Chapter 5. Hydrologic Considerations for Airports

This chapter presents an overview of some of the hydrologic considerations and methods of analysis applicable to design and selection of stormwater BMPs in airport settings in eastern and western Washington.

Designers may also want to consult the following references for additional information related to hydrologic design of stormwater BMPs:

- Continuous simulation hydrologic modeling using the MGSFlood model – see the Highway Runoff Manual (HRM).
- Continuous simulation hydrologic modeling using the Western Washington hydrology Model (WWHM) (western Washington only) – see the SMMWW.
- Single-event models (based on the Natural Resources Conservation Service [NRCS] unit hydrograph and the Santa Barbara Urban Hydrograph [SBUH] Methods) – see the HRM.
- Simplified Method for determining infiltration rates – see the SMMWW.
- Closed depression analysis – see the HRM.

Section 5-1 provides an overview of the differences in hydrologic analysis for airport projects from the analytical methods presented in the SMMWW, SMMEW, and HRM.

Section 5-2 summarizes methods of analysis, depending on the facility type (flow-based or volume-based) and location (eastern or western Washington; on- or off-line).

Section 5-3 provides guidance for design of infiltration facilities, including assessing site suitability criteria and determining the long-term infiltration rate.

Section 5-4 provides specific hydrologic design guidance for some of the BMPs included in Chapter 6.

5-1. Airport-specific Hydrologic Design Considerations

The methods of analysis presented in this chapter are similar to those in the HRM and Ecology manuals with the following exceptions:

- The minimum recommended infiltration rate is 1 inch per hour, rather than 0.5 inches per hour, for use of infiltration facilities for flow control.

- There must be 5 feet minimum distance from the bottom of the infiltration facilities to the groundwater elevation or bedrock. Reducing the distance to 3 feet based on site-specific information as allowed in the HRM and SMMWW is not recommended at airports, providing extra certainty that surface ponding will not occur.

5-2. Methods of Analysis

Tables 5-1 and 5-2 summarize the hydrologic methods of analysis for sizing runoff treatment facilities in western and eastern Washington, respectively.

Table 5-1. Criteria for sizing runoff treatment facilities in western Washington.

Facility Type	Criteria	Model
Flow-based (except for biofiltration swales): upstream of flow control facility (on-line and off-line)	Size treatment facility so that 91 percent of the annual average runoff will receive treatment at or below the design-loading criteria, under postdeveloped conditions. If the flow rate is split upstream of the treatment facility, use the off-line flow rates.	Use an approved continuous simulation model using 15-minute time steps.
Flow-based (except for biofiltration swales): downstream of flow control facility	Size treatment facility using the full 2-year release rate from the detention facility, under postdeveloped conditions.	Use an approved continuous simulation model using 1-hour time steps.
Volume-based (on-line and off-line)	*Wet pool—Volume-based, infiltration, or filtration:* Size the facility to treat 91 percent of the estimated historic runoff file for the postdeveloped conditions. OR *Wet pool*: Size treatment facility using the runoff volume predicted for the 6-month, 24-hour design storm under the postdeveloped conditions. This design storm is approximately 72 percent of the 2-year, 24-hour design storm or 91st percentile, 24-hour runoff volume.	Use an approved continuous simulation model with 1-hour time steps OR Use a single event model (SBUH).
Biofiltration swales	Peak design flow rate estimated by SBUH for a 6-month, 24-hour storm with a Type 1A rainfall distribution. Swale must be designed with a 9-minute residence time under design flow rate.	Use an approved continuous simulation model with 15-minute time steps multiplied by correction factors from Figures 9.6a or 9.6b from Ecology (2005) (depending on whether facility is off-line or on-line) OR Use a single event model (SBUH).

SBUH – Santa Barbara Urban Hydrograph method, based on Natural Resources Conservation Service (NRCS) (formerly the Soil Conservation Service [SCS]) curve number equations.

Table 5-2. Criteria for sizing runoff treatment facilities in eastern Washington.

Facility Type	Criteria	Model
Volume-based	Size facility using the runoff volume predicted for the 6-month, 24-hour storm event under postdeveloped conditions.	Use a single event model (NRCS method or SBUH). Climatic Regions 1 through 4 use a Regional Storm (see Section 5-2.2). Use a Type 1A storm for Climatic Regions 2 and 3.
Flow-based: upstream of detention/retention facility	Size facility using the peak flow rate predicted for the 6-month, short duration storm under postdeveloped conditions.	Use a single event model (NRCS or SBUH). Short duration storm.
Flow-based: downstream of detention facility	Size facility using the full 2-year release rate from the detention facility, under postdeveloped conditions.	Use a single event model (NRCS or SBUH). Short duration storm. Climatic Regions 1 through 4 Regional Storm; OR use a Type 1A storm for Climatic Regions 2 and 3, whichever produces the greatest flow.

SBUH – Santa Barbara Urban Hydrograph method, based on Natural Resources Conservation Service (NRCS) (formerly SCS) curve number equations.

5-2.1. Western Washington

Runoff Treatment and Flow Control BMPs Other Than Wet Pool Treatment Facilities

For all flow control and runoff treatment BMPs in western Washington, Ecology requires that a calibrated continuous simulation hydrologic model based on the U.S. EPA's HSPF (Hydrologic Simulation Program-Fortran) program, or an approved equivalent model, be used to calculate runoff and determine the water quality design flow rates and volumes.

The Western Washington Hydrology Model (WWHM) is one approved model, which is available for download at Ecology's website (Ecology 2008b). WSDOT prefers that project proponents for WSDOT projects use MGSFlood or the public domain version of MGSFlood, known as the Western Washington Highways Hydrology Analysis Model (WHAM). WHAM is available for download at WSDOT's website (WSDOT 2008c).

Wet Pool Facilities

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

5-2.2. Eastern Washington

Runoff Treatment Facilities

Runoff treatment facilities may be analyzed using one of the following methods in eastern Washington:

- Single event hydrograph methods (NRCS hydrograph and Santa Barbara urban hydrograph [SBUH])
- NRCS curve number equations
- Level-pool routing
- Rational method.

Flow Control Facilities

Flow control facilities may be analyzed using one of the following methods in eastern Washington:

- Single event hydrograph methods (NRCS hydrograph and SBUH)
- Level-pool routing
- Continuous runoff model or other hydrograph modeling method, if available.

Eastern Washington Design Storm Events

When WSDOT analyzed rainfall patterns during storms in eastern Washington, it concluded that the NRCS Type II rainfall does not match the historical records. Two types of storms were found to be prominent on the east side of the state: short-duration thunder storms (later spring through early fall seasons) and long-duration winter storms (any time of year, but most common in the late fall through winter period and the late spring and early summer period).

The short-duration storm generates the greatest peak discharges and should be used to design flow-based BMPs. The long duration storm occurs over several days, generating the greatest volume, and should be used to design volume-based BMPs.

When using the long-duration storm, it should be noted that eastern Washington has been divided into the following four climatic regions:

1. East Slope Cascades
2. Central Basin
3. Okanogan, Spokane, Palouse
4. NE and Blue Mountains.

The long-duration storms in Regions 2 and 3 are similar to the NRCS Type 1A storm.

Designers in those regions can choose to use either the long-duration storm or the NRCS Type 1A storm. Eastern Washington design storm events are further discussed in Appendix 4C of the HRM.

5-3. Infiltration Design Guidance

An infiltration facility provides stormwater flow control by containing excess runoff in a storage facility, then percolating that runoff into the surrounding soil. Infiltration facilities can provide runoff treatment and flow control, but to do so requires certain soil characteristics. Section 5-3.1, *Site Suitability Criteria*, provides a detailed discussion of soil characteristics needed to determine which type of infiltration facility is most appropriate for the site.

Chapter 6 lists many types of infiltration BMPs. Some of these facilities include ponds, vaults, trenches, and drywells, along with partial infiltration facilities such as natural and engineered dispersion and compost-amended vegetated filter strips (CAVFS).

This section provides design criteria on the various ways to determine infiltration rates and facility size, dependent on the facility and whether infiltration occurs at the surface or below the surface (subsurface). The simplified approach for determining infiltration rates is not included in this manual. Refer to the HRM or SMMWW for the simplified approach.

Surface infiltration BMP designs and subsurface infiltration BMP designs follow different criteria. Infiltration ponds, infiltration vaults, infiltration trenches (designed to intercept sheet flow), dispersion, and CAVFS are considered surface infiltration BMPs and are based on infiltration rates. To compute these infiltration rates, determination of the soil's saturated hydraulic conductivity must be completed. Infiltration trenches designed as an end-of-pipe application (with underdrain pipe) and drywells are considered subsurface infiltration BMPs and regulated by the Underground Injection Control (UIC) Rule, which is intended to protect underground sources of drinking water. As a result, subsurface infiltration BMPs are known as underground injection facilities and designed dependent on the treatment capacity of the subsurface soil conditions.

The sections that follow provide detailed information on site suitability criteria, saturated hydraulic conductivity determination, determination of infiltration rates, and underground injection facilities.

If the infiltration facility is designed for flow control, the minimum long-term infiltration rate of the native underlying soils must be at least 1 inch per hour, calculated as described in Section 5-3.3 (rather than the HRM requirement of minimum 0.5 inches per hour). This revised guideline is based on the FAA recommendation that open stormwater management facilities at airports be designed to drain within 48 hours of the conclusion of a storm event to eliminate the attraction to waterfowl presented by an open pool of water (FAA 2004a). Based on hydrologic modeling of 50 years of historical rainfall data, it was estimated that infiltration ponds with a design infiltration rate of 0.5 inches per hour would have standing water for greater than 48 hours six times over the 50 years of the record analyzed. The same modeling approach showed that no instances of ponding for more than 48 hours would occur with a design infiltration rate of 1 inch per hour (see Appendix B for details of this ponding analysis).

5-3.1. Site Suitability Criteria (SSC)

This section specifies the site suitability criteria that must be considered for siting stormwater infiltration systems. When a site investigation reveals that any of the following nine applicable criteria cannot be met, appropriate mitigation measures must be implemented so that the infiltration facility will not pose a threat to safety, health, or the environment or an alternative flow control facility should be selected.

For infiltration treatment, site selection, and design decisions, a qualified engineer with geotechnical and hydrogeologic experience should prepare a geotechnical and hydrogeologic report. A comparable professional may also conduct the work if it is under the seal of a registered Professional Engineer (P.E.). The design engineer may use a team of certified or registered professionals in soil science, hydrogeology, geology, and other related fields.

To design infiltration facilities, the following SSC must be followed (if applicable), in addition to those described in the BMP guidelines (Chapter 6).

SSC 1 – Setback Requirements

Setback requirements for infiltration facilities are generally provided in local regulations, Uniform Building Code requirements, or other state regulations. The following setback criteria apply to infiltration facilities at airports, unless otherwise required by critical area ordinance or other jurisdictional authorities:

- Infiltration ponds and other infiltration facilities must be located outside of the RSA and TSA.
- Infiltration facilities should be located a minimum of 20 feet downslope and 100 feet upslope from building foundations, and 50 feet or more from the top of slopes steeper than 15 percent. The designer should request a geotechnical report for the project that would evaluate structural site stability impacts because of extended subgrade saturation and/or head loading of the permeable soil layer, including the potential impacts on downgradient properties (especially on hills with known side-hill seeps). The report should address the adequacy of the proposed BMP locations and recommend any adjustments to the setback distances provided above, either greater or smaller, based on the results of this evaluation.

- Infiltration facilities must be located far enough from runways, taxiways, and other airport facilities as well as buildings to avoid threatening the structural stability. A professional engineer should be consulted for this analysis. In addition, adequate distance for vegetative treatment must be allowed between the receiving water and runways, taxiways, and other areas treated with deicing chemicals, if such chemicals in runoff are not treated with a designed system. Distances between 30 and 600 feet have been reported for effects related to deicers, depending on their type (NCHRP 2005).
- Infiltration facilities should be set back at least 100 feet from drinking water wells, septic tanks or drain fields, and springs used for public drinking water supplies. Infiltration facilities upgradient of drinking water supplies and within the 1-, 5-, and 10-year time of travel zones must comply with health department requirements (Washington Wellhead Protection Program, Washington Administrative Code [WAC] 246-290-135).
- Infiltration facilities must be located at least 20 feet from a native growth protection easement (NGPE).
- Infiltration facilities must be a minimum of 5 feet from any property line and vegetative buffer. This distance may be increased based on permit conditions required by the local jurisdiction.

SSC 2 – Groundwater Protection Areas

A site is not suitable if the infiltration facility will cause a violation of Ecology's groundwater quality standards (WAC 173-200) (see SSC 9 for verification testing guidance). Local jurisdictions should be consulted for applicable pollutant removal requirements upstream of the infiltration facility, and to determine whether the site is located in an aquifer protection area, a sole-source aquifer recharge area, or a wellhead protection zone.

SSC 3 – High Vehicle Traffic Areas

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

High vehicle traffic areas include the following:

- Commercial or industrial sites subject to an expected average daily traffic count (ADT) ≥100 vehicles/1,000 ft² gross building area (trip generation)

- Road intersections with an ADT of ≥ 25,000 on the main roadway, or ≥15,000 on any intersecting roadway
- Loading and unloading areas at airport terminals
- Parking areas at airports
- Aircraft gates.

SSC 4 – Soil Infiltration Rate

For infiltration facilities used for water quality treatment purposes, the short-term soil infiltration rate should be 2.4 inches/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 (see SSC 6). It is comparable to the textures represented by Hydrologic Groups B and C (see *hydrologic soil groups*, in the Glossary). Long-term infiltration rates up to 2.0 inches/hour can also be considered, if the infiltration receptor is not a sole-source aquifer and in the judgment of the site professional, if the treatment soil has characteristics comparable to those specified in SSC 6 to adequately control the target pollutants.

The long-term infiltration rate (calculated in accordance with the methods described in Sections 5-3.2 and 5-3.3 for western Washington; or Section 5-3.4 for eastern Washington) should also be used for maximum drawdown time and routing calculations.

Drawdown Time

If sizing an infiltration facility for treatment in western Washington, the designer should document that the 91st percentile, 24-hour runoff volume (indicated by WWHM or MGSFlood) can infiltrate through the infiltration BMP surface within 36 hours.

If designing an infiltration facility for flow control in eastern Washington, the designer should confirm that the runoff volume associated with the design storm (in accordance with Core Element #6 this would typically be the 25-year, 24-hour design storm unless a higher level of flow control is required by a local jurisdiction) will infiltrate within 48 hours. This can be determined through equation 5-1.

$$t_{dd} = V/(A_{mid} \cdot i) \qquad (5\text{-}1)$$

where:
- t_{dd} = drawdown time (hours)
- V = runoff volume associated with design storm (cubic feet)
- A_{mid} = area of the midpoint of the storage volume of the infiltration facility (square feet)
- i = infiltration rate (inches per hour)

WWHM and MGSFlood do not readily allow the designer to determine runoff volumes (except the water quality volume) or drawdown times. Based on an analysis of 50 years of historic data, Parametrix found that infiltration facilities sized to meet the Ecology duration standard with native underlying soils with an infiltration rate of at least 1.0 inch per hour had no occurrences of inundation for greater than 48 hours during the period of analysis (Parametrix 2007 [included in Appendix B]). Based on this analysis, it is assumed that infiltration facilities designed for flow control in western Washington in accordance with the recommendations of this chapter meet the drawdown time criteria.

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 organic matter in the soil
- Comply with the FAA recommendation that open stormwater management facilities at airports be designed to drain within 48 hours of the conclusion of a storm event to eliminate the attraction to waterfowl presented by an open pool of water (FAA 2004a).

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

The base of all infiltration basins or trench systems shall be ≥ 5 feet above the seasonal high-water table, bedrock (or hardpan), or other low permeability layer.

SSC 6 – Soil Physical and Chemical Suitability for Treatment

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

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. The following soil properties must be carefully considered in making such a determination:

- Cation exchange capacity (CEC) of the treatment soil must be greater than or equal to 5 milliequivalents CEC/100 g dry soil (U.S. EPA Method 9081).
- Depth of soil used for infiltration treatment must be a minimum of 18 inches.
- Organic content of the treatment soil (ASTM D 2974): Organic matter can increase the sorptive capacity of the soil for some pollutants. The designer should evaluate whether the organic matter content is sufficient for control of the target pollutant(s).

- Waste fill materials should not be used as infiltration soil media nor should such media be placed over uncontrolled or nonengineered fill soils.
- Engineered soils may be used to meet the design criteria in this chapter and the performance goals in Chapters 3 and 4 of Volume V of the Ecology manuals. Field performance evaluation(s), using acceptable protocols, would be needed to determine feasibility and acceptability by the local jurisdiction.

SSC 7 – Seepage Analysis and Control

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

Infiltration of stormwater is not recommended on or upgradient of a contaminated site where infiltration of even clean water can cause contaminants to mobilize.

Sidewall seepage is not usually a concern if seepage occurs through the same stratum as the bottom of the facility. However, for engineered soils or soils with very low permeability, the potential to bypass the treatment soil through the sidewalls may be significant. In those cases, the sidewalls must be lined, either with an impervious liner or with at least 18 inches of treatment soil, to prevent seepage of untreated flows through the sidewalls.

SSC 8 – Cold Climate and Impact of Deicers

- For cold climate design criteria (snowmelt/ice impacts), refer to Caraco and Claytor (1997).
- The potential impact of deicers on potable water wells must be considered in the siting determination. Mitigation measures must be implemented if infiltration of deicers could cause a violation of groundwater quality standards.

SSC 9 – Verification Testing of the Completed Facility

Verification testing of the completed full-scale infiltration facility is recommended to confirm that the design infiltration parameters are adequate. The site analysis professional should determine the duration and frequency of the verification testing program, including the monitoring program for the potentially impacted groundwater. Groundwater monitoring wells may be used for this purpose. Long-term (more than 2 years) in-situ drawdown and confirmatory monitoring of the infiltration facility would be preferable.

5-3.2. Simplified Approach to Determining Infiltration Rates (Western Washington)

The Stormwater Management Manual for Western Washington (Ecology 2005) provides two alternatives for determining infiltration rates: the simplified approach and the detailed approach. The simplified approach and the associated three methods for determining long-term infiltration rates are described in Sections 3.3.4 and 3.3.6 of the Ecology manual, respectively.

5-3.3. Detailed Approach to Determining Infiltration Rates (Western Washington)

The detailed approach was obtained from Massmann (2003). Procedures for the detailed approach are as follows (see Figures 5-1 and 5-2 for a flowchart of this process):

1. Select a location:

   This will be based on the ability to convey flow to the location and the expected soil conditions. The minimum setback distances must also be met. (See Site Suitability Criteria, SSC 1.)

2. Estimate volume of stormwater, V_{design}:

   For eastern Washington, a single event hydrograph or value for the volume can be used, allowing a modeling approach such as StormShed to be conducted. For western Washington, a continuous hydrograph should generally be used, requiring a model such as WWHM or MGSFlood to perform the calculations.

3. 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 1.0 inches/hour. This trial geometry should be used to locate the facility, and for planning purposes in developing the geotechnical subsurface investigation plan.

4. 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 other licensed professional judges that conditions are highly variable and make it necessary to increase the depth or the number of explorations to accurately

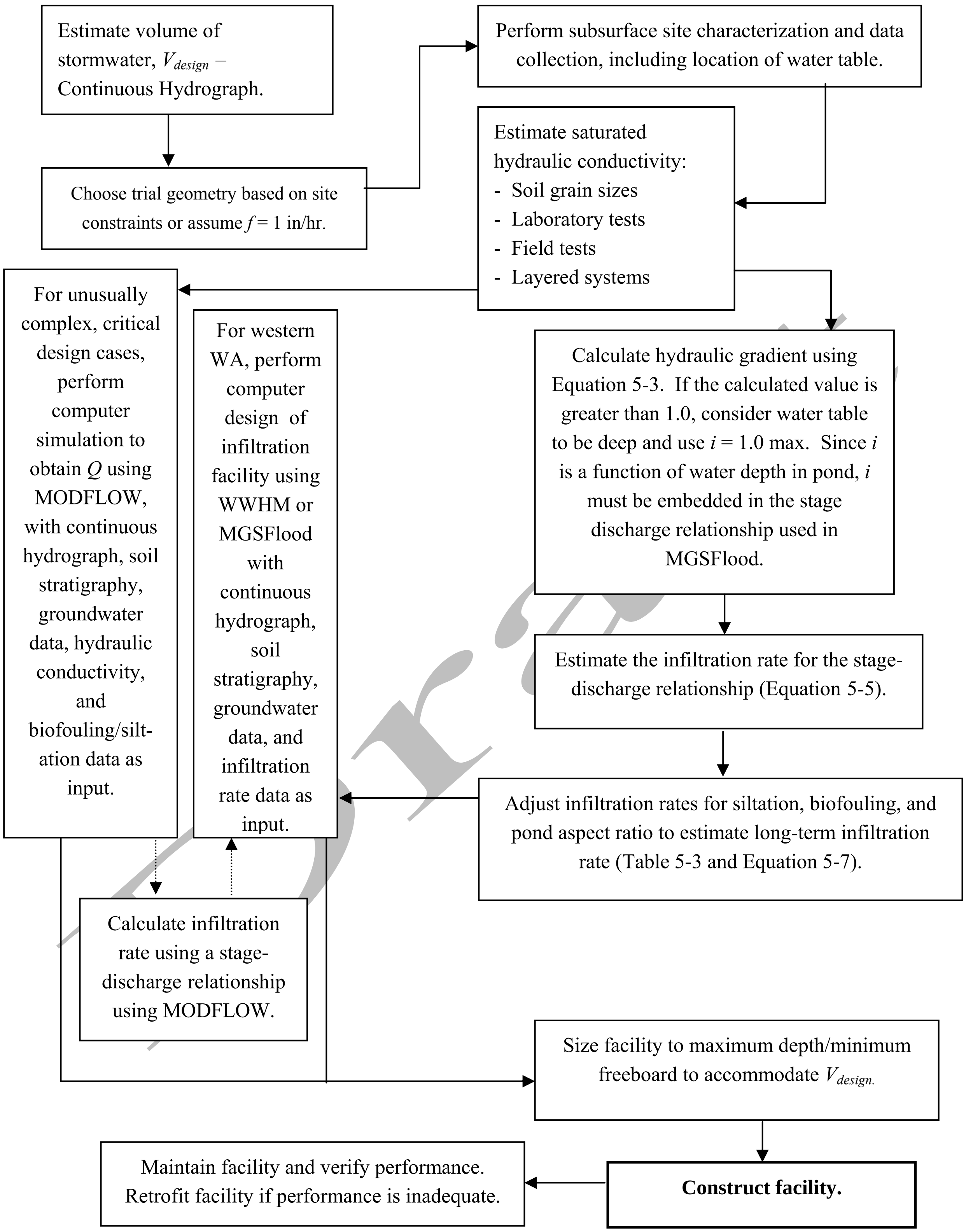

Figure 5-1. Engineering design steps for final design of infiltration facilities using the continuous hydrograph method (western Washington).

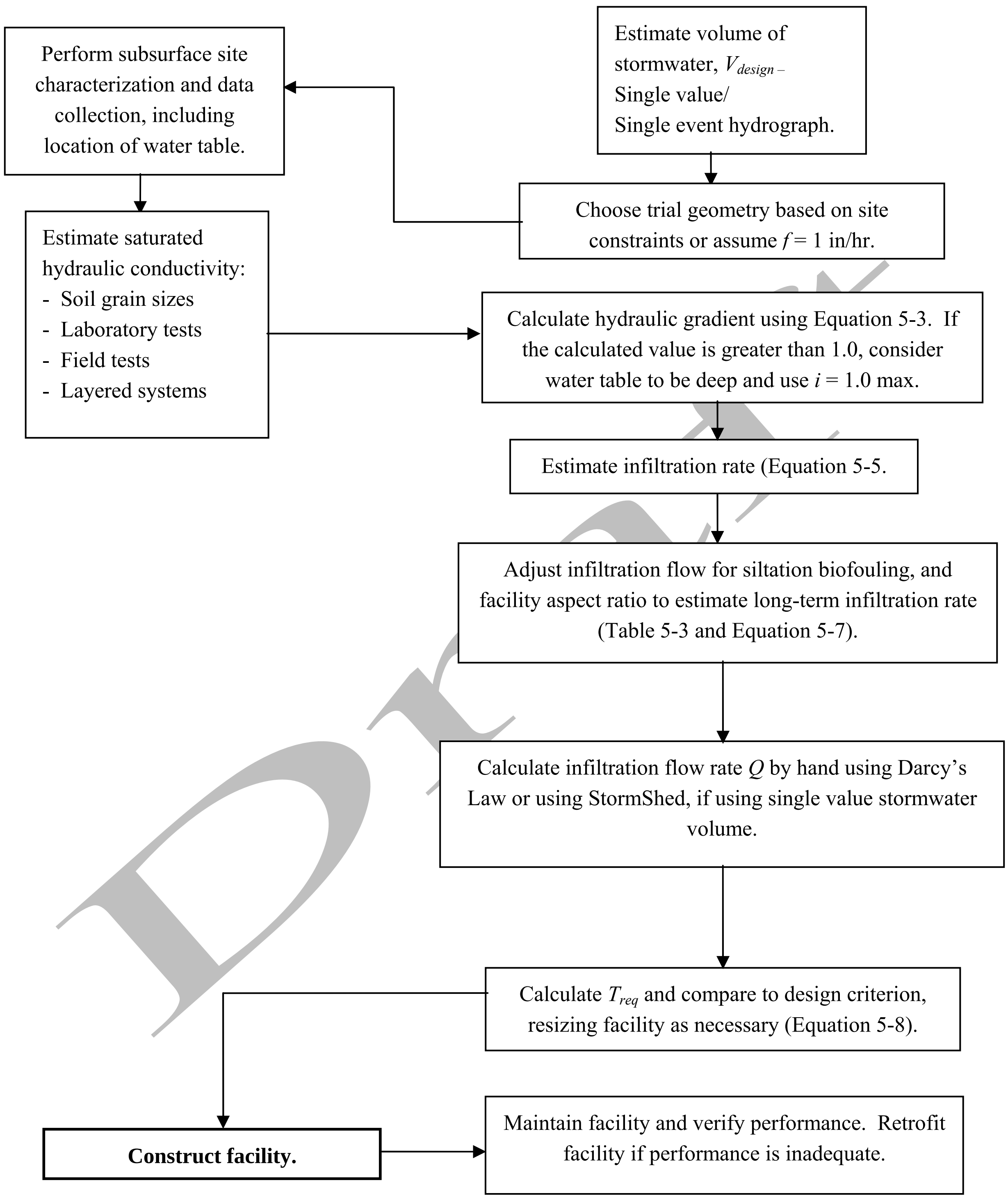

Figure 5-2. Engineering design steps for final design of infiltration facilities using the single hydrograph method (eastern Washington).

estimate the infiltration system's performance. The exploration program described below may be decreased if a licensed engineer with geotechnical expertise (P.E.) or other licensed professional judges that conditions are relatively uniform, or that 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 bottom surface area.
- For infiltration trenches, at least one test pit or test hole per 100 feet of trench length.
- For drywells, samples should be collected from each layer beneath the facility to the depth of groundwater or to approximately 40 feet below the ground surface (approximately 30 feet below the base of the drywell). 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.
- 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.
- Groundwater monitoring wells installed to locate the groundwater table and establish its gradient, direction of flow, and seasonal variations, considering both confined and unconfined aquifers. (Monitoring through at least one wet season is required, unless site historical data regarding groundwater levels are available.) In general, a minimum of three wells per infiltration facility, or three hydraulically connected surface or groundwater features, are needed to determine the direction of flow and gradient. If gradient and flow direction are not required and there is low risk of downgradient impacts, one monitoring well is sufficient. Alternative means of establishing the groundwater levels may be considered. If the groundwater in the area is known to be greater than 50 feet below the proposed facility, detailed investigation of the groundwater regime is not necessary.
- 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.

5. From the geotechnical investigation, determine the following, as applicable:

- 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 groundwater table and to any bedrock/impermeable layers.
- Seasonal variation of the groundwater table.
- The existing groundwater 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.
- The 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.
- For other aspects of the design of infiltration facilities, see Chapter 6.

6. Determine the saturated hydraulic conductivity as follows:

The geotechnical investigation will typically provide a computation of the saturated hydraulic conductivity (K_{sat}) for the area proposed for infiltration. In those cases where the K_{sat} is not provided, the designer can determine the K_{sat} value by referring to the detailed approach in this section or by the Guelph Permeameter described in the HRM (applicable to eastern Washington only).

The K_{sat} derived using the detailed approach can then be used to design the following:

- Infiltration pond (BMP AR.04)
- Infiltration trench (BMP AR.05)
- Infiltration vault (BMP AR.06)
- Underlying soils of CAVFS (BMP AR.12)
- Drywell (BMP AR.07)
- Natural dispersion (BMP AR.01).

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 in cm/second using the following relationship (see Massmann [2003], and Massmann et al. [2003]):

$$\log_{10}(K_{sat}) = -1.57 + 1.90D_{10} + 0.015D_{60} - 0.013D_{90} - 2.08f_{fines} \tag{5-1}$$

where: Ksat = the saturated hydraulic conductivity in cm/second D10, D60, and D90 = grain sizes in mm for which 10%, 60%, and 90% of the sample is more fine
ffines = the fraction of the soil (by weight) that passes the number-200 sieve

Use the following equation to convert K_{sat} from cm/second to ft/day:

K_{sat} (ft/day) = K_{sat} (cm/s) x 2,834.65

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

If the soil layer being characterized has been exposed to heavy compaction, or is overconsolidated because of 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 et al. 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} because of 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, 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 Ecology's SMMWW. Note that these field tests generally provide a hydraulic conductivity combined with a hydraulic gradient (see Equation 5-4). In some of these tests, the hydraulic gradient may be close to 1.0; therefore, in effect, the magnitude of the test result is the same as the hydraulic conductivity. In other cases, the hydraulic gradient may be close to the gradient that is likely to occur in the full-scale infiltration facility. This issue will need to be evaluated on a case-by-case basis when interpreting the results of field tests. It is important to recognize that the gradient in the test may not be the same as the gradient likely to occur in the full-scale infiltration facility in the long-term (i.e., when groundwater mounding is fully developed).

- Once the saturated hydraulic conductivity for each layer has been identified, determine the effective average saturated hydraulic conductivity below the pond. Hydraulic conductivity estimates from different layers can be combined using the harmonic mean:

$$K_{equiv} = \frac{d}{\sum \frac{d_n}{K_{sat\_n}}} \qquad (5\text{-}2)$$

where: K_{equiv} = the average saturated hydraulic conductivity in ft/day
d = the total depth of the soil column in feet
d_n = the thickness of layer "n" in the soil column in feet
K_{sat_n} = the saturated hydraulic conductivity of layer "n" in the soil column in ft/day.

The depth of the soil column, d, typically would include all layers between the pond bottom and the water table. However, for sites with very deep water tables (>100 feet) where groundwater mounding to the base of the pond is not likely to occur, it is recommended that the total depth of the soil column in Equation 5-2 be limited to approximately 20 times the depth of the 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 5-2. Equation 5-2 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 5 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 5-2. The harmonic mean given by Equation 5-2 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 with groundwater mounding).

For the soils underlying a CAVFS, a correction factor should be applied to the saturated hydraulic conductivity to account for compaction in the embankment. A correction factor of 10 (1/10th of the estimated K_{sat} determined by Equation 4-12) should be used for "wellgraded sands and gravels with moderate-to-high silt content." For clean, uniformly graded sands and gravels, a correction factor of 5 should be used, and a correction factor of 15 should be applied to K_{sat} for soils that contain clay.

For drywells, once the saturated hydraulic conductivity for each layer has been identified, the designer must convert the saturated hydraulic conductivity to (ft/min) and then calculate the geometric mean of the multiple saturated hydraulic conductivity values. The HRM has additional guidance on determining the geometric mean of the saturated conductivity values.

7. For unusually complex, critical design cases, develop input data for a simulation model:

   Use MODFLOW, including trial geometry, continuous hydrograph data, soil stratigraphy, groundwater data, hydraulic conductivity data, and reduction in hydraulic conductivity due to siltation or biofouling on the surface of the facility. Use of this approach will generally be fairly rare. Otherwise, skip this step and develop the data needed to estimate the hydraulic gradient, as shown in the following steps.

8. Calculate the hydraulic gradient:

   The steady state hydraulic gradient is calculated as follows:

$$\text{gradient} = \text{i} \approx \frac{D_{wt} + D_{pond}}{138.62(K_{equiv}{}^{0.1})} CF_{size} \tag{5-3}$$

where:
- i = steady state hydraulic gradient
- D_{wt} = the depth from the base of the infiltration facility to the water table in feet
- K_{equiv} = the average saturated hydraulic conductivity in feet/day
- D_{pond} = the depth of water in the facility in feet (see Massmann et al. 2003 for the development of this equation)
- CF_{size} = the correction for pond size.

The correction factor was developed for ponds with bottom areas between 0.6 and 6 acres in size. For small ponds (ponds with area less than or equal to 2/3 acre), the correction factor is equal to 1.0. For large ponds (ponds with area greater than or equal to 6 acres), the correction factor is 0.2, as shown in Equation 5-4.

$$CF_{size} = 0.73(A_{pond})^{-0.76} \tag{5-4}$$

where: A_{pond} = the area of pond bottom in acres.

This equation generally will result in a calculated gradient of less than 1.0 for moderate to shallow groundwater depths (or to a low permeability layer) below the facility, and conservatively accounts for the development of a groundwater mound. A more detailed groundwater mounding analysis, using a program such as MODFLOW, will usually result in a gradient that is equal to or greater than the gradient calculated using Equation 5-3. 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 groundwater of 100 feet or more is required to obtain a gradient of 1.0 or more using Equation 5-3. Since the gradient is a function of depth of water in the facility, the gradient will vary as the pond fills during the season. Therefore, the gradient must be calculated as part of the stage-discharge calculation used in MGSFlood for the continuous hydrograph method. For designs using the single event hydrograph, it is sufficiently accurate to calculate the hydraulic gradient based on one-half of the maximum depth of water in the pond.

9. Calculate the infiltration rate using Darcy's Law as follows:

$$f = 0.5K_{equiv}\left(\frac{dh}{dz}\right) = 0.5K_{equiv}(i) \tag{5-5}$$

where:

- f = the infiltration rate of water through a unit cross section of the infiltration facility (in/hr)
- K_{equiv} = the average saturated hydraulic conductivity (ft/day)
- dh/dz = the steady state hydraulic gradient
- i = the steady state hydraulic gradient
- 0.5 = converts ft/day to in/hr.

10. Adjust the infiltration rate or infiltration stage-discharge relationship obtained in Steps 8 and 9:

This is done to account for reductions in the rate resulting from long-term siltation and biofouling, taking into consideration the degree of long-term maintenance and performance monitoring anticipated, the degree of influent control (e.g., presettling ponds, biofiltration swales), 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 rates estimated in Steps 8 and 9 are multiplied by the reduction factors summarized in Table 5-3.

Table 5-3. Infiltration rate reduction factors to account for biofouling and siltation effects for ponds.

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

Based on Massmann (2003).

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).

An example of a situation with a high potential for biofouling would be a pond located in a shady area 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 over the long term, and no pretreatment (e.g., presettling ponds, biofiltration swales) is provided. 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.

Adjust this infiltration rate for the effect of pond aspect ratio by multiplying the infiltration rate determined in Step 9 (Equation 5-5) by the aspect ratio correction factor CF_{aspect} as shown in the following equation. In no case shall CF_{aspect} be greater than 1.4.

$$CF_{aspect} = 0.02A_r + 0.98 \tag{5-6}$$

where: CF_{aspect} = the aspect ratio correction factor
A_r = the aspect ratio for the pond (length/width).

The final infiltration rate will therefore be as follows:

$$f = (0.5K_{equiv})(\mathrm{i})(CF_{aspect})(CF_{silt/bio}) \tag{5-7}$$

The infiltration rates calculated based on Equations 5-5 and 5-6 are long-term design rates. No additional reduction factor or factor of safety is needed. If the design infiltration rate is less than 1 inch per hour, an infiltration facility may not be used for flow control.

11. Determine the infiltration flow rate Q:

    If the infiltration facility is located in eastern Washington, determine the infiltration flow rate Q using the Infiltration Calculation Spreadsheet in Ecology (2008c). If located in western Washington, determine the infiltration flow rate Q using MGSFlood.

12. Size the facility:

    Use one of the following two approaches to size the facility, depending on the type of hydrograph used:

    - If using a continuous hydrograph for design, size the facility to ensure that the desirable pond depth is 3 feet, with 1-foot-minimum required freeboard. The maximum allowable pond depth is 6 feet.
    - If using a single event/single hydrograph, calculate T_{req}, using StormShed to determine the time it takes the pond to empty, or from the value of Q determined from Step 11 and V_{design} from Step 2 as follows:

$$T_{req} = \frac{V_{design}}{Q} \tag{5-8}$$

where: T_{req} = the time required to infiltrate the design stormwater volume
V_{design} = volume of stormwater in cubic feet
Q = infiltration flow rate in cubic feet per second (cfs).

    This value of T_{req} must be less than or equal to the maximum allowed infiltration time specified in the Site Suitability Criteria.

13. Construct the facility:

    Maintain and monitor the facility for performance.

5-3.4. Design Infiltration Rate Determination (Eastern Washington)

Table 5-4 may be used for determining presumptive rates for surface treatment facilities based on the USDA soil classification or the Unified Soil Classification System. The infiltration rates in Table 5-4 provide conservative estimates based on homogenous soils. They do not consider the effects of site variability and long-term clogging in the infiltration facility.

Table 5-4. Presumptive infiltration rates based on USDA soil classification

USDA Soil Textural Classification	Short-term Infiltration Rate[a]	Correction Factor, CF	Estimated Long-term (Design) Infiltration Rate (inches/hour)
Clean sandy gravels and gravelly sands (i.e., 90% of the total soil sample is retained in the #10 sieve	20	2	10[b]
Sand	8	4	2[c]
Loamy Sand	2	4	0.5
Sandy Loam	1	4	0.25
Loam	0.5	4	0.13

[a] From WEF/ASCE 1998.
[b] Not suitable for infiltration treatment unless justified by geotechnical study and approved by permitting municipality.
[c] Refer to SSC-4 and SSC-6 for treatment acceptability criteria.

For guidance on field tests to determine more accurate, site-specific infiltration rates, refer to Appendix 6B of the SMMEW.

5-4. General BMP Design Guidelines

This section provides hydrologic design guidance for infiltration facilities (Section 5-4.1), compost amended vegetated filter strips (CAVFS) (Section 5-4.2), and volume-based runoff treatment (Section 5-4.3). The information contained in each of these sections is applicable to various BMPs. For example, the infiltration facility design guidance applies to infiltration ponds (BMP AR.04), infiltration trenches (BMP AR.05), infiltration vaults (BMP AR.06), and dry wells (BMP AR.07). Information on determining infiltration rates for soil amendment BMPs in Section 5-4.2 applies to natural and engineered dispersion (BMPs AR.01 and AR.02) as well as to CAVFS. This information is provided in Chapter 5 to minimize redundancy between individual BMP design guidelines (presented in Chapter 6).

5-4.1. Infiltration Facilities

This section covers hydrologic design guidelines and considerations for infiltration basins and trenches.

Design Criteria – Sizing Facilities (Western Washington)

The size of the infiltration facility can be determined by routing the influent runoff file generated by the continuous runoff model through the facility. To prevent the onset of anaerobic conditions, an infiltration facility designed for treatment purposes must be designed to drain the 91st percentile, 24-hour runoff volume within 48 hours (see the explanation under simplified or detailed design procedures). In general, an infiltration facility would have two discharge modes.

The primary mode of discharge from an infiltration facility is infiltration into the ground. However, when the infiltration capacity of the facility is reached, additional runoff to the facility will cause the facility to overflow. Overflows from an infiltration facility must comply with the Minimum Requirement 7 for flow control in Volume I of the Ecology manuals. Infiltration facilities used for runoff treatment must not overflow more than 9 percent of the influent runoff file, by volume.

To determine compliance with the flow control requirements, the WWHM, MGSFlood, or an appropriately calibrated continuous simulation model based on HSPF must be used. Refer to the SMMWW or HRM for more information on specific modeling procedures for infiltration facilities.

Additional Design Criteria

- Slope of the base of the infiltration facility should be <3 percent.
- A nonerodible outlet structure or spillway with a firmly established elevation must be constructed to discharge overflow. Ponding depth, drawdown time, and storage volume are calculated from that reference point.
- For infiltration treatment facilities, side-wall seepage is not a concern if seepage occurs through the same stratum as the bottom of the facility. However, for engineered soils or for soils with very low permeability, the potential to bypass the treatment soil through the sidewalls may be significant. In such cases, the sidewalls must be lined, either with an impervious liner or with at least 18 inches of treatment soil, to prevent seepage of untreated flows through the sidewalls.

Design Criteria – Sizing Facilities (Eastern Washington)

This section describes the iterative process for designing an infiltration facility in eastern Washington.

Step 1. Develop Trial Geometry

The designer should develop a preliminary geometry for the proposed facility. The design guidelines in Chapter 6 will include criteria for maximum and minimum depth for specific BMPs. Select facility dimensions that meet depth requirements and are reasonable in light of the total storm volume associated with the design storm. Often, site constraints will limit the surface are available for siting a facility.

Step 2. Develop Stage-Discharge Relationship for Facility

The stage-discharge relationship may be determined using Darcy's Law;

$$Q = fiA_s, \qquad (5\text{-}9)$$

where: Q = flow rate at which runoff is infiltrated (cfs)
f = infiltration rate of soil (in/hr). Note that the infiltration rate used in this equation should incorporate a safety factor of 2, such that *f = 2 x design infiltration rate*
i = hydraulic gradient
A_s = surface area of the infiltration BMP (sf).

The hydraulic gradient, i, may be calculated as follows:

$$i = (h+L)/L; \qquad (5\text{-}10)$$

where: h = design depth of facility (feet)

L = distance from the bottom of the BMP to the water table, bedrock, impermeable layer, or soil layer of different infiltration rate (ft)

The stage-storage relationship may be calculated as follows:

$$S = A_s \text{ x h x void ratio} \qquad (5\text{-}11)$$

where: A_s and h are as described above.

Step 3. Level Pool Routing

This section presents the methodology for routing a hydrograph through a stormwater facility using hydrograph analysis. Level pool routing is done the same way regardless of the method used to generate the hydrograph; therefore, this part of the analysis is not unique to the SBUH method. The level pool routing technique presented here is one of the simplest and most commonly used hydrograph routing methods.

This technique is based on the following continuity equation:

Inflow – Outflow = Change in Storage

$$((I1 + I2)/2) - ((O1 + O2)/2) = S2 - S1 \qquad (5\text{-}12)$$

where: $I1$, $I2$ = Inflow at time 1 and time 2
$O1$, $O2$ = Outflow at time 1 and time 2
$S1$, $S2$ = Storage at time 1 and time 2

The time interval for the routing analysis must be consistent with the time interval used in developing the inflow hydrograph. The time interval used for a 24-hour storm is 10 minutes.

The variables can be rearranged to obtain the following equation:

$$I1 + I2 + 2S1 - O1 = O2 + 2S2 \tag{5-13}$$

If the time interval is in minutes, the unit of storage (S) is now cubic feet per minute (cf/min), which can be converted to cfs by multiplying by 1 min/60 sec.

The terms on the left-hand side of the equation are known from the inflow hydrograph and from the storage and outflow values of the previous time step. The unknowns O and S can be solved interactively from the given stage-storage and stage-discharge curves. The best way to route a hydrograph through a stormwater facility is to use a computer program. Many hydrologic analysis software programs include features that simplify the hydrograph routing process.

Example

An infiltration trench is proposed to treat the 6-month, 24-hour design storm for a proposed development site. The following conditions apply:

- Design infiltration rate = 1.5 inches/hour for sandy loam soil (extends for at least 5 feet below land surface)
- Depth to water table estimated to be 75 feet
- Depth to impermeable soil layers 10 feet
- Trial geometry = 30 feet long x 3 feet wide x 2 feet deep
- Trench to be filled with rocks, such that void ratio = 0.4

Stage-Discharge

h (ft)	i (ft/ft)	Q (cfs)	S (cf)
0.0	1.0	5.6	0
0.5	1.05	5.9	18
1.0	1.1	6.2	36
1.5	1.15	6.5	54
2.0	1.2	6.8	72

ft = feet
cfs = cubic feet per second
cf = cubic feet

To confirm that the facility will drain within 24 hours, the designer would then need to conduct level pool routing.

Construction Criteria

- Initial basin excavation should be conducted to within 1 foot of the final elevation of the basin floor. Excavate infiltration trenches and basins to final grade only after all disturbed areas in the upgradient project drainage area have been permanently stabilized. The final phase of excavation should remove all accumulation of silt in the infiltration facility before putting it in service. After construction is completed, prevent sediment from entering the infiltration facility by first conveying the runoff water through an appropriate pretreatment system such as a presettling basin, wet pond, or sand filter.
- Infiltration facilities should generally not be used as temporary sediment traps during construction. If an infiltration facility is to be used as a sediment trap, it must not be excavated to final grade until after the upgradient drainage area has been stabilized. Any accumulation of silt in the basin must be removed before putting it in service.
- For traffic control, relatively light-tracked equipment is recommended to avoid compaction of the basin floor. The use of draglines and trackhoes should be considered for constructing infiltration basins. The infiltration area should be flagged or marked to keep heavy equipment away.

Maintenance Criteria

Provision should be made for regular and perpetual maintenance of the infiltration basin/trench, including replacement and/or reconstruction of the soil or other filter media that are relied upon for treatment purposes. Maintenance should be conducted when water remains in the basin or trench for more than 24 hours after the end of a rainfall event, or when overflows occur more frequently than planned. For example, off-line infiltration facilities should not have any overflows. Infiltration facilities designed to completely infiltrate all flows to meet flow control standards should not overflow. An operation and maintenance plan, approved by the local jurisdiction, should ensure that the desired infiltration rate is maintained.

Adequate access for operation and maintenance must be included in the design of infiltration basins and trenches. Removal of accumulated debris/sediment in the basin/trench should be conducted every 6 months or as needed to prevent clogging, or when water remains in the pond for greater than 24 hours after the end of a rainfall event.

Verification of Performance

During the first 1 to 2 years of operation, verification testing (specified in SSC 9) is strongly recommended, along with a maintenance program that results in achieving expected performance levels. Operating and maintaining groundwater monitoring wells is also strongly encouraged.

5-4.2. Design of Compost-Amended Vegetated Filter Strips

This section provides guidance on the hydrologic analysis and soil specifications for compost amended vegetated filter strips (CAVFS).

Determining Infiltration Rates for Soil Amendment BMPs

It is necessary to establish the long-term infiltration rate of an amended soil when it is used as a BMP design component to achieve treatment or flow control requirements. The assumed design infiltration rate should be the lower of the estimated long-term rate of the engineered soil mix or the initial (short-term or measured) infiltration rate of the underlying soil profile. The underlying native soil can be tested using either the detailed approach in Section 5-3.3 or the simplified approach in the SMMWW (Ecology 2005).

The following guidance provides recommended test methods for engineered soil mixes when they are used as part of a stormwater management BMP application. Figure 5-3 illustrates the overall process.

Compost-Amended Engineered Soil Mix

Depending on the size of contributing area, use one of these two recommended test protocols:

Test 1

If the contributing area has less than 5,000 square feet of pollution-generating impervious surface, and less than 10,000 square feet of impervious surface, and less than ¾ acres of lawn and landscape:

- Use ASTM D 2434 Standard Test Method for Permeability of Granular Soils (Constant Head) with a compaction rate of 80 percent using ASTM D1557 Test Method for Laboratory Compaction Characteristics of Soil Using Modified Effort.
- Use 2 as the infiltration reduction factor.

Test 2

If the contributing area is equal to or exceeds any of the following limitations: 5,000 square feet of pollution-generating impervious surface, 10,000 square feet of impervious surface, or ¾ acres of lawn and landscape:

- Use ASTM D 2434 Standard Test Method for Permeability of Granular Soils (Constant Head) with a compaction rate of 80 percent using ASTM D 1557 Test Method for Laboratory Compaction Characteristics of Soil Using Modified Effort.
- Use 4 as the infiltration reduction factor.

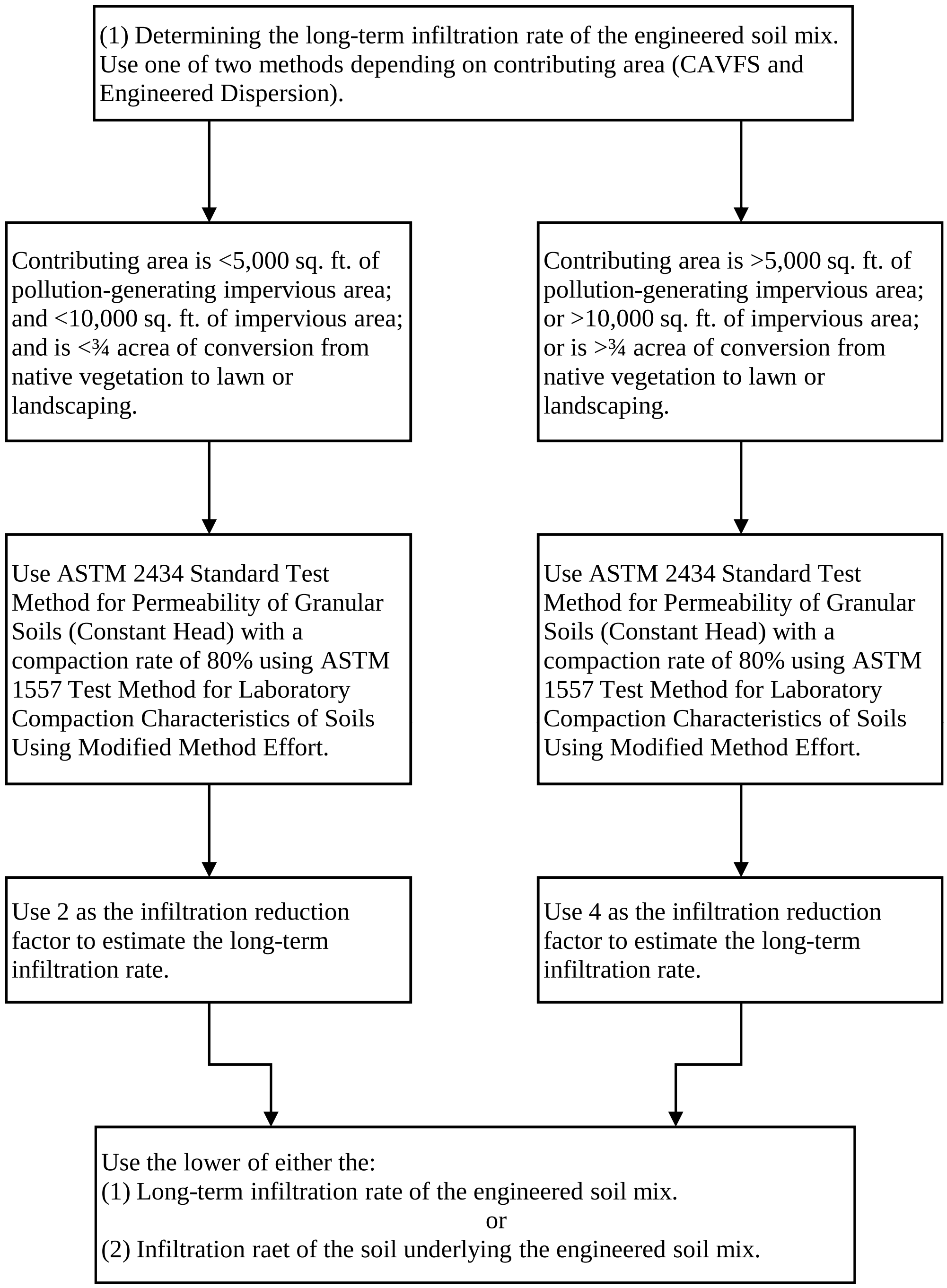

Figure 5-3. Determining the infiltration rate of soil amendments.

- Use the long-term infiltration rate of the engineered soil mix as the assumed infiltration rate of the overlying soil mix if it is higher than the underlying native soil. If the underlying native soil is lower than the engineered soil mix, use either the native soil infiltration rate or a varied infiltration rate that includes both the engineered soil mix infiltration rate and the native soil infiltration according to Step 6 of the detailed approach (Section 5-3.3).

Soil Specification

Proper soil specification, preparation, and installation are the most critical factors for CAVFS BMP performance. Soil specifications can vary according to the design objectives and the in situ soil. For additional information on soil specifications, see Section 5-4.3.2 in the HRM.

Design Procedure for Compost-Amended Vegetated Filter Strips (CAVFS) for Western Washington

This section provides hydrologic modeling guidance for CAVFS, when proposed for flow control in addition to water quality treatment.

CAVFS is most readily modeled in MGSFlood, which has a CAVFS link type (the assumptions and modeling procedures are described below).

The design for CAVFS is an iterative process in MGSFlood to adequately address the infiltrative capacity of both the compost amended layer and the underlying soils to achieve the 91 percent volume treatment criteria.

Flow through CAVFS is simulated using Darcy's Equation (as shown in Figure 5-4), where K_c is the saturated hydraulic conductivity. Note that the width dimension corresponds to the CAVFS width along the slope. Infiltration is accounted for using a constant infiltration rate into the underlying soils. During large storms, the voids in the CAVFS may become full (the CAVFS is saturated) in which case runoff is simulated as overflow down the surface of the CAVFS. The runoff volume filtered by the CAVFS, the volume infiltrated, and the volume flowing over the CAVFS surface are listed in the model output report.

Precipitation and evapotranspiration may (optionally) be applied to the CAVFS. If precipitation and evapotranspiration are applied in the CAVFS link, do not include the area of the CAVFS in the Subbasin Area input.

1. Follow Steps 1 through 11 in the Detailed Approach for Determining Infiltration Rates for the Underlying Soils of a CAVFS (see Section 5-3.3).

2. Follow Section 5-4.2 for CAVFS hydraulic conductivity.

Note: The methods described in Section 5-4.2 provide an infiltration rate. Assuming a hydraulic gradient of one, the infiltration rate is the same as the hydraulic conductivity.

3. Modeling steps for CAVFS.

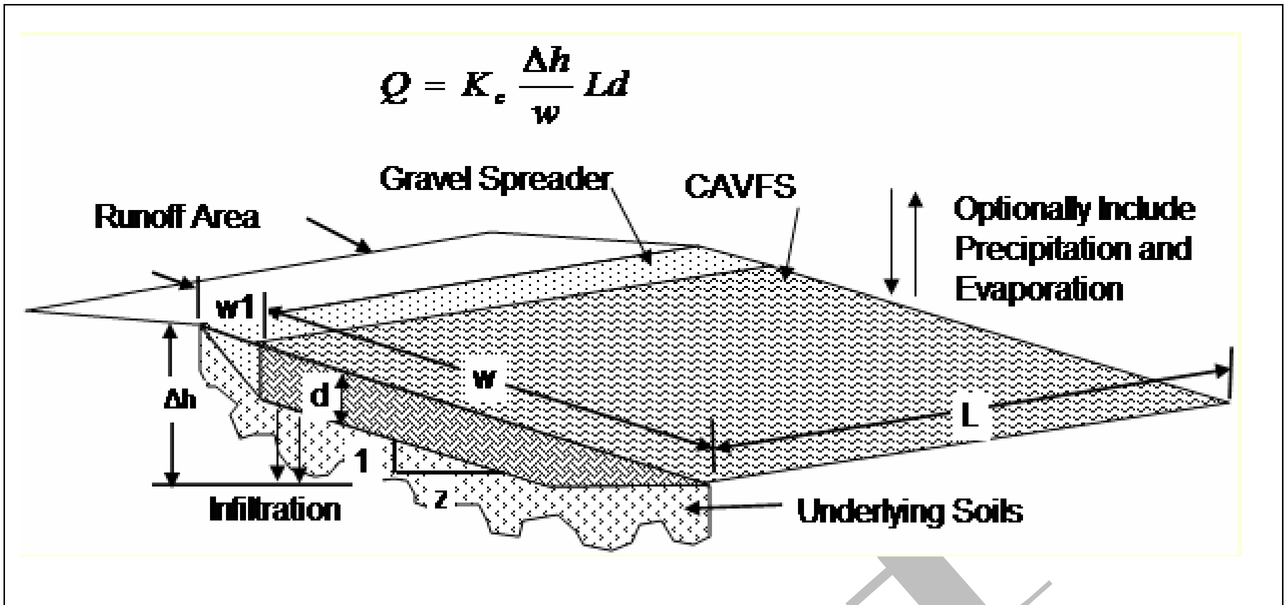

Figure 5-4. Flow-through CAVFS as simulated using Darcy's Equation.

Using MGSFlood, the dimensions of the CAVFS will be set as follows under the Network Tab:

- Select the Link type: CAVFS.
- ***CAVFS Depth d(ft):*** This is a constant depth of 1 foot for all CAVFS designs.
- ***CAVFS Porosity (% by Volume):*** The default value is 20 percent **but must be verified or reestablished by or a licensed geotechnical engineer for the particular site and particular installation.**
- ***CAVFS Hydraulic Conductivity (ft/day):*** The default value is 2 ft/day **and must be verified or reestablished by a licensed geotechnical engineer for the particular site and particular installation.**
- ***CAVFS Length (ft):*** The length parallel to the pavement.
- ***CAVFS Width (ft):*** The width perpendicular to the pavement. This is usually the parameter being solved for.
- ***Underlying Soil Infiltration Rate:*** Refer to Step 1.
- ***CAVFS Slope Z:*** The horizontal slope of the embankment—it cannot be steeper than 4:1.
- ***Gravel Spreader Width (ft):*** The width perpendicular to the pavement.
- ***Gravel Porosity (% by Volume):*** Typical value for gravel porosity is 30.

- ***Gravel Hydraulic Conductivity (ft/day):*** The default value is 4 ft/day **and must be verified or reestablished by a licensed geotechnical engineer for the particular site and particular installation.**

4. Determine that the volume of runoff infiltrated and filtered is 91 percent or greater than the total runoff volume.

MGSFlood will output Postdeveloped CAVFS Treatment Statistics in the MGSFlood Project Report file. The report file will give the percent treated for the structure defined in Step 3. The designer should verify that this number is equal to or greater than 91 percent.

5. Flow Control Compliance.

   After a successful runoff treatment design (Steps 1–4 above), the designer may be able to widen the CAVFS to try to meet the flow duration standard if flow control is required. Otherwise, a flow control structure should be linked downstream of the CAVFS to attenuate the resultant runoff and meet the flow duration standard. For an example problem, refer to Appendix 4A of the HRM.

5-4.3. Design Procedures for Volume-Based Runoff Treatment BMPs

For the purpose of designing runoff treatment BMPs based on volume (wet pool, vaults, tanks, and infiltration treatment facilities), in accordance with Minimum Requirement 6 (see Section 1-3.4), the following two methods can be used to derive the storage volume:

- **Wet Pool and Infiltration**: An approved continuous simulation hydrologic model based on the U.S. EPA's HSPF can be used (WWHM or MGSFlood, for example). The required storage volume is the 91st percentile, 24-hour runoff volume based on the long-term runoff record as predicted based on a 1-hour time step.
- **Wet Pool**: The SBUH method, which is based on NRCS curve number equations, can be used to determine the runoff treatment design storm runoff volume. This is the volume of runoff predicted from the 6-month, 24-hour recurrence interval storm. This design storm is approximated as 72 percent of the 2-year, 24-hour design storm. The size of the wet pool storage volume is the same whether located upstream or downstream of a flow control facility, or whether it is coupled with the flow control facility (e.g., a combination wet/detention facility).

If runoff from the new impervious surfaces and converted pervious surfaces is not separated from runoff from other surfaces on the project site, and/or is combined with run-on from areas outside the right-of-way, volume-based runoff treatment facilities must be sized based on runoff from the entire drainage area. This is because runoff treatment effectiveness can be greatly reduced if inflows to the facility are greater than the flows that the facility was designed to handle. A high-flow bypass (flow splitter) is used to route the incremental flow in excess of the treatment design runoff volume around the treatment facility. Facilities must infiltrate 91 percent of the total runoff volume from the infiltration basin within 36 hours. Under this premise, the storm/runoff ends 12 hours after the runoff period midpoint and combines with the 24-hour drain criteria. Therefore, the actual drawdown time is 36 hours.