APPENDIX F

Hydrologic Analysis and Design

Note:

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SDCI	**Director's Rule 10-2021**
SPU	**Director's Rule DWW-200**

Applicant: Department of Construction & Inspections Seattle Public Utilities	**Page:** 3 of 116	**Supersedes:** **SDCI:** 17-2017 **SPU:** DWW-200 (2017)
	Publication: 05/07/2021	**Effective:** 07/01/2021
Subject: Stormwater Manual Appendix F of I	**Code and Section Reference:** SMC 22.800-22.808	
	Type of Rule: Code interpretation	
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Index: Title 22.800 Stormwater Code	**Approved:** **Date:** ____________________ Nathan Torgelson, Director, SDCI	
	Approved: **Date:** ____________________ Mami Hara, General Manager/CEO, SPU	

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Tables

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F-1. Introduction

This appendix presents hydrologic modeling concepts to support the design of stormwater best management practices (BMPs) that meet minimum requirements in the Stormwater Code and in *Volume 1 — Project Minimum Requirements*. This appendix includes descriptions of acceptable methods for estimating the quantity and hydrologic characteristics of stormwater runoff, and the assumptions and data requirements of these methods. Specifically, hydrologic tools and methods are presented for the following tasks:

- Calculating runoff hydrographs and time series using single-event and continuous rainfall runoff models.
- Calculating peak flows for conveyance, peak flow detention and retention, and water quality rate treatment BMPs.
- Calculating volumes for detention and retention and water quality volume treatment BMPs.
- Calculating flow durations for flow duration detention and retention based requirements.

Flow control and water quality performance standards are presented in *Volume 1*. BMP design requirements and specific modeling methods are provided in *Volume 3, Chapters 4 and 5*. Any request for alternative calculation methods shall follow the principles laid out in this appendix and be approved by the Director.

F-2. Applicability of Hydrologic Analysis Methods

The choice of a hydrologic analysis method depends on the type of facility being designed (conveyance, detention, or water quality) and the required performance standard. The size of the tributary area and watershed characteristics, including backwater effects, should also be considered.

Hydrologic analysis methods may be grouped into three categories:

- **Continuous rainfall-runoff models** use multi-decade precipitation and evaporation time series as input to produce a corresponding multi-decade time series of runoff. Continuous models are used to size stormwater management facilities to meet peak or flow duration performance standards and water quality treatment requirements. Discharge rates computed with continuous models may also be used to size conveyance facilities.
- **Single-event rainfall-runoff models** simulate rainfall-runoff for a single storm, typically 2 hours to 72 hours in length, and usually of a specified exceedance probability (recurrence interval). Single-event methods are applicable for sizing conveyance facilities.
- The **rational method** is appropriate for designing conveyance systems that receive runoff from small, quickly responding areas (less than 10 acres) where short, intense storms generate the highest peak flow. This method only produces a flow peak discharge rate, and routing effects are not included. Advantages of this method are that it is easy to apply and generally produces conservative results. For larger, more complex basins, routing and timing of the flood peaks becomes more important and single-event or continuous rainfall-runoff modeling is required.

The applicability of each method is summarized in Table F.1.

Table F.1. Hydrologic Analysis Method Applicability.

Method	Applicable Models	Constraints	On-site BMP Sizing	FC BMP Sizing	WQ BMP Sizing	Conveyance Sizing	TESC Design Flow Sizing
Continuous Rainfall-runoff Modeling[a]	• HSPF • MGSFlood • WWHM • Other[b]	Refer to Table F.12 for time step requirements	✓	✓	✓	✓	✓
Single-event Rainfall-runoff Modeling	• NRCS TR-55 • SBUH • StormShed • Corps of Engineers HMS and HEC-1 • EPA SWMM, PCSWMM, and XP-SWMM • Other models approved by the Director	Refer to Table F.14	NA	NA	NA	✓	✓
Rational Method	NA	<10 acres (measured to individual conveyance elements) Upstream of storage routing and backwater effects	NA	NA	NA	✓	✓

[a] Refer to the *Approval Status of Continuous Simulation Models* section of the SWMMWW for a list of currently approved models.

[b] The following continuous hydrologic models may also be used for project-specific situations: EPA SWMM5, ModFlow, HMS, PCSWMM, and other models approved by the Director.

BMP – Best Management Practice
FC – Flow Control
HSPF – Hydrologic Simulation Program Fortran (US EPA)
NA – Not Applicable
NRCS – Natural Resources Conservation Service
On-site – On-site Stormwater Management
SBUH – Santa Barbara Urban Hydrograph
SWMM – Storm Water Management Model
TESC – Temporary Erosion and Sediment Control
WQ – Water Quality
WWHM – Western Washington Hydrology Model
✓ = acceptable

F-3. General Modeling Guidance

This section includes general modeling guidance that may apply to all hydrologic analysis methods, including both continuous modeling and single-event modeling using historical precipitation data, watershed characterization, hydrologic soil groups, infiltration equations, and outfalls.

Historical Precipitation Data

Data collected from the Seattle Public Utilities (SPU) rain gauge network may be used in rainfall runoff models to aid in the design process by replicating past floods, to investigate anecdotal flood information, or for use in model calibration. Use of the historical time series is recommended, but is not required for the design of stormwater BMPs.

Continuous historical precipitation data are available from 17 active and 2 closed rain gauges from January 1978 through the present at a time step of 1 minute. Active and closed gauge names and locations are summarized in Table F.2 and active locations are summarized on Figure F.1. Continuous Rainfall-Runoff Methods (*Section F-4*) and Single-event Rainfall-runoff Methods (*Section F-5*) provide additional detail regarding selection of precipitation data.

Table F.2. City of Seattle Rain Gauge Stations.

Station ID	Station Name	Period of Record	Status
45-S001	Haller Lake Shop	1978 – current	Active
45-S002	Magnusson Park	1978 – current	Active
45-S003	UW Hydraulics Lab	1978 – current	Active
45-S004	Maple Leaf Reservoir	1978 – current	Active
45-S005	Fauntleroy Ferry Dock	1978 – current	Active
45-S007	Whitman Middle School	1978 – current	Active
45-S008	Ballard Locks	1978 – current	Active
45-S009	Woodland Park Zoo	1978 – current	Active
45-S010	Rainier View Elementary	1978 – 2008	Closed
45-S011	Metro-KC Denny Regulating	1978 – current	Active
45-S012	Catherine Blaine Elementary School	1978 – current	Active
45-S014	Lafayette Elementary School	1978 – current	Active
45-S015	Puget Sound Clean Air Monitoring Station	1978 – current	Active
45-S016	Metro-KC E Marginal Way	1978 – current	Active
45-S017	West Seattle Reservoir Treatment Shop	1978 – current	Active
45-S018	Aki Kurose Middle School	1978 – current	Active
45-S020	TT Minor Elementary	1978 – 2010	Closed
RG25	Garfield Community Center	2010 – current	Active
RG30	SPL Rainier Beach Branch	2009 – current	Active

Watershed Characterization

Prior to conducting any detailed stormwater runoff calculations, the overall relationship between the proposed project site and upstream and downstream off-site areas must be considered. The general hydrologic characteristics of the project site dictate the amount of runoff that will occur and where stormwater facilities can be placed. It is important to identify the stormwater destination point, including potential backwater effects. Drainage patterns and contributing areas can be determined from preliminary surveys of the area, available topographic contour maps, and SPU drainage system maps. Note that the drainage systems often cross topographic divides within the City of Seattle. Maps can be obtained from the City's GIS web page (www.seattle.gov/utilities/services/gis).

Calculation of Total Impervious Area

Impervious coverage for proposed development must be estimated. Impervious coverage of streets, sidewalks, hard surface trails, etc., shall be taken from plans of the site. Refer to *Volume 1*, *Appendix A*, and the Stormwater Code for definitions and descriptions of all surfaces that must be considered. Impervious coverage for off-site areas contributing flow to the site can be estimated from orthophotos available through GIS.

Calculation of Effective Impervious Area

Effective impervious surface is the fraction of impervious surface connected to a drainage system and is used in hydrologic simulations to estimate runoff. The effective impervious area is the total impervious area multiplied by the effective impervious fraction. Non-effective impervious surface is assumed to have the same hydrologic response as the immediately surrounding pervious area. For the existing condition modeling, areas with unconnected rooftops may be estimated from visual survey as approved by the Director.

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Figure F.1. Active City Rain Gauge Network Stations.

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Soil and Infiltration Parameters

Hydrologic Soil Groups

Hydrologic soil groups for common soil types in the Seattle area are listed in Table F.3.

Infiltration Equations

When computing runoff in models other than those based on HSPF, an infiltration soil loss method should be used. Examples of infiltration methods include the Green-Ampt (Rawls et al. 1993), Philip (Rawls et al. 1993), and Holtan (Holtan 1961) methods. These methods are incorporated into several commonly available computer programs including StormShed, PCSWMM, HEC HMS, and HEC-1. The City recommends the use of Green-Ampt method; however, the other methods listed above can also be used based on project-specific situations.

Table F.3. Hydrologic Soil Group Definition for Common Soils in King County.

Soil Group	Hydrologic Group	Soil Group	Hydrologic Group
Alderwood	C	Orcas Peat	D
Arents, Alderwood Material	C	Oridia	D
Arents, Everett Material	B	Ovalt	C
Beausite	C	Pilchuck	C
Bellingham	D	Puget	D
Briscot	D	Puyallup	B
Buckley	D	Ragnar	B
Coastal Beaches	Variable	Renton	D
Earlmont Silt Loam	D	Riverwash	Variable
Edgewick	C	Salal	C
Everett	A	Sammamish	D
Indianola	A	Seattle	D
Kitsap	C	Shacar	D
Klaus	C	Si Silt	C
Mixed Alluvial Lan	Variable	Snohomish	D
Nellton	A	Sultan	C
Newberg	B	Tukwila	D
Nooksack	C	Urban	Variable
Normal Sandy Loam	D	Woodinville	D

Table F.3 (continued). Hydrologic Soil Group Definition for Common Soils in King County.

HYDROLOGIC SOIL GROUP CLASSIFICATIONS
A. Low runoff potential: Soils having high infiltration rates, even when thoroughly wetted, and consisting chiefly of deep, well-to-excessively drained sands or gravels. These soils have a high rate of water transmission
B. Moderately low runoff potential: Soils having moderate infiltration rates when thoroughly wetted, and consisting chiefly of moderately fine to moderately coarse textures. These soils have a moderate rate of water transmission.
C. Moderately high runoff potential: Soils having slow infiltration rates when thoroughly wetted, and consisting chiefly of soils with a layer that impedes downward movement of water, or soils with moderately fine to fine textures. These soils have a slow rate of water transmission.
D. High runoff potential: Soils having very slow infiltration rates when thoroughly wetted and consisting chiefly of clay soils with a high swelling potential, soils with a permanent high water table, soils with a hardpan or clay later at or near the surface, and shallow soils over nearly impervious material. These soils have a very slow rate of water transmission.

Source: TR-55 (NRCS 1986), Exhibit A-1. Revisions made from Soil Conservation Service (SCS), Soil Interpretation Record, Form #5, September 1988.

Green-Ampt Equation

The Green-Ampt model calculates cumulative infiltration by assuming water flow into a vertical soil profile like a piston flow.

$$f_t = K\left(\frac{\psi\Delta\theta}{F_t} + 1\right) \tag{1}$$

$$F_{t+\Delta t} = F_t + K\Delta t + \psi\Delta\theta \ln\left[\frac{F_{t+\Delta t} + \psi\Delta\theta}{F_t + \psi\Delta\theta}\right] \tag{2}$$

Where:

f_t = infiltration rate (mm/hr or in/hr)

ψ = initial matric potential of the soil (mm or inches)

$\Delta\theta$ = difference of soil water content after infiltration with initial water content

K = hydraulic conductivity (mm/hr or in/hr)

F_t = cumulative infiltration at time t (mm or inches)

$F_{t+\Delta t}$ = cumulative infiltration at time t+ Δt (mm or inches)

Δt = time incremental (hours)

Equation (1) is used for determining ponding situation and (2) is used for calculating the cumulative infiltration after ponding. Trial and error method is the most popular method to solve equation (2) (Chow et al. 1988). Parameters ψ , $\Delta\theta$, and K were tabulated by Chow et al. (1988) for all soil classes. Chow et al. (1988) developed a procedure to solve infiltration with changing rainfall intensity by Green-Ampt method in a table. However, since it simplifies the water movement as a piston flow, the wetting front is distorted.

Typical values suggested by Rawls, Brakensiek, and Miller (as reflected in Chow et al. 1988) are shown in Table F.4 below.

Table F.4. Green-Ampt Infiltration Parameters.

USDA Soil Classification	Suction Head ψ (mm)	Suction Head ψ (in/hr)	Hydraulic Conductivity K (mm/hr)	Hydraulic Conductivity K (in/hr)	Porosity η	Effective Porosity θ_e
Sand	49.5	1.95	117.8	4.64	0.437	0.417
Loamy Sand	61.3	2.42	29.9	1.18	0.437	0.401
Sandy Loam	110.1	4.34	10.9	0.43	0.453	0.412
Loam	88.9	3.50	3.4	0.13	0.463	0.434
Silt Loam	166.8	6.57	6.5	0.26	0.501	0.486
Sandy Clay Loam	218.5	8.61	1.5	0.06	0.398	0.330
Clay Loam	208.8	8.23	1.0	0.04	0.464	0.309
Silty Clay Loam	273.0	10.76	1.0	0.04	0.471	0.432
Sandy Clay	239.0	9.42	0.6	0.02	0.430	0.321
Silty Clay	292.2	11.51	0.5	0.02	0.479	0.423
Clay	316.3	12.46	0.3	0.01	0.475	0.385

in/hr – inches per hour
mm – millimeters
mm/hr – millimeters per hour
USDA – United States Department of Agriculture

Holtan's Equation

The empirical infiltration equation devised by Holtan (1961) is explicitly dependent on soil water conditions in the form of available pore space for moisture storage:

$$F = (GI)(AH)\ SMD^{IEXP} + FC \quad (3)$$

Where:

- F = surface infiltration rate at a given time (in/hr)
- GI = Growth Index representing the relative maturity of the ground cover (0 for newly planted, 1 for mature cover)
- AH = constant as specified below
- SMD = soil moisture deficit at a given time (inches)
- IEXP = infiltration exponent (default value is 1.4)
- FC = minimum surface infiltration rate (in/hr) and occurs when SMD equals zero

Parameters GI, AH, FC, and the initial soil moisture deficit (SMD0) are the principal input parameters and can be determined as follows:

- GI is typically set to 1.0 to represent mature ground cover.
- AH can be determined from Table F.5.

- FC can be approximated from Table F.6 or by using the saturated hydraulic conductivity, which is available from soil survey reports.

Table F.5. Estimates of Holtan AH.

Land Use or Cover	Base Area Rating[a]	
	Poor Condition	Good Condition
Fallow[b]	0.10	0.30
Row crops	0.10	0.20
Small grains	0.20	0.30
Hay (legumes)	0.20	0.40
Hay (sod)	0.40	0.60
Pasture (bunchgrass)	0.20	0.40
Temporary pasture (sod)	0.40	0.60
Permanent pasture (sod)	0.80	1.00
Woods and forests	0.80	1.00

[a] Adjustments needed for “weeds” and “grazing.”

[b] For fallow land only, “poor condition” means “after row crop,” and “good condition” means “after sod.”

Source: Holtan et al. (1975)

Table F.6. Estimates of Holtan FC Values.

NRCS Hydrologic Soil Group	Minimum Infiltration Rates FC (inches/hour)
A	0.30–0.45
B	0.15–0.30
C	0.05–0.15
D	< 0.05

Source: Musgrave (1955)

This equation has been found to be suitable for inclusion in catchment models because of soil water dependence, and satisfactory progress has been reported for runoff predictions (Dunin 1976).

Kostiakov's Equation

Kostiakov (1932) proposed the following equation for estimating infiltration:

$$i(t) = \alpha t^{-\beta} \tag{4}$$

Where:
- t = time
- i = infiltration rate
- α= empirical constant ($\alpha > 0$)
- β= empirical constant ($0 < \beta < 1$)

Upon integration from 0 to t, equation (4) yields equation (5), which is the expression for cumulative infiltration, I(t):

$$I(t) = \frac{\alpha}{1-\beta} t^{(1-\beta)} \tag{5}$$

Where: I(t) = cumulative infiltration

The constants α and β can be determined by curve-fitting equation (5) to experimental data for cumulative infiltration, I(t). Since infiltration rate (i) becomes zero as $t \to \infty$, rather than approach a constant non-zero value, Kostiakov proposed that equations (4) and (5) be used only for $t < t_{\max}$ where $t_{\max}$ is equal to $(\alpha / K_s)^{(1/\beta)}$, and K_s is the saturated hydraulic conductivity of the soil. Kostiakov's equation describes the infiltration quite well at smaller times, but becomes less accurate at larger times (Philip 1957a and 1957b; Parlange and Haverkamp 1989).

Horton's Equation

Horton (1940) proposed to estimate infiltration in the following manner,

$$i(t) = i_f + (i_0 - i_f)e^{-\gamma t} \tag{6}$$

and

$$I(t) = i_f t + \frac{1}{\gamma}(i_0 - i_f)(1 - e^{-\gamma t}) \tag{7}$$

Where: i_0 = measured infiltration rate
i_f = final infiltration rate
γ = empirical constant

It is readily seen that i(t) is non-zero as t approaches infinity, unlike Kostiakov's equation. It does not, however, adequately represent the rapid decrease of i from very high values at small t (Philip 1957a and 1957b). It also requires an additional parameter over the Kostiakov equation. Parlange and Haverkamp (1989), in their comparison study of various empirical infiltration equations, found the performance of Horton's equation to be inferior to that of Kostiakov's equation.

Mezencev's Equation

In order to overcome the limitations of Kostiakov's equation for large times, Mezencev (Philip 1957a and 1957b) proposed the following as modifications to equations (4) and (5). Mezencev proposed infiltration estimated by:

$$i(t) = i_f + \alpha t^{-\beta} \tag{8}$$

and

$$I(t) = i_f t + \frac{\alpha}{1-\beta} t^{(1-\beta)} \tag{9}$$

Where: i_f = final infiltration rate at steady state

Outfalls

Outfalls to Lakes and the Ship Canal

Single-event hydraulic analysis of outfalls that discharge to lakes and the Ship Canal should be performed using high water surface elevation from the observed record. This assumption may lead to conservative results and it is recommended that the designer consider using continuous simulation with a varying receiving water level. Table F.7 shows the maximum observed water levels in Seattle lakes. Water levels may vary from year to year due to sedimentation and season.

For continuous simulations, the designer may choose to use the historical record or the highest observed elevations. Lake Washington and associated waters are controlled at the Hiram M. Chittenden Locks by the US Army Corps of Engineers (USACE). Refer to the USACE Reservoir Control Center website (www.nwd-wc.usace.army.mil/nws/hh/www/index.html) for Lake Washington Ship Canal data and note that elevations given are in USACE datum and should be converted to NAVD88 before use.

Table F.7. Physical Characteristics of Seattle Lakes.

	Bitter Lake	Haller Lake	Green Lake	Lake Union	Lake Washington
Water surface elevation (feet, NAVD88)[a]	434.4	376.9	164.3	18.6~~16.8~~	18.6
Maximum depth (feet)[b]	31.0	36.0	30.0	50.0	214
Mean depth (feet)[b]	16.0	16.0	13.0	34.0	108
Area (acres)[b]	19.0	15.0	259	580	21,500

[a] SPU Engineering Support Division – Survey Field Books, measurements were all converted to NAVD88 from the old City of Seattle Vertical Datum based on a conversion factor of 9.7 feet.

[b] Sources: King County (2015) and King County (2016).

Note: Water levels may vary from year to year by as much as 3 feet.

Tidal Influence/Sea Level Rise

When utilizing single-event hydraulic analysis of the drainage system or combined sewer system with outfalls that discharge to the tidally influenced Duwamish River or Puget Sound, the highest observed tide from the observed record shall be used. Match the peak rainfall intensity to a tide cycle simulation with a peak of 12.14 feet (NAVD88). This assumption may lead to conservative results and it is recommended that the designer consider using continuous simulation with a varying receiving water level.

For continuous simulations, the designer should match, by time, the historical tidal record to the historical rainfall record. For rainfall simulations where there is no observed tidal elevation, use of a tide predictor is recommended. Tidal information is available from National Oceanic and Atmospheric Administration (NOAA) (http://tidesandcurrents.noaa.gov) and from the US Army Corps of Engineer's (www.nws.usace.army.mil/About/Offices/Engineering/HydraulicsandHydrology/HistoricalDatumRegions.aspx). The tidal boundary is simulated as a water surface elevation time series computed using astronomical tide theory (NOAA 1995).

Sea level is rising, and for both continuous and single-event modeling, the designer should evaluate the risks depending on the project design life and objectives. Since 1899, the observed trend was a rise of 2.06 mm per year, which is equivalent to 8 inches in 100 years. The effect of climate change on predicted sea level rise is expected to greatly exceed that rate, but there is uncertainty regarding timing and severity. The Washington Costal Resilience Project (Miller et al. 2018) represents the best available science on sea level rise. The report provides local projections at various likelihoods and time frames (see Figure F.2). For Seattle, the central estimate (i.e., 50 percent probability) is 1.9 to 2.3 feet of rise by 2100, and 3.0 to 3.9 feet by 2150. Upper-end estimates (1 percent probability) project 5.1 feet of rise by 2100, and 10.4 feet of rise by 2150.

For design of tidally impacted public drainage system and public combined sewer system, hydraulic analysis of sea level rise is required. For other projects, it is recommended that designers analyze risk by adjusting the tidal record upwards by 1 to 4 feet, depending on the design life and risk tolerance of the project. Likewise, designers should look to further mitigate risk by considering current design adjustments or identifying possible future modifications. For design of facilities where water level elevation at the outfall is critical, the City recommends that the designer consider storm surge due to low atmospheric pressure and/or wind and wave action.

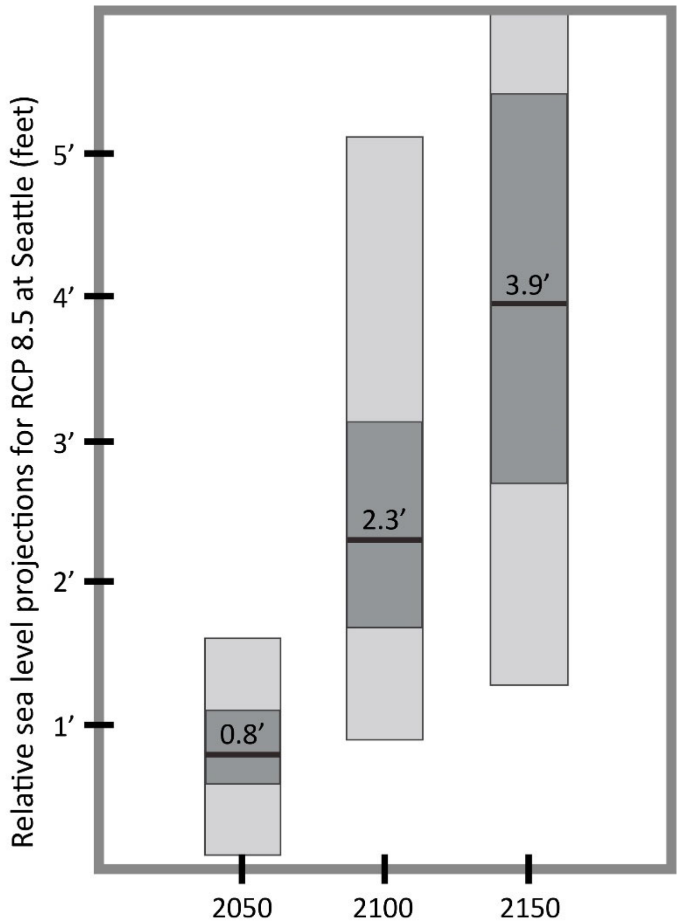


Figure F.2. Projected Sea Level Rise in Washington's Waters Relative to Year 2050.

F-4. Continuous Rainfall-Runoff Methods

This section includes specific modeling guidance that is applicable to continuous rainfall-runoff methods including precipitation input, land cover categorization, soil parameters, infiltration parameters, and modeling guidance.

Precipitation Input

Continuous rainfall-runoff models use multi-year inputs of precipitation and evaporation to compute a multi-year time series of runoff from the site. Using precipitation input that is representative of the site under consideration is critical for the accurate computation of runoff and the design of stormwater facilities.

Two types of precipitation and evaporation data are available for stormwater analysis. The first type is a design precipitation and evaporation time series. The design time series are appropriate for design and analysis of stormwater facilities and were developed by combining and scaling records from distant precipitation stations. The second type of time series is historical precipitation and evaporation time series (described in *Section F-3 — General*

Modeling Guidance). Because the record length of the historical precipitation and evaporation is relatively short, this data should be used for model calibration and not for design.

The City of Seattle Design Time Series consists of a precipitation and evaporation time series that are representative of the climatic conditions in the City of Seattle. The design precipitation time series was developed by combining and scaling precipitation records from widely separated stations to produce an “extended precipitation time series” with a 158-year record length (Schaefer and Barker 2002; Schaefer and Barker 2007). The precipitation scaling was performed such that the scaled precipitation record would possess the regional statistics at durations of 5 minutes, 10 minutes, 15 minutes, 20 minutes, 30 minutes, 45 minutes, 60 minutes, 2 hours, 6 hours, 24 hours, 3 days, 10 days, 30 days, 90 days, 6 months, and 12 months (Refer to www.seattle.gov/sdci/codes/codes-we-enforce-(a-z)/stormwater-code for modeling resources). The precipitation time series was developed at a 5-minute time step. For modeling of the combined sewer system, a shorter precipitation record length may be approved by the Director.

The evaporation time series was developed using a stochastic evaporation generating approach whereby daily evaporation was generated in a manner to preserve the daily and seasonal variability and accounting for differences observed on days with and without rainfall. The evaporation time series was developed from data collected at the Puyallup 2 West Experimental Station (station number 45-6803). Refer to www.seattle.gov/sdci/codes/codes-we-enforce-(a-z)/stormwater-code for modeling resources. The evaporation time series has a 1-hour time step.

Land Cover Categorization

Continuous hydrologic models based on HSPF (e.g., WWHM and MGSFlood) include five land cover types: forest, pasture, lawn (or grass), wetland, and impervious. These cover types shall be applied as specified in Table F.8.

Soil and Infiltration Parameters

Soil Mapping

Mapping of soil types by the Natural Resources Conservation Service (NRCS), or the Washington Department of Natural Resources Geologic Information Portal (www.dnr.wa.gov/geologyportal) may be used as a source of soil/geologic information for use in continuous hydrologic modeling. If using NRCS maps, each soil type defined by the NRCS has been classified into one of four hydrologic soil groups; A, B, C, and D. Table F.3 shows hydrologic soil groups for common soil types in King County. As is common practice in hydrologic modeling in western Washington, the soil groups used in the model generally correspond to the hydrologic soil groups as shown in Table F.9.

Table F.8. Continuous Model Land Cover and Areas of Application.

Continuous Model Land Cover	Application	
	Pre-Developed	**Post-Developed**
Forest	All forest/shrub cover, irrespective of age	All permanent (e.g., protected by covenant) onsite forest/shrub cover, irrespective of age planted at densities sufficient to ensure 80%± canopy cover within 5 years.
Pasture	All grassland, pasture land, lawns, and cultivated or cleared area except for lawns in redevelopment areas with pre-development densities greater than 4 DU/GA	• All areas that are amended using implementation options 2, 3, or 4 from *Volume 3, Section 5.1.5.2* may be modeled as pasture rather than lawn (WWHM) or grass (MGSFlood). • Unprotected forest in rural residential development shall be considered half pasture, half grass.
Lawn (or Grass)	Lawns in redevelopment areas with pre-development densities greater than 4 DU/GA	• All post-development grassland and landscaping that is not amended using implementation options 2, 3, or 4 from *Volume 3, Section 5.1.5.2.* • All onsite forested land not protected by covenant.
Saturated / Wetland	All delineated saturated or wetland areas	All delineated saturated or wetland areas
Impervious	• All impervious surfaces, including heavily compacted gravel and dirt roads, parking areas, etc. • Open receiving waters (ponds and lakes)	• All impervious surfaces (with and without underdrains), including heavily compacted gravel and dirt roads, parking areas, etc. • Pervious surfaces with underdrains • Open receiving waters (ponds, lakes, and onsite detention ponds, and wet ponds)

BMP – Best Management Practice

DU/GA – Dwelling Unit per Gross Acre

Table F.9. Relationship Between Hydrologic Soil Group and Continuous Model Soil Group.

Hydrologic Soil Group	Continuous Model Soil Group
A	Outwash
B	Till or Outwash
C	Till
D	Saturated / Wetland

Type B soils can be classified as either glacial till or outwash depending on the type of soil under consideration. Type B soils underlain by glacial till or bedrock, or with a seasonally high water table would be classified as till. Conversely, well-drained B type soils would be classified as outwash.

The NRCS maps may not be used for determining infiltration capacity or a design infiltration rate.

Infiltration Parameters

The following discussion on HSPF model parameters applies to the use of continuous modeling (e.g., WWHM, MGSFlood). Default model parameters that define interception, infiltration, and movement of moisture through the soil, are based on work by the United States Geological Survey (USGS) (Dinicola 1990, 2001) and King County (2009). Pervious areas have been grouped into three land cover categories (forest, pasture, and lawn) and three soil/geologic categories (till, outwash, and saturated/wetland soil) for a total of seven cover/soil type combinations as shown in Table F.10. The combinations of soil type and land cover are called pervious land segments or PERLNDs. Default runoff parameters for each PERLND are summarized in Table F.11. These parameter values are used automatically by WWHM and MGSFlood programs for each land use type. A complete description of the PERLND parameters can be found in the HSPF User Manual (US EPA 2001). For a general discussion of infiltration equations refer to *Section F-3 — General Modeling Guidance*.

Table F.10. Pervious Land Soil Type/Cover Combinations used with HSPF Model Parameters.

Pervious Land Soil Type/Cover Combinations
1. Till/Forest
2. Till/Pasture
3. Till/Lawn
4. Outwash/Forest
5. Outwash/Pasture
6. Outwash/Lawn
7. Saturated Soil/All Cover Groups

Modeling Guidance

Computational Time Step Selection

An appropriate computational time step for continuous hydrologic models depends on the type of facility under consideration and the characteristics of the tributary watershed. In general, the design of facilities dependent on peak discharge require a shorter time step than facilities dependent on runoff volume. A longer time step is generally desirable to reduce the overall simulation time provided that computational accuracy is not sacrificed. Table F.12 summarizes the allowable computational time steps for various hydrologic design applications.

HSPF Parameter Modification

In HSPF (and MGSFlood and WWHM) pervious land categories are represented by PERLNDs and impervious land categories are represented by IMPLNDs. The only PERLND and IMPLND parameter values that should be adjusted by the user are LSUR (length of surface overland flow plane in feet), SLSUR (slope of surface overland flow plane in feet/feet), and NSUR (roughness of surface overland flow plane). The default HSPF parameter values in MGSFlood and WWHM are appropriate for large sites that are not typical for City of Seattle projects. Users are required to change the values for LSUR, SLSUR, and NSUR per guidance in

Table F.11 or adjust values for LSUR, SLSUR, and NSUR based on site-specific observations. Any changes made to parameter values noted in in Table F.11 shall be recorded in the model output report and included with a project submittal.

Table F.11. Required Runoff Parameter Values for Each Pervious Land Segment (PERLND) and Impervious Land Segment (IMPLND).

Parameter	Pervious Land Segment (PERLND)							Impervious Land Segment (IMPLND)
	Till Soil			Outwash Soil			Saturated Soil	
	Forest	Pasture	Lawn	Forest	Pasture	Lawn	Forest, Pasture, or Lawn	
LZSN	4.5	4.5	4.5	5.0	5.0	5.0	4.0	NA
INFILT	0.08	0.06	0.03	2.0	1.6	0.8	2.0	NA
LSUR[a]	$2 * \sqrt{Contributing\ Area\ (square\ feet)}$							
SLSUR[a]	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05
KVARY	0.5	0.5	0.5	0.3	0.3	0.3	0.5	NA
AGWRC	0.996	0.996	0.996	0.996	0.996	0.996	0.996	NA
INFEXP	2.0	2.0	2.0	2.0	2.0	2.0	10.0	NA
INFILD	2.0	2.0	2.0	2.0	2.0	2.0	2.0	NA
BASETP	0.0	0.0	0.0	0.0	0.0	0.0	0.0	NA
AGWETP	0.0	0.0	0.0	0.0	0.0	0.0	0.7	NA
CEPSC	0.2	0.15	0.1	0.2	0.15	0.1	0.1	NA
UZSN	0.5	0.4	0.25	0.5	0.5	0.5	3.0	NA
NSUR[a]	0.35	0.3	0.25	0.35	0.3	0.25	0.5	0.02
INTFW	6.0	6.0	6.0	0.0	0.0	0.0	1.0	NA
IRC	0.5	0.5	0.5	0.7	0.7	0.7	0.7	NA
LZETP	0.7	0.4	0.25	0.7	0.4	0.25	0.8	NA
RETSC	NA	NA	NA	NA	NA	NA	NA	0.1

[a] LSUR, SLSUR, and NSUR parameter values shall be adjusted based on site-specific observations.

LZSN = lower zone storage nominal (inches)
INFILT = infiltration capacity (in/hr)
LSUR = length of surface overland flow plane (feet)
SLSUR = slope of surface overland flow plane (feet/feet)
KVARY = groundwater exponent variable (inch -1)
AGWRC = active groundwater recession constant (day -1)
INFEXP = infiltration exponent
INFILD = ratio of maximum to mean infiltration
BASETP = base flow evapotranspiration (fraction)
AGWETP = active groundwater evapotranspiration (fraction)
CEPSC = Interception storage (inches)
UZSN = upper zone storage nominal (inches)
NSUR = roughness of surface overland flow plane (Manning's n)
INTFW = interflow index
IRC = interflow recession constant (day $^{-1}$)
LZETP = lower zone evapotranspiration (fraction)
RETSC = retention storage capacity (in)
NA = not applicable

Table F.12. Required Continuous Simulation Model Computational Time Step for Various Stormwater Facilities.

Type of Analysis	Maximum Time Step
Conveyance Sizing (Off-site)	5 minutes[a]
Conveyance Sizing Upstream of Stormwater Detention Facility (Onsite), TESC Design Flows	5 minutes[a]
Conveyance Sizing Downstream of Stormwater Detention Facility (Onsite), TESC Design Flows	15 minutes
Downstream Analysis, Off-site	5 minutes[a]
Flow Control (Detention and/or Infiltration) Facility and On-site BMP Sizing	5 minutes[a]
Water Quality Design Flow Rate	15 minutes
Water Quality Design Flow Volumes/Pollutant Loading	1 hour

[a] A 15-minute time step may be used if the time of concentration computed is 30 minutes or more (refer to *Time of Concentration Estimation* in *Section F-5*).

Steps for Hydrologic Design Using Continuous Rainfall-Runoff Models

This section presents the general process involved in conducting hydrologic analyses using continuous models. The actual design process will vary considerably depending on the project scenario, the applicable requirements, the facility being designed, and the environmental conditions.

Step #	Procedure
C-1	Review all minimum requirements that apply to the proposed project (*Volume 1*)
C-2	Review applicable site assessment requirements (*Volume 1, Chapter 7*)
C-3	Identify and delineate the overall drainage basin for each discharge point from the development site under existing conditions: • Identify existing land use • Identify existing soil types using onsite evaluation, NRCS soil survey, or mapping performed by the University of Washington (http://geomapnw.ess.washington.edu) • Convert hydrologic soil types to HSPF soil classifications (till, outwash, or wetland) • Identify existing drainage features such as streams, conveyance systems, detention facilities, ponding areas, depressions, etc.
C-4	Select and delineate pertinent subbasins based on existing conditions: • Select homogeneous subbasin areas • Select separate subbasin areas for onsite and off-site drainage • Select separate subbasin areas for major drainage features
C-5	Determine hydrologic parameters for each subbasin under existing conditions, if required: • Determine appropriate rainfall time series. For most design applications, the City of Seattle Design Time Series will be required. • Categorize soil types and land cover • Determine total and effective impervious areas within each subbasin • Determine areas for each soil/cover type in each subbasin • Select the required computational time step according to Table F.12
C-6	Compute runoff for the pre-developed condition. The continuous hydrologic model will utilize the selected precipitation time series, compute runoff from each subbasin, and route the runoff through the defined network. Flood-frequency and flow duration statistics will subsequently be computed at points of interest in the study area by the model.

Step #	Procedure
C-7	Determine hydrologic parameters for each subbasin under developed conditions: • Utilize rainfall time series selected for existing conditions • Categorize soil types and land cover • Determine total and effective impervious areas within each subbasin • Determine areas for each soil/cover type in each subbasin • Utilize computational time step selected for existing conditions
C-8	Compute runoff for the developed condition. The continuous hydrologic model will utilize the selected precipitation time series, compute runoff from each subbasin, and route the runoff through the defined network. Flood-frequency and flow duration statistics will subsequently be computed at points of interest in the study area by the model.

Additional design steps specific to flow control and water quality treatment facility design are described below.

Flow Control Facility Design

Peak Standard

Peak flow control-based standards require that the stormwater facilities be designed such that the post-development runoff peak discharge rate is controlled to one or more discharge rates, usually at specified recurrence intervals. An example of this type of standard is the Peak Flow Control Standard.

Flood-frequency analysis seeks to determine the flood flow or water surface elevation with a probability (p) of being equaled or exceeded in any given year. Return period (T_r) or recurrence interval is often used in lieu of probability to describe the frequency of exceedance of a flood of a given magnitude. Return period and annual exceedance probability are reciprocals (equation 10). Flood-frequency analysis is most commonly conducted for flood peak discharge and peak water surface elevation but can also be computed for maximum or minimum values for various durations. Flood-frequency analysis as used here refers to analysis of flood peak discharge or peak water surface elevation.

$$\mathrm{Tr} = \frac{1}{\mathrm{p}} \tag{10}$$

Where: T_r = average recurrence interval in years

p = the annual exceedance probability

The annual exceedance probability of flow (or water surface elevation) may be estimated using the Gringorten (1963) plotting position formula (equation 11), which is a non-parametric approach.

$$Tr = \frac{N + 0.12}{i - 0.44} \tag{11}$$

Where: T_r = recurrence interval of the peak flow or peak elevation in years
i = rank of the annual maxima peak flow from highest to lowest
N = total number of years simulated

A probability distribution, such as the Generalized Extreme Value or Log-Pearson III (Interagency Advisory Committee on Water Data 1981), is not recommended for estimating the frequency characteristics.

Flood frequency analyses are used in continuous flow simulations to determine the effect of land use change and assess the effectiveness of flow control facilities. Flow control facilities are designed such that the post-developed peak discharge rate is at or below a target pre-developed peak discharge rate at one or more recurrence intervals. For example, Figure F.3 shows pre-developed and post-developed flood frequency curves for a stormwater pond designed to control peak discharges at the 2-year and 10-year recurrence intervals. Continuous simulation hydrologic models perform the frequency calculations and present the results in graphical and tabular form.

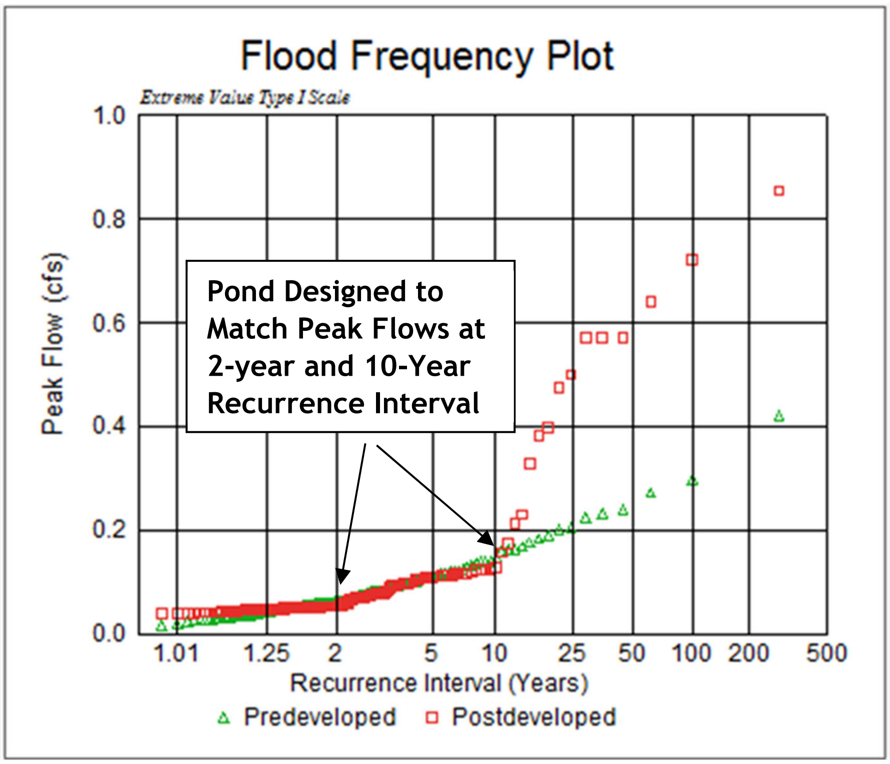


Figure F.3. Example Flood-Frequency Curves for a Stormwater Pond Designed to Control Post-Developed Peak Discharge Rates to Pre-Developed Levels at the 2-Year and 10-Year Recurrence Interval.

Flow Duration Standard

Flow duration statistics provide a convenient tool for characterizing stormwater runoff computed with a continuous hydrologic model. Examples of this type of standard are the Pre-developed Forest Standard and the Pre-developed Pasture Standard. Evaluation of a flow duration design standard requires continuous simulation to compute the pre-development and post-development runoff record. Duration statistics are computed by tracking the fraction of total simulation time that a specified flow rate is equaled or exceeded. Continuous rainfall-runoff models do this by dividing the range of flows simulated into discrete increments, and then tracking the fraction of time that each flow is equaled or exceeded. For example, Figure F.4 shows a 1-year flow time series computed at hourly time steps from a 10-acre forested site and Figure F.5 shows the flow duration curve computed from this time series.

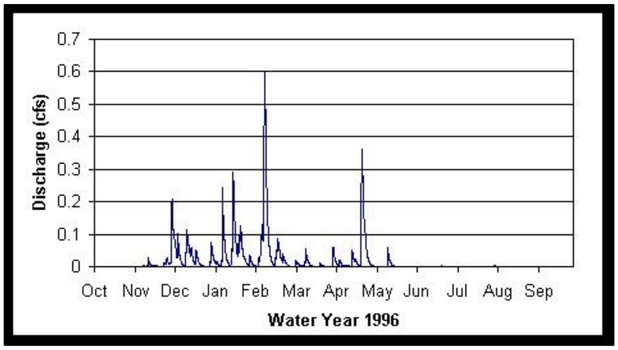


Figure F.4. Runoff from 10-Acre Forested Site.

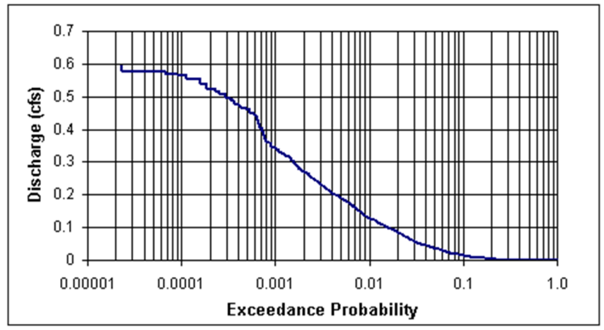


Figure F.5. Flow Duration Curve Computed Using Time Series in Figure F.4.

The fraction of time is termed "exceedance probability" because it represents the probability that a particular flow rate will be equaled or exceeded. It should be noted that exceedance probability for duration statistics is different from the "annual exceedance probability" associated with flood frequency statistics and there is no practical way of converting/relating annual exceedance probability statistics to flow duration statistics.

The flow duration standard can be viewed graphically as shown in Figure F.6. The flow duration curve for the site under pre-developed conditions is computed and is the target to which the post-developed flow duration curve is compared. The flow duration curve for the pond discharge must match the applicable pre-developed curve between 50 percent of the pre-developed 2-year (0.5 Q_2) and an upper limit, either the 2-year (Q_2) or the 50-year (Q_{50}) depending on the flow duration design standard for the facility.

Specified flow levels for the Pre-developed Forest Standard are typically 50 percent of the pre-developed 2-year peak flow (0.5 Q_2), the pre-developed 2-year peak flow (Q_2), and the pre-developed 50-year peak flow (Q_{50}) plus 97 other incremental flow values between 0.5 Q_2 and Q_{50}. Specified flow levels for the Pre-developed Pasture Standard are typically 50 percent of the pre-developed 2-year peak flow (0.5 Q_2) and the pre-developed 2-year peak flow (Q_2) plus 98 other incremental flow values between 0.5 Q_2 and Q_2.

Depending on the flow duration design standard applicable to the stormwater facility, three criteria are evaluated to determine if the standard has been met.

1. Post-development flow duration values may not exceed the pre-development flow duration values between 50 percent of the pre-developed 2-year peak flow (0.5 Q_2) and the pre-developed 2-year peak flow (Q_2).
2. Post-development flow duration values may not exceed pre-development flow duration values between the pre-developed 2-year peak flow (Q_2) and the pre-developed 50-year peak flow (Q_{50}) by more than 10 percent, i.e., a post-development flow duration value may be up to 110 percent of the corresponding pre-development flow duration value.

3. Post-development flow duration values may not exceed pre-development flow duration values for more than 50 percent of flow duration levels, i.e., not more than half of the post-development flow duration values may exceed 100 percent of the corresponding pre-development flow duration value.

General guidance for adjusting the geometry and outlets of stormwater ponds to meet the duration standard were developed by King County (1999) and are summarized in Figure F.7 and described below. Refinements should be made in small increments with one refinement at a time. In general, the recommended approach is to analyze the duration curve from bottom to top, and adjust orifices from bottom to top. Inflection points in the outflow duration curve occur when additional structures (e.g., orifices, notches, overflows) become active. Refer to *Volume 3, Chapter 5* for complete facility design and sizing requirements.

Step #	Parameter	Procedure
P-1	Bottom Orifice Size	Adjust the bottom orifice to control the bottom arc of the post-developed flow duration curve. Reducing the bottom orifice discharge lowers and shortens the bottom arc while increasing the bottom orifice raises and lengthens the bottom arc.
P-2	Height of Second Orifice	The invert elevation of the second orifice affects the point on the flow duration curve where the transition (break in slope) occurs from the curve produced by the low-level orifice. Lower the invert elevation of the second orifice to move the transition point to the right on the lower arc. Raise the height of the second orifice to move the transition point to the left on the lower arc.
P-3	Second Orifice Size	The upper arc represents the combined discharge of both orifices. Adjust the second orifice size to control the arc of the curve for post-developed conditions. Increasing the second orifice raises the upper arc while decreasing the second orifice lowers the arc.
P-4	Pond Volume	Adjust the pond volume to control the upper end of the duration curve. Increase the pond volume to move the entire curve down and to the left to control riser overflow conditions. Decrease the pond volume to move the entire curve up and to the right to ensure that the outflow duration curve extends up to the riser overflow.

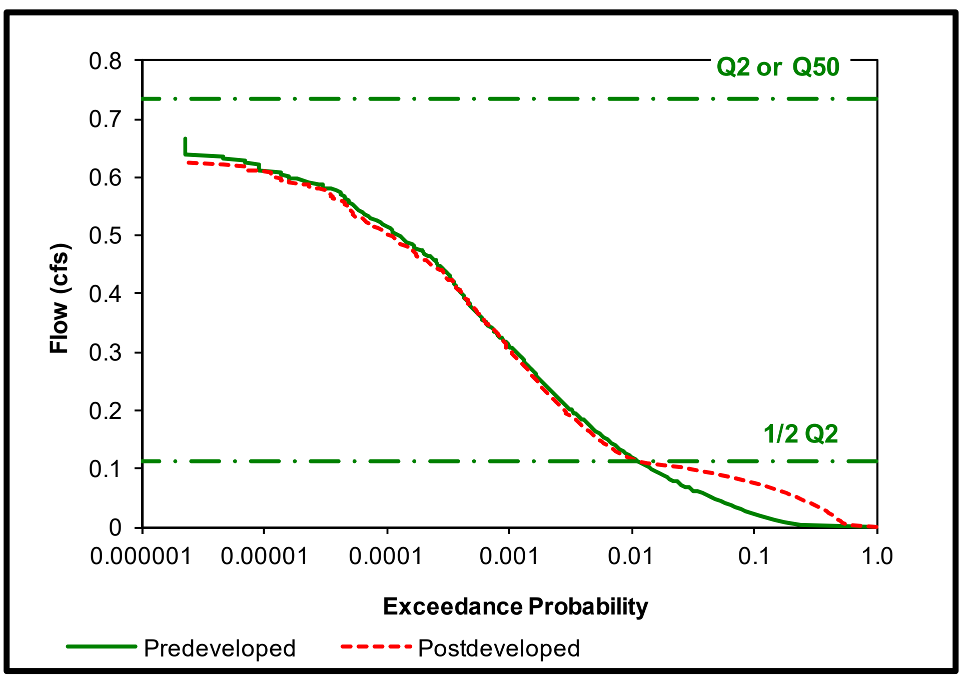


Figure F.6. Comparison of Pre-Developed and Post-Developed Flow Duration Curves.

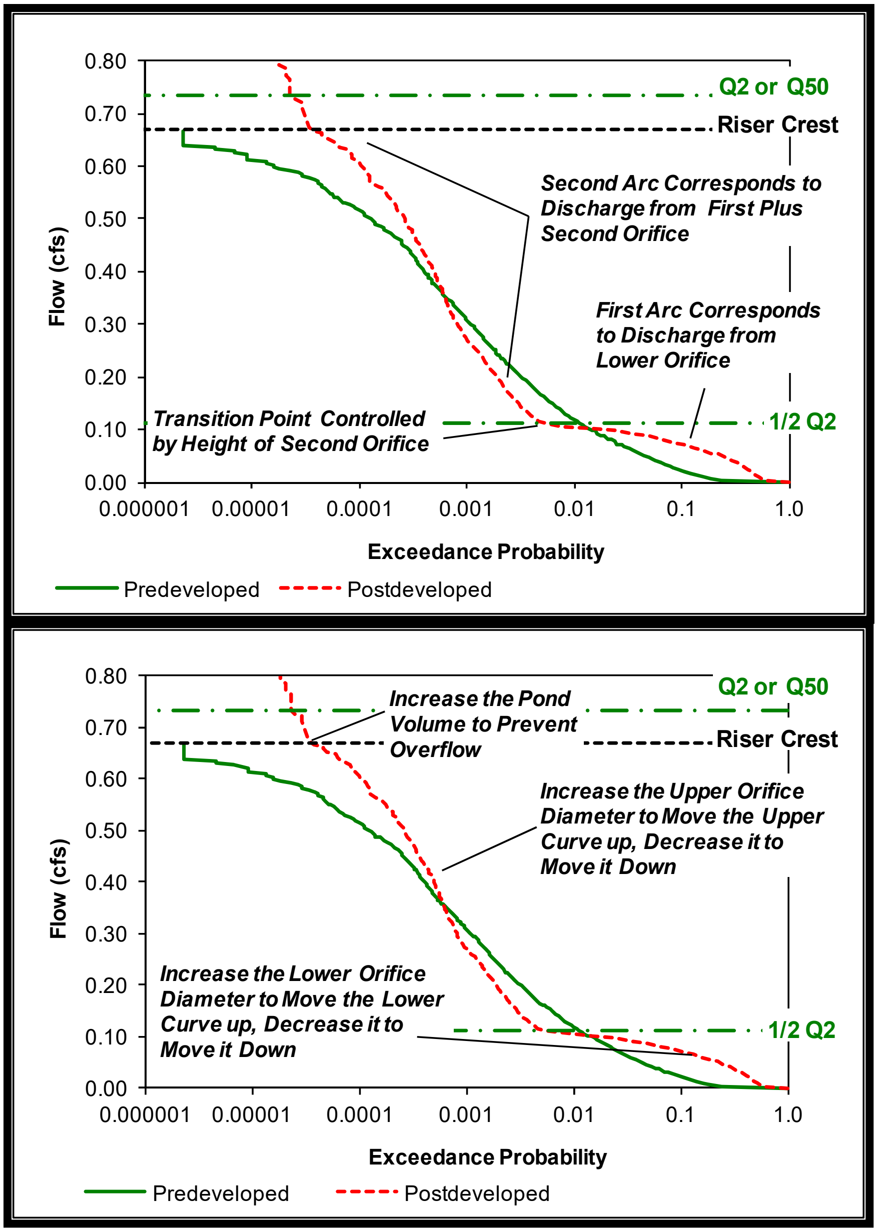


Figure F.7. General Guidance for Adjusting Pond Performance.

On-site Performance Standard BMP Design

This section provides guidance for sizing BMPs to meet the On-site Performance Standard. If the applicant chooses to use the On-site List Approach, modeling is typically not required (refer to sizing requirements in Chapter 5 of Volume 3). If the applicant chooses to use the On-site Performance Standard, the modeling procedures will depend upon the applicable target (i.e., forest or pasture). See Volume 3, Section 5.2.1 to determine the target based on the percent of existing hard surface and the type of drainage basin.

If the project discharge durations must match pre-developed forest flow durations ~~for~~ from 8 percent to 50 percent of the 2-year pre-developed flow, the procedures outlined above in the *Flow Duration Standard* subsection are generally applicable (with duration bounds revised to 8 percent to 50 percent of the 2-year flow). Both WWHM and MGSFlood have the capability to evaluate (and report “pass” and “fail”) for this standard.

If the project discharge durations must match pre-developed pasture flow durations for the range of pre-developed discharge rates between the 1 percent and 10 percent exceedance values, the procedures outlined in this section are applicable.

The “frequency of exceedance” or “percent exceedance” (as referenced in the Stormwater Code), is the percent of time, over the simulation period (e.g., 158 years), that a given flow is equaled or exceeded. MGSFlood and WWHM both report “exceedance probability”- the decimal equivalent of “percent exceedance.” For example, the 1 to 10 percent exceedance range corresponds to the 0.01 and 0.1 exceedance probabilities displayed on the flow duration curves (see Figure F.7a). The standard is achieved if the post-developed flows are less than the pre-developed flows for the 1 to 10 percent exceedance range (red line is beneath the green line for the shaded range of exceedance values).

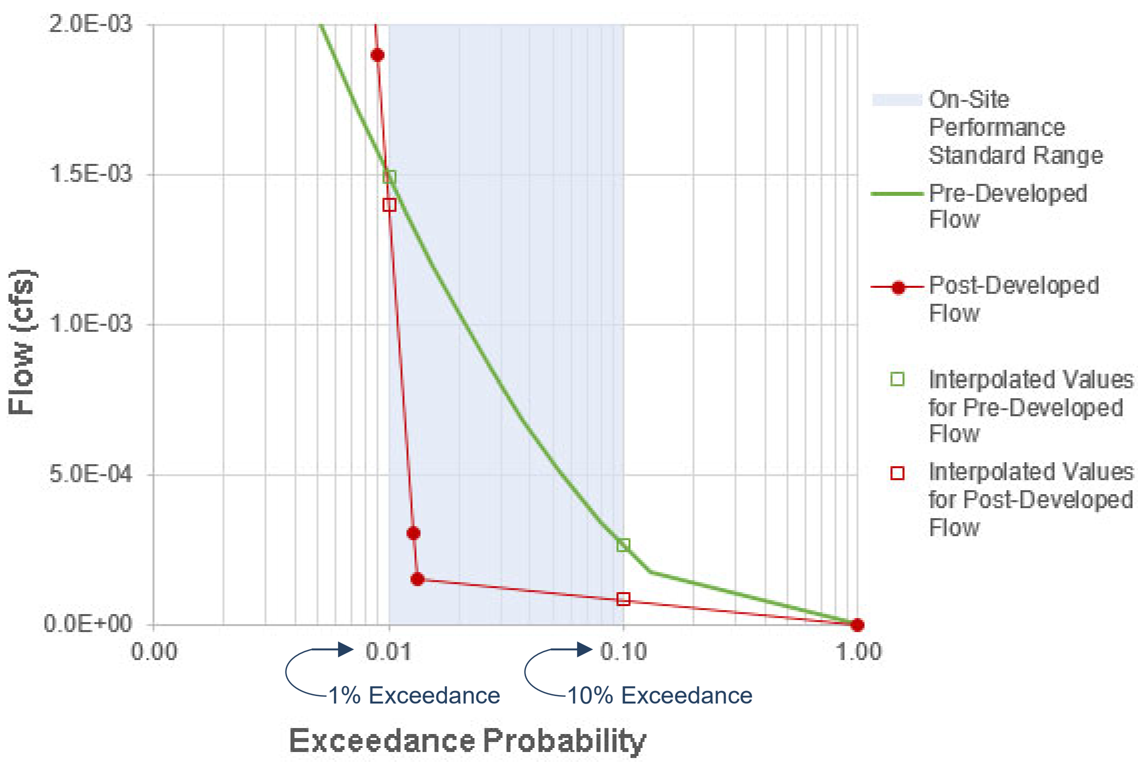


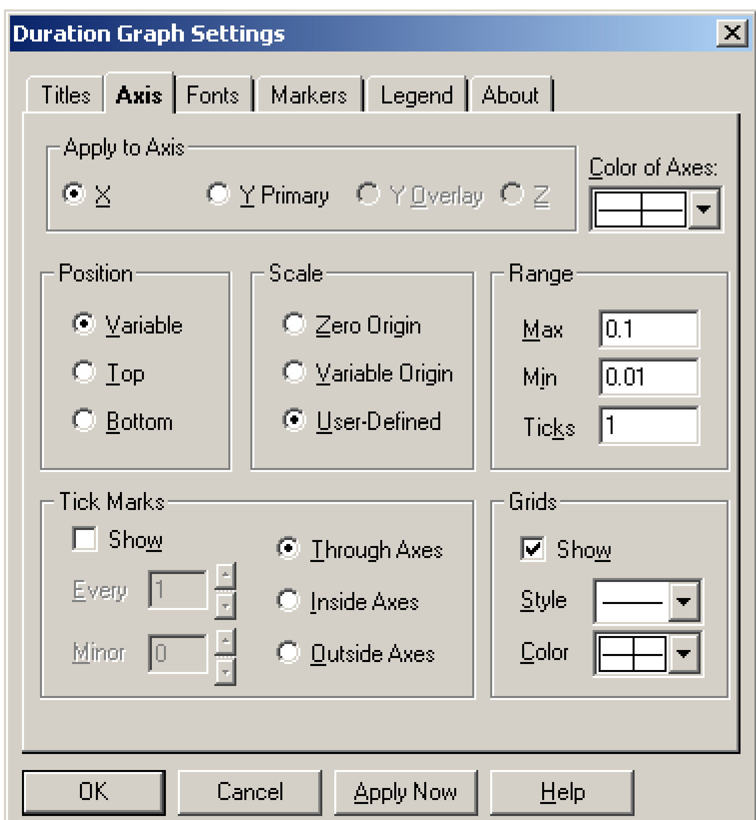
Figure F.7a. On-site Performance Standard Duration Curve.

The latest versions of MGSFlood (Version 4.5.6 ~~and~~or later~~as of April 2021~~) include the option to conduct a flow duration analysis based on the 1 to 10 percent exceedance standard. An MGSFlood user can select the option on the LID Duration tab in the Options menu. MGSFlood will then report “pass” or “fail” for the 1 to 10 percent exceedance standard. WWHM does not ~~currently (as of April 2021)~~ explicitly report “pass” or “fail” for the 1 to 10 percent exceedance standard. However, WWHM allows the user to define the bounds of duration analysis in term of flow rate (cubic feet per second). A user can calculate the pre-developed pasture 1 and 10 percent exceedance flow rates using the software and manually enter them as the bounds for the flow duration analysis on the Duration Criteria tab in the Options menu. WWHM will then report “pass” or “fail” for the 1 to 10 percent exceedance standard. For users with different or older software, the following procedures may be used to determine compliance with Seattle Stormwater Code. Details are provided for determining compliance with both MGSFlood and WWHM but similar procedures may be applicable to other software programs.

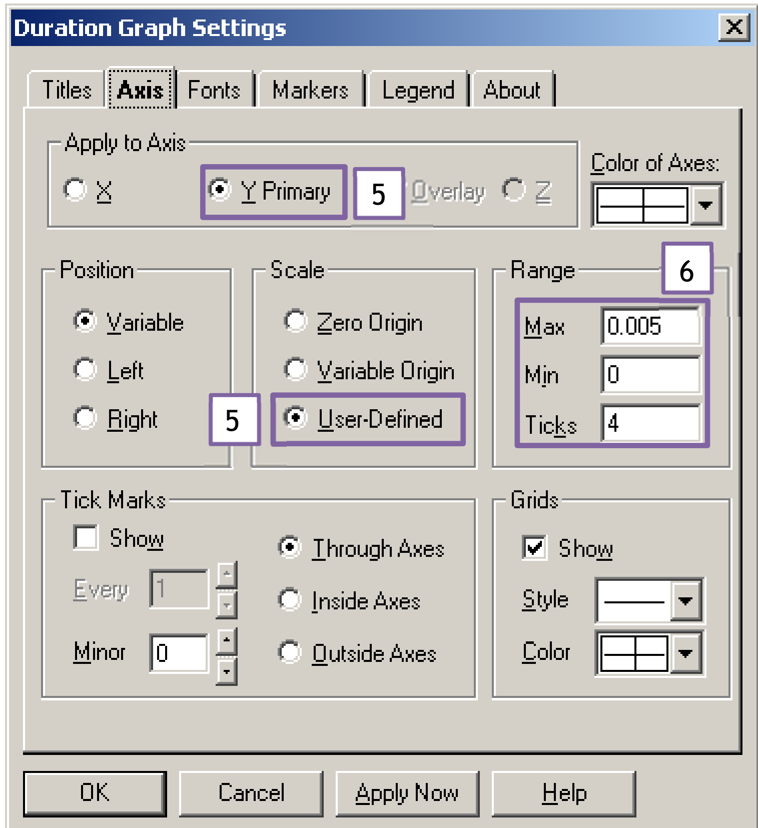
Visual Evaluation of On-site Performance Standard in MGSFlood (Versions prior to 4.5.6)

For versions of MGSFlood released prior to Version 4.5.6, evaluation of the 1 to 10 percent exceedance standard is not automated and ~~Compliance with the 1 to 10 percent exceedance standard~~ may be confirmed by visually observing the MGSFlood Flow Duration Plot. The axes on the plot may be adjusted to clearly display the duration curve from 1 to 10 percent exceedance. Step-by-step instructions are provided below.

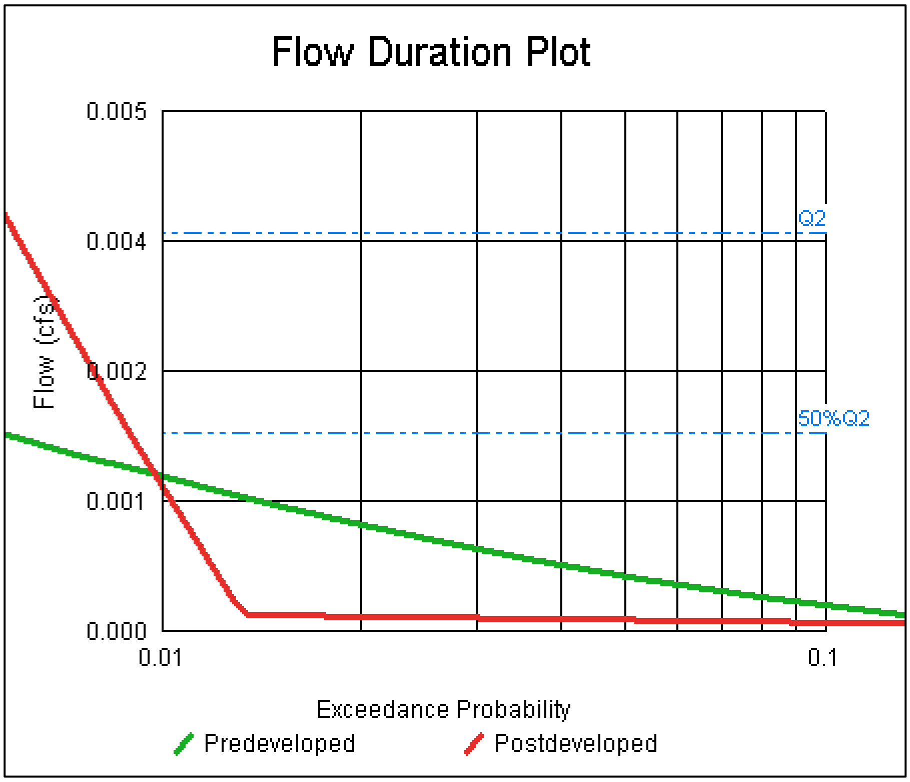
1. Right click on the Flow Duration Plot to open Duration Graph Settings
2. Select “Axis” tab
3. Edit x-axis scale (select “X”, “User Defined”)
4. Update x-axis range of values as follows:
   a. Max = 0.1
   b. Min = 0.01
   c. Ticks = 1



5. Edit y-axis scale (select “Y Primary,” “User Defined”)
6. Update y-axis range of values. Values will vary depending on size of contributing area.



7. Visually inspect to confirm that the post-developed flows are less than the pre-developed flows for the 1 to 10 percent exceedance range (red line is beneath the green line for the range plotted).



Quantitative Evaluation of the On-site Performance Standard in MGSFlood (Versions prior to 4.5.6)

For versions of MGSFlood released prior to Version 4.5.6, ~~If the user wishes to fully optimize~~ BMP sizes can be fully optimized for the 1 to 10 percent exceedance standard by conducting ~~,~~ ~~values must be~~ calculations~~ed and evaluated~~ outside of the model. Step-by-step procedures are provided below with an example:

1. Build and run the model
2. View report file (File>View Report)
3. Select “Full Output” to get full detailed report and click “Refresh”
4. Navigate to “Point of Compliance Flow Duration Data”
5. Determine pre-developed flows associated with 1 percent and 10 percent exceedance probability using the steps below. Note that a higher probability of exceedance corresponds to lower, more frequent, flows.

a. Identify the exceedance probability values immediately higher and immediately lower than the 1 percent exceedance. Record the exceedance probabilities and the associated flows as shown in the example below:

	Pre-development Runoff Discharge (cfs)	Exceedance Probability
Higher than 1%	1.37E-03	1.19%
Lower than 1%	1.54E-03	0.94%

6. Identify the exceedance probability values immediately higher and immediately lower than the 10 percent exceedance. Record the exceedance probabilities and the associated flows as shown in the example below:

	Pre-development Runoff Discharge (cfs)	Exceedance Probability
Higher than 10%	1.71E-04	13.15%
Lower than 10%	3.42E-04	7.97%

7. Logarithmically interpolate flows associated with the 1 and 10 percent exceedance probabilities using Equation 1 and Equation 2, respectively.

$$Flow_{1\%} = Flow_{lower} + \frac{Flow_{lower} - Flow_{higher}}{\log(Exceedance_{lower}) - log(Exceedance_{higher})} \times [log(1\%) - log(Exceedance_{lower})] \qquad \text{Eq 1.}$$

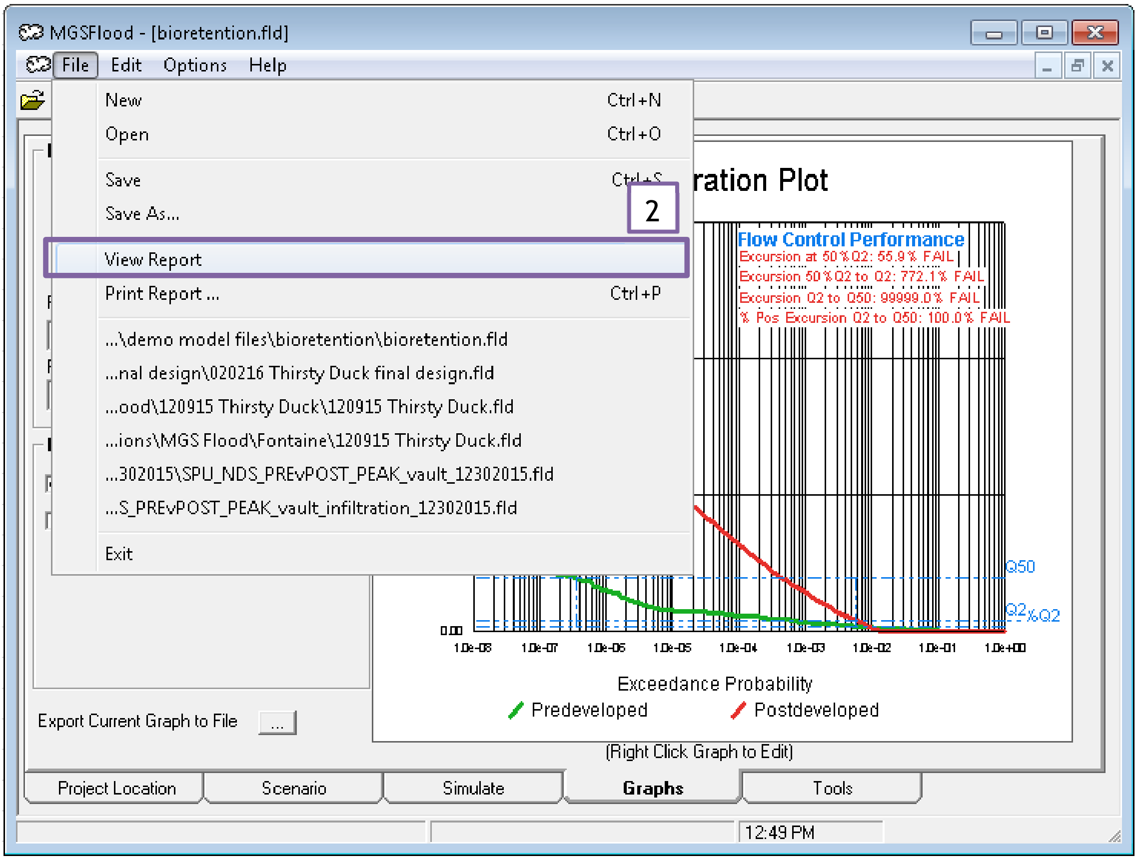
$$Flow_{10\%} = Flow_{lower} + \frac{Flow_{lower} - Flow_{higher}}{\log(Exceedance_{lower}) - log(Exceedance_{higher})} \times [log(10\%) - log(Exceedance_{lower})] \qquad \text{Eq 2.}$$

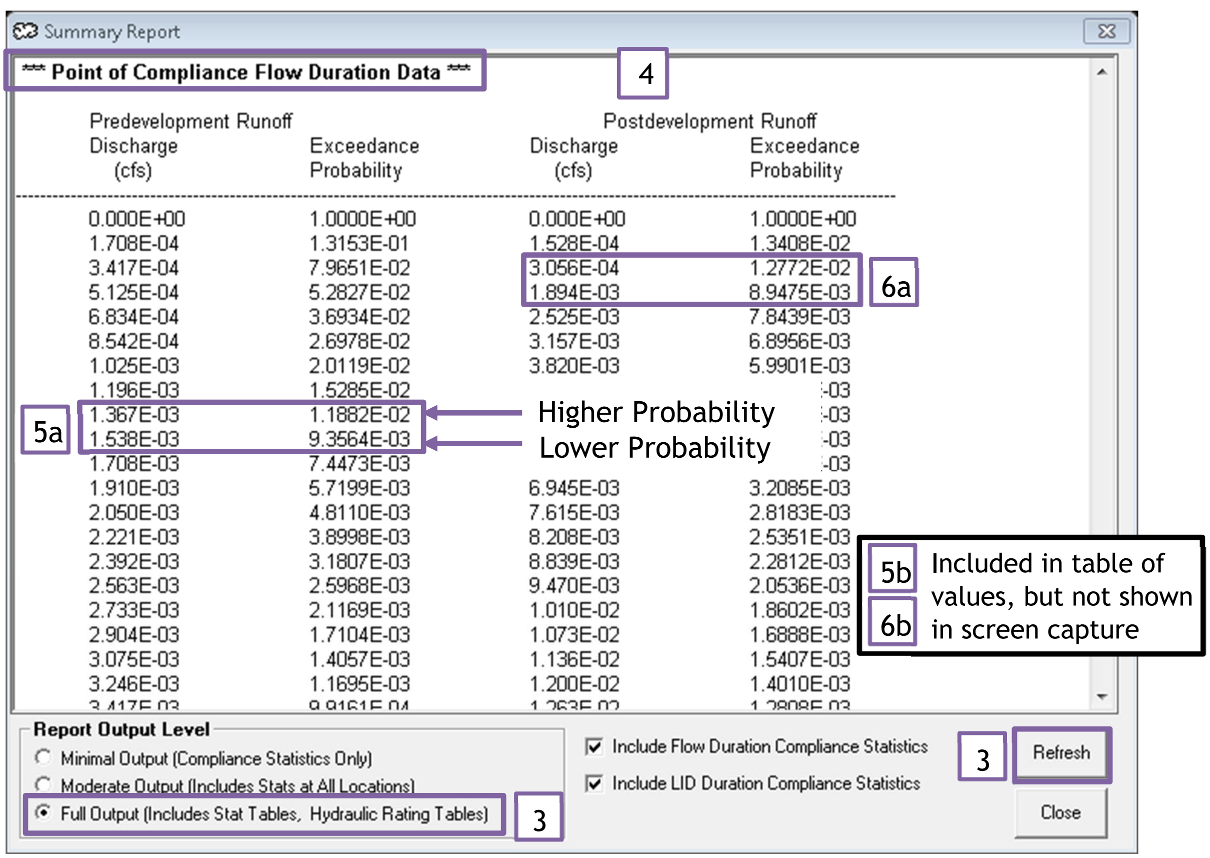
Results for this example are shown below:

	Pre-development Runoff Discharge (cfs)	Exceedance Probability
Interpolated flows at 1%	1.49E-03	1.00%
Interpolated flows at 10%	2.64E-04	10.00%

8. Determine post-developed flows associated with 1 percent and 10 percent exceedance probability. Repeat Steps 5a, 5b, and 5c using post-developed flows.

	Post-development Runoff Discharge (cfs)	Exceedance Probability
Interpolated flows at 1%	1.40E-03	1.00%
Interpolated flows at 10%	8.16E-05	10.00%





Predevelopment Runoff Discharge (cfs)	Exceedance Probability	Postdevelopment Runoff Discharge (cfs)	Exceedance Probability
0.000E+00	1.0000E+00	0.000E+00	1.0000E+00
1.708E-04	1.3153E-01	1.528E-04	1.3408E-02
3.417E-04	7.9651E-02	3.056E-04	1.2772E-02
5.125E-04	5.2827E-02	1.894E-03	8.9475E-03
6.834E-04	3.6934E-02	2.525E-03	7.8439E-03
8.542E-04	2.6978E-02	3.157E-03	6.8956E-03
1.025E-03	2.0119E-02	3.820E-03	5.9901E-03
1.196E-03	1.5285E-02		-03
1.367E-03	1.1882E-02		-03
1.538E-03	9.3564E-03		-03
1.708E-03	7.4473E-03		-03
1.910E-03	5.7199E-03	6.945E-03	3.2085E-03
2.050E-03	4.8110E-03	7.615E-03	2.8183E-03
2.221E-03	3.8998E-03	8.208E-03	2.5351E-03
2.392E-03	3.1807E-03	8.839E-03	2.2812E-03
2.563E-03	2.5968E-03	9.470E-03	2.0536E-03
2.733E-03	2.1169E-03	1.010E-02	1.8602E-03
2.904E-03	1.7104E-03	1.073E-02	1.6888E-03
3.075E-03	1.4057E-03	1.136E-02	1.5407E-03
3.246E-03	1.1695E-03	1.200E-02	1.4010E-03
3.417E-03	9.9161E-04	1.263E-02	1.2808E-03

9. Compare pre-developed flows and post-developed flows at 1 and 10 percent exceedance probabilities and visually confirm, from the flow duration curves in the model, that the post-developed flows are smaller than the pre-developed flows. If post-developed flows at the 1 or 10 percent exceedance probability are higher than the pre-developed flows, or if the post developed flows appear to exceed the pre-developed flows for the 1 to 10 percent exceedance range of the duration curve (refer to procedures for visual observation, above), increase the BMP size(s), run the model, and repeat Steps 2 through 9.

See Figure F.7a for a comparison of pre-and post-developed flow duration curves for the target exceedance probability range. Figure F.7a also includes the interpolated data points described above, shown as hollow squares on the graph. If post-developed flows (shown in red) are smaller than pre-developed flows (shown in green) for the target exceedance probability range (grey hatch), the project satisfies the On-site Performance Standard.

Visual Evaluation of On-site Performance Standard in WWHM

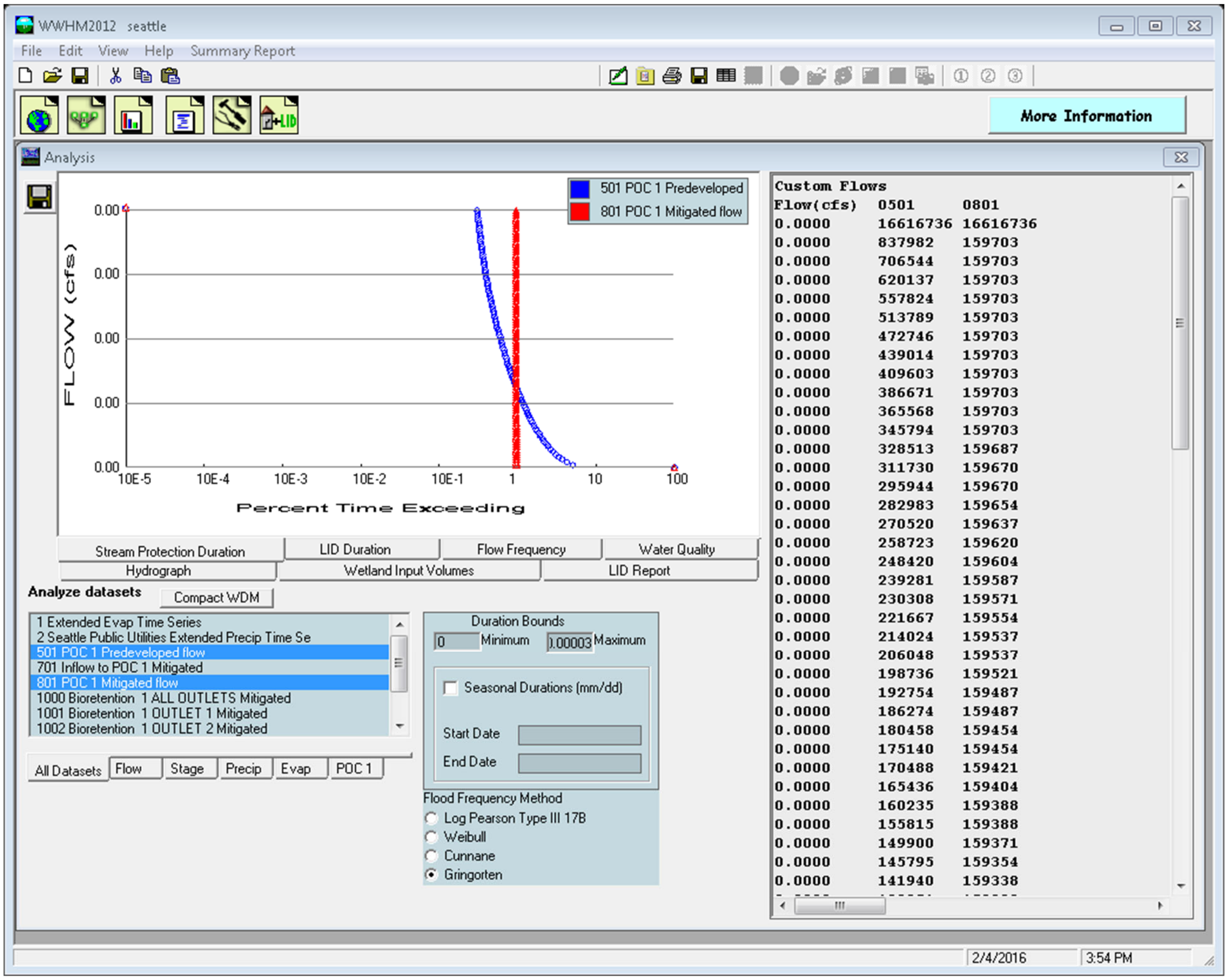
Compliance with the 1 to 10 percent exceedance standard may be estimated by visually observing the WWHM Stream Protection Duration Plot. The axes on the plot must be adjusted and manually evaluated to more clearly display the duration curve from 1 to 10 percent exceedance. Because the graphs are difficult to accurately read, the facility may need to be somewhat oversized to visually confirm compliance. Step-by-step instructions are provided below:

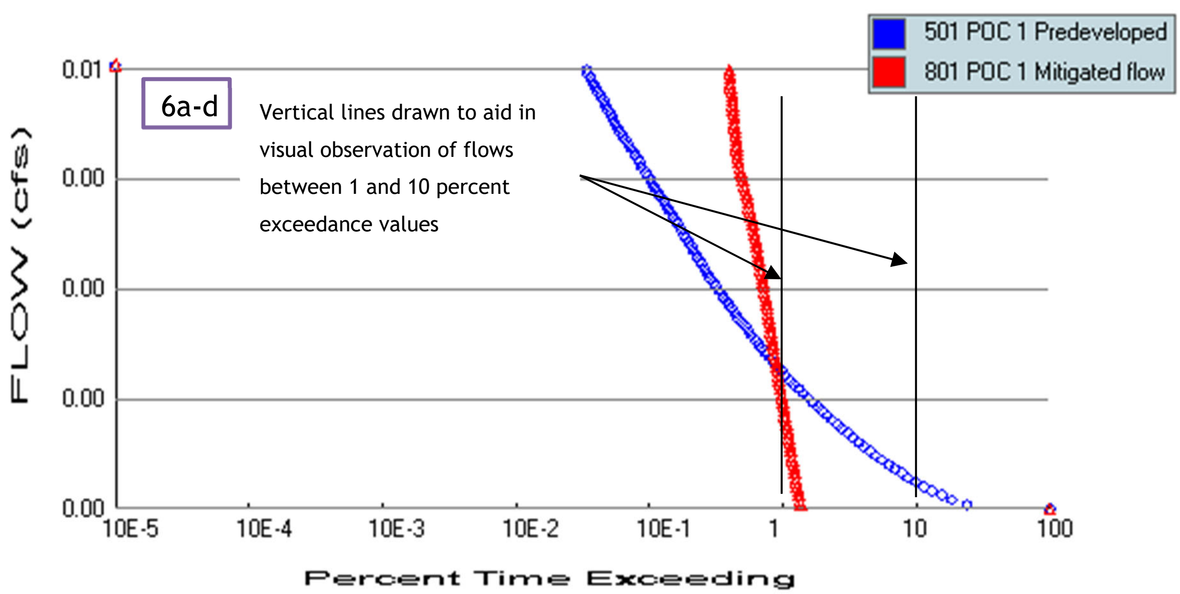
1. Build and run the model
2. View the “Stream Protection Duration” results in the Analysis tab window
3. Select the appropriate points of compliance for the pre-developed scenario and the mitigated (i.e., post-developed) scenario under “All Datasets” (hold CTRL to select more than one)

   501 POC 1 Predeveloped flow
   801 POC 1 Mitigated flow

4. Modify the “Duration Bounds” to include the 1 and 10 percent exceedance values
   a. Minimum = 0 cfs
   b. Maximum = established by trial and error until the pre-developed flows corresponding to the 1 percent exceedance are visible on the graph. To optimize the facility size(s), set the maximum value slightly above the predeveloped flow that is exceeded 1 percent of the time. This value can be approximated as the contributing area in acres times 0.00025 cfs per acre.
5. Select the “Stream Protection Duration” tab to re-calculate the results with the new duration bounds

6. Visually inspect the duration plot to confirm that the mitigated flows are smaller than the pre-developed flows for the 1 to 10 percent exceedance range. Because the plots are difficult to accurately read, the following steps are required to confirm compliance with the 1 to 10 percent exceedance standard:
    a. Take a screenshot of the flow duration curve
    b. Paste the screenshot into a word processing software, e.g., Microsoft Word
    c. Overlay two vertical lines at the 1% and 10% tick marks
    d. Confirm the mitigated flows (red line) are below the pre-developed flows (blue line) within the range of the two horizontal lines. Note: to visually ensure compliance, the facility may need to be somewhat oversized (the screenshot shown below is 10 percent larger than required when quantitative evaluated using the procedure provided below).


WWHM2012 seattle
File Edit View Help Summary Report
More Information
Analysis
501 POC 1 Predeveloped
801 POC 1 Mitigated flow
FLOW (cfs)
0.00
10E-5 10E-4 10E-3 10E-2 10E-1 1 10 100
Percent Time Exceeding
Custom Flows
Flow(cfs) 0501 0801
0.0000 16616736 16616736
0.0000 837982 159703
0.0000 706544 159703
0.0000 620137 159703
0.0000 557824 159703
0.0000 513789 159703
0.0000 472746 159703
0.0000 439014 159703
0.0000 409603 159703
0.0000 386671 159703
0.0000 365568 159703
0.0000 345794 159703
0.0000 328513 159687
0.0000 311730 159670
0.0000 295944 159670
0.0000 282983 159654
0.0000 270520 159637
0.0000 258723 159620
0.0000 248420 159604
0.0000 239281 159587
0.0000 230308 159571
0.0000 221667 159554
0.0000 214024 159537
0.0000 206048 159537
0.0000 198736 159521
0.0000 192754 159487
0.0000 186274 159487
0.0000 180458 159454
0.0000 175140 159454
0.0000 170488 159421
0.0000 165436 159404
0.0000 160235 159388
0.0000 155815 159388
0.0000 149900 159371
0.0000 145795 159354
0.0000 141940 159338
Stream Protection Duration
LID Duration
Flow Frequency
Water Quality
Hydrograph
Wetland Input Volumes
LID Report
Analyze datasets
Compact WDM
1 Extended Evap Time Series
2 Seattle Public Utilities Extended Precip Time Se
501 POC 1 Predeveloped flow
701 Inflow to POC 1 Mitigated
801 POC 1 Mitigated flow
1000 Bioretention 1 ALL OUTLETS Mitigated
1001 Bioretention 1 OUTLET 1 Mitigated
1002 Bioretention 1 OUTLET 2 Mitigated
All Datasets
Flow
Stage
Precip
Evap
POC 1
Duration Bounds
0 Minimum
0.00003 Maximum
Seasonal Durations (mm/dd)
Start Date
End Date
Flood Frequency Method
Log Pearson Type III 17B
Weibull
Cunnane
Gringorten
2/4/2016
3:54 PM


501 POC 1 Predeveloped
801 POC 1 Mitigated flow
6a-d
Vertical lines drawn to aid in visual observation of flows between 1 and 10 percent exceedance values
FLOW (cfs)
0.01
0.00
0.00
0.00
0.00
10E-5 10E-4 10E-3 10E-2 10E-1 1 10 100
Percent Time Exceeding

Evaluation of the On-site Performance Standard in WWHM

To quantitatively evaluate and fully optimize BMP sizes for the 1 to 10 percent exceedance standard, values must be calculated and evaluated outside of the model. Step-by-step procedures are provided below with an example:

1. Build and run the model
2. View the "Stream Protection Duration" results in the Analysis tab window
3. Select the appropriate points of compliance for the pre-developed scenario and the mitigated (i.e., post-developed) scenario under "All Datasets" (hold CTRL to select more than one)

   501 POC 1 Predeveloped flow
   801 POC 1 Mitigated flow

4. Modify the "Duration Bounds" to include the 1 and 10 percent exceedance values
   a. Minimum = 0 cfs
   b. Maximum = established by trial and error until the pre-developed flows corresponding to the 1 percent exceedance are visible on the graph. To optimize the facility size(s), set the maximum value slightly above the predeveloped flow that is exceeded 1 percent of the time. This value can be approximated as the contributing area in acres times 0.00025 cfs per acre.

   0.12 acres x 0.00025 cfs/acre = 0.00003

5. Select the "Stream Protection Duration" tab to re-calculate the results with the new duration bounds
6. Determine the total number of timesteps calculated by the model. Refer to the first line in the "Custom Flows" table (i.e., number of timesteps associated with a flow of zero cfs [flow at every timestep is greater than or equal to zero cfs]).

Custom Flows		
Flow(cfs)	0501	0801
0.0000	16616736	16616736

7. Calculate the number of timesteps that correspond to the 1 percent and 10 percent exceedance values using Equations 3 and 4

$$1 \text{ Percent of Timesteps} = \text{Total number of Timesteps} \times 0.01 \qquad \text{Eq 3.}$$

$$10 \text{ Percent of Timesteps} = \text{Total number of Timesteps} \times 0.1 \qquad \text{Eq 4.}$$

1 Percent of Timesteps = 16,616,736 x 0.01 = 166,167
10 Percent of Timesteps = 16,616,736 x 0.1 = 1,661,674

8. Compare pre-developed flows and post-developed (i.e., mitigated) flows at the 1 percent exceedance probability. While the flow values themselves are often too small to display in the "Custom Flows" table in WWHM, the number of timesteps a given flow is exceeded can be used to evaluate facility performance relative to the pre-developed condition. For the On-site Performance standard, all flows with a probability of exceedance from 1 to 10 percent should be exceeded at the same

frequency, or less frequently than the predeveloped condition. In other words, for a given flow in the target range, the number of timesteps that flow is exceeded should be fewer in the mitigated scenario than the pre-developed scenario. To compare the pre-developed and mitigated flows:

a. Identify the flow values immediately higher and immediately lower than the target 1 percent of timesteps (as determined in Step 7) for the pre-developed scenario

b. Compare the number of timesteps these flow values are exceeded in the mitigated scenario to the pre-developed scenario.

c. If the pre-developed scenario is exceeded less frequently than the mitigated scenario, increase facility size and repeat Step 8.

d. Proceed to Step 9.

Custom Flows		
Flow(cfs)	0501	0801
0.0000	170488	159421
0.0000	165436	159404

The first flow is exceeded for 170,488 timesteps (170,488/16,616,736 = 1.03%) in the pre-developed condition. The second flow is exceeded for 165,436 timesteps (165,436/16,616,736 = 0.996%) in the pre-developed condition. For these flows, the mitigated scenario is exceeded for a fewer number of timesteps than the pre-developed scenario, therefore the mitigated condition meets the On-site Performance Standard at the 1 percent exceedance value.

9. Compare pre-developed flows and post-developed (i.e., mitigated) flows at the 10 percent exceedance probability:

a. Identify the flow values immediately higher and immediately lower than the target 10 percent of timesteps (as determined in Step 7) for the pre-developed scenario.

b. Compare the number of timesteps these flow values are exceeded in the mitigated scenario to the pre-developed scenario.

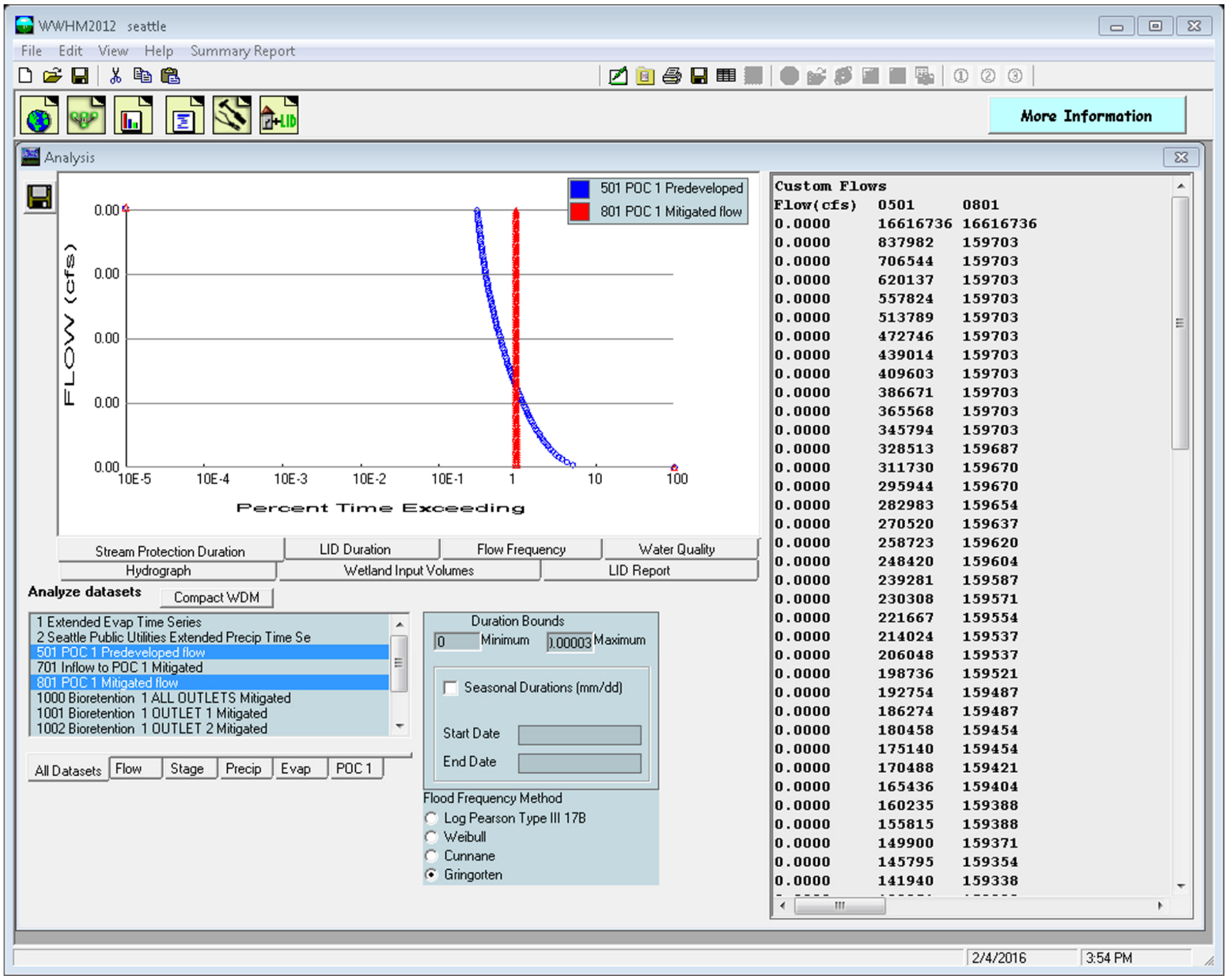
c. If the pre-developed scenario is exceeded less frequently than the mitigated scenario, increase facility size and repeat Step 9.

d. Proceed to Step 10.

Custom Flows		
Flow(cfs)	0501	0801
0.0000	16616736	16616736
0.0000	837982	159703

The first flow is exceeded for 16,616,736 timesteps (16,616,736/16,616,736 = 100%) for both the pre-developed scenario and the mitigated scenario. The second flow is exceeded for 837,982 timesteps (837,982/16,616,736 = 5.04%) in the pre-developed condition and is exceeded for 21,134 timesteps in the mitigated condition. Therefore the mitigated condition meets the on-site standard at the 10 percent exceedance value.

10. Visually confirm, from the flow duration curves in the model, that the mitigated flows are smaller than the pre-developed flows for the 1 to 10 percent exceedance range. If the post developed flows appear to exceed the pre-developed flows for the 1 to 10 percent exceedance range of the duration curve, increase the BMP size(s) and repeat Steps 8 through 10.



Water Quality Treatment BMP Design

Water Quality Design Volume

The water quality design volume for sizing wet ponds is computed as the daily runoff volume that is greater than or equal to 91 percent of all daily values in the simulation period. The continuous model develops a daily runoff time series from the pond inflow time series and scans the computed daily time series to determine the 24-hour volume that is greater than or equal to 91 percent of all daily values in the time series. This value is then used as the volume for a “Basic Wet Pond” and 1.5 times this value is used for sizing a “Large Wet Pond."

The water quality design volume is defined as the daily runoff volume at which 91 percent of the total runoff volume is produced by smaller daily volumes. The procedure can be visualized using Figure F.8 below. The bars on the graph represent daily inflow volume for the entire simulation. The time span along the x-axis in Figure F.8 is for 105 days, but in practice, this would include the entire simulated inflow time series (e.g., 158 years).

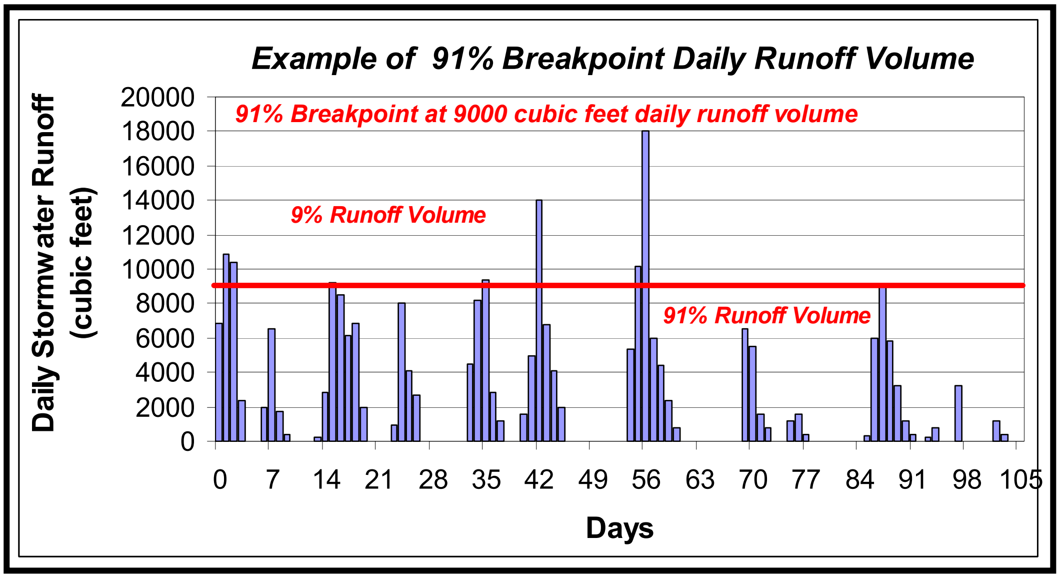


Figure F.8. Example of Portion of Time-Series of Daily Runoff Volume and Depiction of Water Quality Design Volume.

The horizontal line represents the water quality design volume. Its value is calculated such that 91 percent of the total daily runoff volume for the entire simulation resides below this line and 9 percent of the total daily runoff volume for the entire simulation exceeds the water quality design volume. Stated another way, if you total the daily runoff volumes that exceed the 9,000 cubic foot water quality design volume, they represent 9 percent of the total runoff volume.

The process for computing this water quality design volume may vary among continuous simulation models. An example of a typical approach used to compute the water quality design volume (WQDV) is summarized below.

Step #	Procedure
WQDV-1	Compute daily volume to the pond using the inflow time (convert the inflow rate to inflow volume on a midnight to midnight basis using a 1-hour or less time step).
WQDV-2	Compute the total inflow volume by summing all of the daily inflow volume values for the entire simulation.
WQDV-3	Compute a breakpoint value by multiplying the total runoff volume computed in Step WQDV-2 by 9 percent.
WQDV-4	Sort the daily runoff values from Step WQDV-1 in descending order (highest to lowest).
WQDV-5	Sum the sorted daily volume values until the total equals the 9 percent breakpoint. That is, the largest volume is added to the second largest, which is added to the third largest, etc., until the total equals the 9 percent breakpoint.
WQDV-6	The last daily value added to match the 9 percent breakpoint is defined as the water quality design volume.

Water Quality Treatment Design Flow Rate

The flow rate used to design flow rate dependent treatment facilities depends on whether or not the treatment is located upstream of a stormwater detention facility and whether it is an on-line or offline facility (Figure F.9).

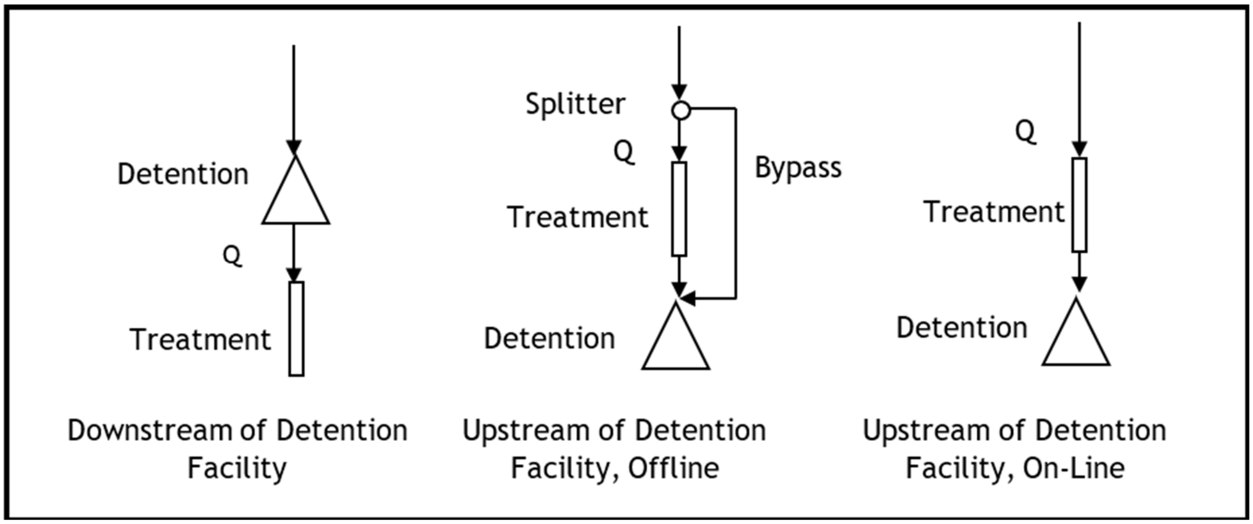


Figure F.9. Water Quality Treatment and Detention Definition.

Downstream of Detention Facilities: If the treatment facility is located downstream of a stormwater detention facility, then the water quality design flow rate is the release rate from the detention facility that has a 50 percent annual probability of occurring in any given year (2-year recurrence interval).

Upstream of Detention Facilities, Offline: Offline water quality treatment located upstream of the detention facility includes a high-flow bypass that routes the incremental flow in excess of the water quality design rate around the treatment facility. It is assumed that flows from the bypass enter the system downstream of the treatment facility but upstream of the detention facility. The continuous model determines the water quality treatment design flow rate as the rate corresponding to the runoff volume that is greater than or equal to 91 percent of the 15-minute runoff volume entering the treatment facility (Figure F.10). If runoff is computed using the City of Seattle Design Time Series with a time step of 15 minutes or less, then no time step adjustment factors are needed for the water quality design discharge.

Upstream of Detention Facilities, On-line: On-line water quality treatment does not include a high-flow bypass for flows in excess of the water quality design flow rate and all runoff is routed through the facility. The continuous model determines the water quality treatment design flow rate as the rate corresponding to the runoff volume that is greater than or equal to 91 percent of the 15-minute runoff volume entering the treatment facility. However, those flows that exceed the water quality design flow are not counted as treated in the calculation (Figure F.11). Therefore, the design flow rate for on-line facilities is higher than for offline facilities. If runoff is computed using the City of Seattle Design Time Series with a time step of 15 minutes or less, then no time step adjustment factors are need for the water quality design discharge.

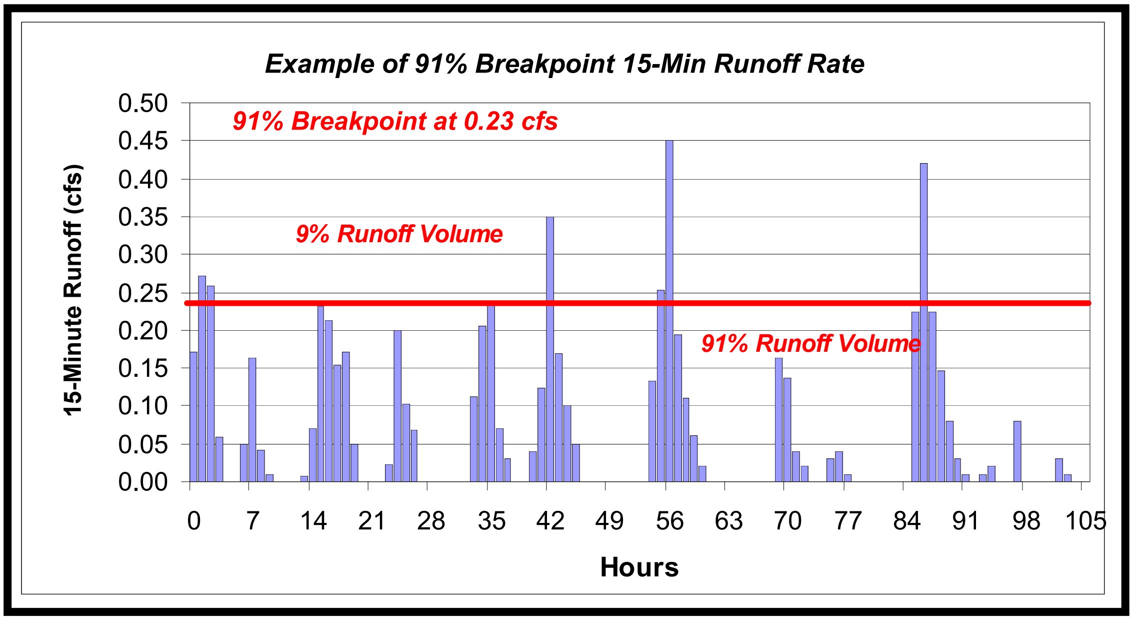


Figure F.10. Offline Water Quality Treatment Discharge Example.

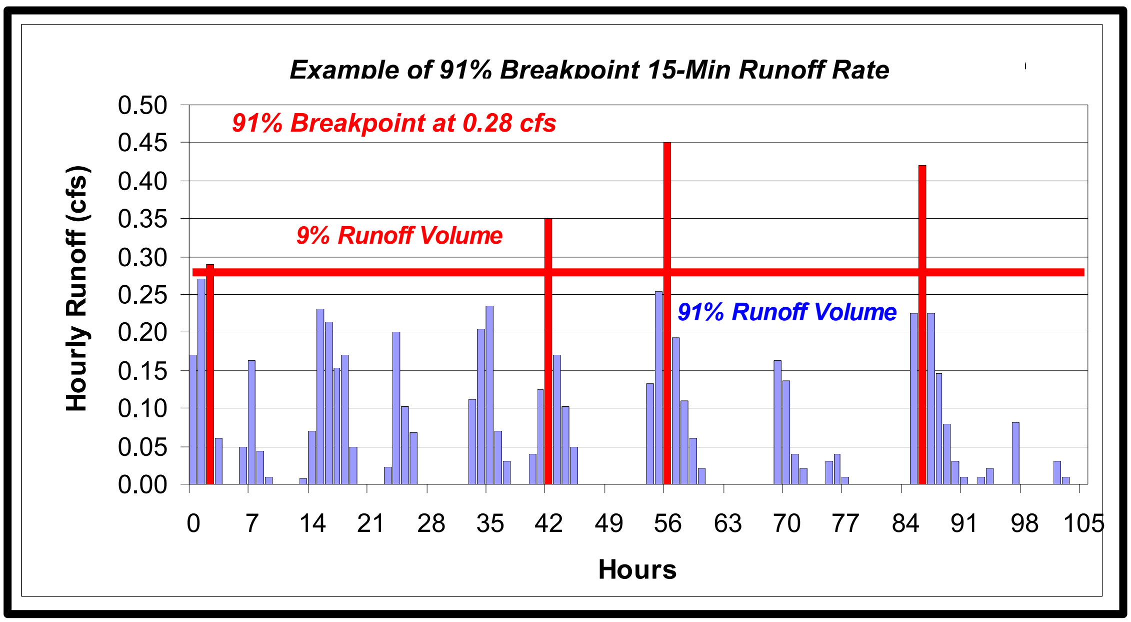


Figure F.11. On-Line Water Quality Treatment Discharge Example.

Infiltration Facilities Providing Water Quality Treatment: Infiltration facilities designed for water quality treatment must infiltrate 91 percent of the total runoff volume through soil meeting the treatment soils requirements outlined in *Volume 3, Section 4.5.2*. The procedure is the same as for designing infiltration for flow control, except that the target is to infiltrate 91 percent of the runoff file without overflow (refer to *Volume 3, Section 4.5.1*). In addition, to prevent the onset of anaerobic conditions, an infiltration facility designed for water quality treatment purposes must be designed to drain the water quality design volume within 48 hours. Drain time can be calculated by using a horizontal projection of the infiltration basin mid-depth dimensions and the design infiltration rate.

Stormwater Conveyance

Storms that produce the highest rates of runoff from developed areas are typically shorter in duration and are characterized by brief periods of high intensity rainfall. A 5-minute time step (refer to Table F.12) is required to adequately simulate the runoff peak discharge and hydrograph shape resulting from these high-intensity storms. A 15-minute time step may be used if the time of concentration computed is 30 minutes or more. Follow the modeling steps outlined in *Steps for Hydrologic Design Using Continuous Models*, and for conveyance-specific designs also perform the following:

Step #	Procedure
SC-1	Identify downstream hydraulic controls, such as outfalls (refer to Outfalls in *Section F-3*), known flooding locations, receiving pipe hydraulic grade line (HGL), pump station, regulator station, weirs, or orifices. Determine if backwater calculations or a dynamic hydraulic routing model are required.
SC-2	Analyze flood frequencies and select the flows representing the level of conveyance service and/or flood protection required.
SC-3	Utilize the peak flows to size or assess the capacity of pipe systems, culverts, channels, spillways and overflow structures.
SC-4	Perform a capacity analysis to verify that there is sufficient capacity in the public drainage system or the public combined sewer system. Refer to *Volume 3, Section 4.3* and SMC, Section 22.805.020.J for specific requirements.
SC-5	Size the pipe to convey the selected peak flows.

Using Continuous Simulation Hydrographs with Dynamic Routing Models

Continuous hydrologic models based on the HSPF program utilize hydrologic (also known as lumped) routing routines to determine the time and magnitude of flow of a watercourse. Because of this, these models cannot simulate complex hydraulics such as where the flow reverses direction or where a downstream channel or pipe influences another upstream in a time dependent way.

For simulation of complex hydraulics in pipe systems or tidally influenced boundaries, a dynamic routing hydraulic program, such as the SWMM Extran routine, may be necessary to accurately determine the discharge rate and the water surface elevation or hydraulic grade line (HGL). Flows simulated using the continuous hydrologic model may be exported and used as input to the dynamic routing hydraulic model.

Dynamic routing models solve the full unsteady flow equations using numeric approximation methods. These methods typically require computational time steps that are relatively short to maintain numerical stability, and it may not be practical to attempt routing of multi-year sequences of runoff produced by the continuous hydrologic model. To reduce the simulation time, flow hydrographs from specific storms of interest computed using the continuous flow model may be used rather than the entire simulated flow time series.

To utilize a dynamic routing model to route hydrographs computed with the continuous hydrologic model, the procedure described in the *Steps for Hydrologic Design Using Continuous Models* should be followed to create the runoff time series. The following additional steps should be followed to identify storms of a particular recurrence interval, export them from the continuous model, and import them into SWMM (or other dynamic routing model):

Step #	Procedure
DR-1	Delineate the watershed with subbasin outlets (runoff collection points) corresponding to the main inflows to the pipe system.
DR-2	Run the continuous hydrologic model for the full period of record. For most design applications, the City of Seattle Design Time Series should be used. The routing effects of the pipe or other conveyance system to be analyzed should not be included in the continuous hydrologic model.
DR-3	Use flood peak discharge statistics computed by the continuous model to identify when floods of various recurrence intervals occur in the simulated time series. Export hydrographs with peak discharge rates corresponding to desired recurrence intervals in a format that can be read by the hydraulic model.

For example, Table F.13 shows flood peak discharge-frequency results for a subbasin. If hydrographs corresponding to the 100-year, 25-year, and 10-year recurrence intervals were needed for conveyance design purposes, then simulated hydrographs with recurrence intervals closest to those required would be exported from the continuous hydrologic model as indicated in the right column of the table. The hydrograph duration would include a period antecedent to the flood peak (typically several days to a week) and several days following the flood peak.

F-5. Single-Event Rainfall–Runoff Methods

Single-event models simulate rainfall-runoff processes for a single-storm, typically 2 hours to 72 hours in length and usually of a specified exceedance probability. Because the primary interest is the flood hydrograph, calculation of evapotranspiration, soil moisture changes between storms, and base flow processes are typically not needed. This is in contrast to continuous rainfall-runoff models (*Section F-4*) where multi-decade precipitation and evaporation time series are used as input to produce a corresponding multi-decade time series of runoff.

Precipitation input to single-event models can include either historical data recorded from a rain gauge or a synthetic design storm hyetograph. This section describes the use of both types of precipitation input.

Design Storm Hyetographs

Design storm hyetographs were developed using noteworthy storms that were recorded by the City of Seattle gauging network. NOAA Atlas 2 precipitation-frequency (isopluvial) maps published in the early 1970s have historically been used in hydrologic analysis and design. These maps are replaced in this manual by precipitation magnitude-frequency estimates more specific to the City of Seattle. These estimates are based on a regional analysis using data from the SPU Rain Gauge Network and gauges from the NOAA national cooperative gaging network in western Washington. The most recent analysis included data from 1940 to 2003. Attachment 2 provides the precipitation data based on the SPU Rain Gauge Network.

Table F.13. Example Simulated Peak Discharge Frequency Table and Hydrographs Exported to SWMM or Other Hydraulic Model for Desired Recurrence Intervals.

Flood Peak Recurrence Interval (years)	Date of Peak[a]	Peak Discharge Rate (cfs)	Desired Recurrence Interval for Analysis
282	06/10/2010	7.62	
101	**11/04/1998**	**6.11**	**100-year**
62	06/29/1952	6.06	
44	02/03/2062	5.38	
35	07/18/2043	4.71	
28	10/06/1981	4.64	
24	**03/03/1950**	**4.54**	**25-year**
21	01/09/1990	4.40	
18	09/30/2011	4.40	
17	11/24/1990	4.27	
15	08/24/2077	4.25	
14	05/03/2002	4.25	
13	10/27/2054	4.15	
12	10/26/1986	4.03	
11	09/01/2061	3.93	
10	**01/20/2013**	**3.92**	**10-year**
9.6	08/23/1968	3.92	
9.0	01/14/2040	3.76	

[a] Simulation was performed using SPU Design Time Series, which is 158 years in length, and has dates spanning 10/1/1939 through 9/30/2097. (Note: This table may be revised in a future version of the 2021 Seattle Stormwater Manual.)

Statistical analyses were conducted for the storm characteristics and dimensionless design storms were developed for short, intermediate, and long duration storm events (Schaefer 2004). The short, intermediate, and long duration design storms can be scaled to any site-specific recurrence interval using precipitation magnitudes at the 2-hour, 6-hour, and 24-hour duration.

Table F.14 summarizes the applicability of the four City design storms. If multiple storm types are listed for a particular application, then all applicable storm types should be considered candidates and used in the hydrologic model. The candidate storm that produces the most severe hydrologic loading and most conservative design is then adopted as the design storm. Note that this table does not override the modeling requirements for specific facilities outlined in *Volume 3*, *Chapters 4* and *5*, or Table F.1. Table F.14 is for general guidance and applicability only.

Table F.14. Applicability of Storm Types for Hydrologic Design Applications.

Storm Type	Description	Applicability	Total Storm Duration	Precipitation from SPU Rain Gauges
Short-duration	• Typically occurs in late spring through early fall • High intensity • Limited volume	• Conveyance (storm drains, ditches, culverts, and other hydraulic structures) • Flow Control	3 hours	2 hours
Intermediate Duration	• Typically occurs in fall through early winter • Low intensity • High volume	• Conveyance (storm drains, ditches, culverts, and other hydraulic structures) • Flow Control	18 hours	6 hours
Seattle 24-hour	NA	Volume Based BMPs	24 hours	24 hours
Long-duration – Front and Back Loaded	• Typically occurs in late fall through early spring • Low intensity • High volume	Flow Control	64 hours	24 hours

NA – not applicable

Short-Duration Storm (3-hour)

Short-duration design storms are used for design situations where peak discharge is of primary interest. The storm temporal pattern is shown in Figure F.12 as a dimensionless unit hyetograph. Tabular values for this hyetograph are listed in Attachment 1. The total storm precipitation is 1.06 times the 2-hour precipitation amount.

Use the following steps to utilize the short-duration storm in hydrologic analyses.

Step #	Procedure
SD-1	Obtain the 2-hour precipitation amount for the recurrence interval of interest (refer to Table 2 in Attachment 2). Note that the 2-hour precipitation values for short-duration storms do not vary across the City.
SD-2	Multiply the 5-minute incremental ordinates of the dimensionless short-duration design storm (Attachment 1, Table 1) by the 2-hour value from Step SD-1. Note that the resulting storm has a duration of 3 hours and the total storm amount will be 1.06 times the volume of the 2-hour precipitation (refer to the SDCI SPU Stormwater webpage for modeling resources).
SD-3	Input the resulting storm hyetograph into the hydrologic model. The resultant incremental precipitation ordinates have units of inches. To obtain the corresponding intensities (in/hr), multiply the precipitation increments by 12.

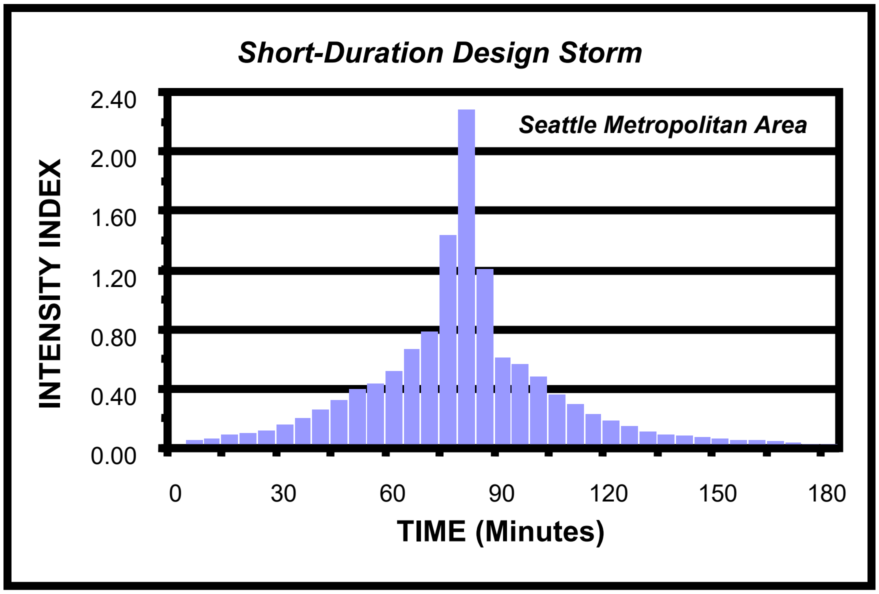


Figure F.12. Dimensionless Short-Duration (3-hour) Design Storm, Seattle Metropolitan Area.

Intermediate-Duration Storm (18 hour)

Intermediate-duration design storms are used in design applications where both peak discharge and runoff volume are important considerations and there is a need for a runoff hydrograph. The storm temporal pattern is shown in Figure F.13 as a dimensionless unit hyetograph. Tabular values for this hyetograph are listed in Attachment 1. The total storm precipitation is 1.51 times the 6-hour precipitation amount.

The following steps describe how to utilize the intermediate-duration storm in hydrologic analyses.

Step #	Procedure
ID-1	Obtain the 6-hour precipitation amount for the recurrence interval of interest for the watershed (refer to Attachment 2 for data from the SPU Gauge(s) of interest).
ID-2	Multiply the 10-minute incremental ordinates of the dimensionless intermediate-duration and long-duration design storms (Attachment 1, Tables 2 and 4) by the 6-hour value from Step ID-1. Note that the resulting storm has a duration of 18 hours and the total storm amount will be 1.51 times the volume of the 6-hour precipitation (refer to the SDCI SPU Stormwater webpage for modeling resources).
ID-3	Input the resulting storm hyetograph into the hydrologic model. The resultant incremental precipitation ordinates have units of inches. To obtain the corresponding intensities (in/hr), multiply the precipitation increments by 6.

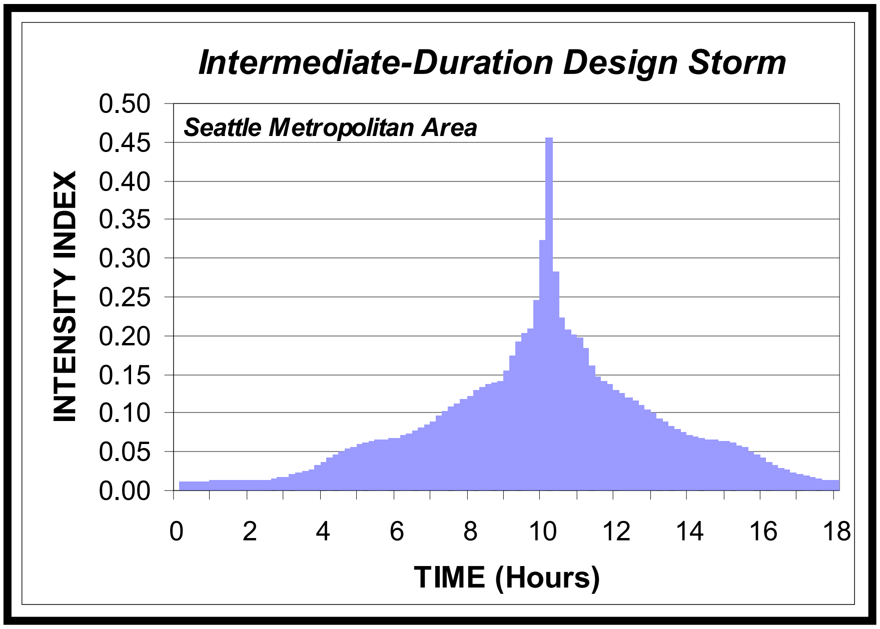


Figure F.13. Dimensionless Intermediate-Duration (18-hour) Design Storm, Seattle Metropolitan Area.

24-Hour Dimensionless Design Storm

Some specific volume-based stormwater facilities require or allow the use of a 24-hour design storm. To meet this need, the 24-hour dimensionless design storm was developed based on the maximum 24-hour period of precipitation within the long-duration design storm. It should be noted that the 24-hour dimensionless design storm has the same temporal shape and ordinates as the period of maximum 24-hour precipitation within the front-loaded and back-loaded long-duration dimensionless design storms. The City of Seattle 24-hour design storm is shown in Figure F.14.

Use the following steps to utilize the 24-hour design storm in hydrologic analyses:

Step #	Procedure
DD-1	Obtain the 24-hour precipitation amount for the recurrence interval of interest for the watershed (refer to Attachment 2 for data from the SPU Gauge(s) of interest).
DD-2	Multiply the 10-minute incremental ordinates of the dimensionless 24-hour duration design storm (Attachment 1, Table 5) by the 24-hour value from Step DD-1 (refer to the SDCI SPU Stormwater webpage for modeling resources).
DD-3	Input the resulting storm hyetograph into the hydrologic model. The resultant incremental precipitation ordinates have units of inches. To obtain the corresponding intensities (in/hr), multiply the precipitation increments by 6.

Long-Duration Storm (64 hour)

Long-duration design storms are primarily used in design of stormwater detention facilities and other projects where runoff volume is a primary consideration. Long-duration storms occur primarily in the late fall into early spring.

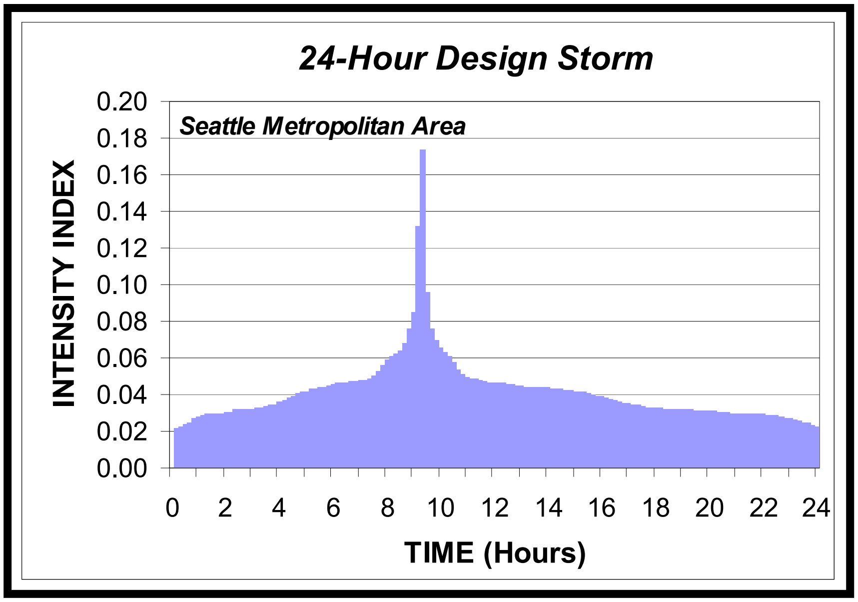


Figure F.14. Dimensionless 24-Hour Design Storm for Seattle Metropolitan Area.

Two long-duration dimensionless design storms are provided: a front-loaded design storm with the highest intensities at the beginning of the storm; and a back-loaded storm with the higher intensities nearer the end of the storm period. Characteristics of the front-loaded design storm have been observed more frequently, and this storm would be expected to produce more "typical" runoff conditions. The back-loaded storm occurs less often and is typically a more conservative event for drainage control facility design.

The long-duration storm hyetographs are 64 hours in duration. The storm temporal patterns for the front loaded and back loaded storms are shown in Figures F.15 and F.16 respectively. Tabular values for these storms are listed in Attachment 1. The total storm precipitation is 1.29 times the 24-hour precipitation amount for both the front and back loaded long-duration storm.

Use the following steps to utilize the long-duration storm in hydrologic analyses.

Step #	Procedure
LD-1	Obtain the 24-hour precipitation amount for the recurrence interval of interest for the watershed (refer to Attachment 2 for data from the SPU Gauge(s) of interest).
LD-2	Multiply the 10-minute incremental ordinates of the dimensionless long-duration design storm (Attachment 1, Table 3 or 4) by the 24-hour value from Step LD-1. Note that the resulting storm has a duration of 64 hours and the total storm amount will be 1.29 times the volume of the 6-hour precipitation (refer to the SDCI SPU Stormwater webpage for modeling resources).
LD-3	Input the resulting storm hyetograph into the hydrologic model. The resultant incremental precipitation ordinates have units of inches. To obtain the corresponding intensities (in/hr), multiply the precipitation increments by 6.

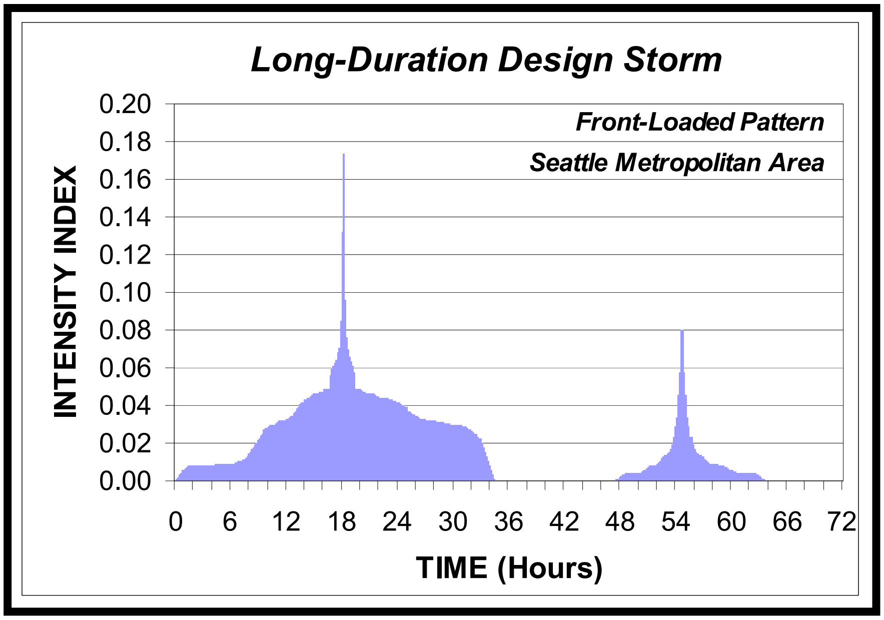


Figure F.15. Dimensionless Front-Loaded Long-Duration (64-hour) Design Storm for the Seattle Metropolitan Area.

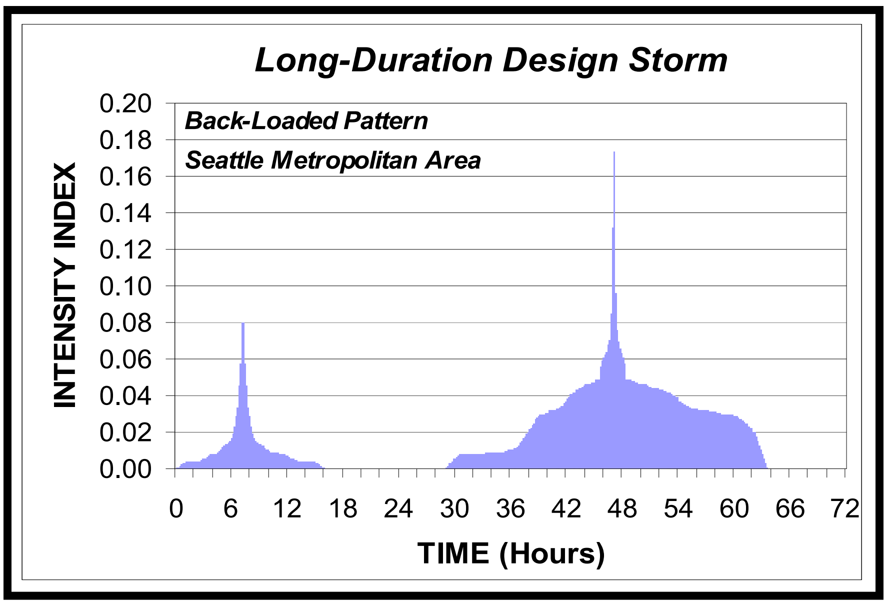


Figure F.16. Dimensionless Back-Loaded Long-Duration (64-hour) Design Storm for the Seattle Metropolitan Area.

Curve Number Equation and Infiltration Parameters

The Curve Number method may be used when computing runoff using the Long-duration storms (24 hours or 66 hours in length). The NRCS developed relationships between land use, soil type, vegetation cover, interception, infiltration, surface storage, and runoff. These relationships have been characterized by a single runoff coefficient called a "curve number" (CN). The National Engineering Handbook — Part 630: Hydrology (NRCS 1997) contains a detailed description of the development and use of the curve number method.

The CN is related to the runoff potential of a watershed according to equations (12) and (13).

$$Q_d = \frac{(P - 0.2\,SMD_{MAX})^2}{(P + 0.8\,SMD_{MAX})} \tag{12}$$

$$SMD_{MAX} = \frac{1000}{CN} - 10 \tag{13}$$

Where:
Q_d = runoff depth (inches)
P = precipitation depth (inches)
SMD_{MAX} = maximum soil moisture deficit (inches)
CN = Curve Number for the soil (Table F.15)

The CN is a combination of a hydrologic soil group and land cover with higher CNs resulting in higher runoff. CN values for combinations of land cover and hydrologic soil group are listed in Table F.15. Refer to Table F.3 in General Modeling Guidance (*Section F-3*) for information on soil groups in King County.

Table F.15. Post-Development Runoff Curve Numbers for Selected Agricultural, Suburban, and Urban Areas.

Land Use Description	Curve Numbers by Hydrologic Soil Group			
Land Cover Type and Hydrologic Condition	**A**	**B**	**C**	**D**
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 Condition (woods are grazed but not burned, and some forest litter covers the soil)	36	60	73	79
Good Condition (woods are protected from grazing, and litter and brush adequately cover the soil)	30	55	70	77
Open space (Lawns, parks, golf courses, cemeteries, landscaping, etc.)				
Fair Condition (grass cover on 50 to 75% of the area)	77	85	90	92
Good Condition (grass cover on greater than 75% of the area)	68	80	86	90

Table F.15 (continued). Post-Development Runoff Curve Numbers for Selected Agricultural, Suburban, and Urban Areas.

Land Use Description	Curve Numbers by Hydrologic Soil Group			
Land Cover Type and Hydrologic Condition	**A**	**B**	**C**	**D**
Impervious areas				
Open receiving waters (lakes, wetlands, ponds, etc.)	100	100	100	100
Paved surfaces (roads, roofs, driveways, etc.)	98	98	98	98
Gravel roads and parking lots	76	85	89	91
Dirt roads and parking lots	72	82	87	89
Permeable pavement				
Porous asphalt, porous concrete, or grid/lattice systems (without underlying perforated drain pipes to collect stormwater)	77	85	90	92
Paving blocks (without underlying perforated drain pipes to collect stormwater)	87	91	94	96
All permeable pavement types (with underlying perforated drain pipes to collect stormwater)	98	98	98	98

Time of Concentration Estimation

The time of concentration for the various surfaces and conveyances should be computed using the following methods, which are based on Chapter 3 of TR-55 (NRCS 1986).

Travel time (T_t) is the time it takes water to travel from one location to another in a watershed. T_t is a component of time of concentration (T_c), which is the time for runoff to travel from the hydraulically most distant point of the watershed. T_c is computed by summing all the travel times for consecutive components of the drainage conveyance system.

Water is assumed to move through a watershed as sheet flow, shallow concentrated flow, open channel flow, or some combination of these. The type that occurs is best determined by field inspection. The time of concentration (T_c) is the sum of T_t values for the various consecutive flow segments.

$$T_c = T_1 + T_2 + T_3 + \ldots T_n \qquad (14)$$

Where:
- T_c = time of concentration (minutes)
- $T_{1,2,3,n}$ = time for consecutive flow path segments with different land cover categories or flow path slope

Travel time for each segment is computed using the following equation:

$$T_t = \frac{L}{60V} \qquad (15)$$

Where:
- T_t = travel time (minutes)
- L = length of flow across a given segment (feet)
- V = average velocity across the land segment (ft/sec)

Sheet Flow: Sheet flow is flow over plane surfaces. Sheet flow travel time is computed using equation (16). This equation is applicable for relatively impervious areas with shallow flow depths up to about 0.1 foot and for travel lengths up to 300 feet. Modified Manning's effective roughness coefficients (n_s) are summarized in Table F.16. These n_s values are applicable for shallow flow depths up to about 0.1 foot and for travel lengths up to 300 feet.

$$T_t = 0.42 * (n_s * L)^{0.8} / ((P_{24})^{0.5} * (S_o)^{0.4}) \quad (16)$$

Where:
- T_t = travel time (minutes)
- n_s = sheet flow Manning's effective roughness coefficient from Table F.16
- L = overland flow length (feet)
- P_{24} = 2-year, 24-hour rainfall (inches)
- S_o = slope of hydraulic grade line or land slope (feet/feet)

Shallow Concentrated Flow: After a maximum of 300 feet, sheet flow is assumed to become shallow concentrated flow. The average velocity for this flow can be calculated using the ks values from Table F.16 in which average velocity is a function of watercourse slope and type of channel. After computing the average velocity using the velocity equation (17), the travel time (T_t) for the shallow concentrated flow segment can be computed using the travel time equation (15).

Velocity Equation: A commonly used method of computing average velocity of flow, once it has measurable depth, is the following equation:

$$V = k_s \sqrt{S_o} \quad (17)$$

Where:
- k_s = velocity factor (Table F.16)
- S_0 = slope of flow path (feet/feet)

"k" values in Table F.16 have been computed for various land covers and channel characteristics with assumptions made for hydraulic radius using the following rearrangement of Manning's equation:

$$k = (1.49 (R) 0.667)/n \quad (18)$$

Where:
- R = assumed hydraulic radius
- n = Manning's roughness coefficient for open channel flow, from Tables F.16 or F.17

Open Channel Flow: Open channels are assumed to begin where flow enters ditches or pipes, where surveyed cross section information has been obtained, where channels are visible on aerial photographs, or where lines indicating streams appear (in blue) on USGS quadrangle sheets. The k_c values from Table F.16 used in velocity equation (17) or water surface profile information can be used to estimate average flow velocity. Average flow velocity is usually determined for bank-full conditions. The travel time (T_t) for the channel segment can be computed using travel time equation (15).

Lakes or Wetlands: Sometimes it is necessary to estimate the velocity of flow through a lake or wetland at the outlet of a watershed. This travel time is normally very small and can be assumed as zero. Where significant attenuation may occur due to storage effects, the flows should be routed using the "level-pool routing" technique described in the *Level-Pool Routing Method* section.

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 drainage systems, carefully identify the appropriate hydraulic flow path to estimate T_c. Drainage systems generally handle only a small portion of a large event. The rest of the peak flow travels by streets, lawns, and other surfaces, to the outlet. Consult a standard hydraulics textbook (e.g., Gray 1961; Linsley et al. 1975; Pilgrim and Cordery 1993; Viessman et al. 1977) to determine average velocity in pipes for either pressure or non-pressure flow.
- A culvert or bridge can act as a reservoir outlet if there is significant storage behind it. A hydrograph should be developed to this point and the "level pool routing" technique should be used to determine the outflow rating curve through the culvert or bridge.

Table F.16. Values of "n" and "k" for use in Computing Time of Concentration.

FOR SHEET FLOW	n_s
Smoot surfaces (concrete, asphalt, gravel, or bare hard soil)	0.011
Fallow fields of loose soil surface (no vegetal residue)	0.05
Cultivated soil with crop residue (slope < 0.20 ft/ft)	0.06
Cultivated soil with crop residue (slope > 0.20 ft/ft)	0.17
Short prairie grass and lawns	0.15
Dense grass	0.24
Bermuda grass	0.41
Range, natural	0.13
Woods or forest, poor cover	0.40
Woods or forest, good cover	0.80
FOR SHALLOW, CONCENTRATED FLOW	k_s
Forest with heavy ground litter and meadows (n = 0.10)	3
Brushy ground with some trees (n = 0.06)	5
Fallow or minimum tillage cultivation (n = 0.04)	8
High grass (n = 0.035)	9
Short grass, pasture and lawns (n = 0.04)	11
Newly-bare ground (n = 0.025)	13
Paved and gravel areas (n = 0.012)	27
CHANNEL FLOW (INTERMITTENT, R = 0.2)	k_c
Forested swale with heavy ground litter (n = 0.10)	5
Forested drainage course/ravine with defined channel bed (n = 0.050)	10
Rock-lined waterway (n = 0.035)	15
Grassed waterway (n = 0.030)	17
Earth-lined waterway (n = 0.025)	20
CMP pipe (n = 0.024)	21
Concrete pipe (n = 0.012)	42
Other waterways and pipes	0.508/n
CHANNEL FLOW (CONTINUOUS STREAM, R = 0.4)	k_c
Meandering stream with some pools (n = 0.040)	20
Rock-lined stream (n = 0.035)	23
Grassed stream (n = 0.030)	27
Other streams, man-made channels and pipe	0.807/n

Source: USDA (1986).

Table F.17. Other Values of the Roughness Coefficient "n" for Channel Flow.

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

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

Single-Event Routing Methods Overview

In the United States, the majority of single-event models for computation of runoff hydrographs are based on unit hydrographs. Most commercial software packages utilize unit hydrographs for making the transformation from computation of runoff volume to generation of the runoff hydrograph. This may require direct input of the ordinates of the unit hydrograph or the unit hydrograph may be computed internally based on watershed characteristics provided by the user. Notable exceptions include event-based models that utilize linear reservoir concepts, such as the Santa Barbara Urban Hydrograph model (SBUH), event-based models that utilize kinematic wave approaches, and continuous flow simulation models such as HSPF.

The *Unit Hydrograph Routing Methods* section describes rainfall-runoff modeling based on unit hydrograph concepts. The reader is referred to any standard hydrology textbook (e.g., Gray 1961; Linsley et al. 1975; Pilgrim and Cordery 1993; Viessman et al. 1977) for a detailed discussion of unit hydrograph theory. The *SBUH Routing Method* section includes a discussion of runoff hydrographs developed using the SBUH model. The *Level-Pool Routing Method* section provides a discussion on the level-pool method, which is appropriate for routing hydrographs through lakes, wetlands, and other areas of standing water.

Unit Hydrograph Routing Methods

The unit hydrograph is defined as the time-distribution of runoff (Figure F.17) measured at the watershed outlet as produced by 1 inch of runoff uniformly generated over the watershed during a specified period of time. Thus, a 10-minute unit hydrograph would be the runoff hydrograph (cfs) observed at the watershed outlet as generated by 1 inch of runoff uniformly produced over the watershed in a 10-minute period.

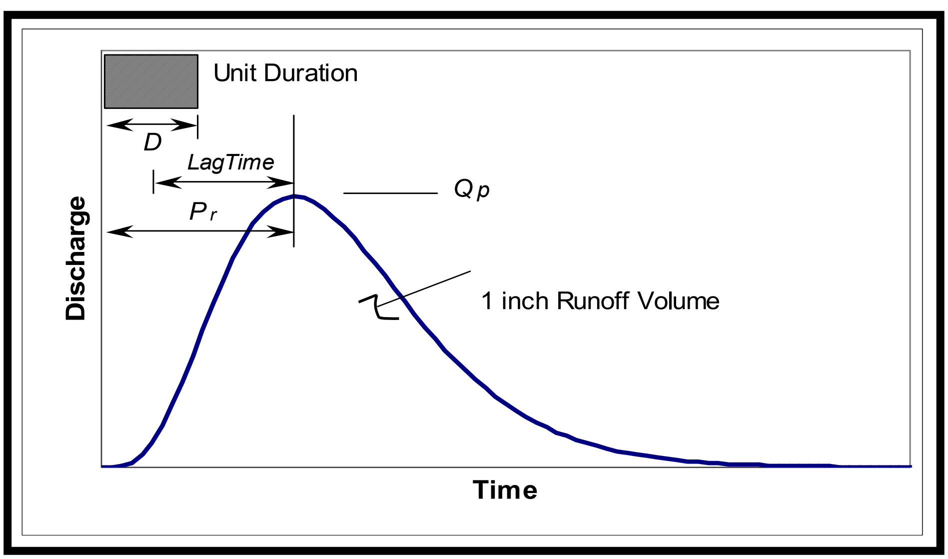


Figure F.17. Characteristics of Unit Hydrographs.

In computation of the runoff hydrograph, the unit hydrograph is scaled by the runoff in each D-minute period, and the resultant hydrographs for each D-minute period are added by superposition to yield the runoff hydrograph from the watershed.

Relationship of Computational Time Step to Time Lag (Lag Time). As indicated above, the ordinates of the unit hydrograph are specified on intervals equal to the computational time step. Recognizing that the time step and unit duration are equal ($\Delta t = D$), the unit duration must be chosen small enough to allow reasonable definition of the rising limb of the unit hydrograph. This is required to provide for adequate definition of the resultant runoff hydrograph in the vicinity of the runoff peak discharge. In addition, the value of D should be an integer multiple of the period of rise P_r so that the computational time step (Δt) falls on the peak discharge of the unit hydrograph.

Selection of Time Step (Δt) Based on Time of Concentration (T_c). The time-of concentration of the watershed (T_c) is often taken to be the elapsed time from the end of the unit duration (D) to the inflection point on the recession limb of the unit hydrograph (NRCS 1997). When the runoff hydrograph is computed based on unit hydrograph concepts utilizing time of concentration, the computational time step should be:

$$\Delta t < T_c/5 \tag{19}$$

To enhance compatibility with the City of Seattle design storms, the computational time step for runoff computations should be a multiple of the time step used to describe the design storm. The short-duration design storm is described in 5-minute intervals and the intermediate and long-duration design storms are described in 10-minute intervals. Therefore, the following additional criteria are required for selection of the time step for use with the short-duration design storm:

$$\Delta t = 5/n \tag{20}$$

And, for use with the intermediate and long-duration design storms:

$$\Delta t = 10/n \tag{21}$$

Where: n = integer greater than or equal to one

The above information should be particularly helpful for use with computer software that allows output of the runoff hydrograph on a time interval other than that used for internal computation of the runoff hydrograph. For those cases, the user may be unaware of the unit duration (D) and internal time step (Δt) being used by the computer program.

SBUH Routing Method

The SBUH method is an adaptation of standard hydrologic routing methods that employ the principle of conservation of mass. The routing equation for the SBUH method may be derived from linear reservoir concepts (Linsley et al. 1975; Fread 1993) where storage is taken to be a linear function of discharge.

The SBUH method uses two steps to synthesize the runoff hydrograph:

Step 1 — Compute the instantaneous hydrograph
Step 2 — Compute the runoff hydrograph

The instantaneous hydrograph is computed as follows:

$$l(t) = 60.5\ R(t)\ A/\Delta t \qquad (22)$$

Where:
- $l(t)$ = instantaneous hydrograph at each time step (Δt) (cfs)
- $R(t)$ = total runoff depth (both impervious and pervious) at time increment Δt (inches)
- A = area (acres)
- Δt = computational time step (minutes)

The runoff hydrograph is then obtained by routing the instantaneous hydrograph through an imaginary reservoir with a time delay equal to the time of concentration of the drainage basin. The following equation estimates the routed flow:

$$Q(t+1) = Q(t) + w[l(t) + l(t+1) - 2Q(t)] \qquad (23)$$

$$w = \Delta t\ /(2T_c + \Delta t) \qquad (24)$$

Where:
- $Q(t)$ = runoff hydrograph or routed flow (cfs)
- T_c = time of concentration (minutes)
- Δt = computational time step (minutes)

Selection of Time Step (Δt) Based on Time of Concentration (T_c). Equation (23) requires that the computational time step be sufficiently short that the change in inflow, outflow, and storage during the time step can be treated as linear. For the case of very small urban watersheds, the low to moderate intensities in the long-duration design storm would typically generate runoff over a longer period than the time of concentration of the watershed. As a result, the elapsed time of the rising limb of the runoff hydrograph (T_r) would likewise be much longer than the time of concentration of the watershed. In addition, the computational time step for routing should be a multiple of the time step used to describe the design storm. Therefore, for intermediate and long-duration storms, the computational time step should satisfy equations (25) and (26):

$$\Delta t < T_c \qquad (25)$$

$$\Delta t = 10/n \qquad (26)$$

Where:
- Δt = computational time step (minutes)
- T_c = time of concentration (minutes)
- n = an integer greater than or equal to one

For short duration design storms, the flood peak of the runoff hydrograph may be quite flashy and produced by high-intensity precipitation during a limited portion of the storm. For this case, the elapsed time for the rising limb of the runoff hydrograph may be similar in magnitude to that of the time-of-concentration of the watershed. In this situation, the time step should be smaller than the time of concentration. In addition, the computational time step for routing should be a multiple of the time step used to describe the design storm. Therefore, for the short-duration storm, the computational time step should satisfy equations (27) and (28):

$$\Delta t < T_c/5 \tag{27}$$

$$\Delta t = 5/n \tag{28}$$

Where:

Δt = computational time step (minutes)
T_c = time of concentration (minutes)
n = an integer greater than or equal to one

Level-Pool Routing Method

This section presents a general description of the methodology for routing a hydrograph through a retention/detention facility, closed depression, or wetland. Note that the City does not allow the use of single-event models for retention/detention facility design. The information presented in this section is for informational purposes only. The level pool routing technique (Fread 1993) is based on the continuity equation:

Inflow-outflow=change in storage

$$\left[\frac{I_1+I_2}{2}-\frac{O_1+O_2}{2}\right]=\frac{\Delta S}{\Delta t}=S_2-S_1 \tag{29}$$

rearranging:

$$I_1 + I_2 + 2S_1 - O_1 = O_2 + 2S_2 \tag{30}$$

Where:

I = inflow at time 1 and time 2
O = outflow at time 1 and time 2
S = storage at time 1 and time 2
Δt = computational time step (minutes)

The time step (Δt) must be consistent with the time interval used in developing the inflow hydrograph.

The following summarizes the steps required in performing level-pool hydrograph routing:

- Develop stage-storage-discharge relationship, which is a function of pond/wetland geometry and outflow
- Route the inflow hydrograph through the structure by applying equation (30) at each time step, where the inflow hydrograph supplies values of I, the stage-storage relationship supplies values of S, and the stage discharge relationship provides values of O.

Commercially available hydrologic computer models perform these calculations automatically.

Modeling Guidance

The following sections present the general process involved in conducting a hydrologic analysis using single-event hydrograph methods to evaluate or design stormwater conveyance systems. Applicability of single-event methods and design standard requirements are discussed in *Section F-2*.

Steps for Hydrologic Design Using Single-Event Methods

The following summarizes the process for conducting hydrologic analyses using single-event models.

Step #	Procedure
SE-1	Review all minimum requirements that apply to the proposed project (*Volume 1*)
SE-2	Review applicable site definition and mapping requirements (*Volume 1*)
SE-3	Identify and delineate the overall drainage basin for each discharge point from the development site under existing conditions: ● Identify existing land use ● Identify existing soil types using on-site evaluation, NRCS soil survey, or mapping performed by the University of Washington (http://geomapnw.ess.washington.edu) ● Identify existing drainage features such as streams, conveyance systems, detention facilities, ponding areas, depressions, etc.
SE-4	Select and delineate pertinent subbasins based on existing conditions: ● Select homogeneous subbasin areas ● Select separate subbasin areas for on-site and off-site drainage ● Select separate subbasin areas for major drainage features.

Stormwater Conveyance

Existing and proposed stormwater conveyance facilities may be analyzed and designed using peak flows from hydrographs derived from single-event approaches described in this appendix. In addition to the steps listed in the *Steps for Hydrologic Design Using Single-event Methods* section, the following steps should be followed for designing/analyzing conveyance facilities:

Step #	Procedure
SC-1	Determine runoff parameters for each subbasin
SC-2	Identify pervious and impervious areas ● The short- or intermediate-duration design storm generally governs the design of conveyance facilities. Both storm durations should be treated as candidate design storms and the one that produces the more conservative design (higher peak discharge rates) used as the design storm (refer to Design Storm Hyetograph section). ● Select runoff parameters per the Infiltration Equation section. ● Compute time of concentration per the Time of Concentration Estimation section.
SC-3	Identify downstream hydraulic controls, such as outfalls (refer to Outfalls in *Section F-3*), known flooding locations, receiving pipe HGL, pump station, regulator station, weirs or orifice. Determine if backwater calculations or a dynamic hydraulic routing model are required.

Step #	Procedure
SC-4	Compute runoff for the drainage system and determine peak discharge at the outlet of each subbasin for the design storm of interest
SC-5	Perform a capacity analysis to verify that there is sufficient capacity in the public drainage system or the public combined sewer system. Refer to *Volume 3, Section 4.3* and SMC, Section 22.805.020.J for specific requirements.
SC-6	Size the pipe based on the designated level of service.

F-6. Rational Method

The rational method is based on the assumption that rainfall intensity for any given duration is uniform over the entire tributary watershed. The rational formula relates peak discharge from the site of interest to rainfall intensity times a coefficient:

$$Q = CiA \tag{31}$$

Where:
- Q = peak discharge from the site of interest
- C = dimensionless runoff coefficient
- i = rainfall intensity for a given recurrence interval (in/hr)
- A = tributary area (acres)

The rainfall intensity (i) is determined from Figure F.18 or Table F.18 for the precipitation recurrence interval of interest and duration corresponding to the calculated time of concentration (refer to *Time of Concentration Estimation* section below).

Peak Rainfall Intensity Duration Frequency (IDF curves)

Rainfall intensity-duration-frequency (IDF) curves allow calculation of average design rainfall intensity for a given exceedance probability (recurrence interval) over a range of durations. Precipitation-frequency statistics presented in this appendix were analyzed using data from the City's 17-gauge precipitation measurement network within the City of Seattle, and the national NOAA cooperative gauge network 13. Durations of 5 minutes, 10 minutes, 15 minutes, 20 minutes, 30 minutes, 45 minutes, 60 minutes, 2 hours, 3 hours, 6 hours, 12 hours, 24 hours, 48 hours, 72 hours, and 7 days were analyzed to develop the IDF curves. IDF curves for storm durations up to 3 hours and applicable to sites within Seattle are shown in Figure F.21.

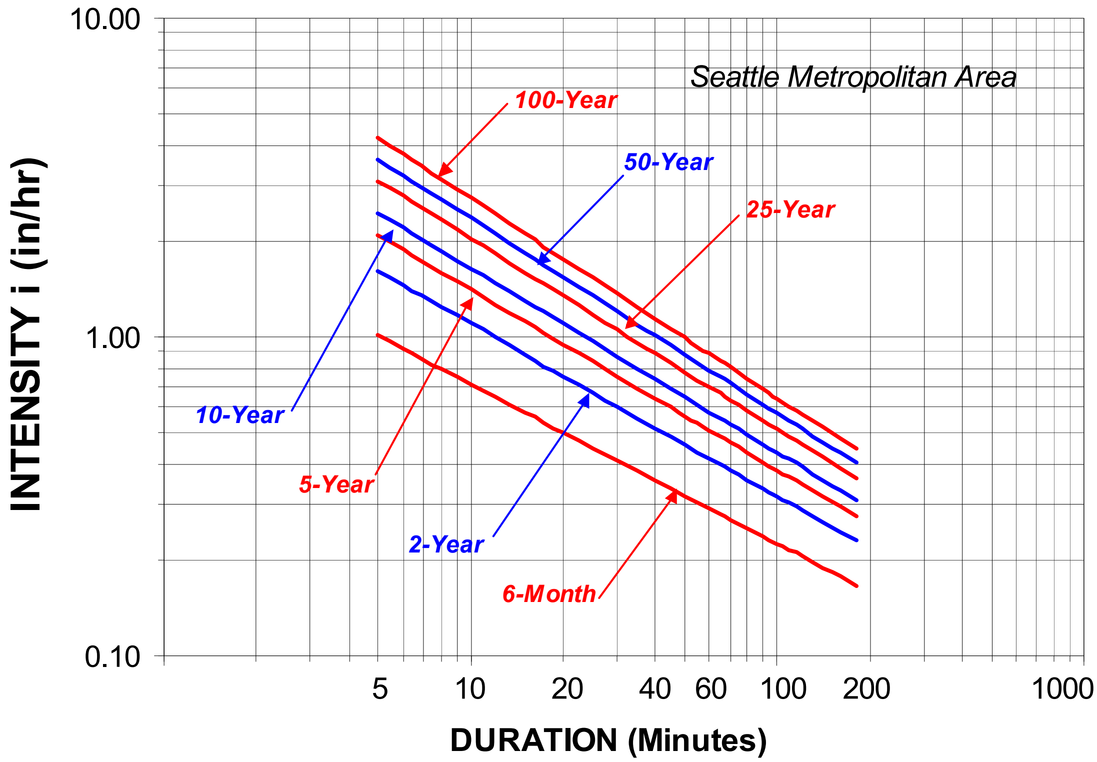


Figure F.18. Intensity-Duration-Frequency Curves for the City of Seattle.

Table F.18. Intensity-Duration-Frequency Values for 5- to 180-Minute Durations for Selected Recurrence Intervals for the City of Seattle.

Duration (minutes)	Precipitation Intensities (in/hr)							
	Recurrence Interval (years)							
	6-mo	2-yr	5-yr	10-yr	20-yr	25-yr	50-yr	100-yr
5	1.01	1.60	2.08	2.45	2.92	3.08	3.61	4.20
6	0.92	1.45	1.87	2.21	2.62	2.76	3.23	3.75
8	0.80	1.24	1.59	1.87	2.21	2.32	2.71	3.13
10	0.71	1.10	1.40	1.64	1.93	2.03	2.36	2.72
12	0.65	1.00	1.27	1.48	1.74	1.82	2.11	2.43
15	0.58	0.88	1.12	1.30	1.52	1.60	1.84	2.11
20	0.50	0.75	0.95	1.10	1.28	1.34	1.54	1.76
25	0.45	0.67	0.84	0.97	1.12	1.18	1.35	1.53
30	0.41	0.61	0.76	0.87	1.01	1.05	1.21	1.37
35	0.38	0.56	0.69	0.80	0.92	0.96	1.10	1.24
40	0.35	0.52	0.64	0.74	0.85	0.89	1.01	1.14
45	0.33	0.49	0.60	0.69	0.79	0.83	0.94	1.06
50	0.32	0.46	0.57	0.65	0.74	0.78	0.88	0.99
55	0.30	0.44	0.54	0.61	0.70	0.73	0.83	0.94
60	0.29	0.42	0.51	0.58	0.67	0.70	0.79	0.89
65	0.28	0.40	0.49	0.56	0.64	0.66	0.75	0.84

Table F.18 (continued). Intensity-Duration-Frequency Values for 5- to 180-minute Durations for Selected Recurrence Intervals for the City of Seattle.

Duration (minutes)	Precipitation Intensities (in/hr) Recurrence Interval (years)							
	6-mo	2-yr	5-yr	10-yr	20-yr	25-yr	50-yr	100-yr
70	0.27	0.38	0.47	0.53	0.61	0.64	0.72	0.80
80	0.25	0.36	0.43	0.49	0.56	0.59	0.66	0.74
90	0.24	0.33	0.41	0.46	0.52	0.55	0.62	0.69
100	0.22	0.32	0.38	0.43	0.49	0.51	0.58	0.64
120	0.20	0.29	0.35	0.39	0.44	0.46	0.52	0.57
140	0.19	0.26	0.32	0.36	0.40	0.42	0.47	0.52
160	0.18	0.24	0.29	0.33	0.37	0.39	0.43	0.48
180	0.17	0.23	0.27	0.31	0.35	0.36	0.40	0.45

Runoff Coefficients

Runoff coefficients vary with the tributary land cover and to a certain extent, the total depth and intensity of the rainfall. The storm depth and intensity is typically neglected, and the runoff coefficient is based on land cover only (Table F.19). For watersheds containing several land cover types, an aggregate runoff coefficient can be developed by computing the area weighted average from all cover types present (equation 32):

$$C_c = (C_1A_1 + C_2A_2 + C_3A_3 + \ldots + C_nA_n)/A_t \qquad (32)$$

Where:
- C_c = composite runoff coefficient for the site
- $C_{1,2,,\ldots n}$ = runoff coefficient for each land cover type
- $A_{1,2,,\ldots n}$ = area of each land cover type (acres)
- A_t = total tributary area (acres)

Table F.19. Rational Equation Runoff Coefficients.

Land Cover	Runoff Coefficient (C)
Dense Forest	0.10
Light Forest	0.15
Pasture	0.20
Lawns	0.25
Gravel Areas	0.80
Pavement and Roofs	0.90
Open Water (Ponds Lakes and Wetlands)	1.00

Time of Concentration Estimation

Time of concentration (T_c) is defined as the time it takes for runoff to travel from the most hydraulically distant point of the drainage area to the outlet. T_c is computed by summing all the travel times for consecutive components of the drainage conveyance system.

$$T_c = T_1 + T_2 + T_3 + \ldots T_n \tag{33}$$

Where:
- T_c = time of concentration (minutes)
- $T_{1,2,3,\ldots n}$ = time for consecutive flow path segments with different land cover categories or flow path slope

Travel time for each segment is computed using the following equation:

$$T_t = L / V$$

Where:
- T_t = travel time (minutes)
- L = length of flow across a given segment (feet)
- V = average velocity across the land segment (ft/sec)

$$V = k_r \sqrt{S_o} \tag{34}$$

Where:
- k_r = Velocity factor (Table F.20)
- S_0 = Slope of flow path (feet/feet)

Table F.20. Coefficients for Average Velocity Equation.

Land Cover	Velocity Factor (k_r)
Forest with Heavy Ground Cover and Meadow	2.5
Grass, Pasture, and Lawns	7.0
Nearly Bare Ground	10.1
Grassed Swale or Channel	15.0
Paved Areas	20.0

F-7. Risk-Based Hydrologic Design Concepts

Risk-based concepts and analytical approaches are being used more frequently in hydrologic design. A risk-based approach focuses on evaluating the two components of risk: the probability, and consequences of failure. Failure may be broadly defined and includes failure to meet a project goal, failure to meet a regulatory requirement, or the physical failure of a project element. Consequences of failure vary with the project type and features and may include economic, life safety, environmental, and political consequences.

Risk can be described qualitatively or quantitatively. For example, qualitative risk is often expressed as low, moderate, high, or very high, based on various combinations of the probability of failure and the consequences of failure. Quantitative risk assessment requires more detailed analysis to provide numerical measures of the probability of failure and consequences of failure. Quantitative units of measure for risk include loss of life per year for life safety risk, and dollars per year for consequences that can be expressed in economic terms.

Risk concepts are often used in design where the design target, level-of-service, etc., is based on the consequences of failure or upon some adopted level of qualitative or quantitative risk. The design targets and level of conservatism of design are typically set based on the tolerable level of risk for a given project type or consideration of the regulatory requirements.

When applying a risk-based approach, engineers and hydrologists primarily evaluate the probability of failure (or probability of being in compliance) and may assess how and which uncertainties affect the probability of failure (or probability of being in compliance). Application of hydrologic computer models and detailed numerical descriptions of hydrologic/hydraulic system components are an integral part of assessing the probability of being in compliance.

Uncertainty

Historically, uncertainty in hydrologic simulation analyses and the consequences for analysis results are rarely quantified as part of stormwater engineering design. Factors of safety have typically been applied at the end of a hydrologic analysis to account for uncertainties in the analysis. The same factor of safety is typically used regardless of the level of uncertainty or the confidence in the hydrologic model's ability to realistically simulate runoff. For many projects, the fixed safety factor approach is adequate. However, for projects where the consequences of failure (an erroneous design) are large, quantifying the analysis uncertainty and risk of not meeting the design standard may be beneficial in selecting an appropriate level of design conservatism.

F-8. References

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Attachment 1

Design Storm Dimensionless Hyetograph Ordinates

Attachment 2

Precipitation Magnitude-Frequency Estimates for SPU Rain Gauge Locations

Filename: ApxF_HydrologicAnalysis_2026_Draft_redlines.docx
Directory: C:\Users\mfox\OneDrive - Herrera Environmental Consultants\Documents
Template: Normal.dotm
Title:
Subject:
Author: Michelle Fox
Keywords:
Comments:
Creation Date: 6/25/2025 5:06:00 PM
Change Number: 3
Last Saved On: 6/25/2025 5:08:00 PM
Last Saved By: Michelle Fox
Total Editing Time: 1 Minute
Last Printed On: 6/25/2025 5:08:00 PM
As of Last Complete Printing
Number of Pages: 80
Number of Words: 19,267 (approx.)
Number of Characters: 109,827 (approx.)