APPENDIX F

Hydrologic Analysis and Design

CONTENTS

TABLES

FIGURES

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 Directors.

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	GSI BMP Sizing	FC BMP Sizing	WQ BMP Sizing	Conveyance Sizing	TESC Design Flow Sizing
Continuous Rainfall-Runoff Modeling	• HSPF • MGSFlood • WWHM • Other [a]	Refer to Table F.12 for time step requirements	✓	✓	✓	✓	✓
Single-event Rainfall-Runoff Modeling	• NRCS (formerly SCS) TR-55 • SBUH • StormShed • Corps of Engineers HMS and HEC-1 • EPA SWMM 5, PCSWMM, and XP-SWMM • Other models approved by the Directors	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] The following continuous hydrologic models may also be used for project-specific situations: EPA SWMM5, ModFlow, HMS, PCSWMM, and other models approved by the Directors.

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

General Modeling Guidance

This section includes general modeling guidance that applies to both continuous modeling and single-event modeling including historic precipitation data, watershed characterization, hydrologic soil groups, infiltration equations, and outfalls.

Historic Precipitation Data

Data collected from the Seattle Public Utilities (SPU) rain gage 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 historic time series is recommended, but is not required for the design of stormwater BMPs.

Continuous historic precipitation data are available from 17 active and 2 closed rain gages from January 1978 through the present at a time step of 5 minutes. Active and closed gage 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 Gage Stations.

Station ID	Station Name	Period of Record	Status
45-S001	Haller Lake Shop	1965 – current	Active
45-S002	Magnusson Park	1969 – current	Active
45-S003	UW Hydraulics Lab	1965 – current	Active
45-S004	Maple Leaf Reservoir	1965 – current	Active
45-S005	Fauntleroy Ferry Dock	1968 – current	Active
45-S007	Whitman Middle School	1965 – current	Active
45-S008	Ballard Locks	1965 – current	Active
45-S009	Woodland Park Zoo	1965 – current	Active
45-S010	Rainier View Elementary	1968 – 2008	Closed
45-S011	Metro-KC Denny Regulating	1970 – current	Active
45-S012	Catherine Blaine Elementary School	1965 – current	Active
45-S014	Lafayette Elementary School	1965 – current	Active
45-S015	Puget Sound Clean Air Monitoring Station	1965 – current	Active
45-S016	Metro-KC E Marginal Way	1965 – current	Active
45-S017	West Seattle Reservoir Treatment Shop	1965 – current	Active
45-S018	Aki Kurose Middle School	1965 – current	Active
45-S020	TT Minor Elementary	1975 – 2010	Closed
RG25	Garfield Community Center	2010 – current	Active
RG30	SPL Rainier Beach Branch	2009 – current	Active

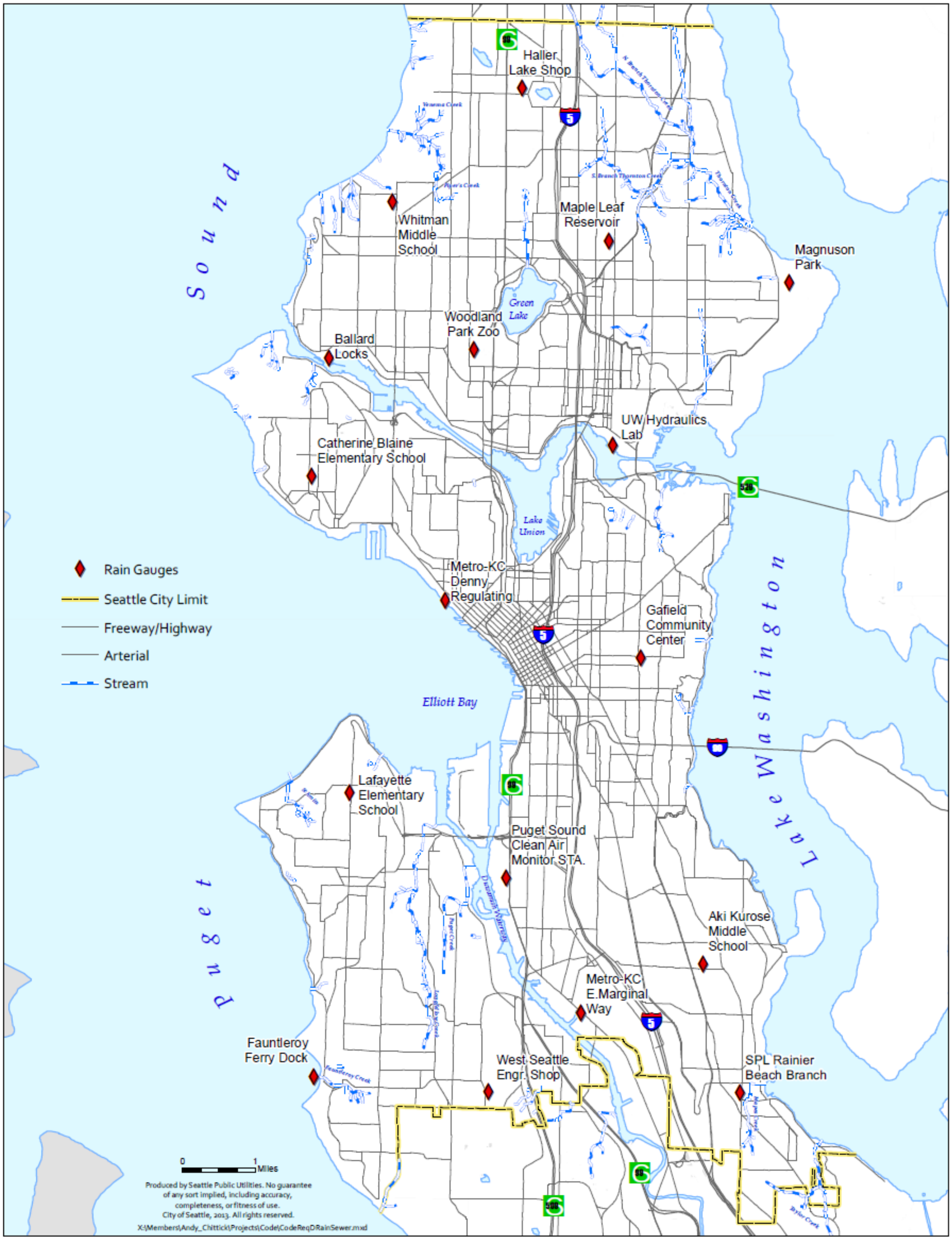


Figure F.1. Active City Rain Gage Network Stations.

Watershed Characterization

Prior to conducting any detailed stormwater runoff calculations, the overall relationship between the proposed project site and upstream and downstream offsite 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 through the City GIS map counter (http://www.ci.seattle.wa.us/GIS/docs/mapctr.htm).

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. Typically, the total impervious surface shall be assumed connected. For the existing condition modeling, areas with unconnected rooftops may be estimated from visual survey as approved by the Directors.

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
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 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.

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

$$F_{t+\Delta t} = F_t + K\Delta t + \psi\Delta\theta \ln[\frac{F_{t+\Delta t} + \psi\Delta\theta}{F_t + \psi\Delta\theta}] \quad (2)$$

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		Hydraulic Conductivity K		Porosity η	Effective Porosity θ_e
	(mm)	(in/hr)	(mm/hr)	(in/hr)		
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:

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

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 [1]	
	Poor Condition	**Good Condition**
Fallow [2]	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

[1] Adjustments needed for "weeds" and "grazing."
[2] 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.

SCS 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

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

Where: t = time

i = infiltration rate

α= empirical constant (α > 0)

β= empirical constant (0 < β < 1)

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

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

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,

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

and

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

Where: i_0 = initial 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

An outfall is defined as a point where collected and concentrated surface and stormwater runoff or combined sewage is discharged into an open drainage feature. These drainage features include streams, rivers, lakes, or Puget Sound.

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 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 historic record or the highest observed elevations. Lake Washington and associated waters are controlled at the Hiram M. Chittenden Locks by the U.S. Army Corps of Engineers. Refer to http://www.nwd-wc.usace.army.mil/nws/nwshh/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. [1]

	Bitter Lake	Haller Lake	Green Lake	Lake Union	Lake Washington
Water surface elevation (ft, NAVD88) [2]	434.4	376.9	164.3	16.8	16.8
Max depth (ft)	31.0	36.0	30.0	50.0	214.0
Mean depth (ft)	16.0	16.0	13.0	34.0	108.0
Area (ac)	19.0	14.9	259	580.0	21,500

[1] Sources: King County (2014a) and King County (2014b).

[2] 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.

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 (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 historic tidal record to the historic 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/index.shtml) and from the U.S. Army Corps of Engineer's (www.nws.usace.army.mil/PublicMenu/Documents/Reg/applications/tides/tides.cfm). 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. The observed trend from 1898 to 1999 was a rise of 2.11 mm per year (0.69 feet total). The effect of climate change on predicted sea level rise is expected to exceed that rate, but there is considerable uncertainty on timing and severity. A report by the University of Washington Climate Impacts Group (Mote et al. 2008) has provided low, medium, and high estimates of local sea level rise as shown in Figure F.2.

For Puget Sound, the "medium" estimate of sea level rise is 6 inches by 2050 and 13 inches by 2100. The low-probability high-impact estimate is for a rise of 22 inches by 2050 and up to 50 inches by 2100.

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.

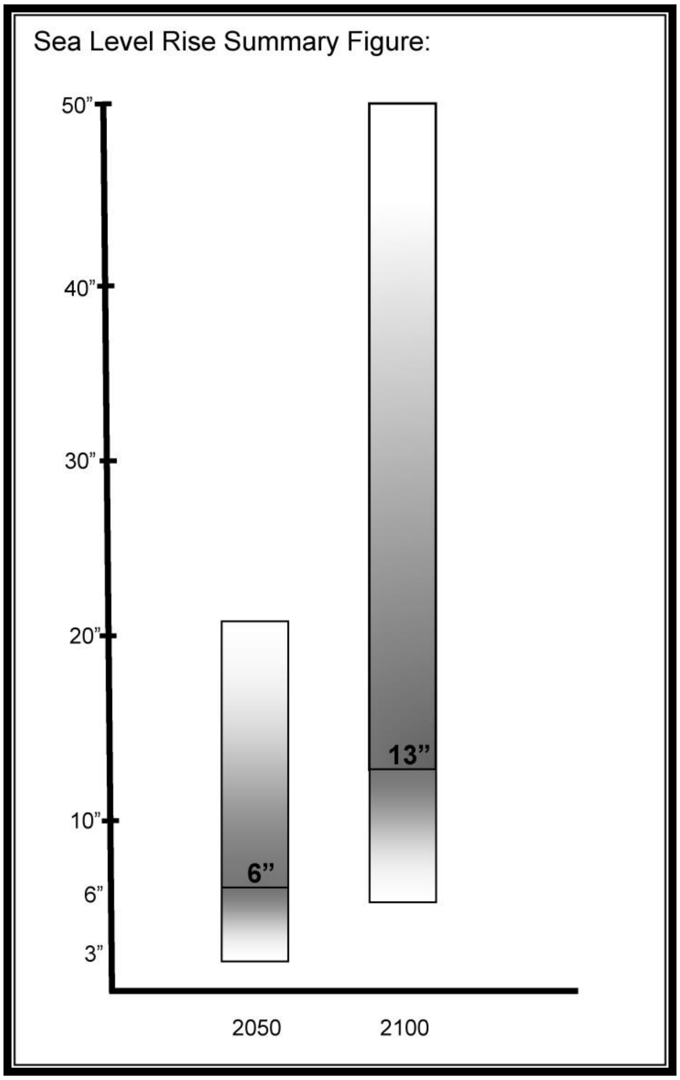


Figure F.2. Projected Sea Level Rise in Washington's Waters Relative to 1980-99, in Inches (shading roughly indicates likelihood).

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 historic precipitation and evaporation time series (described in Section F-3 – General Modeling Guidance). Because the record length of the historic 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-min, 20-min, 30-min, 45-min, 60-min, 2-hour, 6-hours, 24-hours, 3-days, 10-days, 30-days, 90-days, 6-months, and annual (Refer to http://www.seattle.gov/dpd/codesrules/codes/stormwater/default.htm for modeling resources).The precipitation time series was developed at a 5-minute time step.

The evaporation time series were 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 were developed from data collected at the Puyallup 2 West Experimental Station (station number 45-6803). Refer to http://www.seattle.gov/dpd/codesrules/codes/stormwater/default.htm for modeling resources. The evaporation time series has a 1-hour time step.

Land Cover Categorization

Currently approved continuous flow models based on HSPF include five land cover types: forest, pasture, 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 Soil Conservation Service (SCS, now the National Resource Conservation Service [NRCS]), or mapping performed by the University of Washington (http://geomapnw.ess.washington.edu/) may be used as a source of soil/geologic information for use in continuous hydrologic modeling. The interactive online geologic maps for the Seattle area developed by the University of Washington generally provide a higher degree of resolution and better characterization of underlying soil geology. If using SCS maps, each soil type defined by the SCS has been classified into one of four hydrologic soil groups; A, B, C, and D. Table F.3 shows SCS 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 SCS hydrologic soil groups as shown in Table F.9.

Table F.8. Continuous Hydrologic Cover Groups and Areas of Application.

Continuous Model Land Cover	Application	
	Predeveloped	**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 predevelopment densities greater than 4 DU/GA	Unprotected forest in rural residential development shall be considered half pasture, half grass
Grass / Landscape	Lawns in redevelopment areas with predevelopment densities greater than 4 DU/GA	All post-development grassland and landscaping and all onsite forested land not protected by covenant. This includes all disturbed areas required to meet the Soil Amendment BMP requirements (refer to *Volume 1 and Volume 3, Section 5.1*).
Wetland	All delineated wetland areas	All delineated wetland areas
Impervious	All impervious surfaces, including heavily compacted gravel and dirt roads, parking areas, etc., and open receiving waters (ponds and lakes)	All impervious surfaces, including heavily compacted gravel and dirt roads, parking areas, etc., and open receiving waters including onsite detention and water quality ponds

DU/GA – Dwelling Unit per Gross Acre

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

SCS	Model Soil Group
A	Outwash
B	Till or Outwash
C	Till
D	Wetland

SCS 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 have a seasonally high water table would be classified at till. Conversely, well-drained B type soils would be classified as outwash.

Note that neither the University of Washington nor SCS maps may be used for determining infiltration capacity or design infiltration rate.

Infiltration Parameters

The following discussion on HSPF model parameters applies to the use of continuous modeling (e.g., MGSFlood and WWHM). 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 (U.S. 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.

Table F.11. Default Runoff Parameters for Each Pervious Land Segment (PERLND).

Parameter	Pervious Land Segment (PERLND)						
	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
INFILT	0.08	0.06	0.03	2.0	1.6	0.8	2.0
LSUR	400	400	400	400	400	400	100
SLSUR	0.1	0.1	0.1	0.05	0.05	0.05	0.001
KVARY	0.5	0.5	0.5	0.3	0.3	0.3	0.5
AGWRC	0.996	0.996	0.996	0.996	0.996	0.996	0.996
INFEXP	2.0	2.0	2.0	2.0	2.0	2.0	10.0
INFILD	2.0	2.0	2.0	2.0	2.0	2.0	2.0
BASETP	0.0	0.0	0.0	0.0	0.0	0.0	0.0
AGWETP	0.0	0.0	0.0	0.0	0.0	0.0	0.7
CEPSC	0.2	0.15	0.1	0.2	0.15	0.1	0.1
UZSN	0.5	0.4	0.25	0.5	0.5	0.5	3.0
NSUR	0.35	0.3	0.25	0.35	0.3	0.25	0.5
INTFW	6.0	6.0	6.0	0.0	0.0	0.0	1.0
IRC	0.5	0.5	0.5	0.7	0.7	0.7	0.7
LZETP	0.7	0.4	0.25	0.7	0.4	0.25	0.8

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)

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

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

[1] 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: o Identify existing land use o Identify existing soil types using on-site evaluation, NRCS soil survey, or mapping performed by the University of Washington (http://geomapnw.ess.washington.edu) o Convert SCS soil types to HSPF soil classifications (till, outwash, or wetland) o 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: o Select homogeneous subbasin areas o Select separate subbasin areas for on-site and off-site drainage o Select separate subbasin areas for major drainage features.
C-5	Determine hydrologic parameters for each subbasin under existing conditions, if required: o Determine appropriate rainfall time series. For most design applications, the City of Seattle Design Time Series will be required. o Categorize soil types and land cover o Determine total and effective impervious areas within each subbasin o Determine areas for each soil/cover type in each subbasin o Select the required computational time step according to Table F.12.

Step #	Procedure
C-6	Compute runoff for the predeveloped 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.
C-7	Determine hydrologic parameters for each subbasin under developed conditions: o Utilize rainfall time series selected for existing conditions o Categorize soil types and land cover o Determine total and effective impervious areas within each subbasin o Determine areas for each soil/cover type in each subbasin o 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.

$$Tr = \frac{1}{p} \qquad (10)$$

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.

$$Tr = \frac{N + 0.12}{i - 0.44} \quad (11)$$

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 predeveloped peak discharge rate at one or more recurrence intervals. For example, Figure F.3 shows predeveloped and post-developed flood frequency curves for a stormwater pond designed to control peak discharges at the 2-year and 10-year recurrence intervals. Currently approved continuous simulation hydrologic models perform the frequency calculations and present the results in graphical and tabular form.

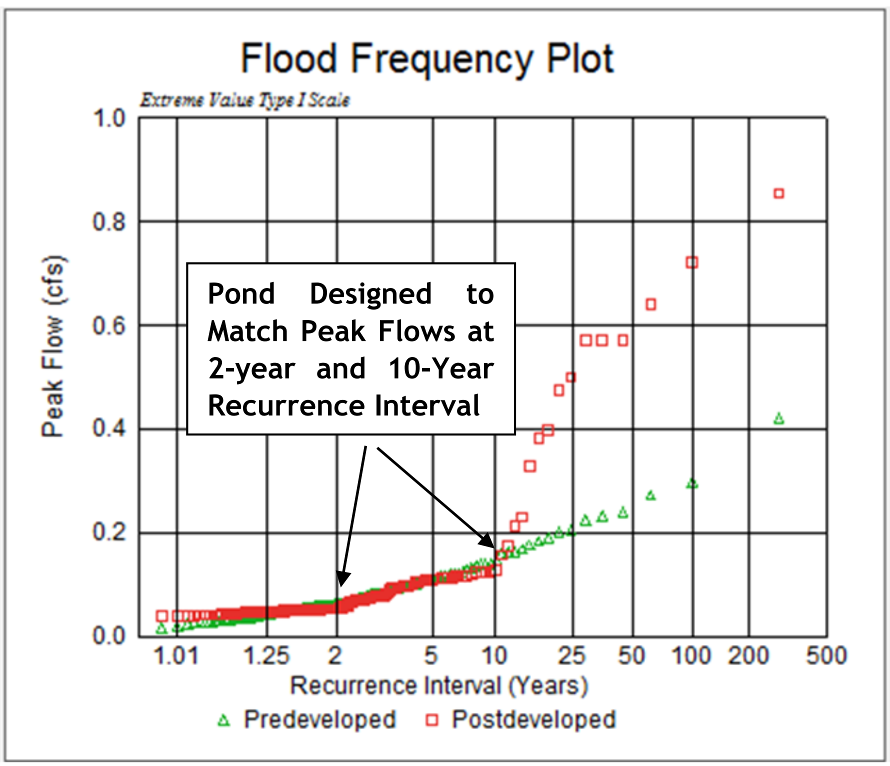


Figure F.3. Example Flood-Frequency Curves for a Stormwater Pond Designed to Control Post-Developed Peak Discharge Rates to Predeveloped Levels at the 2-year and 10-year Recurrence Interval.

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. 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.

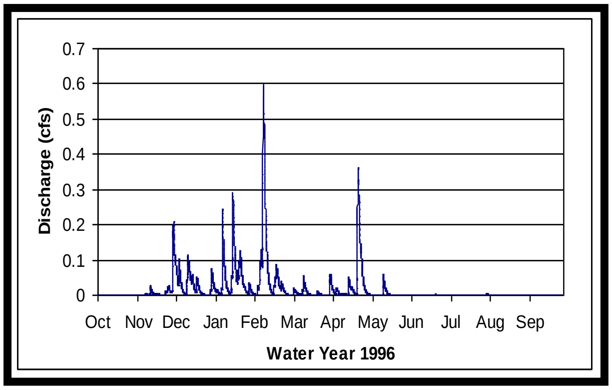


Figure F.4. Runoff from 10-Acre Forested Site.

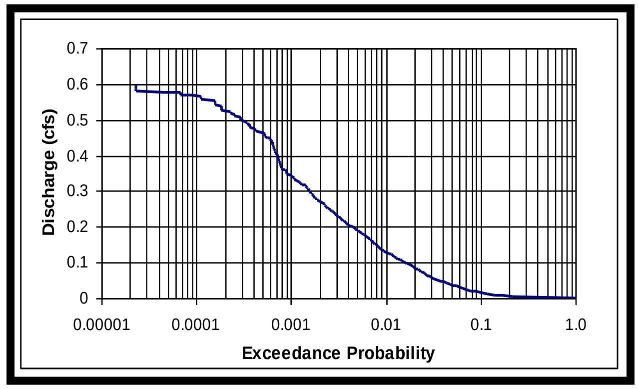


Figure F.5. Flow Duration Curve Computed Using Time Series in Figure F.4.

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 predeveloped 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 predeveloped curve between 1/2 of the predeveloped 2-year (1/2 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.

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.

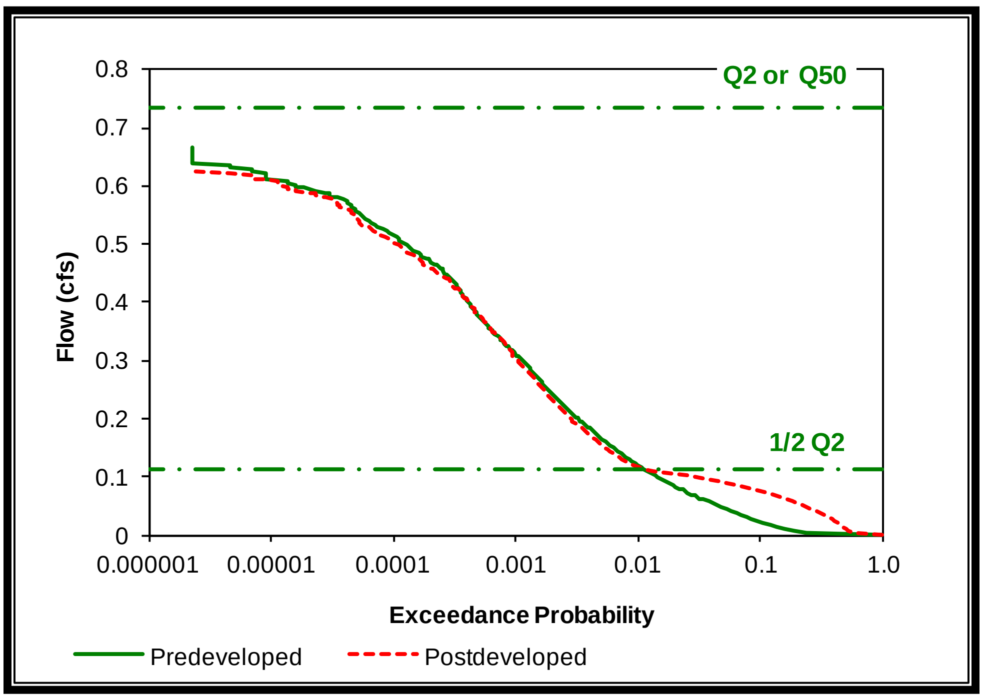
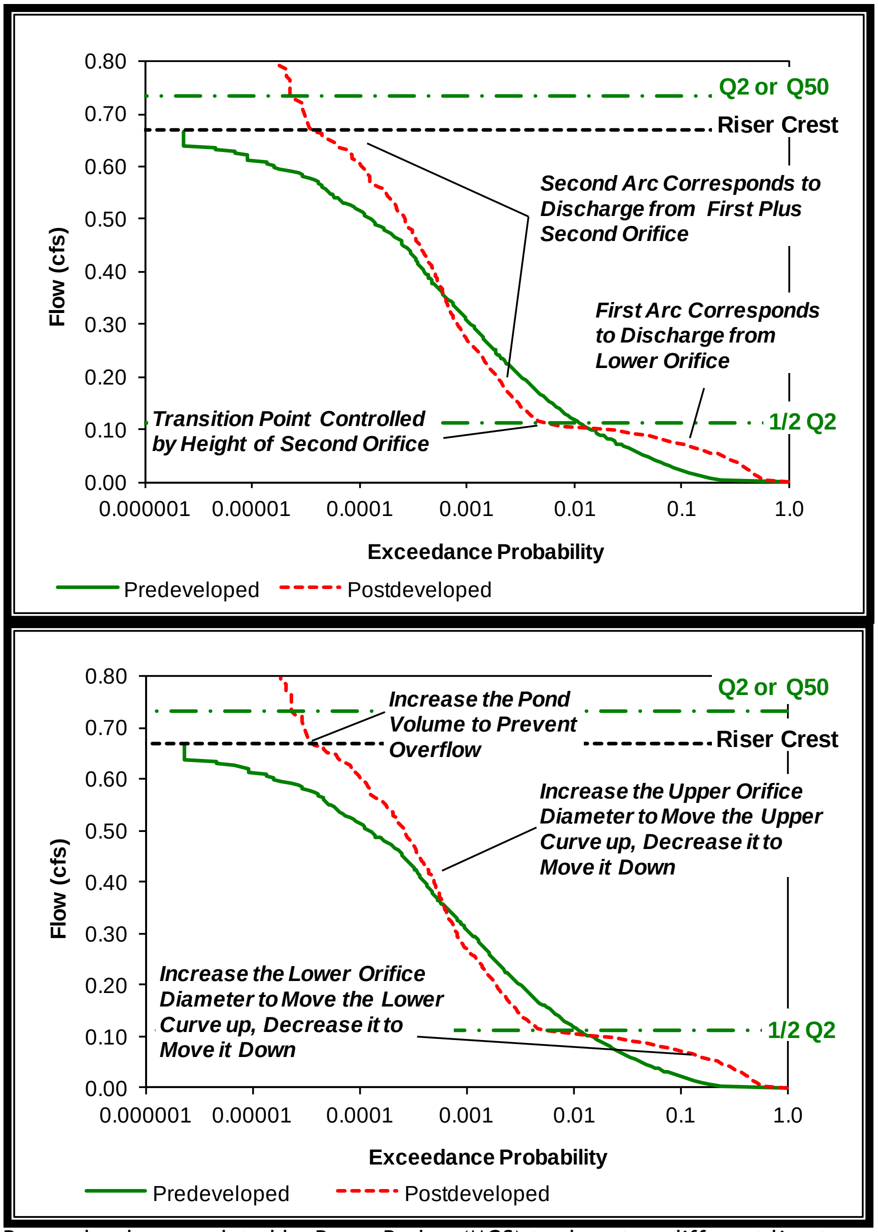


Figure F.6. Comparison of Predeveloped and Post developed Flow Duration Curves.



Beasreshas been updated by Bruce Barker (MGS) to show two different line types

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

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 (typically 60 to 158 years).

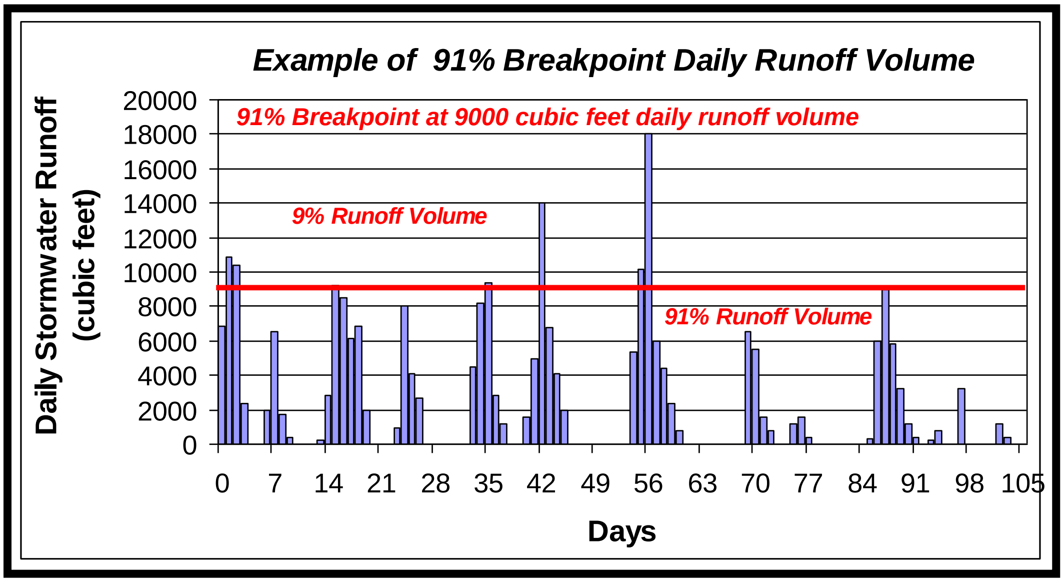


Figure F.8. Example of Portion of Time-Series of Daily Runoff Volume and Depiction of Water Quality Design Volume.

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)./

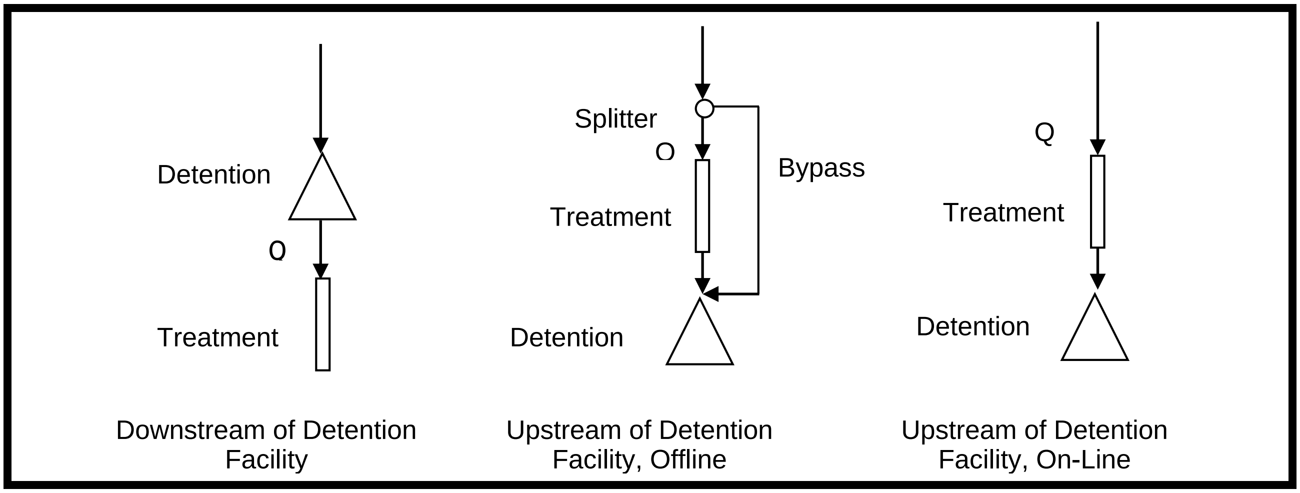


Figure F.9. Water Quality Treatment and Detention Definition.

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 need 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). Thus, 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.

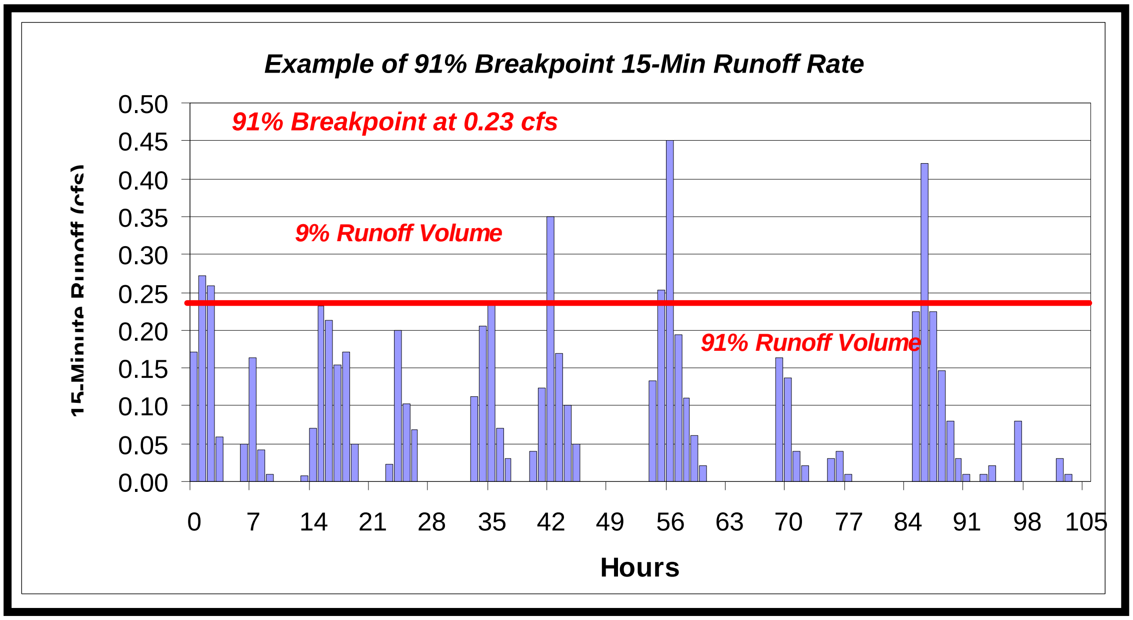


Figure F.10. Offline Water Quality Treatment Discharge Example.

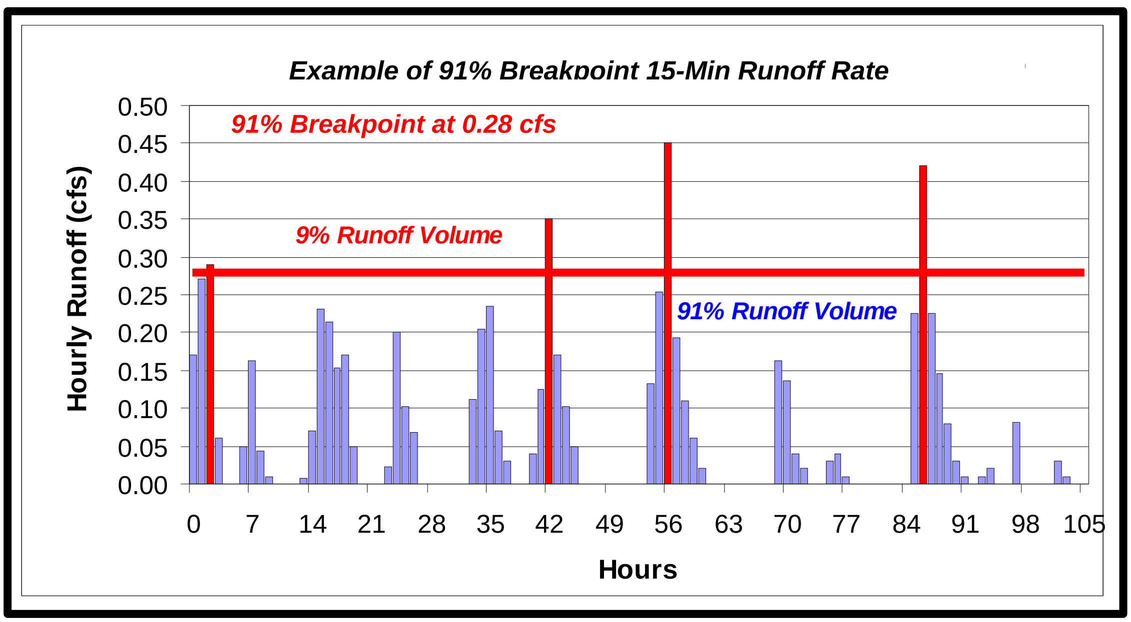


Figure F.11. On-Line Water Quality Treatment Discharge Example.

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 5.4.1*. The procedure is the same as for designing infiltration for flow control (refer to *Volume 3, Section 5.4.1*), except that the target is to infiltrate 91 percent of the runoff file without overflow. 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 estimated long-term 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 on 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.

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 historic data recorded from a rain gage 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 Gage Network and gages 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 Gage 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*	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	

*Note: Simulation was performed using SPU Design Time Series, which is 158 years in length, and has dates spanning 10/1/1939 - 9/30/2097.

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 Gages
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 for the watershed (refer to Attachment 2 and the DPD SPU Stormwater webpage for modeling resources).
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 DPD 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.

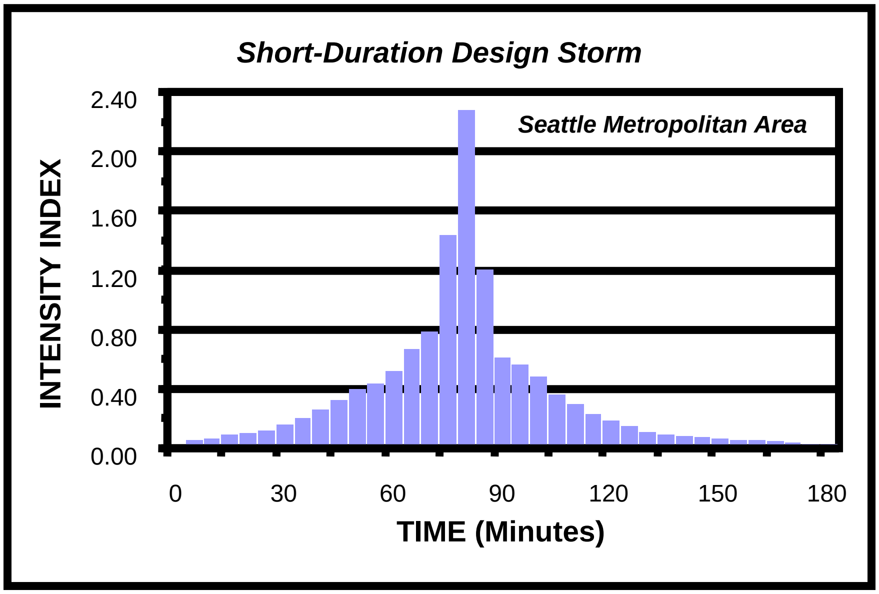


Figure F.12. Dimensionless Short-Duration (3-Hour) Design Storm, Seattle Metropolitan Area.

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 and the DPD SPU Stormwater webpage for modeling resources).
ID-2	Multiply the 10-minute incremental ordinates of the dimensionless intermediate-duration design storm (Attachment 1, Table 2) 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 DPD 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.

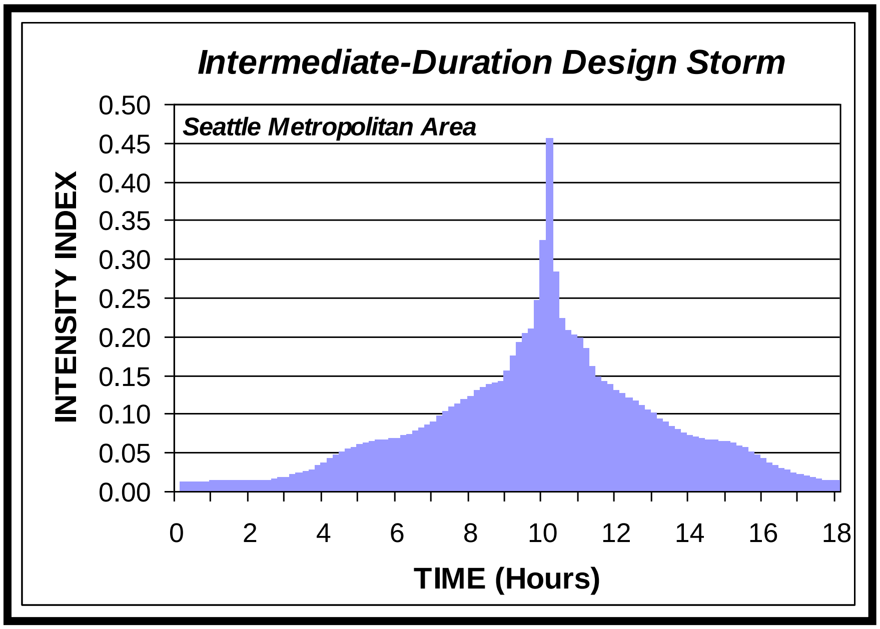


Figure F.13. Dimensionless Intermediate-Duration (18-Hour) Design Storm, Seattle Metropolitan Area.

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 and the DPD SPU Stormwater webpage for modeling resources).
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 DPD 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.

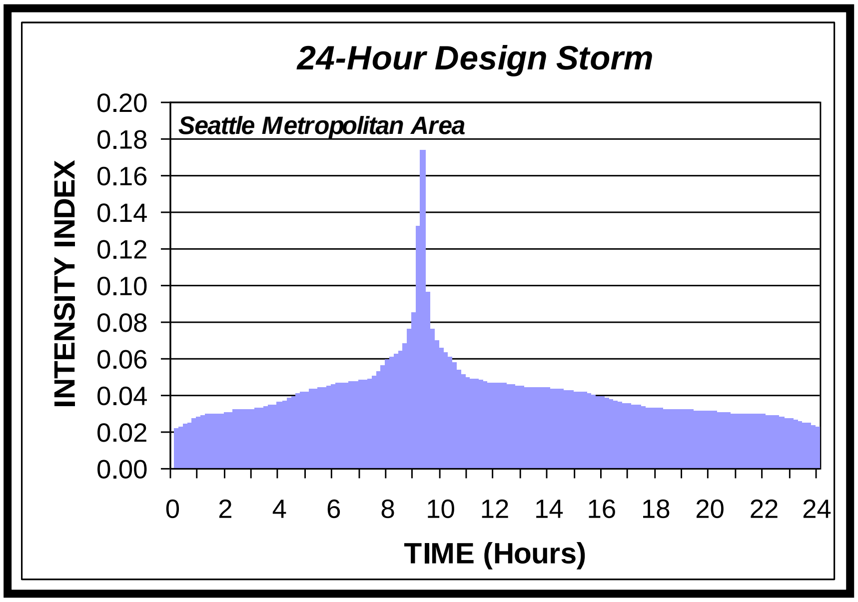


Figure F.14. Dimensionless 24-Hour Design Storm for Seattle Metropolitan Area.

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 and the DPD SPU Stormwater webpage for modeling resources).
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 DPD 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.

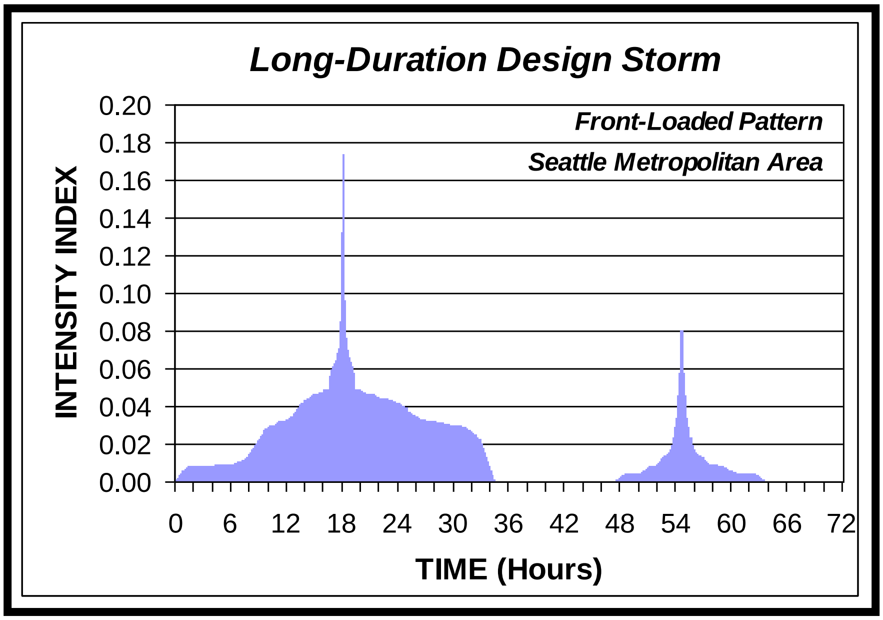


Figure F.15. Dimensionless Front-Loaded Long-Duration (64-Hour) Design Storm for the Seattle Metropolitan Area.

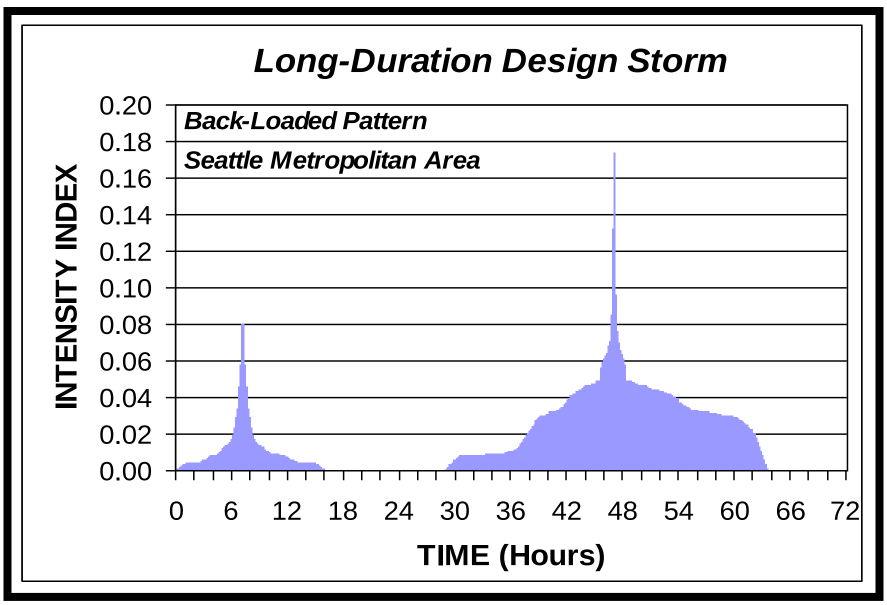


Figure F.16. Dimensionless Back-Loaded Long-Duration (64-Hour) Design Storm for the Seattle Metropolitan Area.

Use of Historic Storms in Analysis

This section includes a catalog of the storms used to derive the design storm patterns described in the previous section. These historic storms can be used in rainfall runoff models to aid in the design process by replicating past floods. For example, an engineer could use the historic storms to demonstrate that a proposed conveyance system design would have adequate capacity to pass a large historic flood that occurred in the watershed. The storms could also be used for calibrating the hydrologic model to recorded flow data. Use of these historic storms to confirm a facility design is recommended but is not required for the design of stormwater facilities.

Tables F.15, F.16, and F.17 summarize historic storms recorded at City gages for durations of 2 hours, 6 hours, and 24 hours respectively. Included in each table is the date when the storm ended, storm recurrence interval, and total precipitation for the duration of interest. The gage locations are shown in Figure F.1. Electronic data for each storm is available in tabular form from SPU (refer to the DPD-SPU Stormwater webpage for modeling resources).

Table F.15. Catalog of Short-Duration (2-hour) Storms at City Rain Gages.

Station ID	Station Name	Storm End Date	Storm Recurrence Interval (years)	2-hour Precipitation (inches)
45-S002	Mathews Beach Pump Stn	06/14/1978	16	0.86
45-S003	UW Hydraulics Lab	11/03/1978	10	0.79
45-S009	Woodland Park Zoo	08/17/1980	20	0.89
45-S008	Ballard Locks	08/28/1980	20	0.89
45-S002	Mathews Beach Pump Stn	05/29/1985	7	0.74
45-S014	West Seattle High School	10/26/1986	15	0.85
45-S020	TT Minor Elementary	10/04/1990	18	0.88
45-S009	Woodland Park Zoo	08/09/1991	6	0.72
45-S008	Ballard Locks	09/23/1992	45	1.02
45-S003	UW Hydraulics Lab	11/23/1997	9	0.77
45-S011	Metro-KC Denny Regulating	02/17/1998	14	0.84
45-S016	Metro-KC E Marginal Way	07/15/2001	6	0.71
45-S012	Catherine Blaine Jr	08/23/2001	14	0.84
45-S020	TT Minor Elementary	05/28/2002	4	0.83
45-S009	Woodland Park Zoo	09/03/2002	10	0.79
45-S004	Maple Leaf Reservoir	10/20/2003	18	0.88
45-S003	UW Hydraulics Lab	12/14/2006	13	0.83

Table F.16. Catalog of Intermediate-Duration (6-hour) Storms at City Rain Gages.

Station ID	Station Name	Storm End Date	Storm Recurrence Interval (years)	6-hour Precipitation (inches)
45-S016	Metro-KC E Marginal Way	9/22/1978	32	1.61
45-S001	Haller Lake Shop	11/04/1978	70	1.74
45-S003	UW Hydraulics Lab	12/03/1982	92	1.82
45-S001	Haller Lake Shop	09/05/1984	5	1.21
45-S020	TT Minor Elementary	01/18/1986	>100	2.27
45-S010	Rainier Ave Elementary	01/09/1990	88	1.83
45-S003	UW Hydraulics Lab	12/29/1996	16	1.45
45-S004	Maple Leaf Reservoir	06/24/1999	7	1.28
45-S004	Maple Leaf Reservoir	10/20/2003	>100	1.96
45-S003	UW Hydraulics Lab	12/14/2006	36	1.62

Table F.17. Catalog of Long-Duration (24-hour) Storms at City Rain Gages.

Station ID	Station Name	Storm End Date	Storm Recurrence Interval (years)	24-hour Precipitation (inches)
45-S008	Ballard Locks	12/17/1979	4	2.40
45-S009	Woodland Park Zoo	10/06/1981	24	3.07
45-S004	Maple Leaf Reservoir	11/01/1984	3	2.11
45-S001	Haller Lake Shop	01/18/1986	96	3.69
45-S016	Metro-KC E Marginal Way	11/23/1986	9	2.70
45-S003	UW Hydraulics Lab	11/24/1990	17	2.91
45-S002	Mathews Beach Pump Stn	04/04/1991	4	2.15
45-S020	TT Minor Elementary	02/08/1996	>100	5.07
45-S020	TT Minor Elementary	04/23/1996	8	2.56
45-S003	UW Hydraulics Lab	03/18/1997	7	2.53
45-S004	Maple Leaf Reservoir	11/25/1998	11	2.68
45-S010	Rainier Ave Elementary	11/14/2001	34	3.31
45-S004	Maple Leaf Reservoir	10/20/2003	>100	4.05

When using historic data from the City rain gage network for model calibration, storms should be selected from stations as close as possible to the center of the watershed tributary to the project site. This will help ensure that the recorded data is representative of precipitation that fell in the watershed for storm of interest. In general, the shorter duration storms typically have smaller areal coverage and greater spatial variability than the longer duration storms. Thus, greater simulation errors would be expected if gage data outside the watershed is used to simulate short-duration storms.

SCS Equation and Infiltration Parameters

The SCS Curve Number loss 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 = SCS Curve Number for the soil (Table F.18)

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.18. Refer to Table F.3 in General Modeling Guidance (*Section F-3*) for information on soil groups in King County.

Table F.18. SCS Western Washington Runoff Curve Numbers.

Land Use Description		Curve Numbers by Hydrologic Soil Group			
Land Cover	**Condition**	**A**	**B**	**C**	**D**
Cultivated land	Winter condition	86	91	94	95
Mountain open areas	Low growing brush and grasslands	74	82	89	92
Meadow or pasture		65	78	85	89
Wood or forest land Wood or forest land Orchard	Undisturbed young second growth or brush with cover crop	42 55 81	64 81 88	76 72 92	81 86 94
Open spaces, lawns, parks, golf courses, cemeteries, landscaping	Good: grass cover on ≥75 percent of the area	68	80	86	90
	Fair: grass cover on 50 to 75 percent of the area	77	85	90	92
Gravel roads and parking lots Dirt roads and parking lots		76 72	85 82	89 87	91 89
Impervious surfaces, pavement, roofs etc., open receiving waters: lakes, wetlands, ponds		98 100	98 100	98 100	98 100

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.19. These n_s values are applicable for shallow flow depths up to about 0.1 foot and for travel lengths up to 300 feet.

$$T_t = 0.42 * (ns * L)0.8 / ((P_{24})0.5 * (So)0.4) \qquad (16)$$

Where: T_t = travel time (minutes)
n_s = sheet flow Manning's effective roughness coefficient from Table F.19
L = overland flow length (feet)
P_{24} = 2-year, 24-hour rainfall (inches)
So = 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.19 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:

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

Where: k_s = velocity factor (Table F.19)
S_0 = slope of flow path (feet/feet)

"k" values in Table F.19 have been computed for various land covers and channel characteristics with assumptions made for hydraulic radius using the following rearrangement of Manning's equation:

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

Where: R = assumed hydraulic radius
n = Manning's roughness coefficient for open channel flow, from Tables F.19 or F.20

Table F.19. 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).

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

Table F.20. 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

sheets. The k_c values from Table 6.14 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.

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.

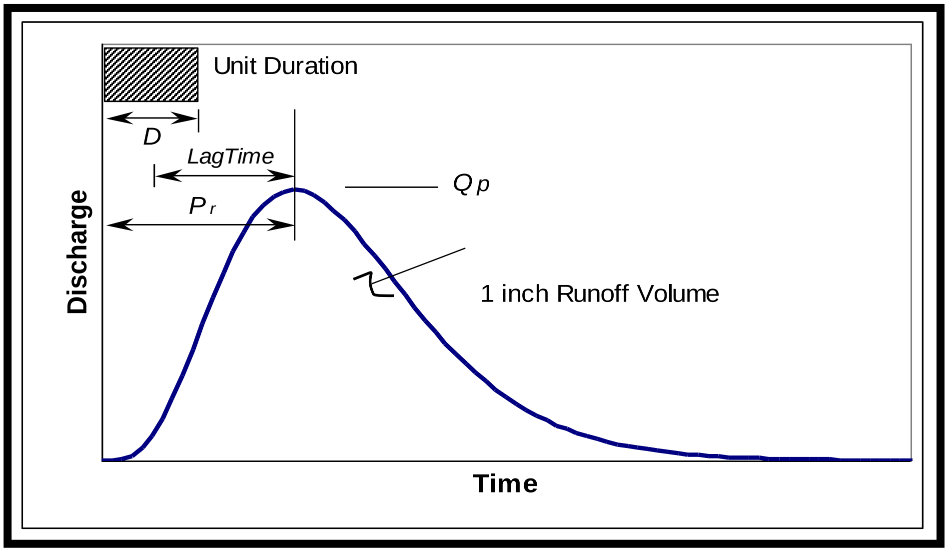


Figure F.17. Characteristics of Unit Hydrographs.

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.

<u>Relationship of Computational Time Step to Time Lag (Lag Time)</u>. 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.

<u>Selection of Time Step (Δt) Based on Time of Concentration (T_c)</u>. 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:

$$\Delta t < T_c/5 \qquad (19)$$

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:

$$\Delta t = 5/n \qquad (20)$$

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

$$\Delta t = 10/n \qquad (21)$$

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, and

Step 2 – Compute the runoff hydrograph.

The instantaneous hydrograph is computed as follows:

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

Where:
- $I(t)$ = 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:

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

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

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):

$$\Delta t < T_c/5 \qquad (27)$$

$$\Delta t = 5/n \qquad (28)$$

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

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

rearranging:

$$I_1 + I_2 + 2S_1 - O_1 = O_2 + 2S_2 \quad (30)$$

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* of this appendix.

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: o Identify existing land use o Identify existing soil types using on-site evaluation, NRCS soil survey, or mapping performed by the University of Washington (http://geomapnw.ess.washington.edu) o Identify existing drainage features such as streams, conveyance systems, detention facilities, ponding areas, depressions, etc.

Step #	Procedure
SE-4	Select and delineate pertinent subbasins based on existing conditions: o Select homogeneous subbasin areas o Select separate subbasin areas for on-site and off-site drainage o 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.
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.

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:

$$Q = CiA \qquad (31)$$

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.21 for the precipitation recurrence interval of interest and duration corresponding to the calculated time of concentration (refer to *Time of Concentration Estimation* section below).

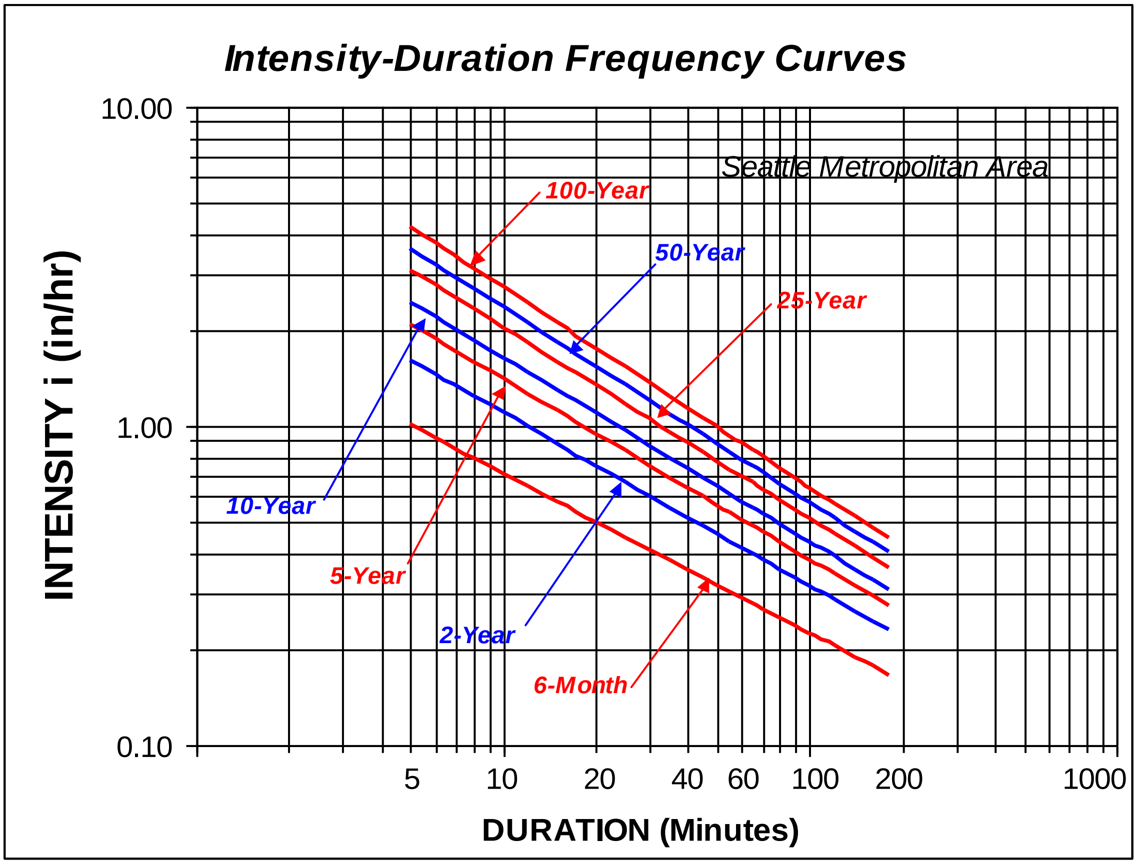


Figure F.18. Intensity-Duration-Frequency Curves for the City of Seattle.

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-gage precipitation measurement network within the City of Seattle, and the national NOAA cooperative gage 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.

Table F.21. 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
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.22). 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
$C_{1,2,,...n}$ = runoff coefficient for each land cover type
$A_{1,2,,...n}$ = area of each land cover type (acres)
A_t = total tributary area (acres)

Table F.22. 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.

$$T_c = T_1 + T_2 + T_3 + \ldots T_n \qquad (33)$$

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:

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

$$V = k_r \sqrt{S_o} \qquad (35)$$

Where: k_r = Velocity factor (Table F.23)
S_0 = Slope of flow path (feet/feet)

Table F.23. 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

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.

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Attachment 1 –

Design Storm Dimensionless Hyetograph Ordinates

Attachment 1- Design Storm Dimensionless Hyetograph Ordinates

Table 1. Dimensionless Ordinates of the Short-Duration Design Storm.

DIMENSIONLESS ORDINATES OF SHORT-DURATION DESIGN STORM		
ELAPSED TIME (min)	INCREMENTAL ORDINATES	CUMULATIVE ORDINATES
0	0.0000	0.0000
5	0.0045	0.0045
10	0.0055	0.0100
15	0.0075	0.0175
20	0.0086	0.0261
25	0.0102	0.0363
30	0.0134	0.0497
35	0.0173	0.0670
40	0.0219	0.0889
45	0.0272	0.1161
50	0.0331	0.1492
55	0.0364	0.1856
60	0.0434	0.2290
65	0.0553	0.2843
70	0.0659	0.3502
75	0.1200	0.4702
80	0.1900	0.6602
85	0.1000	0.7602
90	0.0512	0.8114
95	0.0472	0.8586
100	0.0398	0.8984
105	0.0301	0.9285
110	0.0244	0.9529
115	0.0195	0.9724
120	0.0153	0.9877
125	0.0125	1.0002
130	0.0096	1.0098
135	0.0077	1.0175
140	0.0068	1.0243
145	0.0062	1.0305
150	0.0056	1.0361
155	0.0050	1.0411
160	0.0044	1.0455
165	0.0038	1.0493
170	0.0032	1.0525
175	0.0026	1.0551
180	0.0020	1.0571

Table 2. Dimensionless Ordinates of the Intermediate-Duration Design Storm.

DIMENSIONLESS ORDINATES OF INTERMEDIATE-DURATION DESIGN STORM								
ELAPSED TIME (Hr)	INCRM ORDINATE	SUM ORDINATE	ELAPSED TIME (Hr)	INRM ORDINATE	SUM ORDINATE	ELAPSED TIME (Hr)	INCRM ORDINATE	SUM ORDINATE
0.00	0.0000	0.0000	6.17	0.0118	0.1972	12.17	0.0210	1.1731
0.17	0.0020	0.0020	6.33	0.0123	0.2095	12.33	0.0201	1.1932
0.33	0.0020	0.0040	6.50	0.0129	0.2224	12.50	0.0193	1.2125
0.50	0.0020	0.0060	6.67	0.0136	0.2360	12.67	0.0184	1.2309
0.67	0.0020	0.0080	6.83	0.0142	0.2502	12.83	0.0176	1.2485
0.83	0.0020	0.0100	7.00	0.0150	0.2652	13.00	0.0168	1.2653
1.00	0.0021	0.0121	7.17	0.0163	0.2815	13.17	0.0154	1.2807
1.17	0.0021	0.0142	7.33	0.0171	0.2986	13.33	0.0147	1.2954
1.33	0.0021	0.0163	7.50	0.0180	0.3166	13.50	0.0140	1.3094
1.50	0.0021	0.0184	7.67	0.0188	0.3354	13.67	0.0132	1.3226
1.67	0.0021	0.0205	7.83	0.0197	0.3551	13.83	0.0127	1.3353
1.83	0.0022	0.0227	8.00	0.0205	0.3756	14.00	0.0121	1.3474
2.00	0.0022	0.0249	8.17	0.0215	0.3971	14.17	0.0116	1.3590
2.17	0.0023	0.0272	8.33	0.0224	0.4195	14.33	0.0113	1.3703
2.33	0.0023	0.0295	8.50	0.0229	0.4424	14.50	0.0111	1.3814
2.50	0.0024	0.0319	8.67	0.0232	0.4656	14.67	0.0109	1.3923
2.67	0.0025	0.0344	8.83	0.0237	0.4893	14.83	0.0107	1.4030
2.83	0.0028	0.0372	9.00	0.0257	0.5150	15.00	0.0105	1.4135
3.00	0.0030	0.0402	9.17	0.0290	0.5440	15.17	0.0103	1.4238
3.17	0.0034	0.0436	9.33	0.0320	0.5760	15.33	0.0098	1.4336
3.33	0.0038	0.0474	9.50	0.0338	0.6098	15.50	0.0093	1.4429
3.50	0.0042	0.0516	9.67	0.0349	0.6447	15.67	0.0085	1.4514
3.67	0.0046	0.0562	9.83	0.0411	0.6858	15.83	0.0078	1.4592
3.83	0.0054	0.0616	10.00	0.0540	0.7398	16.00	0.0070	1.4662
4.00	0.0062	0.0678	10.17	0.0760	0.8158	16.17	0.0062	1.4724
4.17	0.0070	0.0748	10.33	0.0470	0.8628	16.33	0.0054	1.4778
4.33	0.0079	0.0827	10.50	0.0372	0.9000	16.50	0.0049	1.4827
4.50	0.0085	0.0912	10.67	0.0347	0.9347	16.67	0.0044	1.4871
4.67	0.0090	0.1002	10.83	0.0337	0.9684	16.83	0.0039	1.4910
4.83	0.0095	0.1097	11.00	0.0330	1.0014	17.00	0.0035	1.4945
5.00	0.0100	0.1197	11.17	0.0308	1.0322	17.17	0.0032	1.4977
5.17	0.0104	0.1301	11.33	0.0269	1.0591	17.33	0.0029	1.5006
5.33	0.0107	0.1408	11.50	0.0247	1.0838	17.50	0.0026	1.5032
5.50	0.0109	0.1517	11.67	0.0237	1.1075	17.67	0.0024	1.5056
5.67	0.0110	0.1627	11.83	0.0228	1.1303	17.83	0.0024	1.5080
5.83	0.0113	0.1740	12.00	0.0218	1.1521	18.00	0.0023	1.5103
6.00	0.0114	0.1854						

Table 3. Dimensionless Ordinates of Front-Loaded Long-Duration Design Storm.

DIMENSIONLESS ORDINATES OF INTERMEDIATE-DURATION DESIGN STORM								
ELAPSED TIME (Hr)	INCRM ORDINATE	SUM ORDINATE	ELAPSED TIME (Hr)	INRM ORDINATE	SUM ORDINATE	ELAPSED TIME (Hr)	INCRM ORDINATE	SUM ORDINATE
0.00	0.0000	0.0000	7.17	0.0018	0.0569	14.17	0.0072	0.2570
0.17	0.0001	0.0001	7.33	0.0019	0.0588	14.33	0.0073	0.2643
0.33	0.0003	0.0004	7.50	0.0019	0.0607	14.50	0.0074	0.2717
0.50	0.0005	0.0009	7.67	0.0020	0.0627	14.67	0.0075	0.2792
0.67	0.0007	0.0016	7.83	0.0022	0.0649	14.83	0.0076	0.2868
0.83	0.0009	0.0025	8.00	0.0024	0.0673	15.00	0.0077	0.2945
1.00	0.0010	0.0035	8.17	0.0026	0.0699	15.17	0.0078	0.3023
1.17	0.0011	0.0046	8.33	0.0028	0.0727	15.33	0.0078	0.3101
1.33	0.0012	0.0058	8.50	0.0030	0.0757	15.50	0.0078	0.3179
1.50	0.0013	0.0071	8.67	0.0032	0.0789	15.67	0.0079	0.3258
1.67	0.0013	0.0084	8.83	0.0034	0.0823	15.83	0.0079	0.3337
1.83	0.0013	0.0097	9.00	0.0036	0.0859	16.00	0.0079	0.3416
2.00	0.0013	0.0110	9.17	0.0038	0.0897	16.17	0.0081	0.3497
2.17	0.0013	0.0123	9.33	0.0040	0.0937	16.33	0.0082	0.3579
2.33	0.0013	0.0136	9.50	0.0042	0.0979	16.50	0.0082	0.3661
2.50	0.0014	0.0150	9.67	0.0045	0.1024	16.67	0.0093	0.3754
2.67	0.0014	0.0164	9.83	0.0047	0.1071	16.83	0.0099	0.3853
2.83	0.0014	0.0178	10.00	0.0048	0.1119	17.00	0.0102	0.3955
3.00	0.0014	0.0192	10.17	0.0049	0.1168	17.17	0.0104	0.4059
3.17	0.0014	0.0206	10.33	0.0049	0.1217	17.33	0.0107	0.4166
3.33	0.0014	0.0220	10.50	0.0049	0.1266	17.50	0.0114	0.4280
3.50	0.0014	0.0234	10.67	0.0050	0.1316	17.67	0.0118	0.4398
3.67	0.0014	0.0248	10.83	0.0051	0.1367	17.83	0.0142	0.4540
3.83	0.0014	0.0262	11.00	0.0051	0.1418	18.00	0.0220	0.4760
4.00	0.0014	0.0276	11.17	0.0053	0.1471	18.17	0.0290	0.5050
4.17	0.0014	0.0290	11.33	0.0053	0.1524	18.33	0.0160	0.5210
4.33	0.0015	0.0305	11.50	0.0054	0.1578	18.50	0.0127	0.5337
4.50	0.0015	0.0320	11.67	0.0054	0.1632	18.67	0.0116	0.5453
4.67	0.0015	0.0335	11.83	0.0054	0.1686	18.83	0.0110	0.5563
4.83	0.0015	0.0350	12.00	0.0055	0.1741	19.00	0.0106	0.5669
5.00	0.0015	0.0365	12.17	0.0055	0.1796	19.17	0.0102	0.5771
5.17	0.0015	0.0380	12.33	0.0056	0.1852	19.33	0.0096	0.5867
5.33	0.0015	0.0395	12.50	0.0057	0.1909	19.50	0.0082	0.5949
5.50	0.0015	0.0410	12.67	0.0058	0.1967	19.67	0.0082	0.6031
5.67	0.0015	0.0425	12.83	0.0060	0.2027	19.83	0.0082	0.6113
5.83	0.0015	0.0440	13.00	0.0062	0.2089	20.00	0.0081	0.6194
6.00	0.0015	0.0455	13.17	0.0064	0.2153	20.17	0.0080	0.6274
6.17	0.0015	0.0470	13.33	0.0066	0.2219	20.33	0.0079	0.6353
6.33	0.0015	0.0485	13.50	0.0068	0.2287	20.50	0.0079	0.6432

Table 3 (continued). Dimensionless Ordinates of Front-Loaded Long-Duration Design Storm.

DIMENSIONLESS ORDINATES OF INTERMEDIATE-DURATION DESIGN STORM								
ELAPSED TIME (Hr)	INCRM ORDINATE	SUM ORDINATE	ELAPSED TIME (Hr)	INRM ORDINATE	SUM ORDINATE	ELAPSED TIME (Hr)	INCRM ORDINATE	SUM ORDINATE
6.50	0.0016	0.0501	13.67	0.0069	0.2356	20.67	0.0078	0.6510
6.67	0.0016	0.0517	13.83	0.0070	0.2426	20.83	0.0078	0.6588
6.83	0.0017	0.0534	14.00	0.0072	0.2498	21.00	0.0077	0.6665
7.00	0.0017	0.0551						
21.17	0.0077	0.6742	30.17	0.0050	1.0069	39.17	0.0000	1.0984
21.33	0.0077	0.6819	30.33	0.0049	1.0118	39.33	0.0000	1.0984
21.50	0.0077	0.6896	30.50	0.0049	1.0167	39.50	0.0000	1.0984
21.67	0.0076	0.6972	30.67	0.0049	1.0216	39.67	0.0000	1.0984
21.83	0.0075	0.7047	30.83	0.0049	1.0265	39.83	0.0000	1.0984
22.00	0.0075	0.7122	31.00	0.0048	1.0313	40.00	0.0000	1.0984
22.17	0.0074	0.7196	31.17	0.0048	1.0361	40.17	0.0000	1.0984
22.33	0.0074	0.7270	31.33	0.0048	1.0409	40.33	0.0000	1.0984
22.50	0.0073	0.7343	31.50	0.0047	1.0456	40.50	0.0000	1.0984
22.67	0.0073	0.7416	31.67	0.0046	1.0502	40.67	0.0000	1.0984
22.83	0.0073	0.7489	31.83	0.0045	1.0547	40.83	0.0000	1.0984
23.00	0.0072	0.7561	32.00	0.0044	1.0591	41.00	0.0000	1.0984
23.17	0.0072	0.7633	32.17	0.0043	1.0634	41.17	0.0000	1.0984
23.33	0.0072	0.7705	32.33	0.0042	1.0676	41.33	0.0000	1.0984
23.50	0.0071	0.7776	32.50	0.0041	1.0717	41.50	0.0000	1.0984
23.67	0.0071	0.7847	32.67	0.0039	1.0756	41.67	0.0000	1.0984
23.83	0.0070	0.7917	32.83	0.0038	1.0794	41.83	0.0000	1.0984
24.00	0.0070	0.7987	33.00	0.0037	1.0831	42.00	0.0000	1.0984
24.17	0.0069	0.8056	33.17	0.0033	1.0864	42.17	0.0000	1.0984
24.33	0.0068	0.8124	33.33	0.0029	1.0893	42.33	0.0000	1.0984
24.50	0.0067	0.8191	33.50	0.0025	1.0918	42.50	0.0000	1.0984
24.67	0.0067	0.8258	33.67	0.0021	1.0939	42.67	0.0000	1.0984
24.83	0.0066	0.8324	33.83	0.0017	1.0956	42.83	0.0000	1.0984
25.00	0.0065	0.8389	34.00	0.0013	1.0969	43.00	0.0000	1.0984
25.17	0.0062	0.8451	34.17	0.0009	1.0978	43.17	0.0000	1.0984
25.33	0.0062	0.8513	34.33	0.0005	1.0983	43.33	0.0000	1.0984
25.50	0.0060	0.8573	34.50	0.0001	1.0984	43.50	0.0000	1.0984
25.67	0.0059	0.8632	34.67	0.0000	1.0984	43.67	0.0000	1.0984
25.83	0.0059	0.8691	34.83	0.0000	1.0984	43.83	0.0000	1.0984
26.00	0.0058	0.8749	35.00	0.0000	1.0984	44.00	0.0000	1.0984
26.17	0.0057	0.8806	35.17	0.0000	1.0984	44.17	0.0000	1.0984
26.33	0.0056	0.8862	35.33	0.0000	1.0984	44.33	0.0000	1.0984
26.50	0.0055	0.8917	35.50	0.0000	1.0984	44.50	0.0000	1.0984
26.67	0.0055	0.8972	35.67	0.0000	1.0984	44.67	0.0000	1.0984

Table 3 (continued). Dimensionless Ordinates of Front-Loaded Long-Duration Design Storm.

DIMENSIONLESS ORDINATES OF INTERMEDIATE-DURATION DESIGN STORM								
ELAPSED TIME (Hr)	INCRM ORDINATE	SUM ORDINATE	ELAPSED TIME (Hr)	INRM ORDINATE	SUM ORDINATE	ELAPSED TIME (Hr)	INCRM ORDINATE	SUM ORDINATE
26.83	0.0055	0.9027	35.83	0.0000	1.0984	44.83	0.0000	1.0984
27.00	0.0055	0.9082	36.00	0.0000	1.0984	45.00	0.0000	1.0984
27.17	0.0054	0.9136	36.17	0.0000	1.0984	45.17	0.0000	1.0984
27.33	0.0054	0.9190	36.33	0.0000	1.0984	45.33	0.0000	1.0984
27.50	0.0054	0.9244	36.50	0.0000	1.0984	45.50	0.0000	1.0984
27.67	0.0053	0.9297	36.67	0.0000	1.0984	45.67	0.0000	1.0984
27.83	0.0053	0.9350	36.83	0.0000	1.0984	45.83	0.0000	1.0984
28.00	0.0053	0.9403	37.00	0.0000	1.0984	46.00	0.0000	1.0984
28.17	0.0053	0.9456	37.17	0.0000	1.0984	46.17	0.0000	1.0984
28.33	0.0052	0.9508	37.33	0.0000	1.0984	46.33	0.0000	1.0984
28.50	0.0052	0.9560	37.50	0.0000	1.0984	46.50	0.0000	1.0984
28.67	0.0052	0.9612	37.67	0.0000	1.0984	46.67	0.0000	1.0984
28.83	0.0052	0.9664	37.83	0.0000	1.0984	46.83	0.0000	1.0984
29.00	0.0052	0.9716	38.00	0.0000	1.0984	47.00	0.0000	1.0984
29.17	0.0051	0.9767	38.17	0.0000	1.0984	47.17	0.0000	1.0984
29.33	0.0051	0.9818	38.33	0.0000	1.0984	47.33	0.0000	1.0984
29.50	0.0051	0.9869	38.50	0.0000	1.0984	47.50	0.0000	1.0984
29.67	0.0050	0.9919	38.67	0.0000	1.0984	47.67	0.0001	1.0985
29.83	0.0050	0.9969	38.83	0.0000	1.0984	47.83	0.0002	1.0987
30.00	0.0050	1.0019	39.00	0.0000	1.0984	48.00	0.0003	1.0990
48.17	0.0004	1.0994	56.17	0.0026	1.2422			
48.33	0.0005	1.0999	56.33	0.0024	1.2446			
48.50	0.0006	1.1005	56.50	0.0023	1.2469			
48.67	0.0007	1.1012	56.67	0.0023	1.2492			
48.83	0.0007	1.1019	56.83	0.0022	1.2514			
49.00	0.0007	1.1026	57.00	0.0021	1.2535			
49.17	0.0007	1.1033	57.17	0.0019	1.2554			
49.33	0.0007	1.1040	57.33	0.0017	1.2571			
49.50	0.0007	1.1047	57.50	0.0016	1.2587			
49.67	0.0007	1.1054	57.67	0.0015	1.2602			
49.83	0.0007	1.1061	57.83	0.0015	1.2617			
50.00	0.0007	1.1068	58.00	0.0015	1.2632			
50.17	0.0007	1.1075	58.17	0.0015	1.2647			
50.33	0.0008	1.1083	58.33	0.0015	1.2662			
50.50	0.0009	1.1092	58.50	0.0015	1.2677			
50.67	0.0010	1.1102	58.67	0.0014	1.2691			
50.83	0.0011	1.1113	58.83	0.0014	1.2705			
51.00	0.0012	1.1125	59.00	0.0013	1.2718			

Table 3 (continued). **Dimensionless Ordinates of Front-Loaded Long-Duration Design Storm.**

DIMENSIONLESS ORDINATES OF INTERMEDIATE-DURATION DESIGN STORM								
ELAPSED TIME (Hr)	INCRM ORDINATE	SUM ORDINATE	ELAPSED TIME (Hr)	INRM ORDINATE	SUM ORDINATE	ELAPSED TIME (Hr)	INCRM ORDINATE	SUM ORDINATE
51.17	0.0013	1.1138	59.17	0.0013	1.2731			
51.33	0.0014	1.1152	59.33	0.0012	1.2743			
51.50	0.0014	1.1166	59.50	0.0012	1.2755			
51.67	0.0014	1.1180	59.67	0.0011	1.2766			
51.83	0.0014	1.1194	59.83	0.0010	1.2776			
52.00	0.0015	1.1209	60.00	0.0009	1.2785			
52.17	0.0016	1.1225	60.17	0.0009	1.2794			
52.33	0.0018	1.1243	60.33	0.0008	1.2802			
52.50	0.0020	1.1263	60.50	0.0008	1.2810			
52.67	0.0021	1.1284	60.67	0.0007	1.2817			
52.83	0.0023	1.1307	60.83	0.0007	1.2824			
53.00	0.0023	1.1330	61.00	0.0007	1.2831			
53.17	0.0024	1.1354	61.17	0.0007	1.2838			
53.33	0.0026	1.1380	61.33	0.0007	1.2845			
53.50	0.0028	1.1408	61.50	0.0007	1.2852			
53.67	0.0032	1.1440	61.67	0.0007	1.2859			
53.83	0.0039	1.1479	61.83	0.0007	1.2866			
54.00	0.0048	1.1527	62.00	0.0007	1.2873			
54.17	0.0056	1.1583	62.17	0.0007	1.2880			
54.33	0.0076	1.1659	62.33	0.0007	1.2887			
54.50	0.0096	1.1755	62.50	0.0007	1.2894			
54.67	0.0133	1.1888	62.67	0.0006	1.2900			
54.83	0.0133	1.2021	62.83	0.0005	1.2905			
55.00	0.0096	1.2117	63.00	0.0004	1.2909			
55.17	0.0076	1.2193	63.17	0.0003	1.2912			
55.33	0.0056	1.2249	63.33	0.0002	1.2914			
55.50	0.0048	1.2297	63.50	0.0001	1.2915			
55.67	0.0039	1.2336	63.67	0.0000	1.2915			
55.83	0.0032	1.2368	63.83	0.0000	1.2915			
56.00	0.0028	1.2396	64.00	0.0000	1.2915			

Table 4. Dimensionless Ordinates of Back-Loaded Long-Duration Design Storm.

DIMENSIONLESS ORDINATES OF BACK-LOADED LONG-DURATION DESIGN STORM								
ELAPSED TIME (Hr)	INCRM ORDINATE	SUM ORDINATE	ELAPSED TIME (Hr)	INCRM ORDINATE	SUM ORDINATE	ELAPSED TIME (Hr)	INCRM ORDINATE	SUM ORDINATE
0.00	0.0000	0.0000	8.17	0.0039	0.1352	16.17	0.0000	0.1931
0.17	0.0001	0.0001	8.33	0.0032	0.1384	16.33	0.0000	0.1931
0.33	0.0002	0.0003	8.50	0.0028	0.1412	16.50	0.0000	0.1931
0.50	0.0003	0.0006	8.67	0.0026	0.1438	16.67	0.0000	0.1931
0.67	0.0004	0.0010	8.83	0.0024	0.1462	16.83	0.0000	0.1931
0.83	0.0005	0.0015	9.00	0.0023	0.1485	17.00	0.0000	0.1931
1.00	0.0006	0.0021	9.17	0.0023	0.1508	17.17	0.0000	0.1931
1.17	0.0007	0.0028	9.33	0.0022	0.1530	17.33	0.0000	0.1931
1.33	0.0007	0.0035	9.50	0.0021	0.1551	17.50	0.0000	0.1931
1.50	0.0007	0.0042	9.67	0.0019	0.1570	17.67	0.0000	0.1931
1.67	0.0007	0.0049	9.83	0.0017	0.1587	17.83	0.0000	0.1931
1.83	0.0007	0.0056	10.00	0.0016	0.1603	18.00	0.0000	0.1931
2.00	0.0007	0.0063	10.17	0.0015	0.1618	18.17	0.0000	0.1931
2.17	0.0007	0.0070	10.33	0.0015	0.1633	18.33	0.0000	0.1931
2.33	0.0007	0.0077	10.50	0.0015	0.1648	18.50	0.0000	0.1931
2.50	0.0007	0.0084	10.67	0.0015	0.1663	18.67	0.0000	0.1931
2.67	0.0007	0.0091	10.83	0.0015	0.1678	18.83	0.0000	0.1931
2.83	0.0008	0.0099	11.00	0.0015	0.1693	19.00	0.0000	0.1931
3.00	0.0009	0.0108	11.17	0.0014	0.1707	19.17	0.0000	0.1931
3.17	0.0010	0.0118	11.33	0.0014	0.1721	19.33	0.0000	0.1931
3.33	0.0011	0.0129	11.50	0.0013	0.1734	19.50	0.0000	0.1931
3.50	0.0012	0.0141	11.67	0.0013	0.1747	19.67	0.0000	0.1931
3.67	0.0013	0.0154	11.83	0.0012	0.1759	19.83	0.0000	0.1931
3.83	0.0014	0.0168	12.00	0.0012	0.1771	20.00	0.0000	0.1931
4.00	0.0014	0.0182	12.17	0.0011	0.1782	20.17	0.0000	0.1931
4.17	0.0014	0.0196	12.33	0.0010	0.1792	20.33	0.0000	0.1931
4.33	0.0014	0.0210	12.50	0.0009	0.1801	20.50	0.0000	0.1931
4.50	0.0015	0.0225	12.67	0.0009	0.1810	20.67	0.0000	0.1931
4.67	0.0016	0.0241	12.83	0.0008	0.1818	20.83	0.0000	0.1931
4.83	0.0018	0.0259	13.00	0.0008	0.1826	21.00	0.0000	0.1931
5.00	0.0020	0.0279	13.17	0.0007	0.1833	21.17	0.0000	0.1931
5.17	0.0021	0.0300	13.33	0.0007	0.1840	21.33	0.0000	0.1931
5.33	0.0023	0.0323	13.50	0.0007	0.1847	21.50	0.0000	0.1931
5.50	0.0023	0.0346	13.67	0.0007	0.1854	21.67	0.0000	0.1931
5.67	0.0024	0.0370	13.83	0.0007	0.1861	21.83	0.0000	0.1931
5.83	0.0026	0.0396	14.00	0.0007	0.1868	22.00	0.0000	0.1931
6.00	0.0028	0.0424	14.17	0.0007	0.1875	22.17	0.0000	0.1931
6.17	0.0032	0.0456	14.33	0.0007	0.1882	22.33	0.0000	0.1931
6.33	0.0039	0.0495	14.50	0.0007	0.1889	22.50	0.0000	0.1931
6.50	0.0048	0.0543	14.67	0.0007	0.1896	22.67	0.0000	0.1931

Table 4 (continued). Dimensionless Ordinates of Back-Loaded Long-Duration Design Storm.

DIMENSIONLESS ORDINATES OF BACK-LOADED LONG-DURATION DESIGN STORM								
ELAPSED TIME (Hr)	INCRM ORDINATE	SUM ORDINATE	ELAPSED TIME (Hr)	INCRM ORDINATE	SUM ORDINATE	ELAPSED TIME (Hr)	INCRM ORDINATE	SUM ORDINATE
6.67	0.0056	0.0599	14.83	0.0007	0.1903	22.83	0.0000	0.1931
6.83	0.0076	0.0675	15.00	0.0007	0.1910	23.00	0.0000	0.1931
7.00	0.0096	0.0771	15.17	0.0006	0.1916	23.17	0.0000	0.1931
7.17	0.0133	0.0904	15.33	0.0005	0.1921	23.33	0.0000	0.1931
7.33	0.0133	0.1037	15.50	0.0004	0.1925	23.50	0.0000	0.1931
7.50	0.0096	0.1133	15.67	0.0003	0.1928	23.67	0.0000	0.1931
7.67	0.0076	0.1209	15.83	0.0002	0.1930	23.83	0.0000	0.1931
7.83	0.0056	0.1265	16.00	0.0001	0.1931	24.00	0.0000	0.1931
8.00	0.0048	0.1313						
24.17	0.0000	0.1931	32.17	0.0014	0.2137	40.17	0.0053	0.3402
24.33	0.0000	0.1931	32.33	0.0014	0.2151	40.33	0.0053	0.3455
24.50	0.0000	0.1931	32.50	0.0014	0.2165	40.50	0.0054	0.3509
24.67	0.0000	0.1931	32.67	0.0014	0.2179	40.67	0.0054	0.3563
24.83	0.0000	0.1931	32.83	0.0014	0.2193	40.83	0.0054	0.3617
25.00	0.0000	0.1931	33.00	0.0014	0.2207	41.00	0.0055	0.3672
25.17	0.0000	0.1931	33.17	0.0014	0.2221	41.17	0.0055	0.3727
25.33	0.0000	0.1931	33.33	0.0015	0.2236	41.33	0.0056	0.3783
25.50	0.0000	0.1931	33.50	0.0015	0.2251	41.50	0.0057	0.3840
25.67	0.0000	0.1931	33.67	0.0015	0.2266	41.67	0.0058	0.3898
25.83	0.0000	0.1931	33.83	0.0015	0.2281	41.83	0.0060	0.3958
26.00	0.0000	0.1931	34.00	0.0015	0.2296	42.00	0.0062	0.4020
26.17	0.0000	0.1931	34.17	0.0015	0.2311	42.17	0.0064	0.4084
26.33	0.0000	0.1931	34.33	0.0015	0.2326	42.33	0.0066	0.4150
26.50	0.0000	0.1931	34.50	0.0015	0.2341	42.50	0.0068	0.4218
26.67	0.0000	0.1931	34.67	0.0015	0.2356	42.67	0.0069	0.4287
26.83	0.0000	0.1931	34.83	0.0015	0.2371	42.83	0.0070	0.4357
27.00	0.0000	0.1931	35.00	0.0015	0.2386	43.00	0.0072	0.4429
27.17	0.0000	0.1931	35.17	0.0015	0.2401	43.17	0.0072	0.4501
27.33	0.0000	0.1931	35.33	0.0015	0.2416	43.33	0.0073	0.4574
27.50	0.0000	0.1931	35.50	0.0016	0.2432	43.50	0.0074	0.4648
27.67	0.0000	0.1931	35.67	0.0016	0.2448	43.67	0.0075	0.4723
27.83	0.0000	0.1931	35.83	0.0017	0.2465	43.83	0.0076	0.4799
28.00	0.0000	0.1931	36.00	0.0017	0.2482	44.00	0.0077	0.4876
28.17	0.0000	0.1931	36.17	0.0018	0.2500	44.17	0.0078	0.4954
28.33	0.0000	0.1931	36.33	0.0019	0.2519	44.33	0.0078	0.5032
28.50	0.0000	0.1931	36.50	0.0019	0.2538	44.50	0.0078	0.5110
28.67	0.0000	0.1931	36.67	0.0020	0.2558	44.67	0.0079	0.5189
28.83	0.0000	0.1931	36.83	0.0022	0.2580	44.83	0.0079	0.5268
29.00	0.0000	0.1931	37.00	0.0024	0.2604	45.00	0.0079	0.5347

Table 4 (continued). Dimensionless Ordinates of Back-Loaded Long-Duration Design Storm.

DIMENSIONLESS ORDINATES OF BACK-LOADED LONG-DURATION DESIGN STORM								
ELAPSED TIME (Hr)	INCRM ORDINATE	SUM ORDINATE	ELAPSED TIME (Hr)	INCRM ORDINATE	SUM ORDINATE	ELAPSED TIME (Hr)	INCRM ORDINATE	SUM ORDINATE
29.17	0.0001	0.1932	37.17	0.0026	0.2630	45.17	0.0081	0.5428
29.33	0.0003	0.1935	37.33	0.0028	0.2658	45.33	0.0082	0.5510
29.50	0.0005	0.1940	37.50	0.0030	0.2688	45.50	0.0082	0.5592
29.67	0.0007	0.1947	37.67	0.0032	0.2720	45.67	0.0093	0.5685
29.83	0.0009	0.1956	37.83	0.0034	0.2754	45.83	0.0099	0.5784
30.00	0.0010	0.1966	38.00	0.0036	0.2790	46.00	0.0102	0.5886
30.17	0.0011	0.1977	38.17	0.0038	0.2828	46.17	0.0104	0.5990
30.33	0.0012	0.1989	38.33	0.0040	0.2868	46.33	0.0107	0.6097
30.50	0.0013	0.2002	38.50	0.0042	0.2910	46.50	0.0114	0.6211
30.67	0.0013	0.2015	38.67	0.0045	0.2955	46.67	0.0118	0.6329
30.83	0.0013	0.2028	38.83	0.0047	0.3002	46.83	0.0142	0.6471
31.00	0.0013	0.2041	39.00	0.0048	0.3050	47.00	0.0220	0.6691
31.17	0.0013	0.2054	39.17	0.0049	0.3099	47.17	0.0290	0.6981
31.33	0.0013	0.2067	39.33	0.0049	0.3148	47.33	0.0160	0.7141
31.50	0.0014	0.2081	39.50	0.0049	0.3197	47.50	0.0127	0.7268
31.67	0.0014	0.2095	39.67	0.0050	0.3247	47.67	0.0116	0.7384
31.83	0.0014	0.2109	39.83	0.0051	0.3298	47.83	0.0110	0.7494
32.00	0.0014	0.2123	40.00	0.0051	0.3349	48.00	0.0106	0.7600
48.17	0.0102	0.7702	56.17	0.0054	1.1067			
48.33	0.0096	0.7798	56.33	0.0054	1.1121			
48.50	0.0082	0.7880	56.50	0.0054	1.1175			
48.67	0.0082	0.7962	56.67	0.0053	1.1228			
48.83	0.0082	0.8044	56.83	0.0053	1.1281			
49.00	0.0081	0.8125	57.00	0.0053	1.1334			
49.17	0.0080	0.8205	57.17	0.0053	1.1387			
49.33	0.0079	0.8284	57.33	0.0052	1.1439			
49.50	0.0079	0.8363	57.50	0.0052	1.1491			
49.67	0.0078	0.8441	57.67	0.0052	1.1543			
49.83	0.0078	0.8519	57.83	0.0052	1.1595			
50.00	0.0077	0.8596	58.00	0.0052	1.1647			
50.17	0.0077	0.8673	58.17	0.0051	1.1698			
50.33	0.0077	0.8750	58.33	0.0051	1.1749			
50.50	0.0077	0.8827	58.50	0.0051	1.1800			
50.67	0.0076	0.8903	58.67	0.0050	1.1850			
50.83	0.0075	0.8978	58.83	0.0050	1.1900			
51.00	0.0075	0.9053	59.00	0.0050	1.1950			
51.17	0.0074	0.9127	59.17	0.0050	1.2000			
51.33	0.0074	0.9201	59.33	0.0049	1.2049			
51.50	0.0073	0.9274	59.50	0.0049	1.2098			

Table 4 (continued). Dimensionless Ordinates of Back-Loaded Long-Duration Design Storm.

DIMENSIONLESS ORDINATES OF BACK-LOADED LONG-DURATION DESIGN STORM								
ELAPSED TIME (Hr)	INCRM ORDINATE	SUM ORDINATE	ELAPSED TIME (Hr)	INCRM ORDINATE	SUM ORDINATE	ELAPSED TIME (Hr)	INCRM ORDINATE	SUM ORDINATE
51.67	0.0073	0.9347	59.67	0.0049	1.2147			
51.83	0.0073	0.9420	59.83	0.0049	1.2196			
52.00	0.0072	0.9492	60.00	0.0048	1.2244			
52.17	0.0072	0.9564	60.17	0.0048	1.2292			
52.33	0.0072	0.9636	60.33	0.0048	1.2340			
52.50	0.0071	0.9707	60.50	0.0047	1.2387			
52.67	0.0071	0.9778	60.67	0.0046	1.2433			
52.83	0.0070	0.9848	60.83	0.0045	1.2478			
53.00	0.0070	0.9918	61.00	0.0044	1.2522			
53.17	0.0069	0.9987	61.17	0.0043	1.2565			
53.33	0.0068	1.0055	61.33	0.0042	1.2607			
53.50	0.0067	1.0122	61.50	0.0041	1.2648			
53.67	0.0067	1.0189	61.67	0.0039	1.2687			
53.83	0.0066	1.0255	61.83	0.0038	1.2725			
54.00	0.0065	1.0320	62.00	0.0037	1.2762			
54.17	0.0062	1.0382	62.17	0.0033	1.2795			
54.33	0.0062	1.0444	62.33	0.0029	1.2824			
54.50	0.0060	1.0504	62.50	0.0025	1.2849			
54.67	0.0059	1.0563	62.67	0.0021	1.2870			
54.83	0.0059	1.0622	62.83	0.0017	1.2887			
55.00	0.0058	1.0680	63.00	0.0013	1.2900			
55.17	0.0057	1.0737	63.17	0.0009	1.2909			
55.33	0.0056	1.0793	63.33	0.0005	1.2914			
55.50	0.0055	1.0848	63.50	0.0001	1.2915			
55.67	0.0055	1.0903	63.67	0.0000	1.2915			
55.83	0.0055	1.0958	63.83	0.0000	1.2915			
56.00	0.0055	1.1013	64.00	0.0000	1.2915			

Table 5. Dimensionless Ordinates of 24-Hour Design Storm.

DIMENSIONLESS ORDINATES OF 24-HOUR DESIGN STORM								
ELAPSED TIME (Hr)	INCRM ORDINATE	SUM ORDINATE	ELAPSED TIME (Hr)	INCRM ORDINATE	SUM ORDINATE	ELAPSED TIME (Hr)	INCRM ORDINATE	SUM ORDINATE
0.00	0.0000	0.0000	7.17	0.0080	0.2596	14.17	0.0072	0.6769
0.17	0.0036	0.0036	7.33	0.0082	0.2678	14.33	0.0072	0.6841
0.33	0.0038	0.0074	7.50	0.0084	0.2762	14.50	0.0072	0.6913
0.50	0.0040	0.0114	7.67	0.0088	0.2850	14.67	0.0071	0.6984
0.67	0.0042	0.0156	7.83	0.0093	0.2943	14.83	0.0071	0.7055
0.83	0.0045	0.0201	8.00	0.0099	0.3042	15.00	0.0070	0.7125
1.00	0.0047	0.0248	8.17	0.0102	0.3144	15.17	0.0070	0.7195
1.17	0.0048	0.0296	8.33	0.0104	0.3248	15.33	0.0069	0.7264
1.33	0.0049	0.0345	8.50	0.0107	0.3355	15.50	0.0068	0.7332
1.50	0.0049	0.0394	8.67	0.0114	0.3469	15.67	0.0067	0.7399
1.67	0.0049	0.0443	8.83	0.0127	0.3596	15.83	0.0066	0.7465
1.83	0.0050	0.0493	9.00	0.0142	0.3738	16.00	0.0065	0.7530
2.00	0.0051	0.0544	9.17	0.0220	0.3958	16.17	0.0064	0.7594
2.17	0.0051	0.0595	9.33	0.0290	0.4248	16.33	0.0063	0.7657
2.33	0.0053	0.0648	9.50	0.0160	0.4408	16.50	0.0062	0.7719
2.50	0.0053	0.0701	9.67	0.0127	0.4535	16.67	0.0060	0.7779
2.67	0.0054	0.0755	9.83	0.0116	0.4651	16.83	0.0059	0.7838
2.83	0.0054	0.0809	10.00	0.0110	0.4761	17.00	0.0059	0.7897
3.00	0.0054	0.0863	10.17	0.0106	0.4867	17.17	0.0058	0.7955
3.17	0.0055	0.0918	10.33	0.0102	0.4969	17.33	0.0057	0.8012
3.33	0.0055	0.0973	10.50	0.0096	0.5065	17.50	0.0056	0.8068
3.50	0.0056	0.1029	10.67	0.0089	0.5154	17.67	0.0055	0.8123
3.67	0.0057	0.1086	10.83	0.0085	0.5239	17.83	0.0055	0.8178
3.83	0.0058	0.1144	11.00	0.0083	0.5322	18.00	0.0055	0.8233
4.00	0.0060	0.1204	11.17	0.0082	0.5404	18.17	0.0055	0.8288
4.17	0.0062	0.1266	11.33	0.0081	0.5485	18.33	0.0054	0.8342
4.33	0.0064	0.1330	11.50	0.0080	0.5565	18.50	0.0054	0.8396
4.50	0.0066	0.1396	11.67	0.0079	0.5644	18.67	0.0054	0.8450
4.67	0.0068	0.1464	11.83	0.0078	0.5722	18.83	0.0053	0.8503
4.83	0.0069	0.1533	12.00	0.0078	0.5800	19.00	0.0053	0.8556
5.00	0.0070	0.1603	12.17	0.0077	0.5877	19.17	0.0053	0.8609
5.17	0.0072	0.1675	12.33	0.0077	0.5954	19.33	0.0053	0.8662
5.33	0.0072	0.1747	12.50	0.0076	0.6030	19.50	0.0052	0.8714
5.50	0.0073	0.1820	12.67	0.0076	0.6106	19.67	0.0052	0.8766
5.67	0.0074	0.1894	12.83	0.0075	0.6181	19.83	0.0052	0.8818
5.83	0.0075	0.1969	13.00	0.0075	0.6256	20.00	0.0052	0.8870
6.00	0.0076	0.2045	13.17	0.0074	0.6330	20.17	0.0052	0.8922
6.17	0.0077	0.2122	13.33	0.0074	0.6404	20.33	0.0051	0.8973
6.33	0.0078	0.2200	13.50	0.0074	0.6478	20.50	0.0051	0.9024
6.50	0.0078	0.2278	13.67	0.0073	0.6551	20.67	0.0051	0.9075

Table 5 (continued). Dimensionless Ordinates of 24-Hour Design Storm.

DIMENSIONLESS ORDINATES OF 24-HOUR DESIGN STORM								
ELAPSED TIME (Hr)	INCRM ORDINATE	SUM ORDINATE	ELAPSED TIME (Hr)	INCRM ORDINATE	SUM ORDINATE	ELAPSED TIME (Hr)	INCRM ORDINATE	SUM ORDINATE
6.67	0.0079	0.2357	13.83	0.0073	0.6624	20.83	0.0050	0.9125
6.83	0.0079	0.2436	14.00	0.0073	0.6697	21.00	0.0050	0.9175
7.00	0.0080	0.2516						
21.17	0.0050	0.9225						
21.33	0.0050	0.9275						
21.50	0.0049	0.9324						
21.67	0.0049	0.9373						
21.83	0.0049	0.9422						
22.00	0.0049	0.9471						
22.17	0.0048	0.9519						
22.33	0.0048	0.9567						
22.50	0.0048	0.9615						
22.67	0.0047	0.9662						
22.83	0.0046	0.9708						
23.00	0.0045	0.9753						
23.17	0.0044	0.9797						
23.33	0.0043	0.9840						
23.50	0.0042	0.9882						
23.67	0.0041	0.9923						
23.83	0.0039	0.9962						
24.00	0.0038	1.0000						

Attachment 2 –

Precipitation Magnitude-Frequency Estimates for SPU Rain Gage Locations (up to 2003 data only)

Attachment 2 - Precipitation Magnitude-Frequency Estimates for SPU Rain Gage Locations (up to 2003 data only)

This appendix contains adapted text and excerpted tables and figures from *Analysis of Precipitation-Frequency and Storm Characteristics for the City of Seattle* (MGS Engineering Consultants, Inc. for Seattle Public Utilities, December 2003). The analysis presented here is from rain gage data ending in 2003. Analysis of data from later years was not available at the time of publication of the 2008 Directors' Rules. Updated information may be obtained from the SPU Rain Gage Network Data Steward as it becomes available.

The results of homogeneity analyses indicate that at-site mean values for precipitation do not vary across the Seattle Metropolitan Area for durations of 3 hours and less. Accordingly, one set of intensity-duration-frequency (IDF) curves can be developed that are applicable to the Seattle Metropolitan Area. Table 1 and Figures 1 and 2 provide precipitation intensities and IDF curves representative of the Seattle Metropolitan Area for durations from 5 to 180 minutes.

Table 1. Intensity-Duration-Frequency Values for Durations from 5-Minutes through 180-Minutes for Selected Recurrence Intervals for the Seattle Metropolitan Area.

DURATION (minutes)	PRECIPITATION INTENSITIES (in/hr)							
	RECURRENCE INTERVAL (Years)							
	6-Month	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
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

Table 2. Two-hour Precipitation Magnitude-Frequency Values for Selected Recurrence Intervals for the Seattle Metropolitan Area.

Recurrence Interval	2-Hour Total (inches)
6-month	0.40
2-yr	0.58
5-yr	0.70
10-yr	0.78
20-yr	0.88
25-yr	0.92
50-yr	1.04
100-yr	1.14

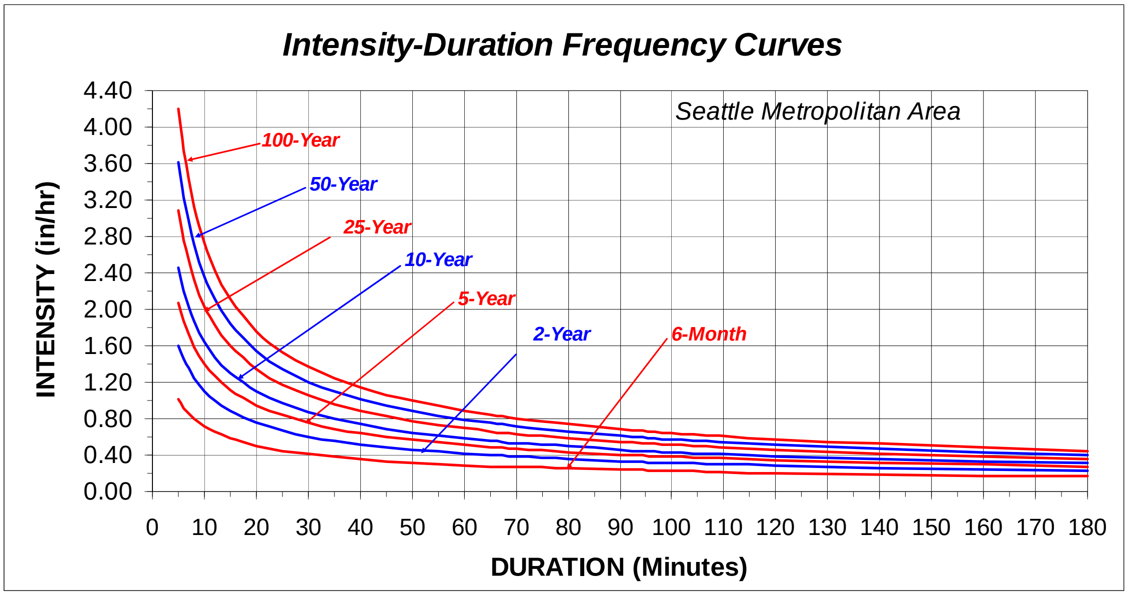

Figure 1. Intensity-Duration-Frequency Curves for the Seattle Metropolitan Area.

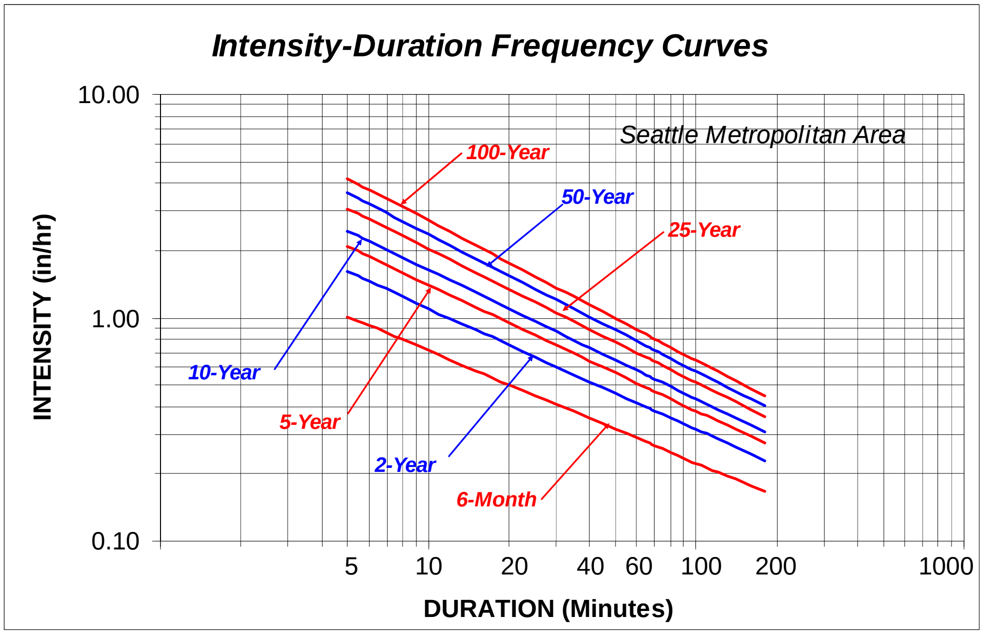

Figure 2. Intensity-Duration-Frequency Curves for the Seattle Metropolitan Area.

The following tables and figures contain estimates of precipitation-frequency values for durations of 6 hours, 12 hours, 24 hours, 48 hours, and 7 days for locations of SPU precipitation gages (Table 2) in both tabular format and as magnitude-frequency curves. These precipitation values are based on estimates of the at-site mean values for the location of SPU gages (Table 3) based on the spatial analysis of precipitation (gridded datasets) and the applicable regional growth curves obtained from the regional frequency analyses. Corrections have been applied to provide equivalent partial duration series estimates for frequently occurring events (5 times/year, 2 times/year, once/year, 2-year, and 5-year recurrence intervals).

Table 3. Listing of City of Seattle (SPU) Precipitation Gages.

Station ID	Station Name	Latitude	Longitude	Year Start	Year End	Gage Type
45-S001	Haller Lake Shop	47.7211	122.3431	1965	2003	TB
45-S002	Magnusson Park	47.6950	122.2731	1969	2003	TB
45-S003	UW Hydraulics Lab	47.6481	122.3081	1965	2003	TB
45-S004	Maple Leaf Reservoir	47.6900	122.3119	1965	2003	TB
45-S005	Fauntleroy Ferry Dock	47.5231	122.3919	1968	2003	TB
45-S007	Whitman Middle School	47.6961	122.3769	1965	2003	TB
45-S008	Ballard Locks	47.6650	122.3969	1965	2003	TB
45-S009	Woodland Park Zoo	47.6681	122.3539	1965	2003	TB
45-S010	Rainier View Elementary	47.5000	122.2600	1968	2003	TB
45-S011	Metro-KC Denny Regulating	47.6169	122.3550	1970	2003	TB

45-S012	Catherine Blaine Elementary School	47.6419	122.3969	1965	2003	TB
45-S014	Lafayette Elementary School	47.5781	122.3819	1965	2003	TB
45-S015	Puget Sound Clean Air Monitoring Station	47.5619	122.3400	1965	2003	TB
45-S016	Metro-KC E Marginal Way	47.5350	122.3139	1970	2003	TB
45-S017	West Seattle Reservoir Treatment Shop	47.5211	122.3450	1965	2003	TB
45-S018	Aki Kurose Middle School	47.5481	122.2750	1965	2003	TB
45-S020	TT Minor Elementary	47.6119	122.3069	1975	2003	TB
45-7473	Seattle Tacoma Airport	47.4500	122.3000	1965	2002	HR

Table 4. Listing of At-Site Mean Values for City of Seattle (SPU) Precipitation Gages.

At-Site Mean Values (in)							
Station ID	**Station Name**	**6-Hr**	**12-Hr**	**24-Hr**	**48-Hr**	**72-Hr**	**7-Day**
45-S001	Haller Lake Shop	1.00	1.44	1.87	2.56	2.91	4.10
45-S002	Magnusson Park	1.00	1.43	1.85	2.55	2.89	4.07
45-S003	UW Hydraulics Lab	1.01	1.45	1.90	2.60	2.95	4.18
45-S004	Maple Leaf Reservoir	1.00	1.44	1.87	2.57	2.91	4.11
45-S005	Fauntleroy Ferry Dock	1.06	1.58	2.14	2.89	3.32	4.80
45-S007	Whitman Middle School	1.01	1.45	1.89	2.59	2.94	4.16
45-S008	Ballard Locks	1.03	1.50	1.99	2.71	3.08	4.41
45-S009	Woodland Park Zoo	1.01	1.45	1.89	2.59	2.94	4.16
45-S010	Rainier View Elementary	1.02	1.47	1.94	2.65	3.01	4.28
45-S011	Metro-KC Denny Regulating	1.01	1.46	1.91	2.61	2.97	4.21
45-S012	Catherine Blaine Elementary School	1.03	1.50	1.99	2.71	3.09	4.41
45-S014	Lafayette Elementary School	1.03	1.51	2.00	2.73	3.11	4.44
45-S015	Puget Sound Clean Air Monitoring Station	1.01	1.46	1.91	2.61	2.96	4.20
45-S016	Metro-KC E Marginal Way	1.02	1.47	1.94	2.65	3.02	4.29
45-S017	West Seattle Reservoir Treatment Shop	1.03	1.51	2.02	2.74	3.13	4.48
45-S018	Aki Kurose Middle School	1.01	1.46	1.91	2.61	2.97	4.21
45-S020	TT Minor Elementary	1.00	1.44	1.88	2.58	2.92	4.12
45-7473	Seattle Tacoma Airport	1.04	1.54	2.06	2.80	3.20	4.60

Table 5. Precipitation-Magnitude-Frequency Estimates for of SPU Gage 01.

Duration (hr)	Precipitation (in)									
	Recurrence Interval (years)									
	0.2-Yr	0.5-Yr	1-Yr	2-Yr	5-Yr	10-Yr	20-Yr	25-Yr	50-Yr	100-Yr
6	0.58	0.75	0.88	1.02	1.20	1.33	1.48	1.52	1.67	1.82
12	0.76	1.05	1.26	1.47	1.76	1.96	2.19	2.27	2.49	2.72
24	0.93	1.32	1.61	1.91	2.31	2.61	2.94	3.04	3.37	3.71
48	1.34	1.84	2.22	2.61	3.14	3.53	3.97	4.11	4.56	5.02
72	1.53	2.11	2.53	2.97	3.56	3.98	4.47	4.62	5.10	5.58
168	2.11	3.00	3.62	4.23	5.01	5.53	6.10	6.28	6.81	7.32

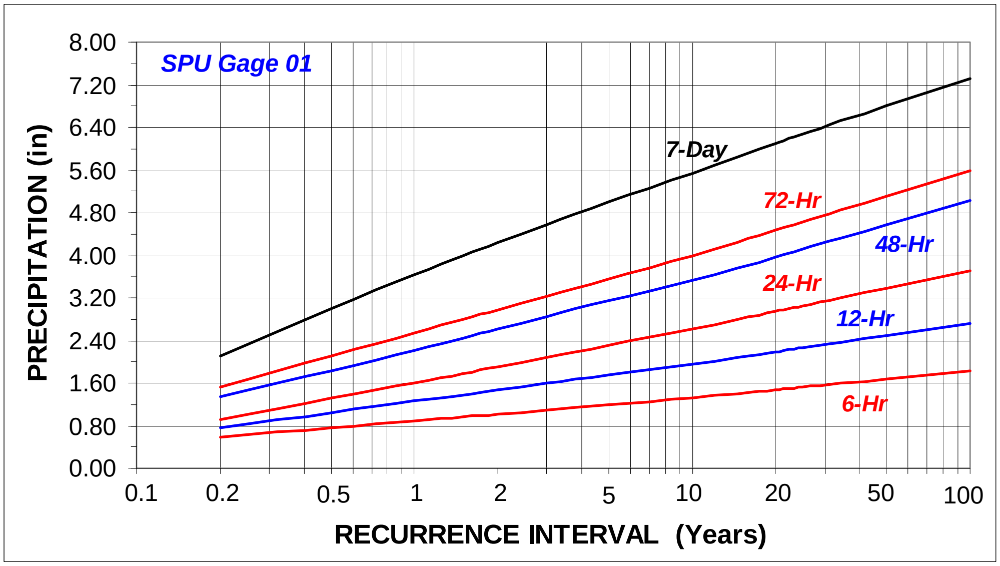

Figure 3. Precipitation-Magnitude-Frequency Estimates for of SPU Gage 01.

Table 6. Precipitation-Magnitude-Frequency Estimates for SPU Gage 02.

Duration (hr)	Precipitation (in)									
	Recurrence Interval (years)									
	0.2-Yr	0.5-Yr	1-Yr	2-Yr	5-Yr	10-Yr	20-Yr	25-Yr	50-Yr	100-Yr
6	0.58	0.75	0.88	1.02	1.20	1.33	1.48	1.52	1.67	1.82
12	0.76	1.04	1.25	1.46	1.75	1.95	2.18	2.25	2.48	2.70
24	0.92	1.31	1.59	1.89	2.29	2.58	2.90	3.01	3.34	3.67
48	1.33	1.83	2.21	2.60	3.13	3.51	3.95	4.10	4.55	5.00
72	1.52	2.09	2.51	2.95	3.54	3.96	4.43	4.59	5.06	5.55
168	2.09	2.98	3.60	4.20	4.97	5.49	6.06	6.23	6.76	7.27

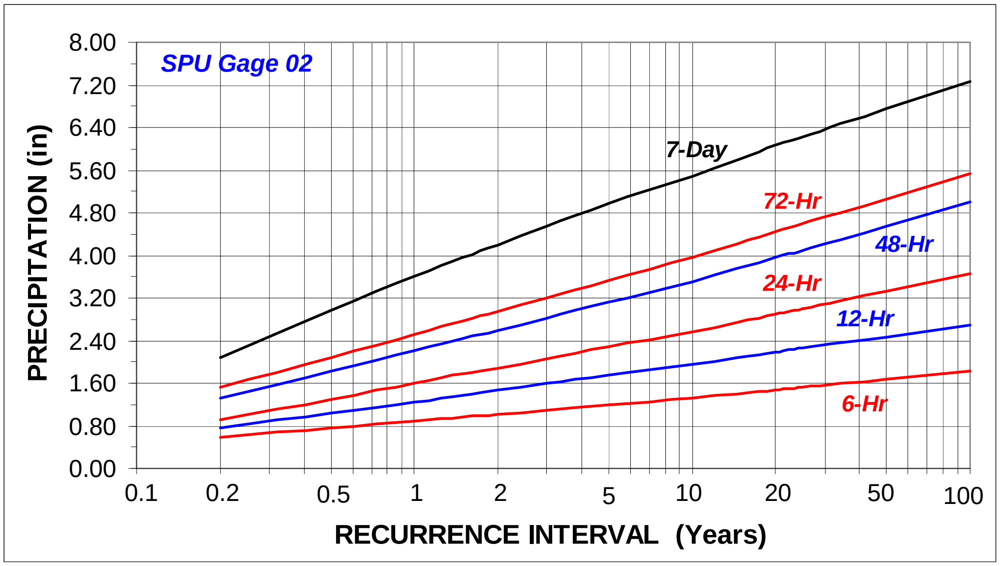

Figure 4. Precipitation-Magnitude-Frequency Estimates for SPU Gage 02.

Table 7. Precipitation-Magnitude-Frequency Estimates for SPU Gage 03.

Duration (hr)	Precipitation (in)									
	Recurrence Interval (years)									
	0.2-Yr	0.5-Yr	1-Yr	2-Yr	5-Yr	10-Yr	20-Yr	25-Yr	50-Yr	100-Yr
6	0.58	0.76	0.89	1.03	1.21	1.34	1.49	1.54	1.69	1.84
12	0.77	1.06	1.27	1.48	1.77	1.97	2.21	2.28	2.51	2.74
24	0.94	1.34	1.64	1.94	2.35	2.65	2.98	3.09	3.43	3.77
48	1.36	1.87	2.25	2.65	3.19	3.58	4.03	4.18	4.63	5.10
72	1.55	2.14	2.57	3.01	3.61	4.04	4.53	4.68	5.17	5.66
168	2.15	3.06	3.69	4.32	5.10	5.64	6.22	6.40	6.94	7.47

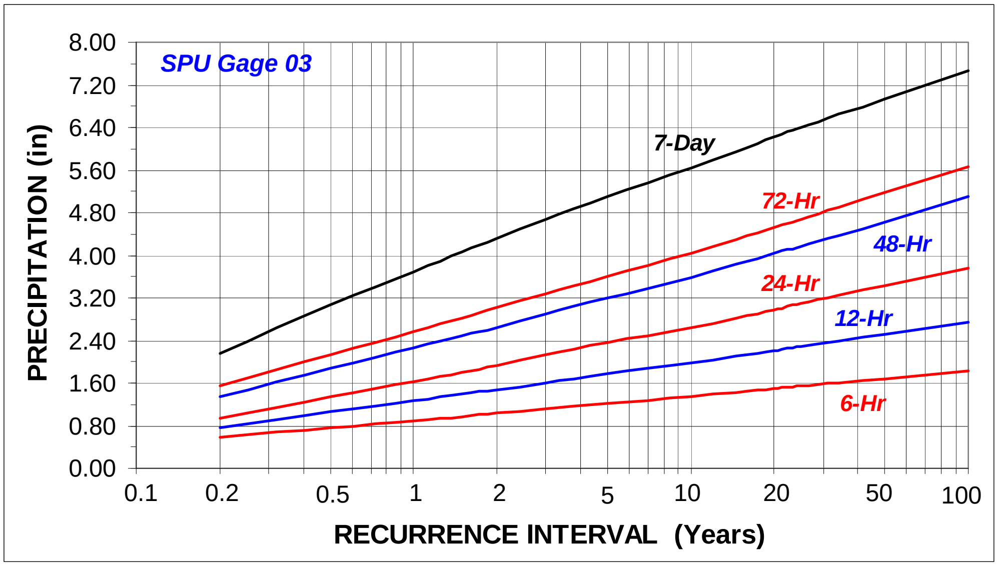

Figure 5. Precipitation-Magnitude-Frequency Estimates for SPU Gage 03.

Table 8. Precipitation-Magnitude-Frequency Estimates for SPU Gage 04.

Duration (hr)	Precipitation (in)									
	Recurrence Interval (years)									
	0.2-Yr	0.5-Yr	1-Yr	2-Yr	5-Yr	10-Yr	20-Yr	25-Yr	50-Yr	100-Yr
6	0.58	0.75	0.88	1.02	1.20	1.33	1.48	1.52	1.67	1.82
12	0.76	1.05	1.26	1.47	1.76	1.96	2.19	2.27	2.49	2.72
24	0.93	1.32	1.61	1.91	2.31	2.61	2.94	3.04	3.37	3.71
48	1.34	1.85	2.22	2.62	3.15	3.54	3.99	4.13	4.58	5.04
72	1.53	2.11	2.53	2.97	3.56	3.98	4.47	4.62	5.10	5.58
168	2.11	3.01	3.63	4.24	5.02	5.55	6.12	6.29	6.83	7.34

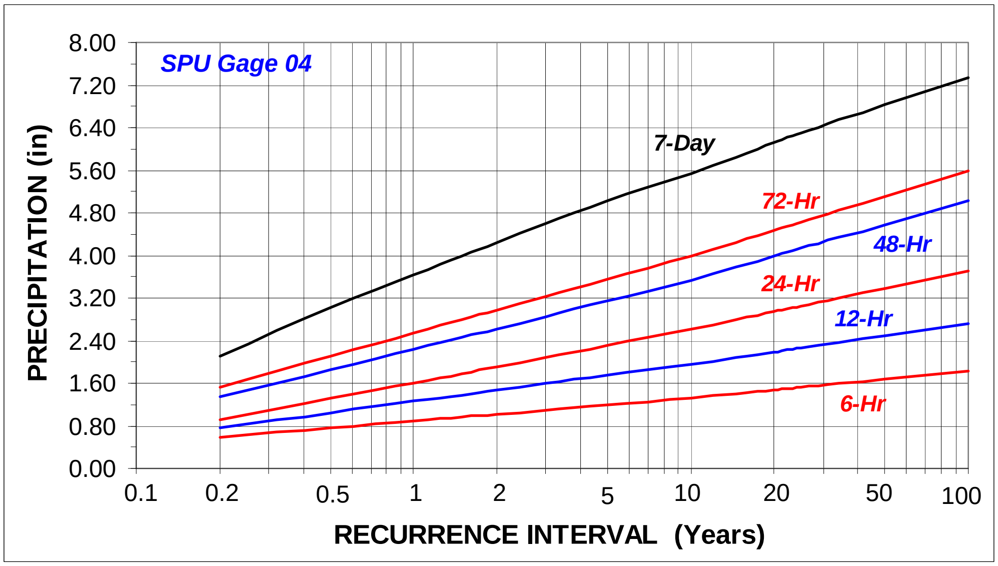

Figure 6. Precipitation-Magnitude-Frequency Estimates for SPU Gage 04.

Table 9. Precipitation-Magnitude-Frequency Estimates for SPU Gage 05.

Duration (hr)	Precipitation (in)									
	Recurrence Interval (years)									
	0.2-Yr	0.5-Yr	1-Yr	2-Yr	5-Yr	10-Yr	20-Yr	25-Yr	50-Yr	100-Yr
6	0.61	0.80	0.94	1.08	1.27	1.41	1.57	1.62	1.77	1.93
12	0.84	1.15	1.38	1.61	1.93	2.15	2.40	2.49	2.74	2.99
24	1.06	1.51	1.84	2.19	2.65	2.98	3.36	3.48	3.86	4.24
48	1.51	2.08	2.50	2.94	3.54	3.98	4.48	4.64	5.15	5.67
72	1.74	2.40	2.89	3.39	4.06	4.55	5.09	5.27	5.82	6.37
168	2.47	3.52	4.24	4.96	5.86	6.48	7.14	7.35	7.97	8.57

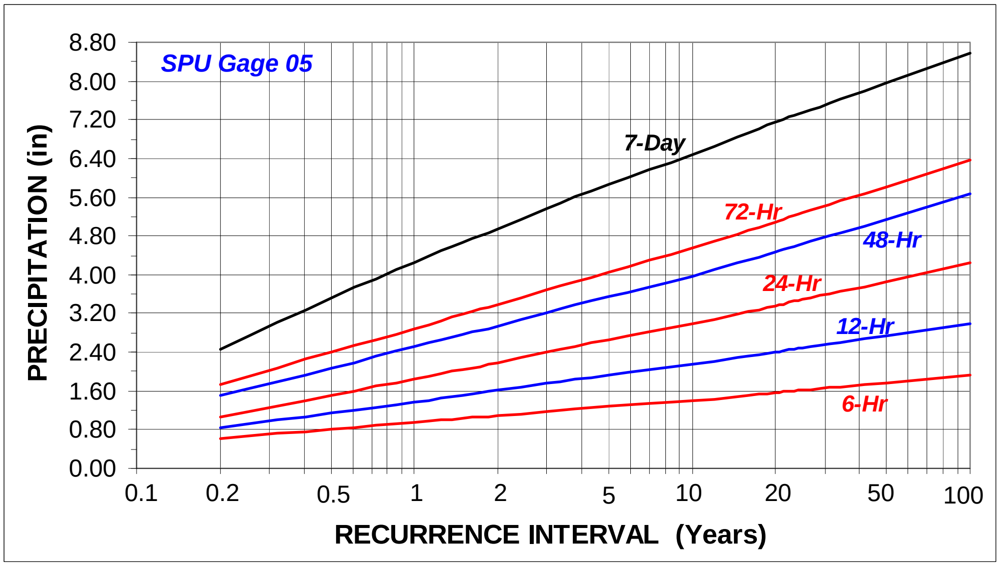

Figure 7. Precipitation-Magnitude-Frequency Estimates for SPU Gage 05.

Table 10. Precipitation-Magnitude-Frequency Estimates for SPU Gage 07.

Duration (hr)	Precipitation (in)									
	Recurrence Interval (years)									
	0.2-Yr	0.5-Yr	1-Yr	2-Yr	5-Yr	10-Yr	20-Yr	25-Yr	50-Yr	100-Yr
6	0.58	0.76	0.89	1.03	1.21	1.34	1.49	1.54	1.69	1.84
12	0.77	1.06	1.27	1.48	1.77	1.97	2.21	2.28	2.51	2.74
24	0.94	1.33	1.63	1.93	2.34	2.63	2.97	3.07	3.41	3.75
48	1.35	1.86	2.24	2.64	3.18	3.57	4.02	4.16	4.62	5.08
72	1.54	2.13	2.56	3.00	3.60	4.03	4.51	4.67	5.15	5.64
168	2.14	3.05	3.68	4.30	5.08	5.61	6.19	6.37	6.91	7.43

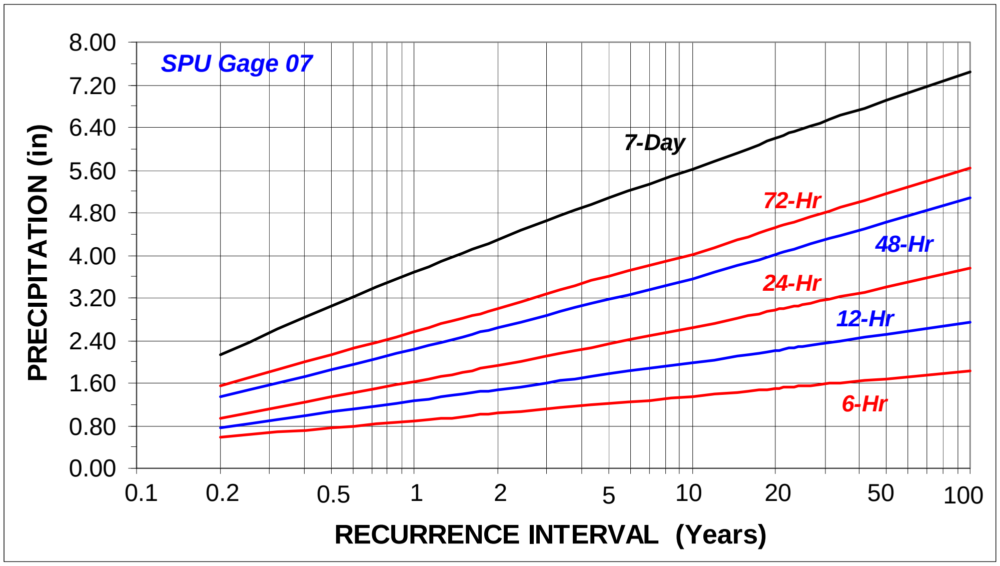

Figure 8. Precipitation-Magnitude-Frequency Estimates for SPU Gage 07.

Table 11. Precipitation-Magnitude-Frequency Estimates for SPU Gage 08.

Duration (hr)	Precipitation (in)									
	Recurrence Interval (years)									
	0.2-Yr	0.5-Yr	1-Yr	2-Yr	5-Yr	10-Yr	20-Yr	25-Yr	50-Yr	100-Yr
6	0.59	0.78	0.91	1.05	1.24	1.37	1.52	1.57	1.72	1.87
12	0.80	1.09	1.31	1.53	1.83	2.04	2.28	2.36	2.60	2.84
24	0.98	1.41	1.71	2.03	2.46	2.77	3.12	3.24	3.59	3.94
48	1.41	1.95	2.35	2.76	3.32	3.73	4.20	4.35	4.83	5.32
72	1.62	2.23	2.68	3.14	3.77	4.22	4.73	4.89	5.40	5.91
168	2.27	3.23	3.90	4.55	5.39	5.95	6.56	6.75	7.33	7.88

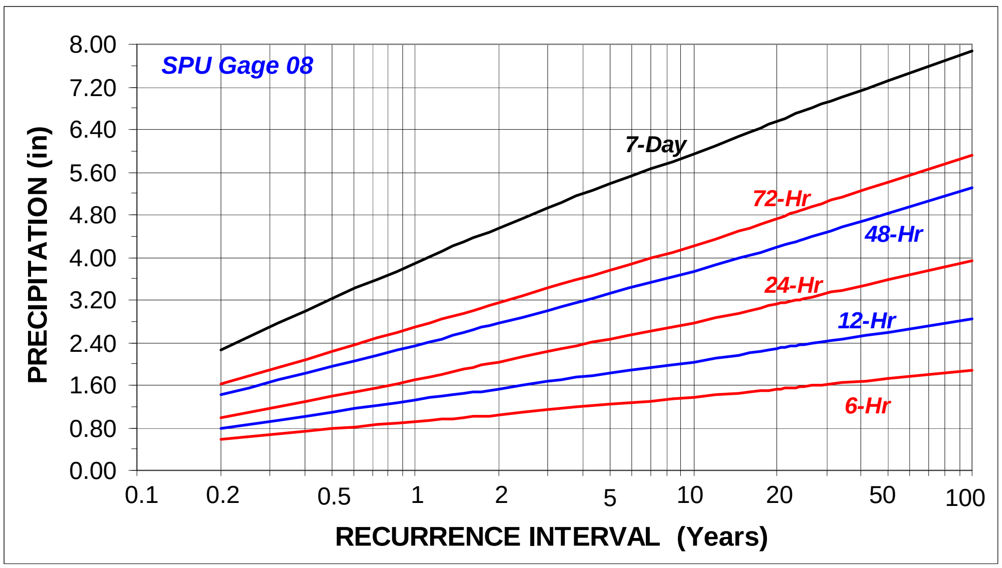

Figure 9. Precipitation-Magnitude-Frequency Estimates for SPU Gage 08.

Table 12. Precipitation-Magnitude-Frequency Estimates for SPU Gage 09.

Duration (hr)	Precipitation (in)									
	Recurrence Interval (years)									
	0.2-Yr	0.5-Yr	1-Yr	2-Yr	5-Yr	10-Yr	20-Yr	25-Yr	50-Yr	100-Yr
6	0.58	0.76	0.89	1.03	1.21	1.34	1.49	1.54	1.69	1.84
12	0.77	1.06	1.27	1.48	1.77	1.97	2.21	2.28	2.51	2.74
24	0.94	1.33	1.63	1.93	2.34	2.63	2.97	3.07	3.41	3.75
48	1.35	1.86	2.24	2.64	3.18	3.57	4.02	4.16	4.62	5.08
72	1.54	2.13	2.56	3.00	3.60	4.03	4.51	4.67	5.15	5.64
168	2.14	3.05	3.68	4.30	5.08	5.61	6.19	6.37	6.91	7.43

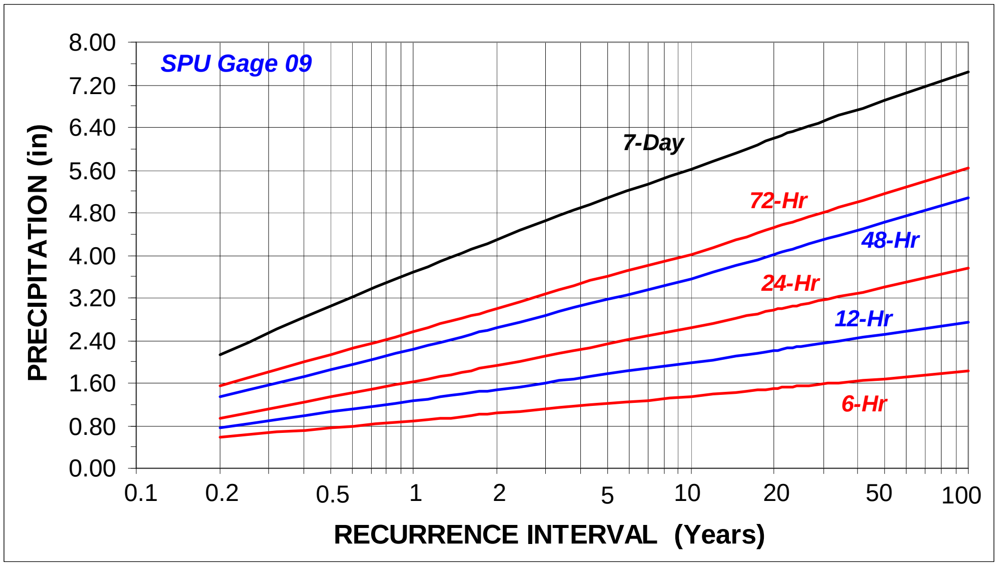

Figure 10. Precipitation-Magnitude-Frequency Estimates for SPU Gage 09.

Table 13. Precipitation-Magnitude-Frequency Estimates for SPU Gage 10.

Duration (hr)	Precipitation (in)									
	Recurrence Interval (years)									
	0.2-Yr	0.5-Yr	1-Yr	2-Yr	5-Yr	10-Yr	20-Yr	25-Yr	50-Yr	100-Yr
6	0.59	0.77	0.90	1.04	1.22	1.36	1.51	1.55	1.70	1.86
12	0.78	1.07	1.28	1.50	1.79	2.00	2.24	2.31	2.55	2.78
24	0.96	1.37	1.67	1.98	2.40	2.70	3.05	3.16	3.50	3.85
48	1.38	1.91	2.29	2.70	3.25	3.65	4.11	4.26	4.72	5.20
72	1.58	2.18	2.62	3.07	3.68	4.12	4.62	4.78	5.27	5.78
168	2.20	3.14	3.78	4.42	5.23	5.78	6.37	6.55	7.11	7.64

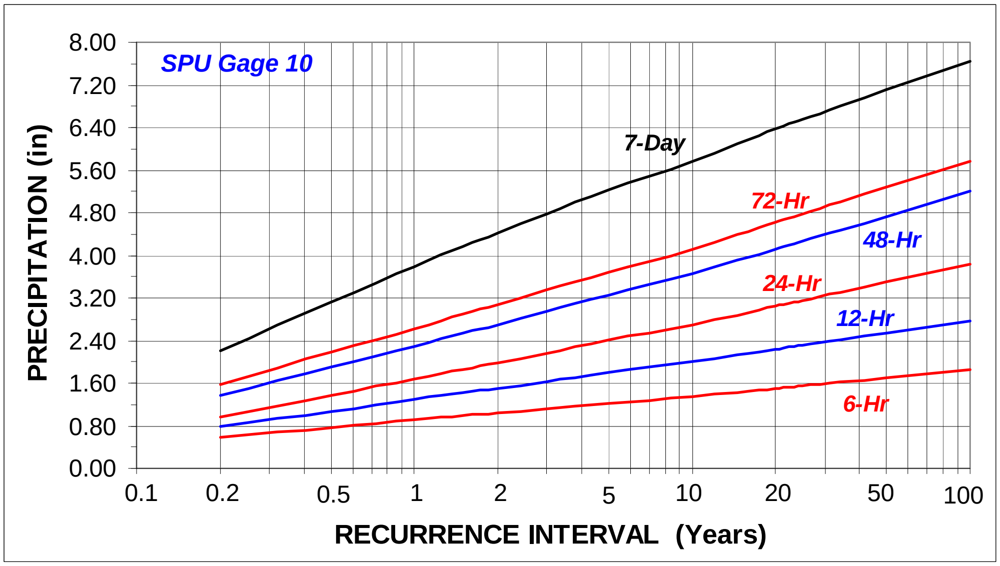

Figure E-11. Precipitation-Magnitude-Frequency Estimates for SPU Gage 10.

Table 14. Precipitation-Magnitude-Frequency Estimates for SPU Gage 11.

Duration (hr)	Precipitation (in)									
	Recurrence Interval (years)									
	0.2-Yr	0.5-Yr	1-Yr	2-Yr	5-Yr	10-Yr	20-Yr	25-Yr	50-Yr	100-Yr
6	0.58	0.76	0.89	1.03	1.21	1.34	1.49	1.54	1.69	1.84
12	0.77	1.06	1.27	1.49	1.78	1.99	2.22	2.30	2.53	2.76
24	0.95	1.35	1.64	1.95	2.36	2.66	3.00	3.11	3.44	3.79
48	1.36	1.88	2.26	2.66	3.20	3.60	4.05	4.19	4.65	5.12
72	1.56	2.15	2.58	3.03	3.63	4.07	4.56	4.71	5.20	5.70
168	2.16	3.08	3.72	4.35	5.14	5.68	6.27	6.45	6.99	7.52

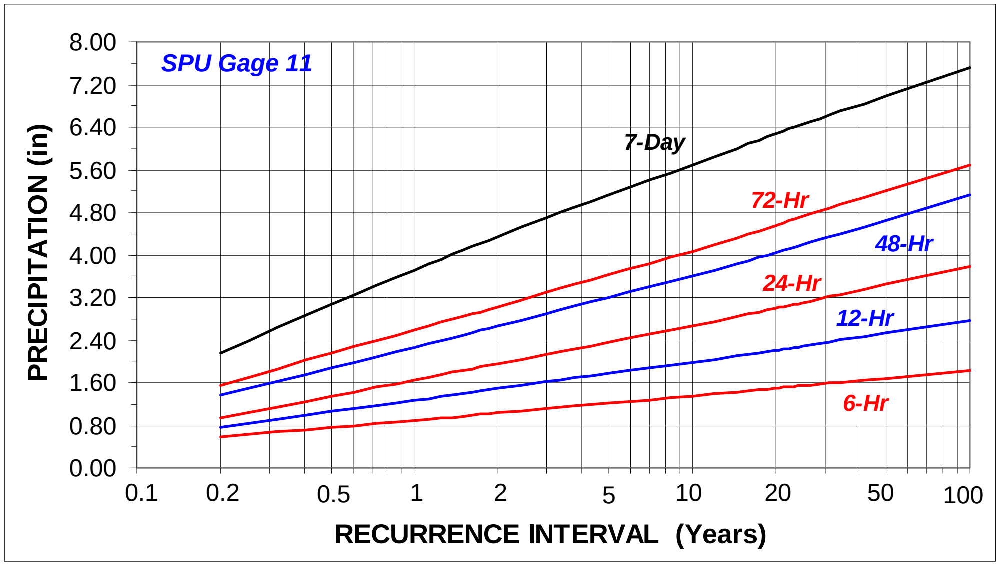

Figure E-11. Precipitation-Magnitude-Frequency Estimates for SPU Gage 11.

Table 15. Precipitation-Magnitude-Frequency Estimates for SPU Gage 12.

Duration (hr)	Precipitation (in)									
	Recurrence Interval (years)									
	0.2-Yr	0.5-Yr	1-Yr	2-Yr	5-Yr	10-Yr	20-Yr	25-Yr	50-Yr	100-Yr
6	0.59	0.78	0.91	1.05	1.24	1.37	1.52	1.57	1.72	1.87
12	0.80	1.09	1.31	1.53	1.83	2.04	2.28	2.36	2.60	2.84
24	0.98	1.41	1.71	2.03	2.46	2.77	3.12	3.24	3.59	3.94
48	1.41	1.95	2.35	2.76	3.32	3.73	4.20	4.35	4.83	5.32
72	1.62	2.24	2.69	3.16	3.78	4.23	4.74	4.90	5.41	5.93
168	2.27	3.23	3.90	4.55	5.39	5.95	6.56	6.75	7.33	7.88

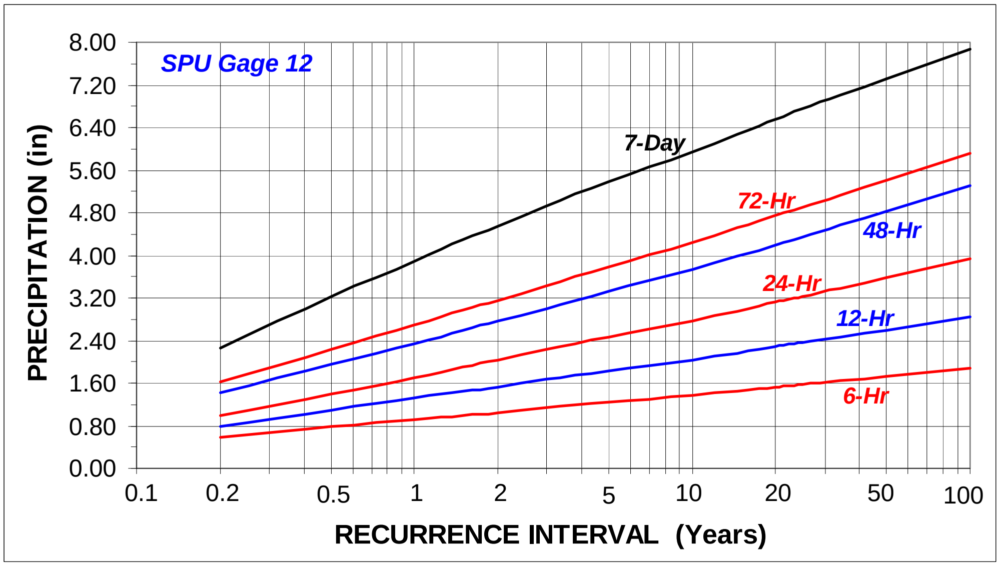

Figure 13. Precipitation-Magnitude-Frequency Estimates for SPU Gage 12.

Table 16. Precipitation-Magnitude-Frequency Estimates for SPU Gage 14.

Duration (hr)	Precipitation (in)									
	Recurrence Interval (years)									
	0.2-Yr	0.5-Yr	1-Yr	2-Yr	5-Yr	10-Yr	20-Yr	25-Yr	50-Yr	100-Yr
6	0.59	0.78	0.91	1.05	1.24	1.37	1.52	1.57	1.72	1.87
12	0.80	1.10	1.32	1.54	1.84	2.06	2.30	2.38	2.61	2.86
24	0.99	1.41	1.72	2.04	2.48	2.79	3.14	3.25	3.61	3.96
48	1.42	1.96	2.36	2.78	3.35	3.76	4.23	4.39	4.87	5.36
72	1.63	2.25	2.71	3.18	3.81	4.26	4.77	4.94	5.45	5.97
168	2.28	3.25	3.92	4.58	5.42	5.99	6.61	6.80	7.38	7.93

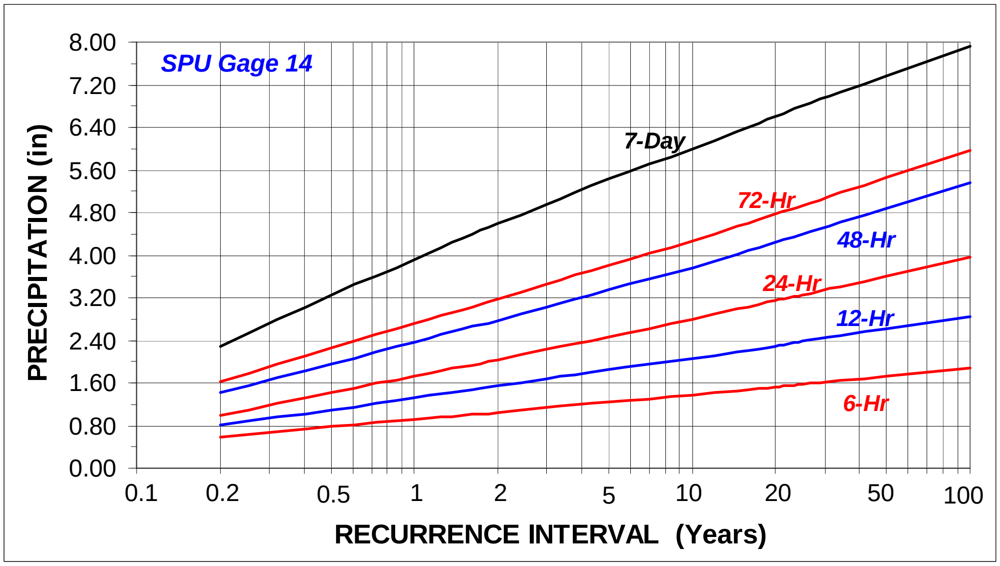

Figure 14. Precipitation-Magnitude-Frequency Estimates for SPU Gage 14.

Table 17. Precipitation-Magnitude-Frequency Estimates for SPU Gage 15.

Duration (hr)	Precipitation (in)									
	Recurrence Interval (years)									
	0.2-Yr	0.5-Yr	1-Yr	2-Yr	5-Yr	10-Yr	20-Yr	25-Yr	50-Yr	100-Yr
6	0.58	0.76	0.89	1.03	1.21	1.34	1.49	1.54	1.69	1.84
12	0.77	1.06	1.27	1.49	1.78	1.99	2.22	2.30	2.53	2.76
24	0.95	1.35	1.64	1.95	2.36	2.66	3.00	3.11	3.44	3.79
48	1.36	1.88	2.26	2.66	3.20	3.60	4.05	4.19	4.65	5.12
72	1.55	2.14	2.58	3.02	3.62	4.05	4.54	4.70	5.19	5.68
168	2.16	3.08	3.71	4.34	5.13	5.67	6.25	6.43	6.98	7.50

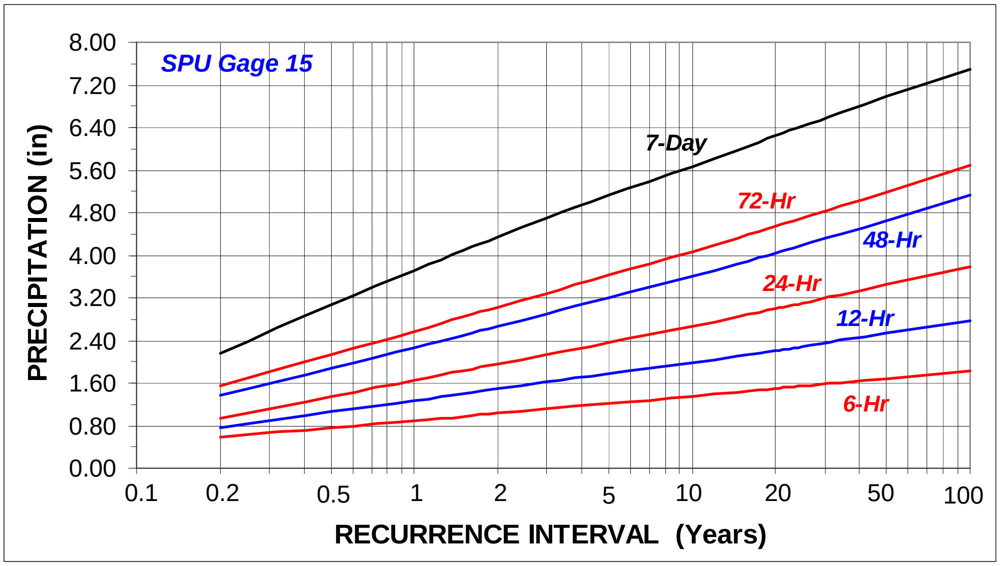

Figure 15. Precipitation-Magnitude-Frequency Estimates for SPU Gage 15.

Table 18. Precipitation-Magnitude-Frequency Estimates for SPU Gage 16.

Duration (hr)	Precipitation (in)									
	Recurrence Interval (years)									
	0.2-Yr	0.5-Yr	1-Yr	2-Yr	5-Yr	10-Yr	20-Yr	25-Yr	50-Yr	100-Yr
6	0.59	0.77	0.90	1.04	1.22	1.36	1.51	1.55	1.70	1.86
12	0.78	1.07	1.28	1.50	1.79	2.00	2.24	2.31	2.55	2.78
24	0.96	1.37	1.67	1.98	2.40	2.70	3.05	3.16	3.50	3.85
48	1.38	1.91	2.29	2.70	3.25	3.65	4.11	4.26	4.72	5.20
72	1.58	2.19	2.63	3.08	3.70	4.13	4.63	4.79	5.29	5.80
168	2.20	3.14	3.79	4.43	5.24	5.79	6.39	6.57	7.13	7.66

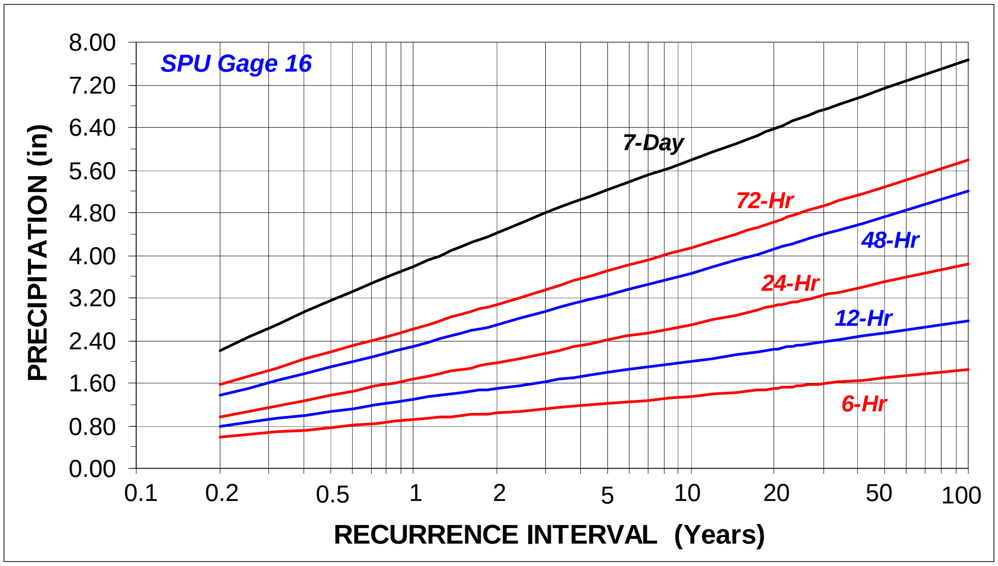

Figure 16. Precipitation-Magnitude-Frequency Estimates for SPU Gage 16.

Table 19. Precipitation-Magnitude-Frequency Estimates for SPU Gage 17.

Duration (hr)	Precipitation (in)									
	Recurrence Interval (years)									
	0.2-Yr	0.5-Yr	1-Yr	2-Yr	5-Yr	10-Yr	20-Yr	25-Yr	50-Yr	100-Yr
6	0.59	0.78	0.91	1.05	1.24	1.37	1.52	1.57	1.72	1.87
12	0.80	1.10	1.32	1.54	1.84	2.06	2.30	2.38	2.61	2.86
24	1.00	1.43	1.74	2.06	2.50	2.81	3.17	3.29	3.64	4.00
48	1.43	1.97	2.37	2.79	3.36	3.77	4.25	4.40	4.88	5.37
72	1.64	2.27	2.72	3.20	3.83	4.29	4.80	4.97	5.49	6.01
168	2.30	3.28	3.96	4.63	5.47	6.05	6.67	6.86	7.44	8.00

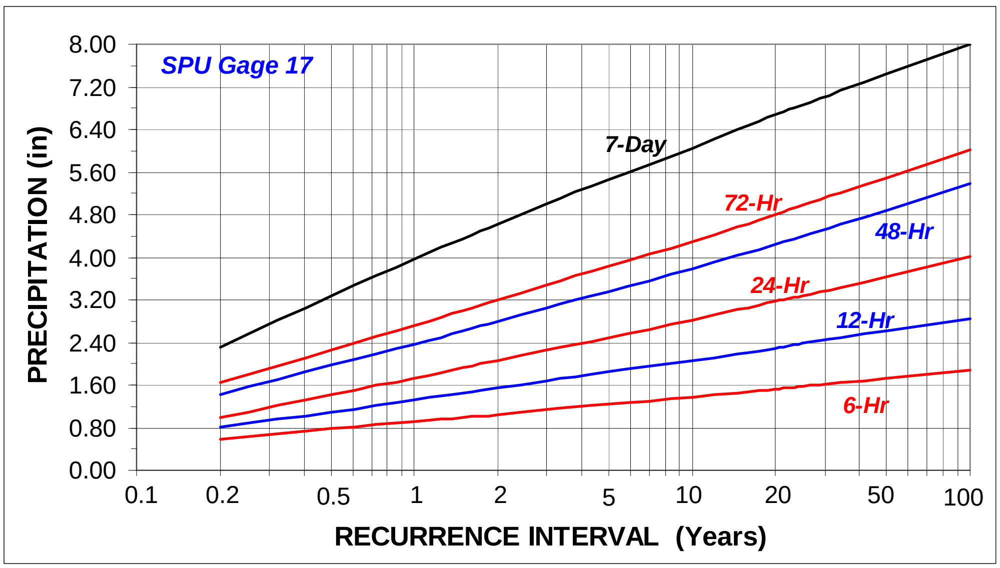

Figure 17. Precipitation-Magnitude-Frequency Estimates for SPU Gage 17.

Table 20. Precipitation-Magnitude-Frequency Estimates for SPU Gage 18.

Duration (hr)	Precipitation (in)									
	Recurrence Interval (years)									
	0.2-Yr	0.5-Yr	1-Yr	2-Yr	5-Yr	10-Yr	20-Yr	25-Yr	50-Yr	100-Yr
6	0.58	0.76	0.89	1.03	1.21	1.34	1.49	1.54	1.69	1.84
12	0.77	1.06	1.27	1.49	1.78	1.99	2.22	2.30	2.53	2.76
24	0.95	1.35	1.64	1.95	2.36	2.66	3.00	3.11	3.44	3.79
48	1.36	1.88	2.26	2.66	3.20	3.60	4.05	4.19	4.65	5.12
72	1.56	2.15	2.58	3.03	3.63	4.07	4.56	4.71	5.20	5.70
168	2.16	3.08	3.72	4.35	5.14	5.68	6.27	6.45	6.99	7.52

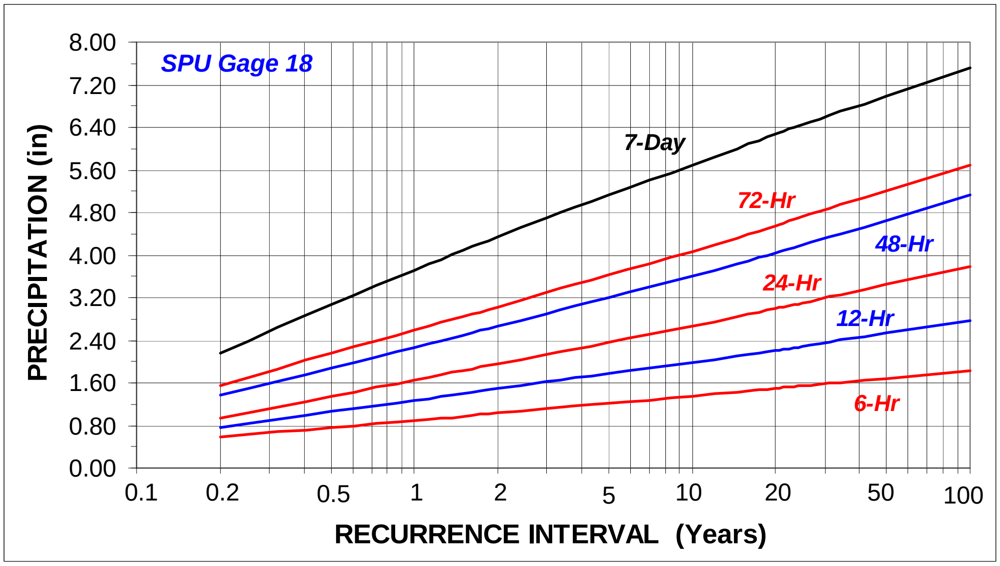

Figure 18. Precipitation-Magnitude-Frequency Estimates for SPU Gage 18.

Table 21. Precipitation-Magnitude-Frequency Estimates for SPU Gage 20.

Duration (hr)	Precipitation (in)									
	Recurrence Interval (years)									
	0.2-Yr	0.5-Yr	1-Yr	2-Yr	5-Yr	10-Yr	20-Yr	25-Yr	50-Yr	100-Yr
6	0.58	0.75	0.88	1.02	1.20	1.33	1.48	1.52	1.67	1.82
12	0.76	1.05	1.26	1.47	1.76	1.96	2.19	2.27	2.49	2.72
24	0.93	1.33	1.62	1.92	2.33	2.62	2.95	3.06	3.39	3.73
48	1.35	1.86	2.23	2.63	3.16	3.55	4.00	4.15	4.60	5.06
72	1.53	2.11	2.54	2.98	3.57	4.00	4.48	4.64	5.12	5.60
168	2.12	3.02	3.64	4.25	5.03	5.56	6.13	6.31	6.84	7.36

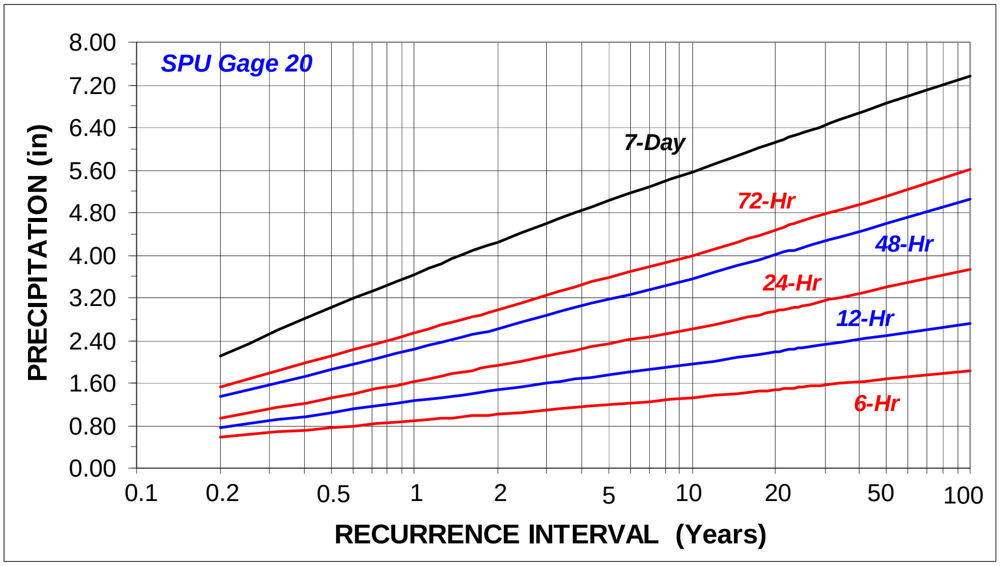

Figure 19. Precipitation-Magnitude-Frequency Estimates for SPU Gage 20.

Table 22. Precipitation-Magnitude-Frequency Estimates for SeaTac.

Duration (hr)	Precipitation (in)									
	Recurrence Interval (years)									
	0.2-Yr	0.5-Yr	1-Yr	2-Yr	5-Yr	10-Yr	20-Yr	25-Yr	50-Yr	100-Yr
6	0.60	0.78	0.92	1.06	1.25	1.38	1.54	1.58	1.74	1.89
12	0.82	1.12	1.34	1.57	1.88	2.10	2.34	2.42	2.67	2.91
24	1.02	1.45	1.77	2.11	2.55	2.87	3.23	3.35	3.72	4.08
48	1.46	2.01	2.42	2.85	3.43	3.86	4.34	4.50	4.99	5.49
72	1.68	2.32	2.78	3.27	3.92	4.38	4.91	5.08	5.61	6.14
168	2.36	3.37	4.06	4.75	5.62	6.21	6.85	7.04	7.64	8.22

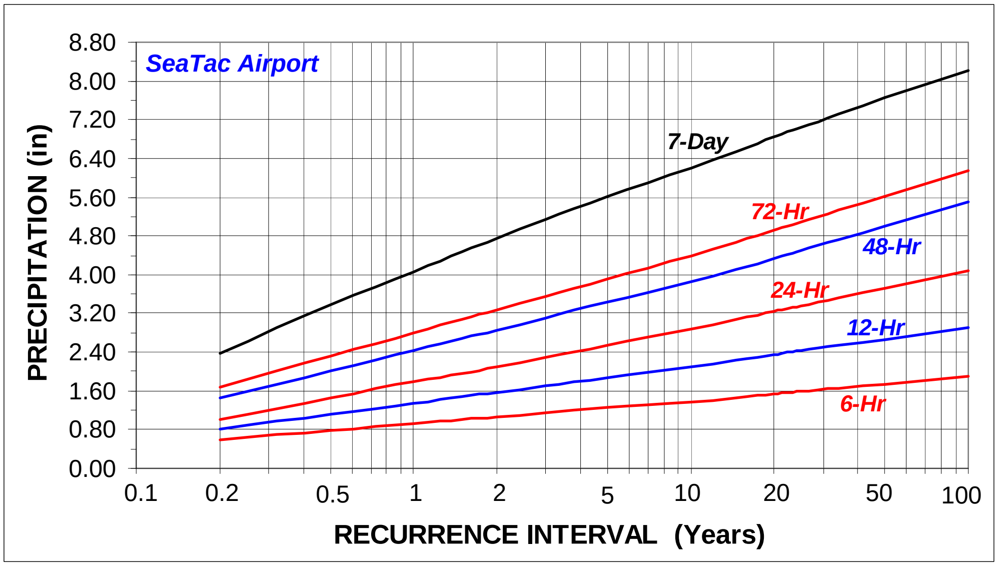

Figure 20. Precipitation-Magnitude-Frequency Estimates for SeaTac.