-hour runoff volume (indicated by WWHM or MGS Flood) can infiltrate through the infiltration basin surface within 48 hours (using the long-term infiltration rate). This can be calculated using a horizontal projection of the infiltration basin mid-depth dimensions and the estimated long-term infiltration rate. 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.

In order to determine compliance with the flow control requirements, the Western Washington Hydrology Model (WWHM), or an appropriately calibrated continuous simulation runoff model based on HSPF, must be used. When using WWHM for simulating flow through an infiltrating facility, the facility is represented by using the Pond Icon and entering the pre-determined infiltration rates. Below are the procedures for sizing a pond to completely infiltrate 100 percent of runoff.

For 100 Percent Infiltration

- Input dimensions of your infiltration pond.

- Input infiltration rate and safety (rate reduction) factor. In general, the rate reduction factor is 1 if the design infiltration rate is used with the applicable adjustment factors described in Appendix III-A. If amended soils or engineered soils are used for treatment in the bottom of the facility, an adjustment factor would be applied to the infiltration rate as described in Volume V.

- 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).

- Run only HSPF for Developed Mitigated Scenario (if that is where you put the infiltration pond). It is not necessary to run duration.
- Go back to your infiltration pond and look at the Percentage Infiltrated at the bottom right. If less than 100 percent infiltrated, increase pond dimension until you get 100 percent.

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)
- Culverts (Section 3.9)
- Open conveyances (Section 3.10)
- Private Drainage Systems (Section 3.11)
- Floodplains/floodways (covered in TCC 17.15).

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 and maximum allowable impervious surface area. 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 2.1.4. 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’ unsurcharged capacity, a backwater (pressure sewer) analysis shall be required. Results shall be submitted in tabular and graphic format showing hydraulic and energy gradient.

3.4.1 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 3.1, and for piped conveyances in Table 3.2. A more extensive table of Manning's roughness factors can be found in Table B-3 in Appendix III-B.

Table 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 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

3.4.2 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 below.

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

Development projects are required to handle offsite drainage in the same manner as exists in the predeveloped condition. In other words, after development of the subject site, offsite 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, the potential for increased future flow volumes should be factored in to the design of facilities. This may require modeling the off-site land area as if it were developed with a detention facility discharging per the minimum requirements of this manual. 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 offsite drainage is to be infiltrated on site, the infiltration facilities shall be sized to accommodate the correct proportion of offsite flows.

Offsite 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 offsite flows is impracticable, then offsite flows may be routed through the project's stormwater management systems. Those systems affected by the offsite flows shall be sized as if the offsite 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 Unit for additional information on easement marker requirements.

3.6.1 .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. Drainage facilities shall not be located in dedicated public road right-of-way areas, with the exception of County and highway facilities.

The dedicated tract for a stormwater facility shall include a minimum 15-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 access 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 without vehicular access must be channeled.

3.6.2 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 15 feet. All pipes and channels must be located within the easement in accordance with Table 3.3. If circumstances require the location of the pipe or channel within the easement to differ from the requirements of Table 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 3.3 are minimums for drainage facilities and may be increased depending on pipe/channel size, depth or other factors.

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

Conveyance Width	Easement/Tract Width
Channels ≤ 30 feet wide	Channel Width + 15 feet from top, one side
Channels > 30 feet wide	Channel Width + 15 feet from top, both sides
Pipes/Outfalls ≤ 36 inches	15 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.

3.6.3 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 with a measurable annual discharge such as a Department of Natural Resources (DNR) Type 5 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.

3.7.1 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.

3.7.2 Acceptable Pipe Sizes

Storm drainage pipe are subject to the following minimum diameters:

- Private drainage system = 4 inches
- Public right-of-way = 12 inches (exception: laterals connecting catch basins to main lines may be 8 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 3.9.

3.7.3 Pipe Materials

All storm drainage pipe, except as otherwise indicated, shall be a rubber-gasketed concrete pipe, or double-walled, corrugated, polyethylene pipe, with a smooth internal diameter (AASHTO M-294 Type-S) ADS N-12 plastic pipe (up to twenty-four (24) inch diameter only) or approved equal, coupled with a company produced PVC coupling or approved equal, except for perforated pipe and major underground detention facilities. ADS N-12 pipe shall have a minimum cover of two (2) feet measured from the top of pipe to the top of paved surface. The rubber-gasket requirement may be waived if it can be shown that joint leakage will not be detrimental to the road prism.

When extreme slope conditions or other unusual topographic conditions exist, other pipe materials and methods such as (but not limited to) PVC, HDPE, or ductile iron pipe may be used with prior County approval.

If other pipe materials are used, they shall meet the following minimum requirements and shall have prior County approval:

- Ductile Iron, Class 50 or 52
- Reinforced concrete pipe
- Galvanized corrugated iron or steel pipe (with Treatment 1 through 6)
- Galvanized steel spiral rib pipe (with Treatment 1 through 6)
- Corrugated aluminum pipe
- Aluminum spiral rib pipe
- Aluminized Type 2 corrugated steel (meeting AASHTO treatment M274 and M56)
- Corrugated high density polyethylene pipe (CPEP) - smooth interior (maximum 24 inch diameter) meeting AASHTO standard M-294
- Corrugated high density polyethylene pipe (CPEP) - single wall, fully corrugated meeting AASHTO standard M-252 (permitted only outside public right-of-way and for use in temporary storm sewer systems and as downspout/footing/yard drain collectors on private property)
- Polyvinyl chloride (PVC) sewer pipe (SDR 35, meeting requirements of ASTM D3034)
- High density polyethylene (HDPE) pipe. Pipe must comply with requirement of Type III C5P34 per ASTM D1248 and have the PPI recommended designation of PE3408 and have an ASTM D3350 cell classification of 345434C or 345534C. Pipe shall have a manufacturer's recommended hydrostatic design stress rating of 800 psi based on a material with a 1600 psi design basis determined in accordance with ASTM D2837-69. Pipe shall have a suggested design working pressure of 50 psi at 73.4 degrees F and SDR of 32.5. Designs utilizing HDPE pipe shall include consideration of the material's thermal expansion/contraction properties for anchoring.

Pipe material, joints, and protective treatment shall meet WSDOT Standard Specifications, Sections 7-04 and 9-05 and AASHTO and ASTM treatment standards as amended by the County. The Applicant is responsible for contacting the County to determine the allowable pipe materials which can be used.

3.7.4 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 3.4. If velocities exceed 15 feet per second for the conveyance system design event, provide anchors at bends and junctions.

Table 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-singlewall(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 200% 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) Restrainted 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.

3.7.5 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. If slopes exceed 40 percent, then pipe shall be installed above ground and anchored (see Table 3.4). Additional anchoring design may be required for these pipes.

3.7.6 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) 754-3355 (x6518) 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.

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.

3.7.7 Pipe Structure Criteria

Catch Basins and Manholes

All catch basins and manholes shall meet WSDOT standards such as Type 1L, Type 1, and Type 2. 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.

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 3.5).

Table 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 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 350 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 will be installed on all structures containing restrictor or flow devices. The locking lids shall be of a quality and design acceptable to the County.

A metal frame and grate for catch basin and inlet, WSDOT Standard Plan B-2a or B-2b or 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, an asphalt berm shall be installed around the inlet of the structure or the catch basin may be recessed into the curb per this detail.

Table 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 4.24.

Table 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 3.1 and Figure 3.2. Other equivalent designs that achieve the result of splitting low flows and diverting higher flows around the facility are also acceptable.

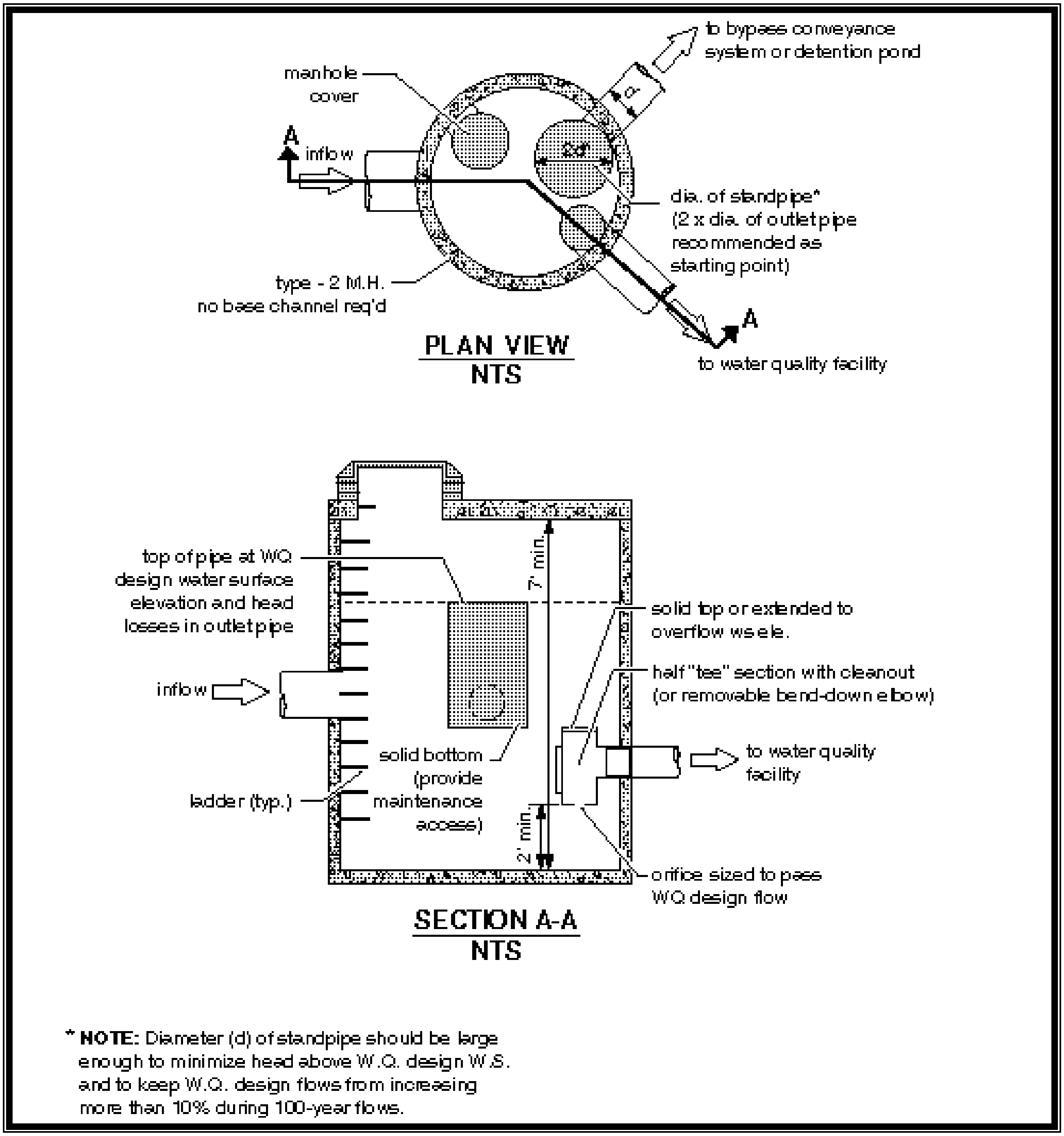

Figure 3.1. Flow Splitter, Option A.

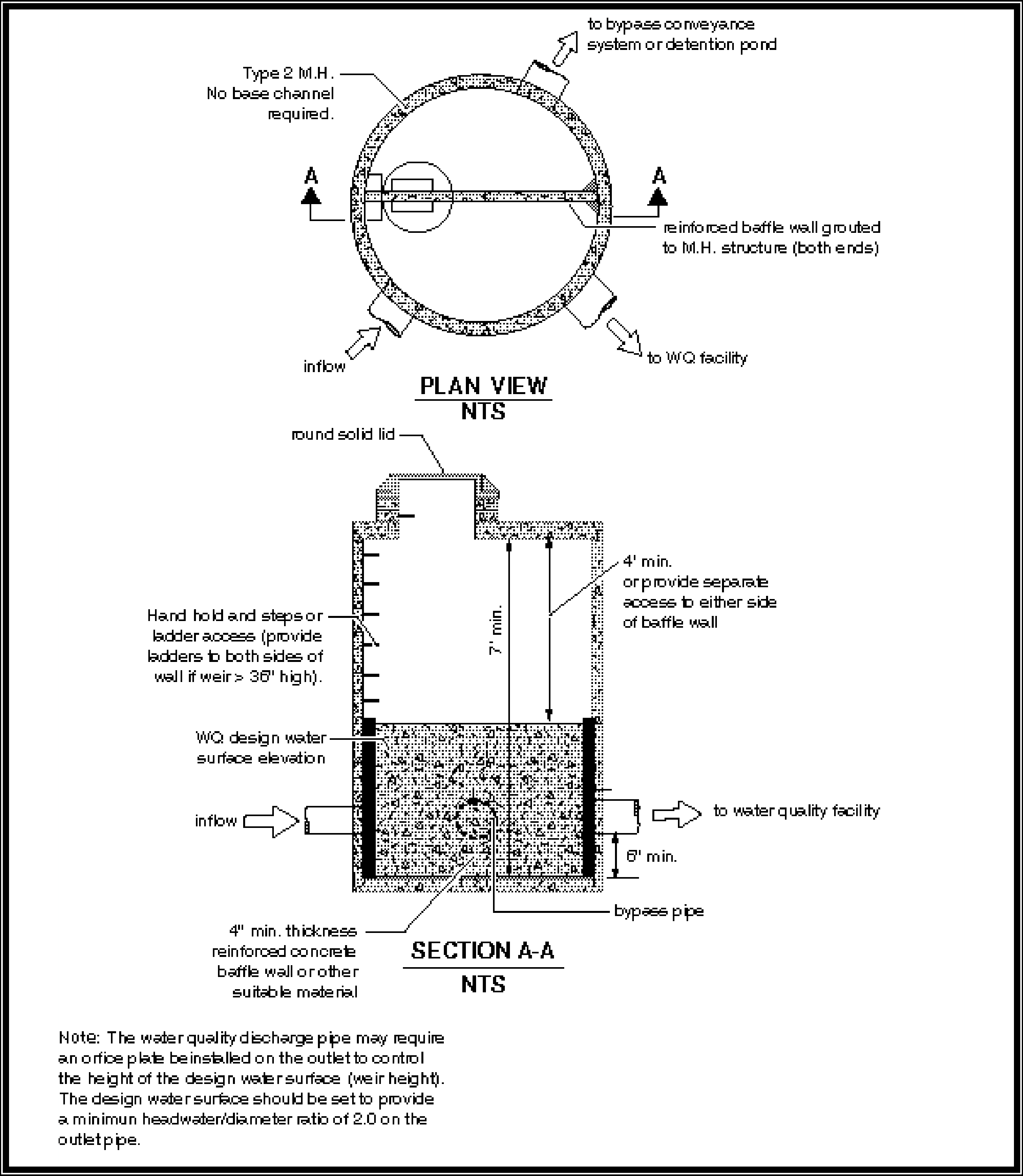

Figure 3.2. Flow Splitter, Option B.

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 (see also Volume V). Flows modeled using a continuous simulation runoff model should 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 3.1 or Figure 3.2 or an equivalent design may be used.
- As an alternative to using a solid top plate in Figure 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 onsite 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 Marine Bluff 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.

3.8.1 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.

The standard for outfall design is as shown in Figure 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., see alternate outfall designs shown in Figures 3.6 and 3.7. Other outfall designs will be allowed upon acceptance of the Administrator or designee.

See Table 3.7 for a summary of the rock protection requirements at outfalls.

Table 3.7. 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^+ - 12	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
12^+	Engineered Design	As required	As required	As required	Crown + 1 foot

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 12 feet per second or greater for the conveyance system design event (Section 3.2), an engineered energy dissipater is required. Examples are gabion splash blocks, 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 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 3.3). See Table 3.7 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 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 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. Contact the County for specific requirements.

Flow Dispersal Trench

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

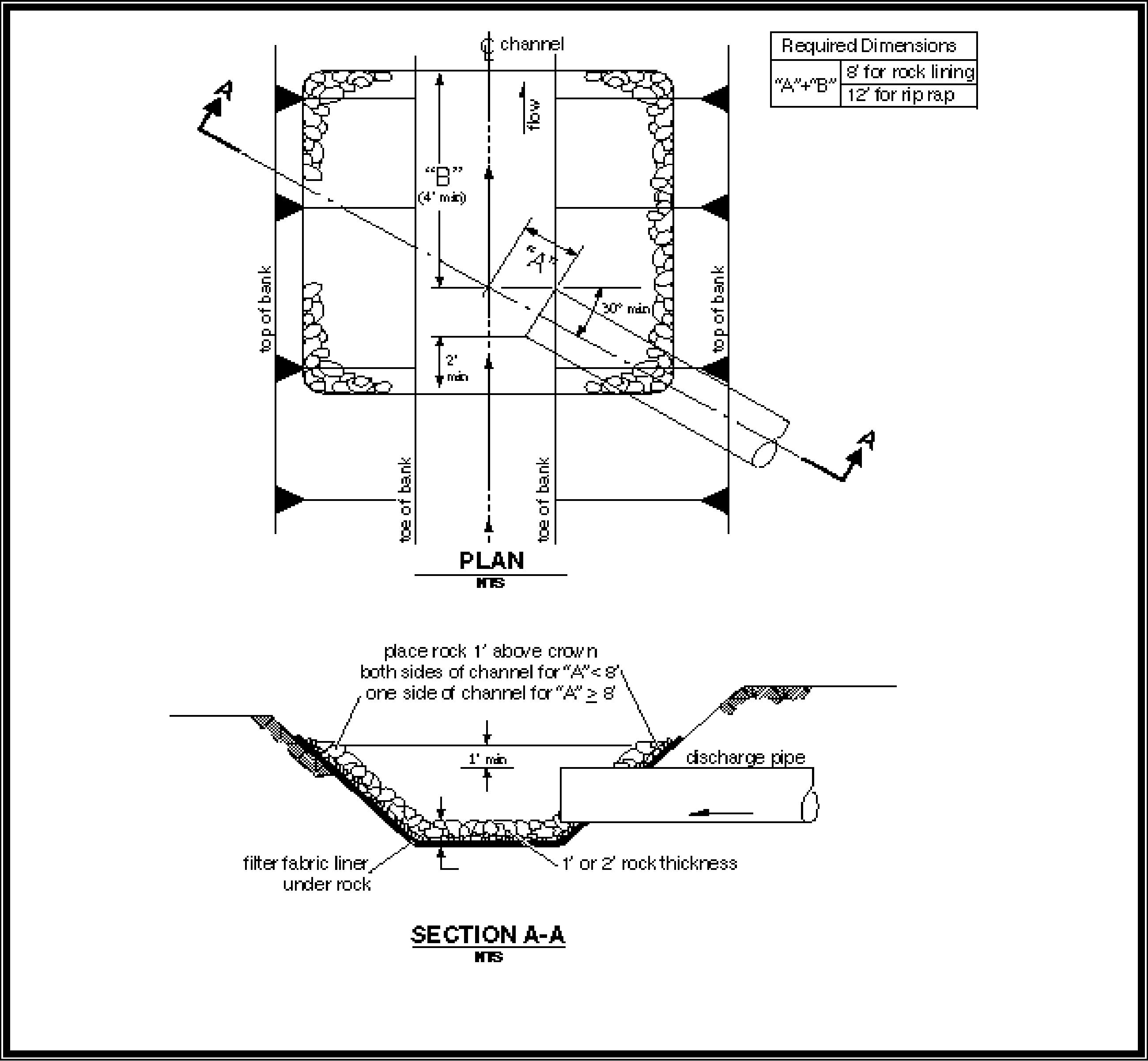

Figure 3.3. Pipe/Culvert Outfall Discharge Protection.

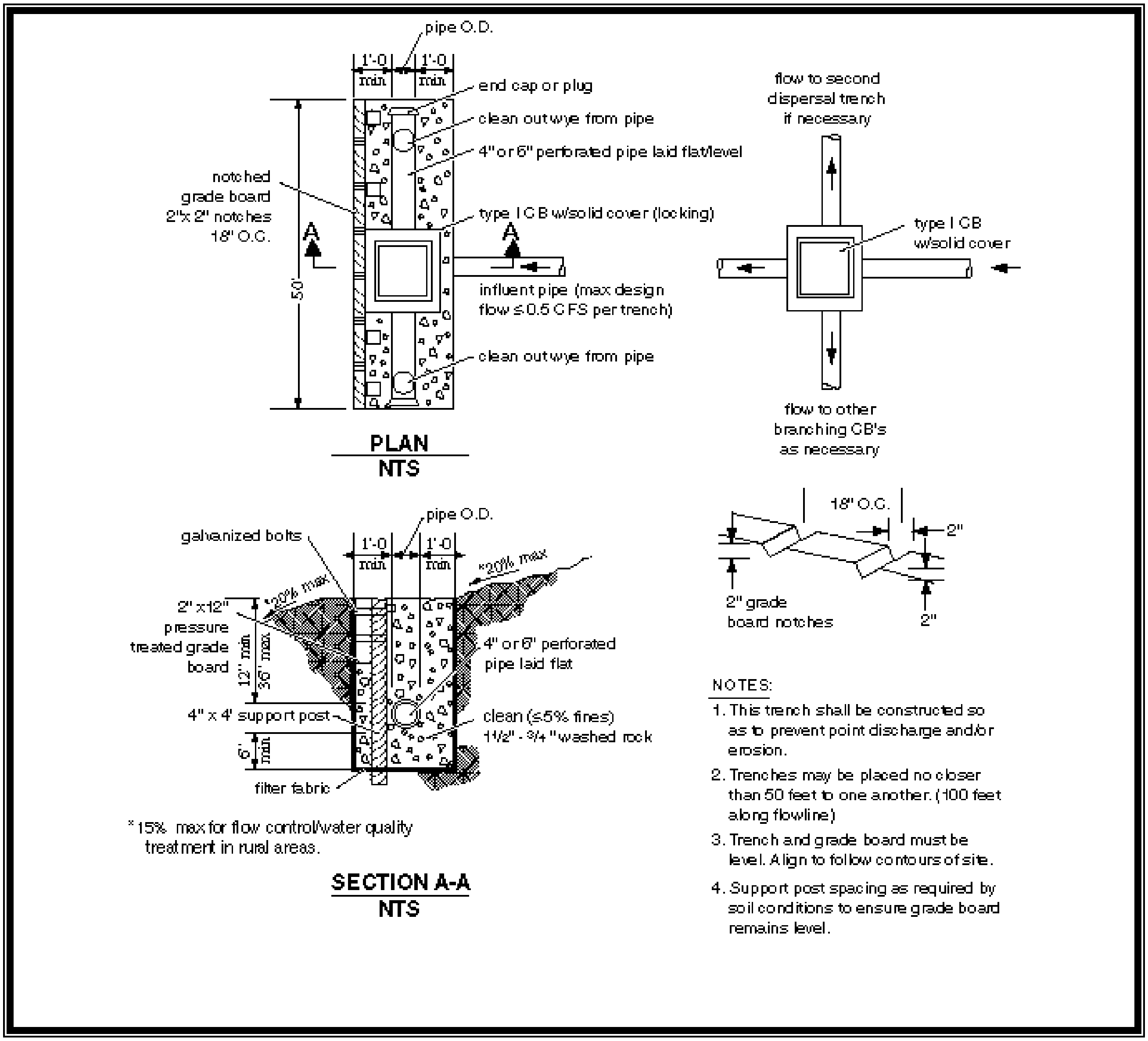

Figure 3.4. Flow Dispersal Trench.

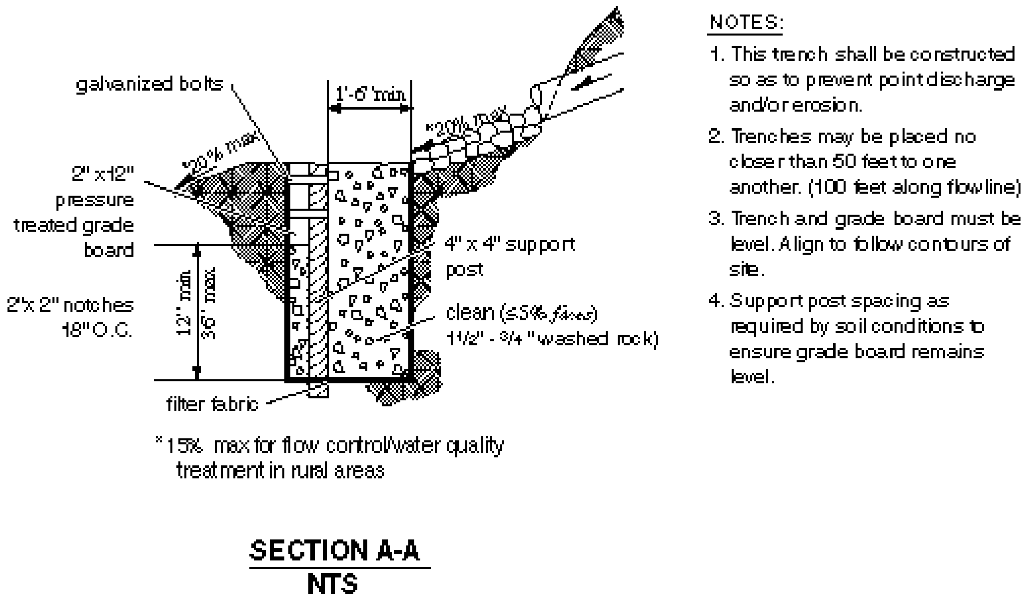

Figure 3.5. Alternative Flow Dispersal Trench.

3.8.2 Tightline Systems

Tightline systems may be needed to prevent aggravation or creation of a downstream erosion problem. The following design criteria apply to tightline systems:

- Outfall tightlines 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 tightlines be placed at grade with proper pipe anchorage and support.

- Except as indicated above, tightlines 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 tightline, such material should be covered with at least 3 feet of native bed material or equivalent.

- High density polyethylene pipe (HDPP) tightlines must be designed to address the material limitations, particularly thermal expansion and contraction and pressure design, as specified by the manufacturer. The coefficient of thermal expansion and contraction for solid wall polyethylene pipe (SWPE) is on the order of 0.001 inch per foot per Fahrenheit degree. Sliding sleeve connections should be used to address this thermal expansion and contraction. These sleeve connections consist of a section of the appropriate length of the next larger size diameter of pipe into which the outfall pipe is fitted. These sleeve connections should be located as close to the discharge end of the outfall system as is practical.

- Due to the ability of HDPP tightlines to transmit flows of very high energy, special consideration for energy dissipation must be made. Details of a sample gabion mattress energy dissipater have been provided as Figure 3.6. Flows of very high energy will require a specifically engineered energy dissipater structure.

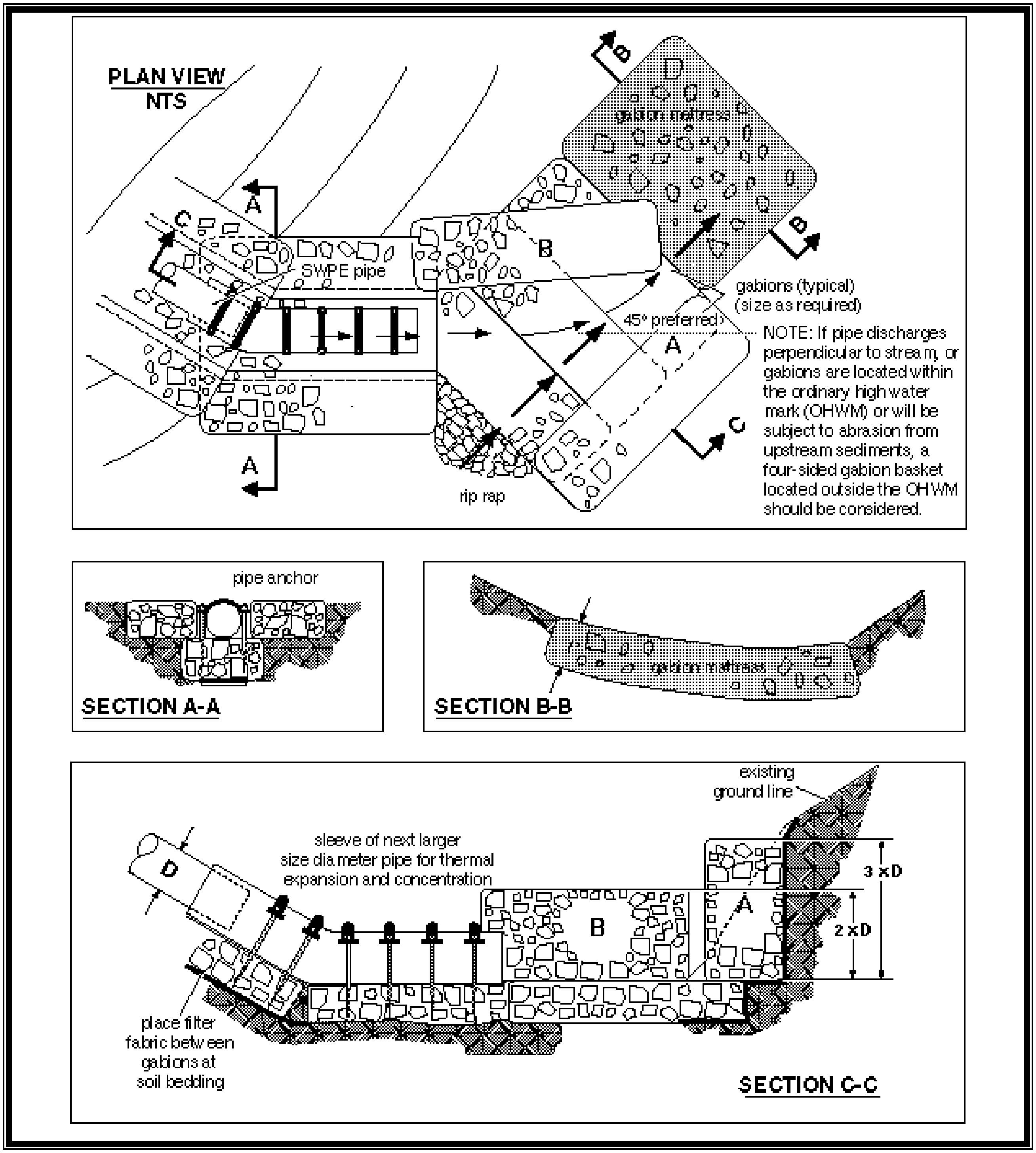

Figure 3.6. Gabion Outfall Detail.

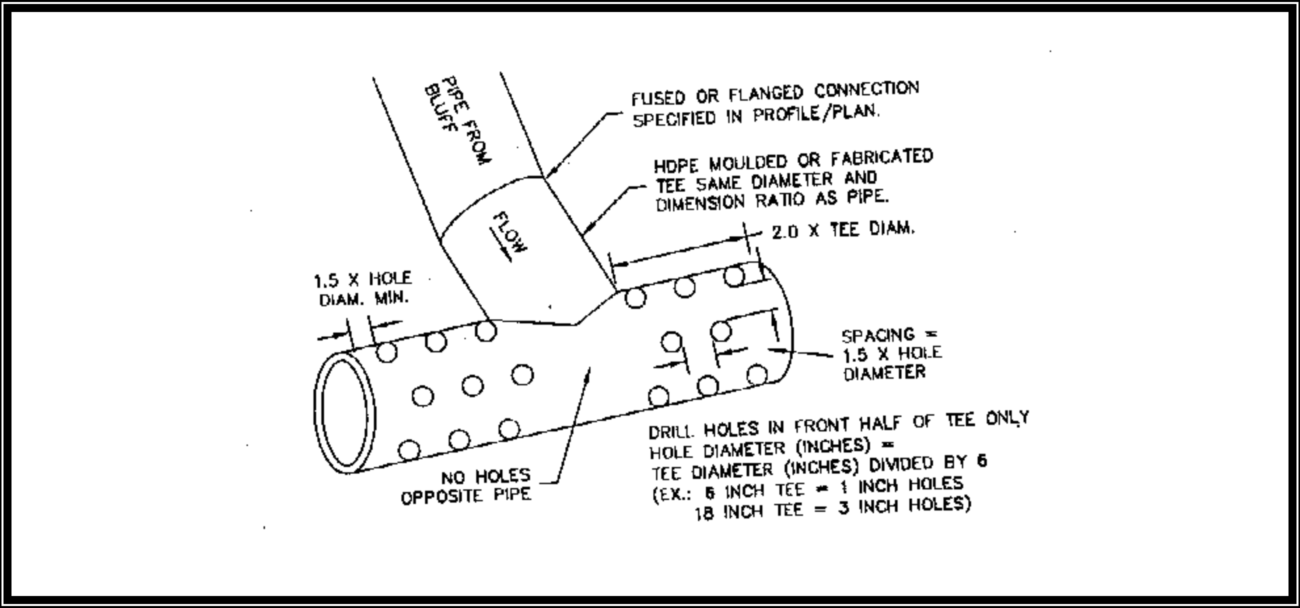

Figure 3.7. Diffuser TEE (an example of energy dissipating end feature).

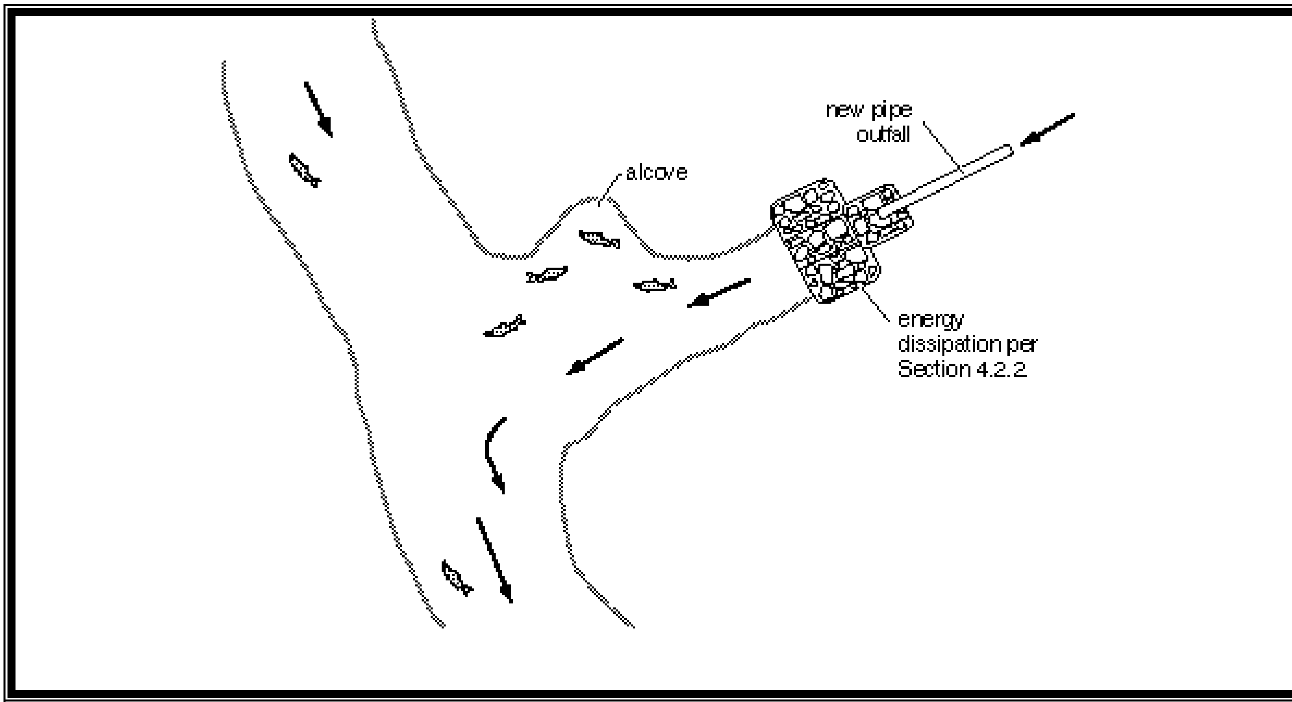

Figure 3.8. Fish Habitat Improvement at New Outfalls.

3.9 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 3.7.3. Galvanized or aluminized pipe is not permitted in marine environments or where contact with salt water may occur, even infrequently through backwater events.

3.9.1 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 3.8, 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 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 sectioned to match the side slope.

3.9.2 Fish Passage Criteria

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

3.10 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 preferred over the following design standards for open channel conveyance (see Volume V).

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 3.8.

Table 3.8. 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	1 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 <u>and</u> 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.

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.

3.11 Private Drainage Systems

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

3.11.1 Discharge Locations

Stormwater cannot discharge directly onto County roads or into a County system without prior County approval, with the exception of single family residences. 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.

3.11.2 Drainage Stub-outs

If drainage outlets (stub outs) are to be provided for each individual lot, the stub outs shall conform to the following requirements:

- 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.

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Appendix III-A
Methods for Determining Design Infiltration Rates

This appendix provides details on methods to estimate the design infiltration rate for infiltration facilities. The methods described include:

- Simple Method 1 – Field Testing Procedures
- Simple Method 2 – Soil Property Relationships
- Detailed Method – Based on Massmann (2003).

Simple Method 1 – Field Testing Procedures (In-Situ)

1. Excavate to the bottom elevation of the proposed infiltration facility.

2. 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 EPA falling head percolation test procedure as modified by Clark County (2008) (refer to Clark County Drainage Manual), the double ring infiltrometer test (ASTM D3385)(not described here, use ASTM procedure), or the Department of Ecology large scale Pilot Infiltration Test (PIT) described below and presented in the *Stormwater Management Manual for Western Washington* (Ecology 2005).

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

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

5. Perform the minimum required number of infiltration tests at the proposed infiltration facility location as specified in Volume III, Chapter 2, Section 2.3.3 and by recommendations of the geotechnical professional.

6. Determine a representative infiltration rate.

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. 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 EPA method, the SDI (ASTM D3385) method, or large-scale PIT testing, $F_{testing} = 0.50$.

$F_{geometry}$ 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) (Source: EPA, *On-site Wastewater Treatment and Disposal Systems,* 1980)

1. **Number and Location of Tests**

   A minimum of three tests shall be performed within the area proposed for an infiltration facility. Tests shall be spaced uniformly throughout the area. For larger facilities or if soil conditions are highly variable, more tests may be required (see minimum testing requirements in Volume III).

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.

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 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. Following is a step-by-step description of the testing procedure.

Infiltration Test

1. Excavate the test pit to the depth of the bottom of the proposed infiltration facility. Lay back the slopes sufficiently to avoid caving and erosion during the test.

2. The horizontal surface area of the bottom of the test pit should be approximately 100 square feet. For small drainages and where water availability is a problem, smaller areas may be considered as determined by the site professional.

3. Accurately document the size and geometry of the test pit.

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

5. 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 3 and 4 feet above the bottom of the pit. A rotameter can be used to measure the flow rate into the pit.

Note: A water level of 3 to 4 feet provides for easier measurement and flow stabilization control. 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 (between 3 and 4 feet) 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) while maintaining the same pond water level (usually 17 hours).

9. After the flow rate has stabilized, turn off the water and record the rate of infiltration in inches per hour from the measuring rod data, until the pit is empty.

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.

Note: 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.

Simple 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 A-1.

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 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 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.

ASTM Gradation Testing

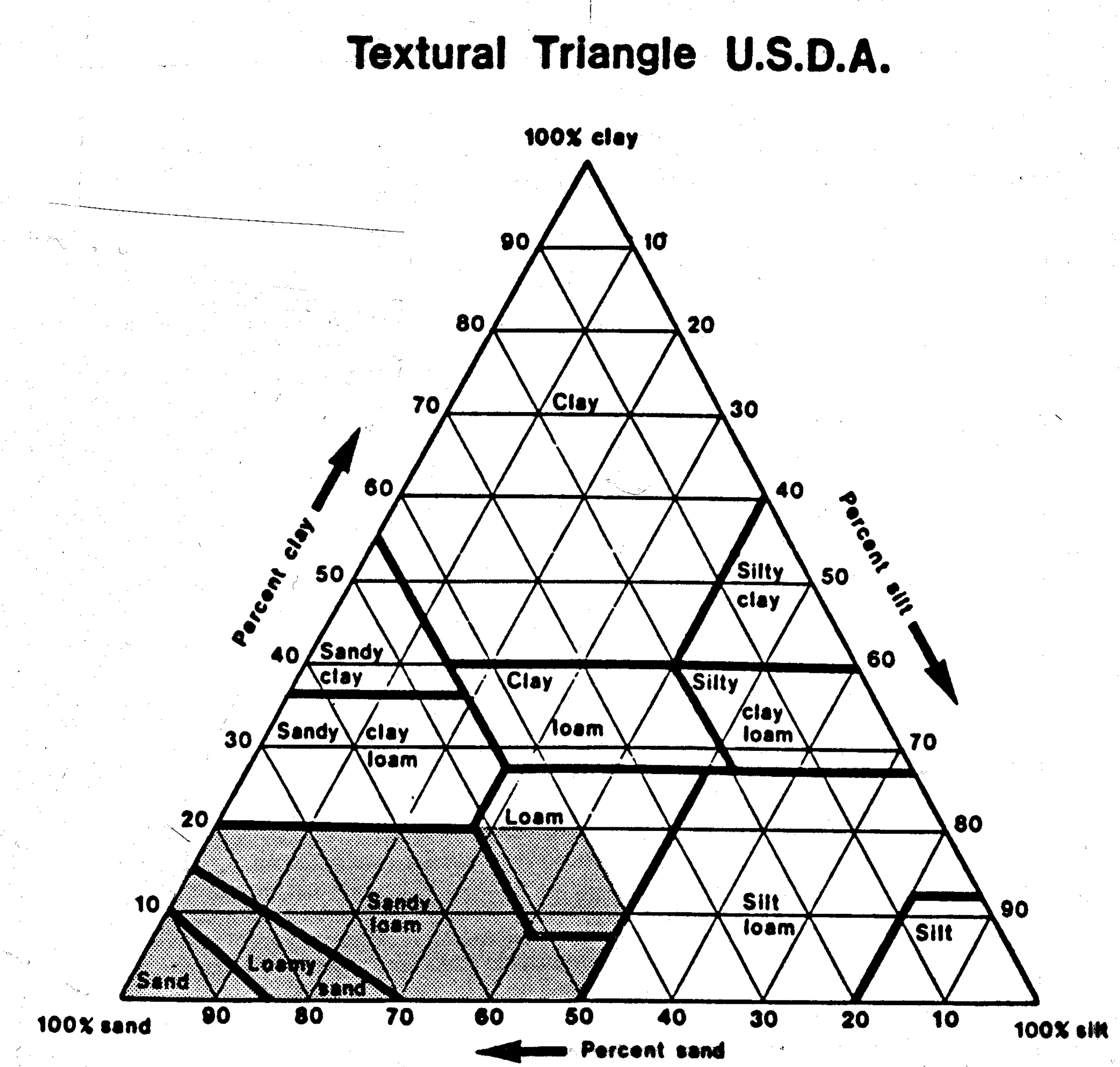
For sites with soils that would be classified as sands or sandy gravels ($D_{10}\geq0.05$ mm, US Standard Sieve), Table A.2 may be used to estimate design infiltration rates. These rates may need to be reduced if the site is highly variable or if maintenance and influent characteristics are not well controlled.

For finer soils ($D_{10}<0.05$ mm, US Standard Sieve), consult Volume III of the *Stormwater Management Manual for Western Washington* (Ecology 2005).

Table A.2. Alternative Recommended Infiltration Rates based on ASTM Gradation Testing

D_{10} Size from ASTM D422 Soil Gradation Test (mm)	Estimated Design (Long-Term) Infiltration Rate (in./hr)
$\geq$ 0.4	9
0.3	6.5
0.2	3.5
0.1	2.0
0.05	0.8

Source: *Stormwater Management Manual for Western Washington* (Ecology 2005).



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

Figure A-1. USDA Textural Triangle.

Detailed Method

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):

$$\log_{10}(K_{sat}) = -1.57 + 1.90D_{10} + 0.015D_{60} - 0.013D_{90} - 2.08f_{fines} \qquad (1)$$

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 number-200 sieve.

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 ground water 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 field tests such as the packer permeability test (above or below the water table), the piezocone (below the water table), an air conductivity test (above the water table), or through the use of a pilot infiltration test (PIT) as described in Appendix III-A. 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 ground water mounding is fully developed).

Once the saturated hydraulic conductivity for each layer has been identified, determine the effective average saturated hydraulic conductivity below the pond. Hydraulic conductivity estimates from different layers can be combined into an equivalent hydraulic conductivity (K_{equiv}) using the harmonic mean:

$$K_{equiv} = \frac{d}{\sum \frac{d_i}{K_i}} \tag{2}$$

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 ground water 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. 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 pond. For sites where the lowest conductivity layer is within five feet of the base of the pond, it is suggested that this lowest hydraulic conductivity value be used as the equivalent hydraulic conductivity rather than the value from Equation 2.

The harmonic mean given by Equation 2 is the appropriate effective hydraulic conductivity for flow that is perpendicular to stratigraphic layers, and will produce conservative results when flow has a significant horizontal component such as could occur due to ground water mounding.

Calculate the Hydraulic Gradient

The steady state hydraulic gradient (i) is calculated as follows:

$$i = \frac{D_{wt} + D_{pond}}{138.62(K^{0.1})} \times CF_{size} \qquad (3)$$

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 equal to 2/3 acre), the correction factor is equal to 1.0. For large ponds (ponds with area equal to 6 acres), the correction factor is 0.2, as shown in Equation 4.
>
> $$CF_{size} = 0.73(A_{pond})^{-0.76} \qquad (4)$$
>
> 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 ground water depths (or to a low permeability layer) below the facility, and conservatively accounts for the development of a ground water mound.

A more detailed ground water mounding analysis using a program such as MODFLOW will usually result in a gradient that is equal to or greater than the gradient calculated using Equation 3. If the calculated gradient is greater than 1.0, the water table is considered to be deep, and a maximum gradient of 1.0 must be used. Typically, a depth to ground water of 100 feet or more is required to obtain a gradient of 1.0 or more using this equation.

Since the gradient is a function of depth of water in the facility, the gradient will vary as the pond fills during the season. The gradient could be calculated as part of the stage-discharge calculation used in the continuous runoff models. As of the date of this update, neither the WWHM or MGSFlood have that capability. However, updates to those models may soon incorporate the capability. Until that time, use a steady-state hydraulic gradient that corresponds with a ponded depth of ¼ of the maximum ponded depth – as measured from the basin floor to the overflow.

Calculate the Infiltration Rate using Darcy's Law

$$f = K\left(\frac{dh}{dz}\right) = Ki \tag{5}$$

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)

Adjustments to Infiltration Rate

Adjustments to the infiltration rate calculated above are required to adjust for biofouling, siltation and pond aspect ratio.

To account for reductions in the rate resulting from long-term siltation and biofouling, take into consideration the degree of long-term maintenance and performance monitoring anticipated, the degree of influent control (e.g., pre-settling ponds biofiltration swales, etc.), and the potential for siltation, litterfall, moss buildup, etc. based on the surrounding environment.

It should be assumed that an average to high degree of maintenance will be performed on these facilities. A low degree of maintenance should be considered only when there is no other option (e.g., access problems). The infiltration rate estimated in the step above is multiplied by the reduction factors summarized in Table A.3.

Table A.3. Infiltration Rate Reduction Factors to Account for Biofouling and Siltation Effects for Ponds (Massmann, 2003)

Potential for Biofouling	Degree of Long-Term Maintenance/Performance Monitoring	Infiltration Rate Reduction Factor, $CF_{silt/bio}$
Low	Average to High	0.9
Low	Low	0.6
High	Average to High	0.5
High	Low	0.2

The values in this table assume that final excavation of the facility to the finished grade is deferred until all disturbed areas in the upgradient drainage area have been stabilized or protected (e.g., construction runoff is not allowed into the facility after final excavation of the facility).

Ponds located in shady areas where moss and litterfall from adjacent vegetation can build up on the pond bottom and sides, the upgradient drainage area will remain in a disturbed condition long-term, and no pretreatment (e.g., pre-settling ponds, biofiltration swales, etc.) is provided, are one example of a situation with a high potential for biofouling.

A low degree of long-term maintenance includes, for example, situations where access to the facility for maintenance is very difficult or limited, or where there is minimal control of the party responsible for enforcing the required maintenance. A low degree of maintenance should be considered only when there is no other option.

Adjustment for Pond Aspect Ratio

Adjust the infiltration rate for the effect of pond aspect ratio by multiplying the infiltration rate determined above by the aspect ratio correction factor CF_{aspect} as shown in the following equation:

$$CF_{aspect} = 0.02A_r + 0.98 \qquad (6)$$

Where, A_r is the aspect ratio for the pond (length/width). In no case shall CF_{aspect} be greater than 1.4. The final infiltration rate will therefore be as follows:

$$f = K \times i \times CF_{aspect} \times CF_{silt/bio} \qquad (7)$$

The rates calculated based on Equation 7 are long-term design rates. No additional reduction factor or factor of safety is needed.

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 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.

NRCS has developed "curve number" (CN) values based on soil type and land use. They can be found in *Urban Hydrology for Small Watersheds*, Technical Release 55 (TR-55), June 1986, published by the NRCS. The combination of these two factors is called the "soil-cover complex." The soil-cover complexes have been assigned to one of four hydrologic soil groups, according to their runoff characteristics. NRCS has classified over 4,000 soil types into these four soil groups. **Table 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 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 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 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 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 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 T_c.
- 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 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 such as the hyetograph provided in the 1994 Thurston County Drainage Design and Erosion Control Manual will be accepted with adequate justification.

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 Routing

All hydrographs shall be routed through retention and/or detention facilities or closed depressions by use of a level pool routing technique. Methods are described in "Handbook of Applied Hydrology", by Chow, V. Te, 1964, and elsewhere.

It is recommended that all such routing be conducted with the use of a computer program.

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 B-6 for a range of land cover types. Regression coefficients (m, n) for determining rainfall intensity can be found in Table 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

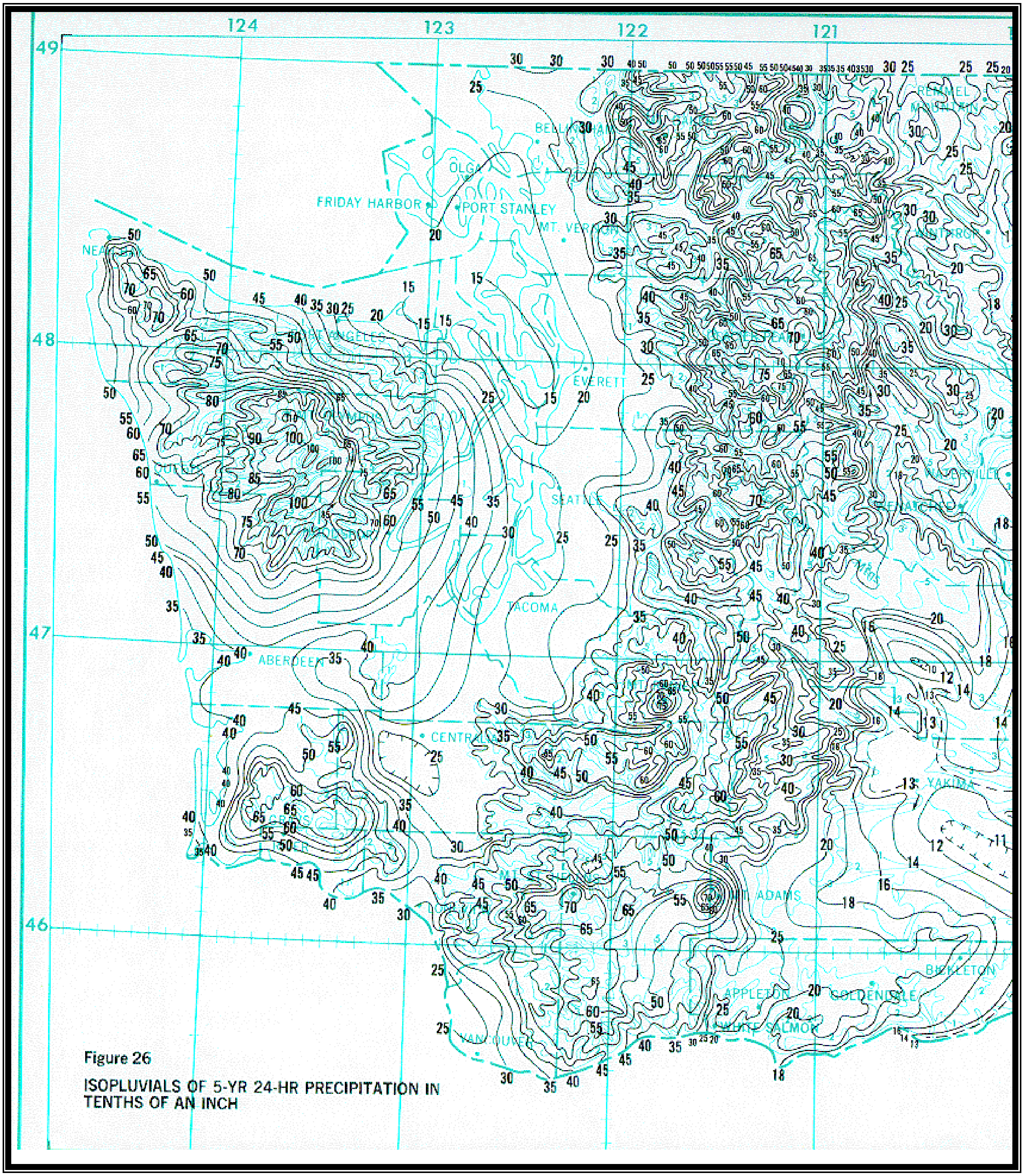


Figure 26

ISOPLUVIALS OF 5-YR 24-HR PRECIPITATION IN TENTHS OF AN INCH

Western Washington Isopluvial 5-year, 24-hour

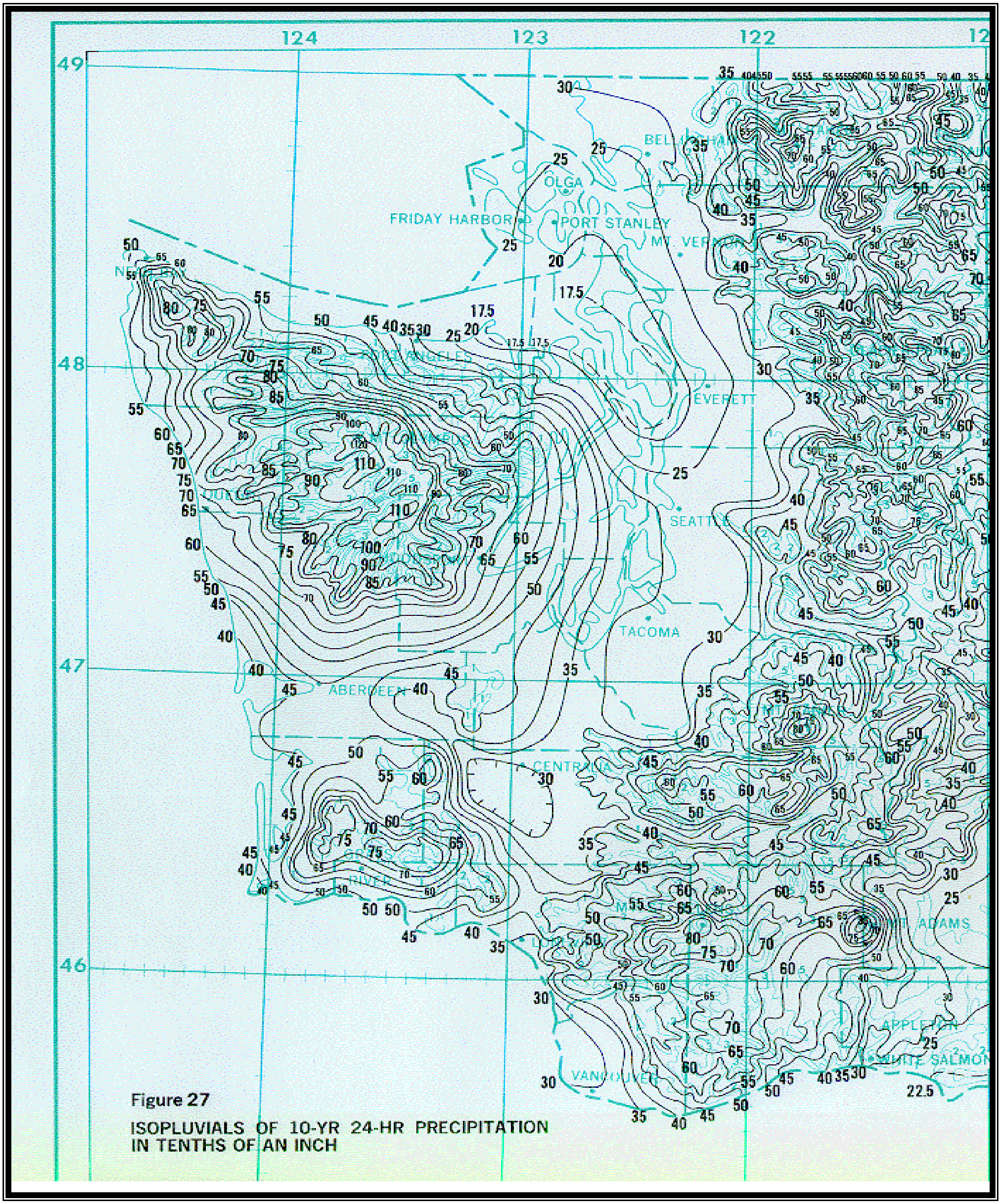
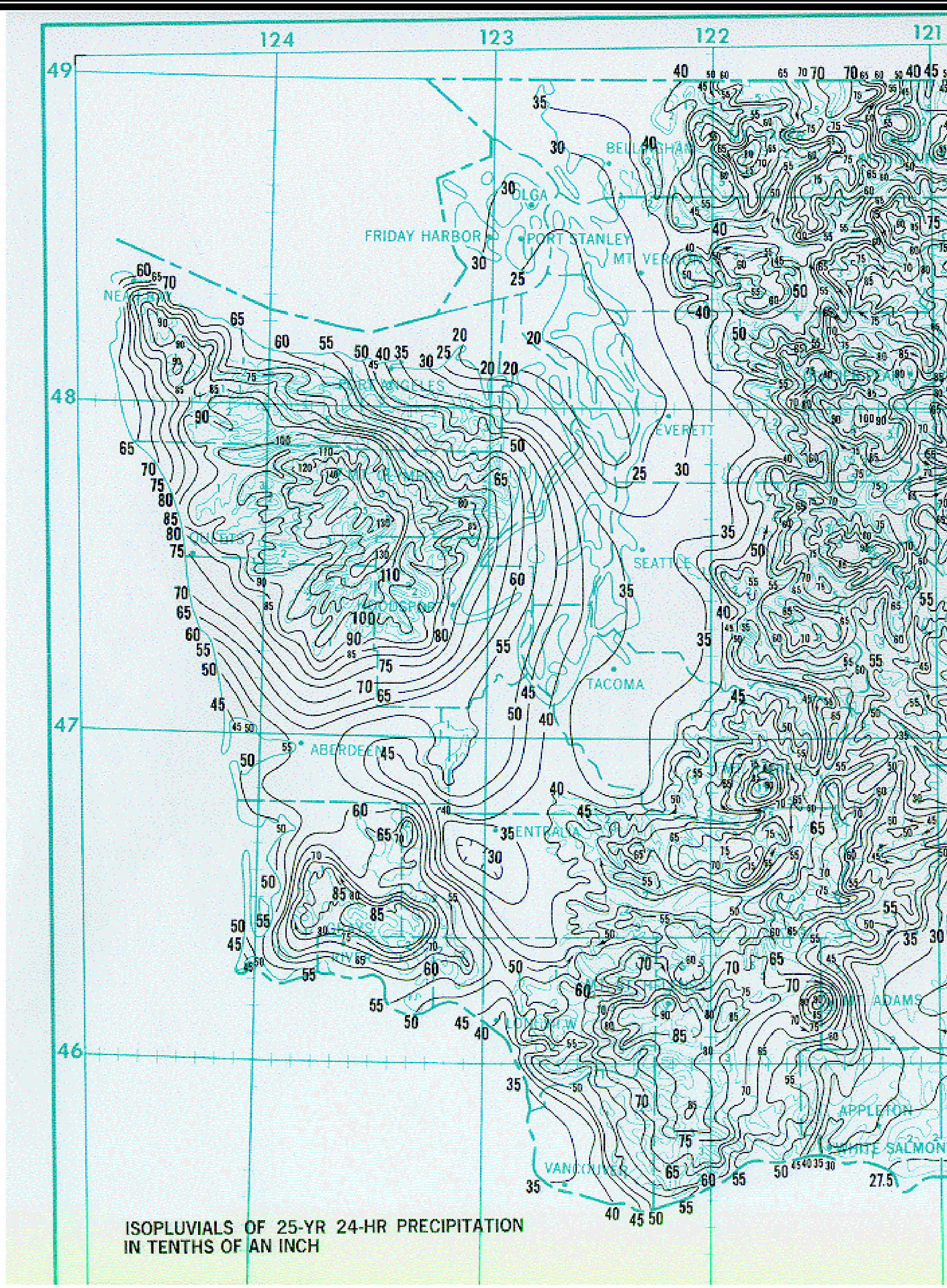


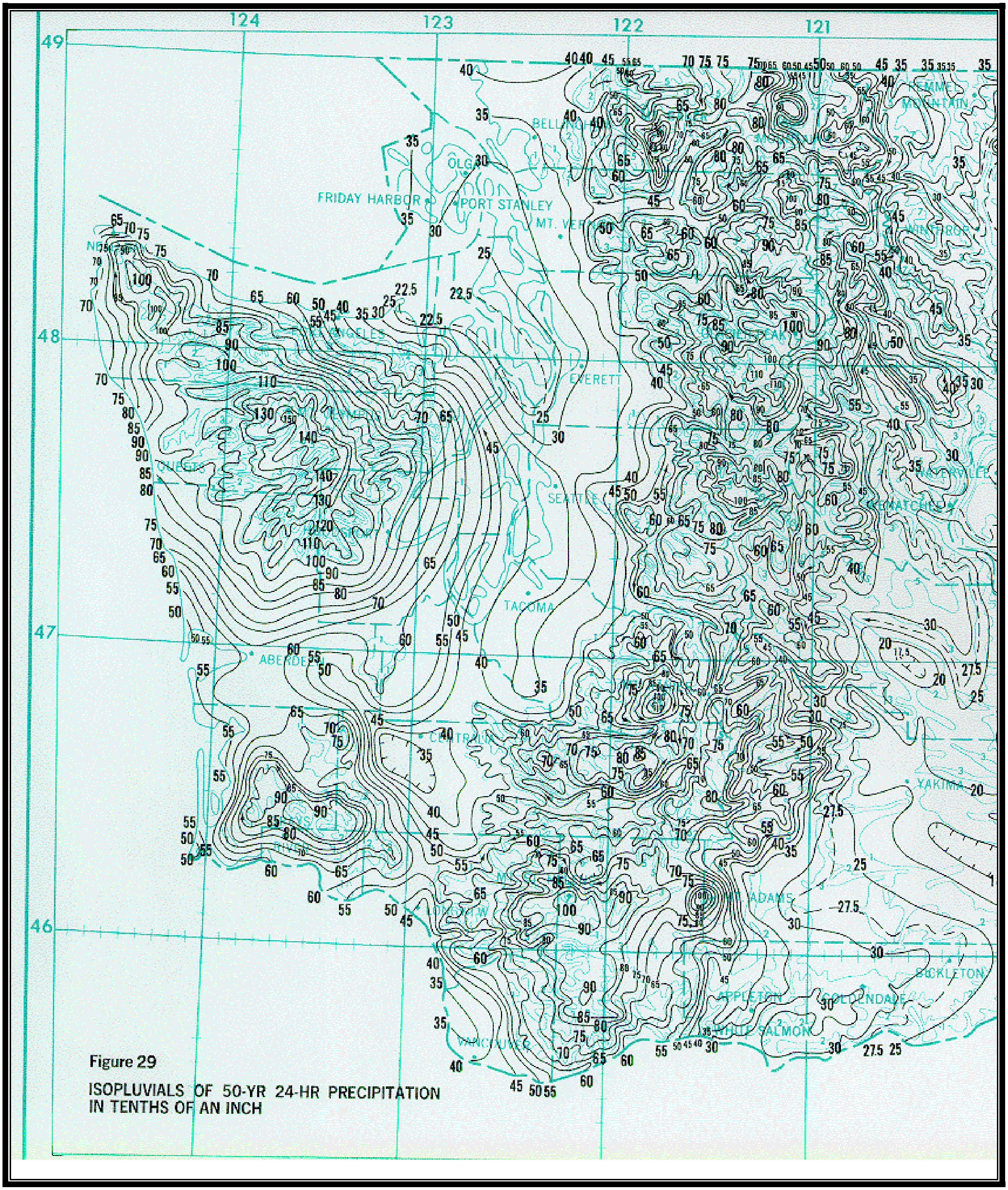
Figure 27
ISOPLUVIALS OF 10-YR 24-HR PRECIPITATION
IN TENTHS OF AN INCH

Western Washington Isopluvial 10-year, 24-hour



Source: NOAA

Western Washington Isopluvial 25-year, 24-hour



Western Washington Isopluvial 50-year, 24-hour

Western Washington Isopluvial 100-year, 24-hour

Table 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≤ 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 = 0.25)	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 6.2	0.807/n**

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

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

Type of Channel and Description	Manning's "n"
A. Constructed Channels	
a. Earth, straight and uniform	
1. Clean, recently completed	0.018
2. Gravel, uniform section, clean	0.025
3. With short grass, few weeds	0.027
b. Earth, winding and sluggish	0.025
1. No vegetation	0.025
2. Grass, some weeds	0.030
3. Dense weeds or aquatic plants in deep channels	0.035
4. Earth bottom and rubble sides	0.030
5. Stony bottom and weedy banks	0.035
6. Cobble bottom and clean sides	0.040
c. Rock lined	
1. Smooth and uniform	0.035
2. Jagged and irregular	0.040
d. Channels not maintained, weeds and brush uncut	
1. Dense weeds, high as flow depth	0.080
2. Clean bottom, brush on sides	0.050
3. Same as above, highest stage of flow	0.070
4. Dense brush, high stage	0.100
B. Natural Streams	
B-1 Minor streams (top width at flood stage < 100 ft.)	
a. Streams on plain	
1. Clean, straight, full stage no rifts or deep pools	0.030
2. Same as above, but more stones and weeds	0.035
3. Clean, winding, some pools and shoals	0.040
4. Same as above, but some weeds	0.040
5. Same as 4, but more stones	0.050

Type of Channel and Description	Manning's "n"
6. Sluggish reaches, weedy deep pools	0.070
7. Very weedy reaches, deep pools, or floodways with heavy stand of timber and underbrush	0.100
b. Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stages	
1. Bottom: gravel, cobbles, and few boulders	0.040
2. Bottom: cobbles with large boulders	0.050
B-2 Flood plains	
a. Pasture, no brush	
1. Short grass	0.030
2. High grass	0.035
b. Cultivated areas	
1. No crop	0.030
2. Mature row crops	0.035
3. Mature field crops	0.040
c. Brush	
1. Scattered brush, heavy weeds	0.050
2. Light brush and trees	0.060
3. Medium to dense brush	0.070
4. Heavy, dense brush	0.100
d. Trees	
1. Dense willows, straight	0.150
2. Cleared land with tree stumps, no sprouts	0.040
3. Same as above, but with heavy growth of sprouts	0.060
4. Heavy stand of timber, a few down trees, little undergrowth, flood stage below branches	0.100
5. Same as above, but with flood stage reaching branches	0.120

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

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

(Sources: TR 55, 1986, and Stormwater Management Manual, 1992.)				
	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
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
Paved	98	98	98	98
Gravel (including right-of-way)	76	85	89	91
Dirt (including right-of-way)	72	82	87	89
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).	45	66	77	83
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 businesses, 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 under "Flow Credit for Roof Downspout Infiltration" and "Flow Credit for Roof Downspout 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 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):

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 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 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 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

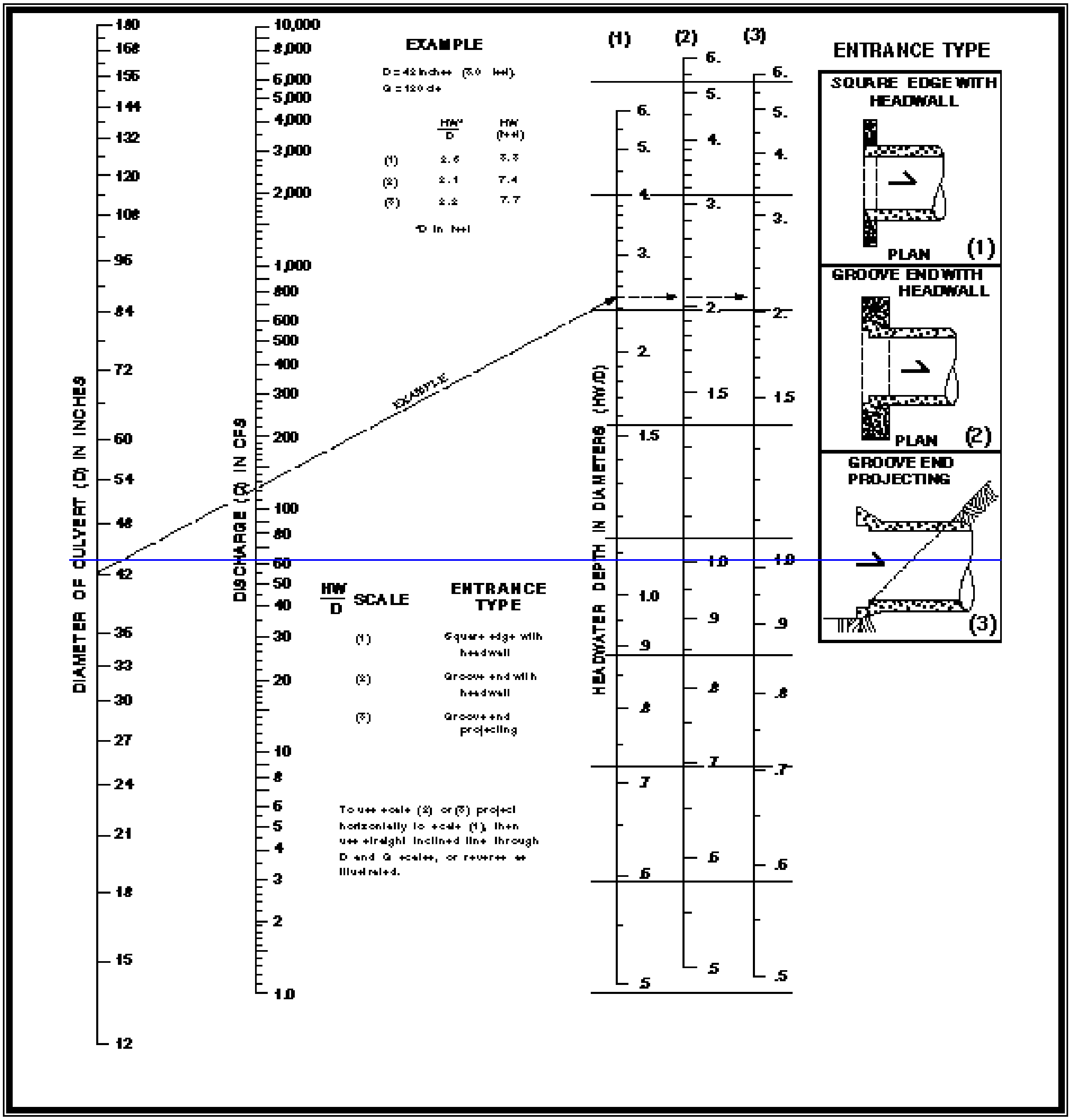


Figure C.1. Headwater Depth for Smooth Interior Pipe Culverts with Inlet Control.

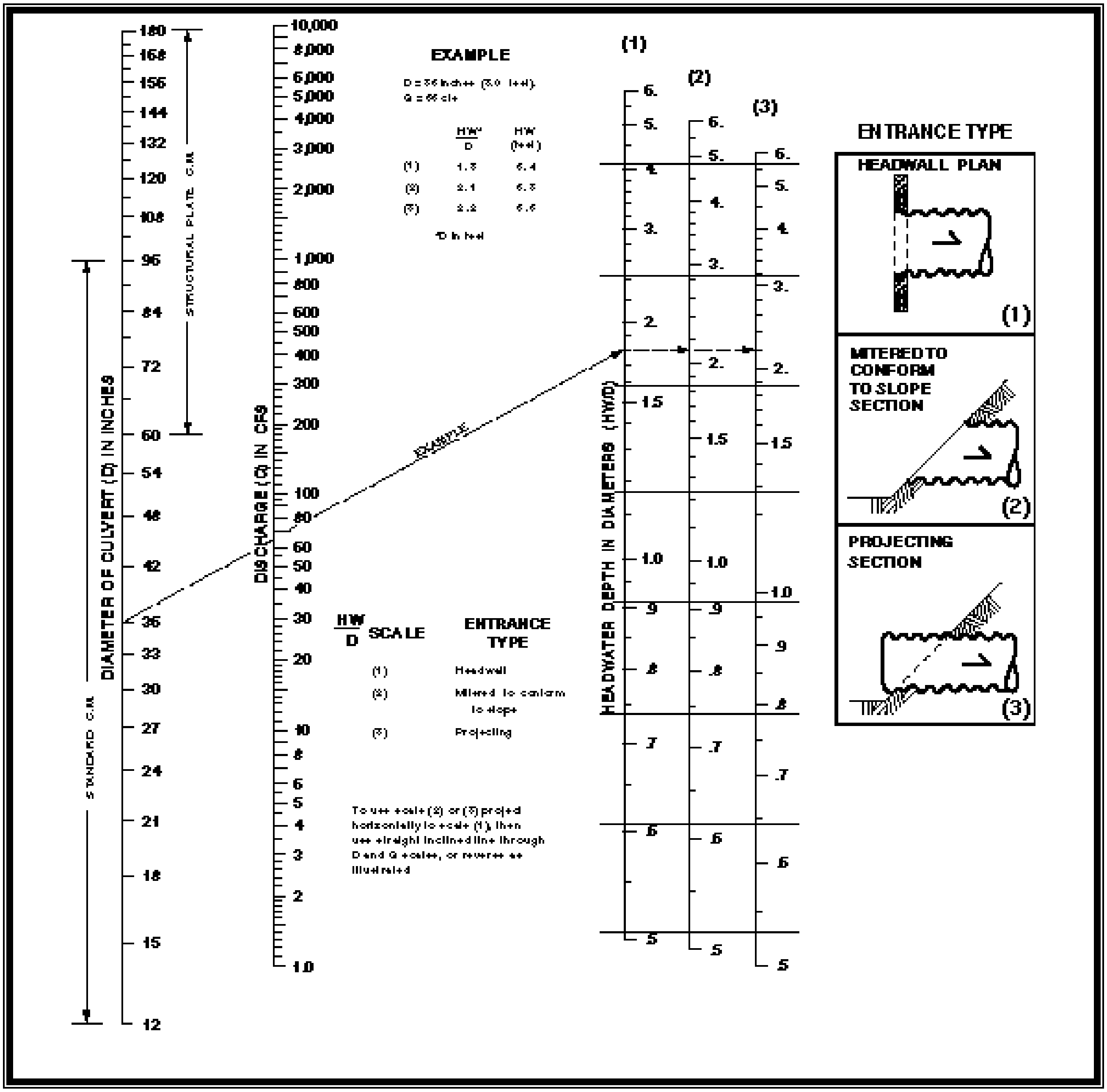


Figure C.2. Headwater Depth for Corrugated Pipe Culverts with Inlet Control.

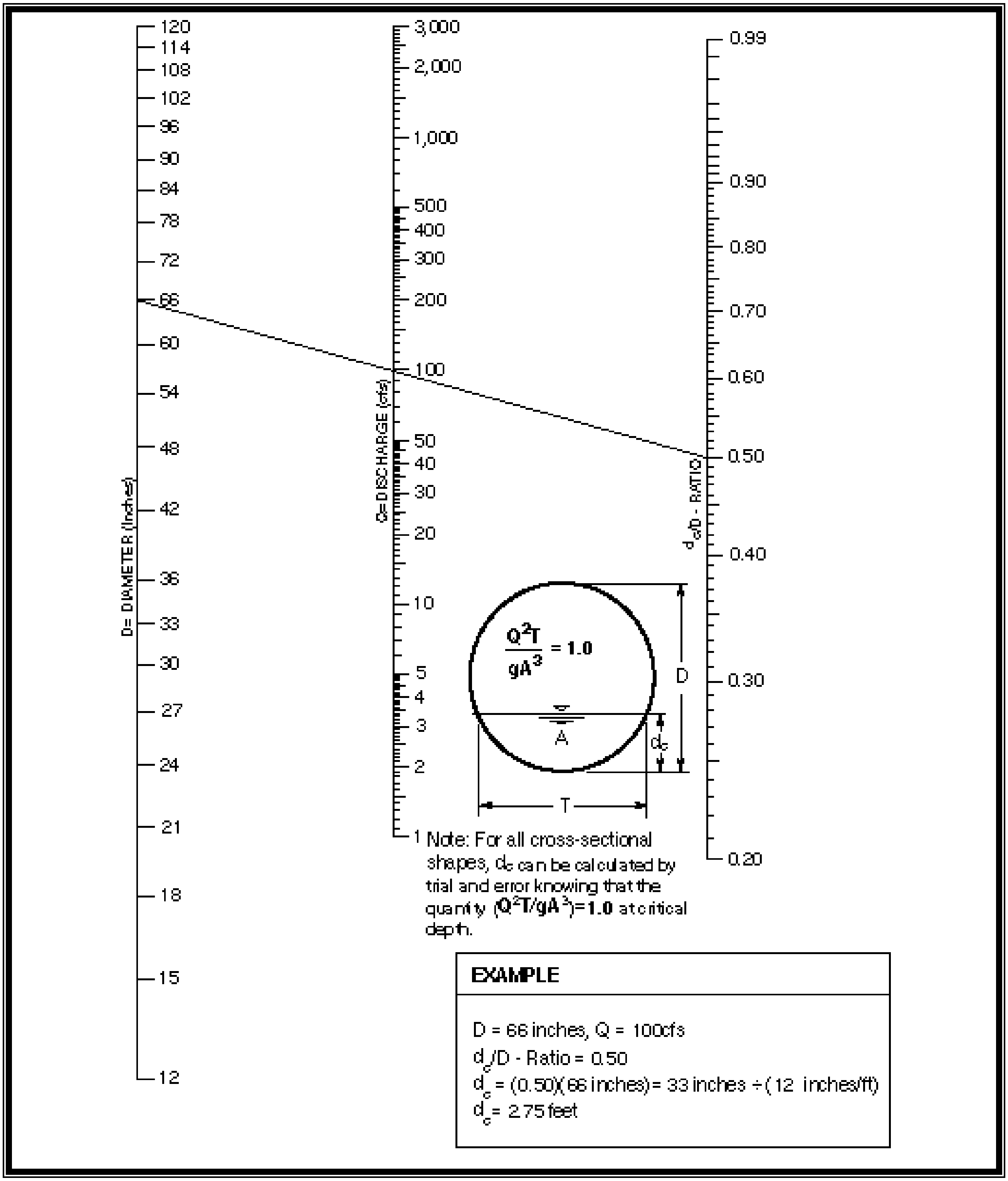


Figure C.3. Critical Depth of Flow for Circular Culverts.

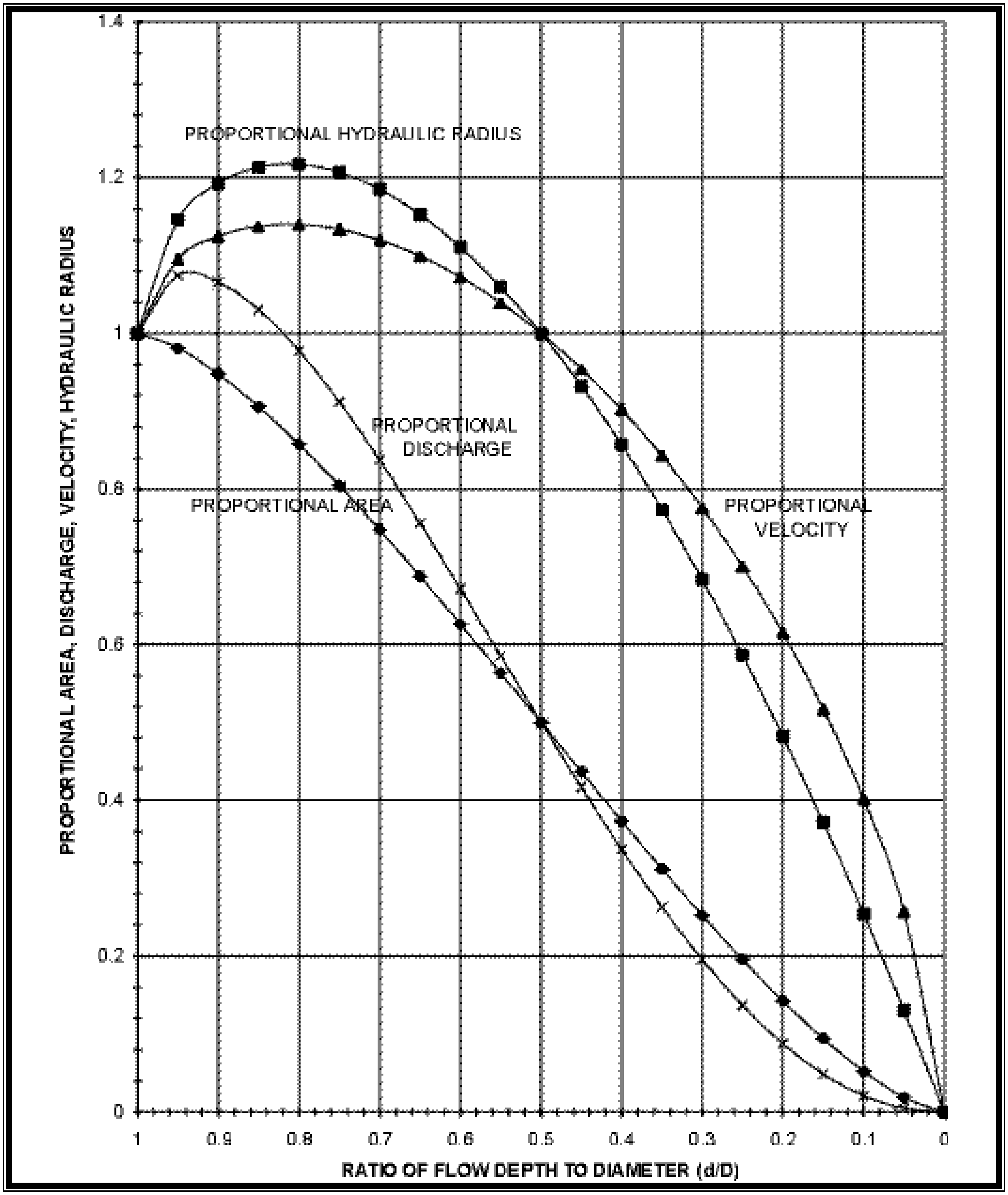


Figure C.4. Circular Channel Ratios.