CHAPTER 3

HYDROLOGIC ANALYSIS & DESIGN

CITY OF RENTON

SURFACE WATER DESIGN MANUAL

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CHAPTER 3

HYDROLOGIC ANALYSIS & DESIGN

This chapter presents the concepts and rationale for the surface water controls and designs required by this manual and the acceptable methods for estimating the quantity and characteristics of surface water runoff. These methods are used to analyze existing and to design proposed drainage systems and facilities.

Hydrologic concepts, tools and methodologies, and an overview of the assumptions and data requirements of the methods, are described for the following tasks:

- Calculating runoff time series and flow statistics
- Designing detention and infiltration facilities

Approved hydrologic modeling software are listed in Reference Section 6-D. Tools and methodologies specific to the software can be obtained from the software documentation and trainings provided by the software providers. At this writing, the approved models for stormwater runoff and water quality design include WWHM2012 and WWHM4, available from the Washington State Department of Ecology (Ecology), MGS Flood, available from MGS Engineering Consultants, Inc., and the Hydrologic Simulation Program (Fortran) (HSPF). **The King County Reduced Time Series (KCRTS) software is no longer maintained by King County and is not an approved model for use with the Surface Water Design Manual (SWDM).**

Hydrologic tools and methodologies, and the assumptions and data requirements of the methods, are presented for the following tasks:

- Sizing conveyance facilities
- Analyzing conveyance capacities.

Chapter Organization

The information presented in this chapter is organized into three main sections:

- Section 3.1, “Hydrologic Design Standards and Principles”
- Section 3.2, “Runoff Computation and Analysis Methods”
- Section 3.3, “Hydrologic Design Procedures and Considerations”

These sections begin on odd pages so the user can insert tabs if desired for quicker reference.

Other Supporting Information

For specific guidance on the mechanics of using the approved modeling software for hydrologic analysis and design, refer to the associated approved model website and program documentation. See Reference Section 6-D for limited modeling guidance and requirements as applicable for specific tasks in this manual.

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3.1 HYDROLOGIC DESIGN STANDARDS AND PRINCIPLES

This section presents the rationale for and approach to hydrologic analysis and design. Topics covered include the following:

- "Hydrologic Impacts and Mitigation," Section 3.1.1
- "Flow Control Standards," Section 3.1.2
- "Hydrologic Analysis Using Continuous Models," Section 3.1.3

3.1.1 HYDROLOGIC IMPACTS AND MITIGATION

Hydrologic Effects of Urbanization

The hydrologic effects of development can cause a multitude of problems, including minor nuisance flooding, degradation of public resources, diminished fish production, and significant flooding endangering life and property. Increased stormwater flows expand floodplains, bringing flooding to locations where it did not occur before and worsening flood problems in areas already flood-prone. Increased stormwater flows also hasten channel erosion, alter channel structure, and degrade fish habitat.

Human alteration of the landscape, including clearing, grading, paving, building construction, and landscaping, changes the physical and biological features that affect hydrologic processes. Soil compaction and paving reduce the infiltration and storage capacity of soils. This leads to a runoff process called *Horton overland flow* whereby the rainfall rate exceeds the infiltration rate, and the excess precipitation flows downhill over the soil surface. This type of flow rapidly transmits rainfall to the stream or conveyance system, causing much higher peak flow rates than would occur in the unaltered landscape.

Horton overland flow is almost nonexistent in densely vegetated areas, such as forest or shrub land, where the vast majority of rainfall infiltrates into the soil. Some of this infiltrated water is used by plants, and depending on soil conditions, some of it percolates until it reaches the groundwater table. Sometimes the percolating soil water will encounter a low-permeability soil or rock layer. In this case, it flows laterally as interflow over the low-permeability layer until it reaches a stream channel. Generally, forested lands deliver water to streams by subsurface pathways, which are much slower than the runoff pathways from cleared and landscaped lands. Therefore, urbanization of forest and pasture land leads to increased stormwater flow volumes and higher peak flow rates.

Land development increases not only peak flow rates but also changes annual and seasonal runoff volumes. *In forested basins in King County, about 55% of the rain that falls each year eventually appears as streamflow.* This percentage is called the *yield of a basin*. The remaining 45% of the rain evaporates and returns to the atmosphere. As trees are cleared and the soil is graded to make way for lawns and pastures, and as part of the land is covered with asphalt or concrete, the basin yield increases. More of the rain becomes streamflow, and less evaporates. In lowland King County, the yield of a basin covered with landscaped lawns would be about 65%, while the yield of an impervious basin would be about 85 to 90%.

For these reasons, development without mitigation increases peak stormwater rates, stormwater volumes, and annual basin yields. Furthermore, the reduction of groundwater recharge decreases summer base flows.

In summary, the following are the hydrologic impacts of unmitigated development:

- Increased peak flows
- Increased durations of high flows
- Increased stormwater runoff volumes
- Decreased groundwater recharge and base flows
- Seasonal flow volume shifts
- Altered wetland hydroperiods.

The resulting economic and ecological consequences of these hydrologic changes include the following:

- Increased flooding
- Increased stream erosion
- Degraded aquatic habitat
- Changes to wetland species composition.

Mitigation of Hydrologic Effects of Urbanization

Engineered facilities can mitigate many of the hydrologic changes associated with development. Detention facilities can maintain the rates and/or durations of high flows at predevelopment levels. Infiltration facilities can control flow volumes and increase groundwater recharge as well as control flow rates and durations. Conveyance problems can be avoided through analysis and appropriate sizing and design of conveyance facilities. Engineered mitigation of the hydrologic impacts of development include the following:

- Managing peak flow rates with detention facilities
- Managing high flow durations with detention facilities
- Reducing flow volumes and maintaining or enhancing groundwater recharge with infiltration facilities
- Avoiding flooding problems with appropriately sized and designed conveyance systems
- Bypassing erosion problems with tightlines.

Engineered facilities cannot mitigate all of the hydrologic impacts of development. Detention facilities do not mitigate seasonal volume shifts, wetland water level fluctuations, groundwater recharge reductions, or base flow changes. Such impacts can be further reduced through the use of low impact development (LID) techniques, beginning with careful ***site*** planning. For instance, clustering of units to reduce impervious cover while maintaining ***site*** density is an effective way to limit hydrologic change. Preserving native vegetation and minimizing soil disturbance or compaction in pervious areas also reduces hydrologic change. Such non-engineered mitigation measures are encouraged by the City and are discussed in Core Requirement #9 and Appendix C of this manual and are referred to as ***on-site BMPs***.

Other ***on-site BMPs***, such as permeable pavements, bioretention, vegetated roofs, and rainwater harvesting can be effective in reducing increases in surface water volumes. The incorporation of these concepts in the design of the project is required, as detailed in Core Requirement #9 and Appendix C. Many of these approaches will result in a reduction in flow control facility size, so the ***on-site BMP*** requirements in Core Requirement #9 and Appendix C should be carefully considered and applied to maximize the benefits of this approach.

Detention Facility Concepts

The basic concept of a *detention facility* is simple: water is collected from developed areas and released at a slower rate than it enters the collection system. The excess of inflow over outflow is temporarily stored in a pond or a vault and is typically released over a few hours or a few days. The volume of storage needed is determined by (1) how much stormwater enters the facility (determined by the size and density of the contributing area), (2) how rapidly water is allowed to leave the facility, and (3) the level of hydrologic control the facility is designed to achieve.

To prevent increases in the frequency of flooding due to new development, detention facilities are often designed to **maintain peak flow rates at their predevelopment levels** for recurrence intervals of concern (e.g., 2- and 10-year). Such mitigation can prevent increases in the frequency of downstream flooding. Facilities that control only peak flow rates, however, usually allow the duration of high flows to increase, which may cause increased erosion of the downstream system. For example, the magnitude of a 2-year flow may not increase, but the amount of time that flow rate occurs may double. Therefore, stream systems, including those with salmonid habitat, which require protection from erosion warrant detention systems that **control the durations of geomorphically significant flows** (flows capable of moving sediment). Such detention systems employ lower release rates and are therefore larger in volume.

3.1.2 FLOW CONTROL STANDARDS

Refer to Chapter 1, Section 1.2.3, for flow control standards.[1,2]

3.1.3 HYDROLOGIC ANALYSIS USING CONTINUOUS MODELS

The Need for Continuous Hydrologic Modeling

This manual prescribes the use of a **continuous hydrologic model** for most hydrologic analyses rather than an event model. Event models such as the Santa Barbara Urban Hydrograph (SBUH), King County Runoff Series (KCRTS) and the Soil Conservation Service (SCS)[3] method were used in previous versions of the King County Surface Water Design Manual. A continuous model was selected for the current version of the City of Renton SWDM because hydrologic problems in western Washington are associated with the high volumes of flow from sequential winter storms rather than high peak flows from short duration, high intensity rainfall events.

The continuous hydrologic analysis tools prescribed in this manual are generically described as the "**approved model"**; a list of the approved models is found in Reference Section 6-D (as updated). At this writing, the approved continuous hydrologic models[4] include the **Western Washington Hydrologic Model (WWHM)** and **MGS Flood**, both of which are variants of the Hydrologic Simulation Program-FORTRAN (HSPF) model. HSPF is also an approved model, but is more complex than other approved models and is typically used for basin planning and master drainage plan analyses.

Continuous models are well suited to accounting for the climatological conditions in the lowland Puget Sound area. Continuous models include algorithms that maintain a continuous water balance for a catchment to account for soil moisture and hydraulic conditions antecedent to each storm event (Linsley, Kohler, Paulhus, 1982), whereas event models assume initial conditions and only address single hypothetical storm events. As a result, continuous hydrologic models are more appropriate for evaluating runoff during the extended wet winters typical of the Puget Sound area.

The **drawbacks of event models** are summarized as follows:

- Event methods inherently overestimate peak flows from undeveloped land cover conditions. The overestimation is due, in part, to the assumption that runoff from forest and pasture land covers flows across the ground surface. In actuality, the runoff from forests and pastures, on till soils, is dominated by shallow subsurface flows (interflow) which have hydrologic response times much longer than those used in event methods. This leads to an over estimation of predeveloped peak flows, which results in detention facility release rates being overestimated and storage requirements being underestimated.
- A single event cannot represent the sequential storm characteristics of Puget Sound winters.
- Event models assume detention facilities are empty at the start of a design event, whereas actual detention facilities may be partially full as a result of preceding storms.
- Testing of event-designed detention facilities with calibrated, long-term continuous hydrologic simulations demonstrates that these facilities do not achieve desired performance goals.
- Event methods do not allow analysis of flow durations or water level fluctuations.

The **benefits of continuous hydrologic modeling** are summarized as follows:

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[1] Footnote 1 does not apply.

[2] Footnote 2 does not apply.

[3] The Soil Conservation Service (SCS) is now known as the National Resources Conservation Service (NRCS). The method described in *Urban Hydrology for Small Watersheds*, Technical Release 55 (TR-55), June 1986, published by the NRCS, is commonly referred to as the "SCS method."

[4] KCRTS is no longer maintained by King County and is not an approved model for use with the SWDM.

- A continuous model accounts for the long duration and high precipitation volume of winter wet periods characterized by sequential, low-intensity rainfall events. Continuous simulation uses continuous long-term records of observed rainfall rather than short periods of data representing hypothetical storm events. As a result, continuous simulation explicitly accounts for the long duration rainfall events typically experienced in the Pacific Northwest as well as the effects of rainfall antecedent to major storm events.
- HSPF has been shown to more accurately simulate runoff from basins with a wide range of sizes and land covers using the regional parameters developed by the United States Geologic Survey (USGS).
- Continuous simulation allows direct examination of flow duration data for assessing the impacts of development on stream erosion and morphology. An event model, whether using a 1-day or a 7-day storm, cannot provide such information.
- A continuous model allows water level analysis for wetlands, lakes, and closed depressions whose water level regime is often dependent on seasonal runoff rather than on 1-day or 7-day event runoff.
- Continuous models produce flow control facilities that more accurately and effectively achieve desired performance goals.

The importance of continuous modeling in the Puget Sound area is illustrated in Figure 3.1.3.A, which shows a small basin's runoff response to a series of winter storms and the outflow from a detention pond designed to control the peak annual flows from this basin. Note that the largest outflow from the detention pond corresponds not to the peak inflow on 11/6/86, but rather to the high volume of flow from the sequential storms beginning on 11/19/86. This demonstrates a key difference between continuous and event based models.

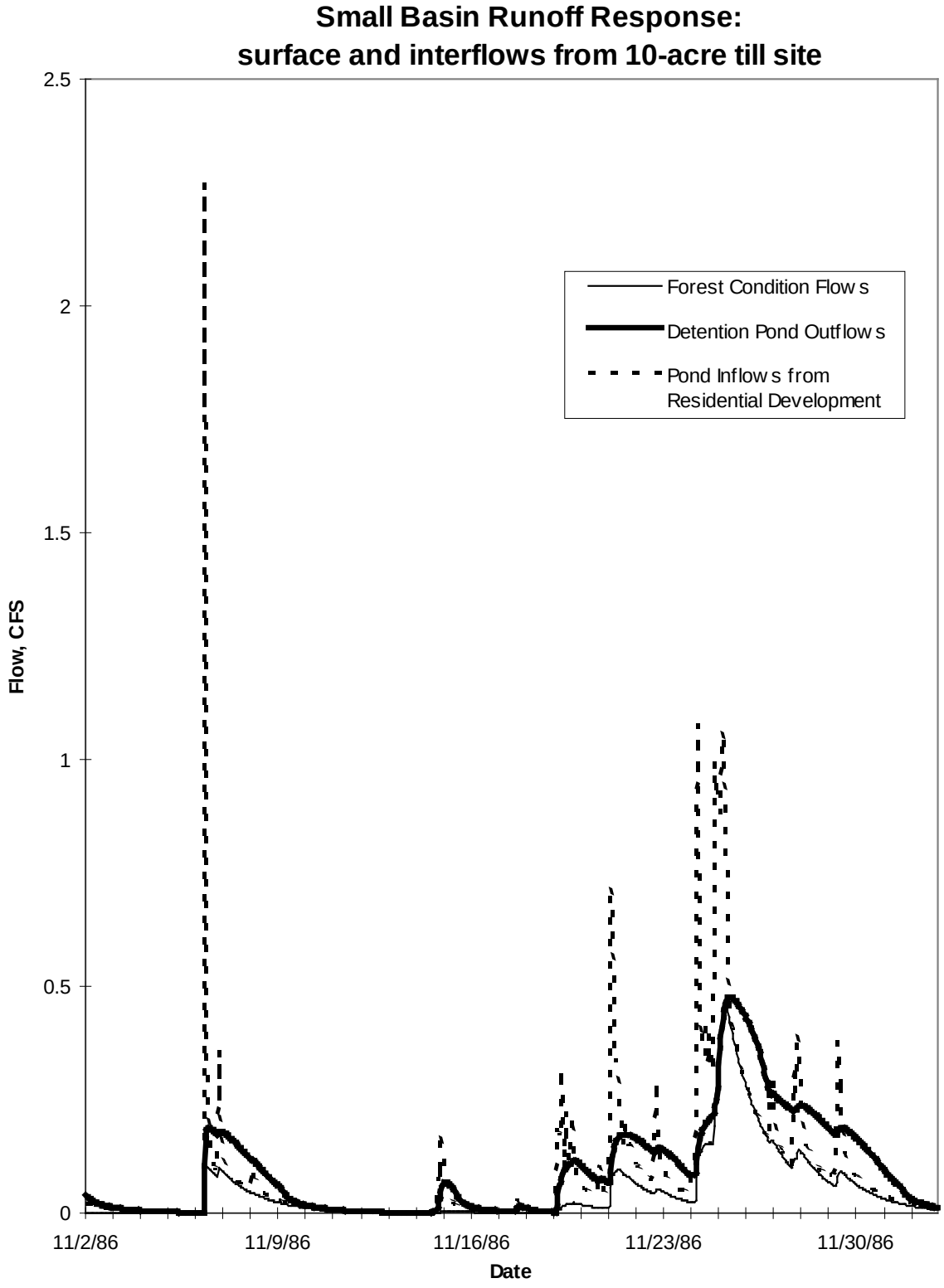
With an event model, designers are accustomed to working with a single design storm event (e.g., 10-year), which by definition has the same return period once routed through a reservoir (10-year inflow will always generate 10-year outflow). With a continuous model, flow recurrence estimates are based on annual peak flow rates, with each time series being analyzed independently. Events that generate annual peak inflows to a reservoir may not generate annual peak discharges from the reservoir. In other words, the runoff event containing the 10-year inflow peak, when routed, may not create the 10-year outflow peak. This is due to natural variability of storm peaks and volumes (e.g., high intensity/short duration thunderstorms as compared to moderate intensity/long duration winter storms) contained within a continuous record.

Requirements of Continuous Hydrologic Modeling

For the entire period of simulation, a continuous hydrologic model requires a continuous record of precipitation and evaporation at discrete time steps small enough to capture the temporal variability of hydrologic response, and it provides a continuous record of simulated flows at the same time step. The quicker a basin responds hydrologically (e.g., due to small size, land cover, or lack of detention), the smaller the time step should be. Time steps of 15 minutes are sufficient for most basins in the Puget Sound area.

The continuous hydrologic model must include mathematical representations of hydrologic processes to determine the fate and movement of rainfall. For example, a good continuous hydrologic model must include representations of infiltration processes to determine how much water infiltrates the soil and how much runs off the surface. It must represent shallow and deep soil storage as well as the release of subsurface water to streams via interflow and groundwater flow, and it must also account for the loss of soil water to the atmosphere via evapotranspiration between rainfall events. The benefit of all this computation is a complete hydrologic assessment including information on peak flow rates, flow durations, storm volumes, seasonal volumes, annual volumes, and water levels of receiving bodies.

FIGURE 3.1.3.A EFFECTS OF SEQUENTIAL STORMS ON DETENTION PERFORMANCE



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3.2 RUNOFF COMPUTATION AND ANALYSIS METHODS

This section presents the following four runoff computation methods accepted for hydrologic analysis and design:

- The **Rational Method** described below and detailed in Section 3.2.1
- The **TR-55 or SBUH methods** described below.
- The **Runoff Files Method** described below and detailed in Section 3.2.2
- The **Hydrologic Simulation Program-FORTRAN (HSPF) model** described below and detailed in Section 3.2.4.

❑ ACCEPTABLE USES OF RUNOFF COMPUTATION METHODS

Acceptable uses of the four runoff computation methods are summarized below and in Table 3.2:

- Rational Method: This method is most appropriate for sizing new conveyance systems that drain smaller, quickly responding tributary areas (i.e., less than 10 acres) where very short, intense storms tend to generate the highest peak flows. The Rational Method may also be used for conveyance sizing in any size basin if the attenuation effects of existing storage features within the basin are ignored.
- TR-55/SBUH Methods: The Natural Resources Conservation Service (NRCS, formerly the Soil Conservation Service (SCS)) TR-55 method or the SBUH method of the 1990 King County *Surface Water Design Manual* may be used for conveyance sizing where tributary areas are greater than or equal to 10 acres and if storage features are ignored. The peak flows from these single-event models are considered conservative for larger tributary areas if the flows are not routed through existing storage features. The TR-55 method is also used for water quality volume calculation in this manual. For more background information, refer to NRCS Publication 210-VI-TR-55, Second Edition (June 1986) or the 1990 *SWDM*.
- The Runoff Files Method: This continuous modeling method using the approved model is the most versatile for quickly performing many of the computations summarized in Table 3.2. For conveyance sizing and analysis, the peak flows from the approved model are most accurate when the shortest possible time step is used. Unlike the Rational Method, the approved model may be used for tributary areas less than 10 acres where there is a significant storage feature(s). In previous editions of this manual, sizing and analysis of storage features and volume-based water quality facilities used hourly time steps for determination of predevelopment discharges and for routing purposes. As of this edition, the City requires **15-minute time steps[5] for sizing of all flow control facilities, water quality facilities and conveyance** to provide consistent management of surface water and protect against cumulative increases in peak flows on a basin-wide basis (see Sections 3.3.1 and 3.3.2). Methods for analysis and design of detention storage and water levels[6] require the use of the approved model. See the user's documentation for background and guidance.

- HSPF Model: For projects in Large Project Drainage Review (see Section 1.1.2.5), the City may require HSPF modeling for formulating a Master Drainage Plan (see *Master Drainage Planning for Large Site Developments – Process and Requirement Guidelines* available from King County). The City also generally encourages use of HSPF for tributary areas larger than 200 acres. The HSPF model can be used wherever the approved model is allowed for sizing and analysis of conveyance systems, flow control facilities, and water quality facilities using a 15-minute time step. For such projects draining to a wetland or potentially impacting groundwater resources or stream base flows, the City

[5] Footnote 5 does not apply.

[6] One of the simplest and most commonly used level pool routing methods is described in the *Handbook of Applied Hydrology* (Chow, Ven Te, 1964) and elsewhere, and summarized in Reference Section 6-C, It is based on the continuity equation and can be completed with a spreadsheet. Although not approved for design with this manual, it provides a background for modeled routing techniques.

may require the collection of actual rainfall and runoff data to be used in developing and calibrating the HSPF model.

TABLE 3.2 ACCEPTABLE USES OF RUNOFF COMPUTATION METHODS

TYPE OF COMPUTATION	APPLIED TO	RATIONAL METHOD	TR 55/SBUH	THE APPROVED MODEL	HSPF
PEAK FLOW CONVEYANCE SIZING INC. TESC[1] (DESIGN FLOWS) (See Chapter 4 for hydraulic analysis procedures)	**Tributary Areas < 10 ac** (measured to individual conveyance elements)	REQUIRED for undetained areas,[2] and OKAY for detained areas if no storage routing[3] is performed		OKAY if majority of tributary area is detained[4]	OKAY if majority of tributary area is detained[4]
	Tributary Areas ≥ 10 ac	OKAY if no storage routing[3] is performed	OKAY if no storage routing[3] is performed	OKAY (storage routing is allowed)	OKAY (storage routing is allowed)
LEVEL-POOL ROUTING FLOW CONTROL (NEW/EXIST.) & WQ FACILITY SIZING AND ANALYSIS	**Projects in Full Drainage Review**			OKAY	OKAY
	Projects in Large Project Drainage Review			MAY BE ALLOWED[5]	MAY BE REQUIRED[5]
DOWNSTREAM ANALYSIS	**Projects in Full or Targeted Drainage Review**	OKAY if no storage routing[3] is performed	OKAY for tributary areas ≥ 10 ac. if no storage routing[3] is performed	OKAY	OKAY
	Projects in Large Project Drainage Review	MAY BE ALLOWED[5] if used as described in the box above	MAY BE ALLOWED[5] if as described in the box above	MAY BE ALLOWED[5] if used as described in the box above	
PEAK FLOWS FOR APPLYING EXEMPTIONS & THRESHOLDS	**All Projects**			OKAY	OKAY

Notes:

(1) Water quality design flow rates are determined as described in Section 6.2.1.

(2) *Undetained areas* are those upstream of detention facilities or other storage features.

(3) *Storage routing* uses the Level Pool Routing technique (described in Reference Section 6-C) or other similar method to account for the attenuation of peak flows passing through a detention facility or other storage feature.

(4) The majority of the tributary area is considered *detained* if the runoff from more than 50% of the tributary area is detained by a detention facility or other storage facility.

(5) For projects in Large Project Drainage Review, the selection of methodology for detention sizing and/or downstream analysis becomes a site-specific or basin-specific decision that is usually made by CED during the scoping process for master drainage plans. Guidelines for selecting the approved model, HSPF, or calibrated HSPF are found in the King County publication *Master Drainage Planning for Large or Complex Site Developments*, available from King County.

3.2.1 RATIONAL METHOD

The Rational Method is a simple, conservative method for analyzing and sizing conveyance elements serving small drainage subbasins, subject to the following specific **limitations**:

- Only for use in predicting peak flow rates for sizing conveyance elements
- Drainage subbasin area A cannot exceed 10 acres for a single peak flow calculation
- The time of concentration T_c must be computed using the method described below and cannot exceed 100 minutes. It is also set equal to 6.3 minutes when computed to be less than 6.3 minutes.

Note: Unlike other methods of computing times of concentration, the 6.3 minutes is not an initial collection time to be added to the total computed time of concentration.

❑ RATIONAL METHOD EQUATION

The following is the traditional Rational Method equation:

$$Q_R = CI_RA \tag{3-1}$$

where Q_R = peak flow (cfs) for a storm of return frequency R
C = estimated runoff coefficient (ratio of rainfall that becomes runoff)
I_R = peak rainfall intensity (inches/hour) for a storm of return frequency R
A = drainage subbasin area (acres)

"*C*" Values

The allowable runoff coefficients to be used in this method are shown in Table 3.2.1.A by type of land cover. These values were selected following a review of the values previously accepted by King County for use in the Rational Method and as described in several engineering handbooks. The values for single family residential areas were computed as composite values (as illustrated in the following equation) based on the estimated percentage of coverage by roads, roofs, yards, and unimproved areas for each density. For drainage basins containing several land cover types, the following formula may be used to compute a composite runoff coefficient, C_c:

$$C_c = (C_1A_1 + C_2A_2 + \ldots + C_nA_n)/A_t \tag{3-2}$$

where A_t = total area (acres)
$A_{1,2,\ldots n}$ = areas of land cover types (acres)
$C_{1,2,\ldots n}$ = runoff coefficients for each area land cover type

"IR" Peak Rainfall Intensity

The peak rainfall intensity I_R for the specified design storm of return frequency R is determined using a unit peak rainfall intensity factor i_R in the following equation:

$$I_R = (P_R)(i_R) \tag{3-3}$$

where P_R = the total precipitation at the ***project site*** for the 24-hour duration storm event for the given return frequency. Total precipitation is found on the Isopluvial Maps in Figure 3.2.1.A through Figure 3.2.1.D.
i_R = the unit peak rainfall intensity factor

The unit peak rainfall intensity factor i_R is determined by the following equation:

$$i_R = (a_R)(T_c)^{(-b_R)} \tag{3-4}$$

where T_c = time of concentration (minutes), calculated using the method described below and subject to equation limitations ($6.3 \le T_c \le 100$)

a_R, b_R = coefficients from Table 3.2.1.B used to adjust the equation for the design storm return frequency R

This "i_R" equation was developed by DNRP from equations originally created by Ron Mayo, P.E. It is based on the original **Renton/Seattle Intensity/Duration/Frequency (I.D.F.) curves**. Rather than requiring a family of curves for various locations, this equation adjusts proportionally the Renton/Seattle I.D.F. curve data by using the 24-hour duration total precipitation isopluvial maps. This adjustment is based on the assumption that the localized geo-climatic conditions that control the total volume of precipitation at a specific location also control the peak intensities proportionally.

Note: Due to the mathematical limits of the equation coefficients, values of T_c less than 6.3 minutes or greater than 100 minutes cannot be used. Therefore, real values of T_c less than 6.3 minutes must be assumed to be equal to 6.3 minutes, and values greater than 100 minutes must be assumed to be equal to 100 minutes.

"T_c" Time of Concentration

The time of concentration is defined as the time it takes runoff to travel overland (from the onset of precipitation) from the most hydraulically distant location in the drainage basin to the point of discharge. *Note: When C_c (see Equation 3-2) of a drainage basin exceeds 0.60, it may be important to compute T_c and peak rate of flow from the impervious area separately.* The computed peak rate of flow for the impervious surface alone may exceed that for the entire drainage basin using the value at T_c for the total drainage basin. The higher of the two peak flow rates shall then be used to size the conveyance element.

T_c is computed by summation of the travel times T_t of overland flow across separate flowpath segments defined by the six categories of land cover listed in Table 3.2.1.C, which were derived from a chart published by the Soil Conservation Service in 1975. The equation for time of concentration is:

$$T_c = T_1 + T_2 + \ldots + T_n \tag{3-5}$$

where $T_{1,2,\ldots n}$ = travel time for consecutive flowpath segments with different land cover categories or flowpath slope

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

$$T_t = \frac{L}{60V} \tag{3-6}$$

where T_t = travel time (minutes) Note*: T_t through an open water body (such as a pond) shall be assumed to be zero with this method*

L = the distance of flow across a given segment (feet)

V = average velocity (fps) across the land cover = $k_R \sqrt{s_o}$

where k_R = time of concentration velocity factor; see Table 3.2.1.C

s_o = slope of flowpath (feet/feet)

TABLE 3.2.1.A RUNOFF COEFFICIENTS – "*C*" VALUES FOR THE RATIONAL METHOD

General Land Covers		Single Family Residential Areas*	
Land Cover	***C***	**Land Cover Density**	***C***
Dense forest	0.10	0.20 DU/GA (1 unit per 5 ac.)	0.17
Light forest	0.15	0.40 DU/GA (1 unit per 2.5 ac.)	0.20
Pasture	0.20	0.80 DU/GA (1 unit per 1.25 ac.)	0.27
Lawns	0.25	1.00 DU/GA	0.30
Playgrounds	0.30	1.50 DU/GA	0.33
Gravel areas	0.80	2.00 DU/GA	0.36
Pavement and roofs	0.90	2.50 DU/GA	0.39
Open water (pond, lakes, wetlands)	1.00	3.00 DU/GA	0.42
		3.50 DU/GA	0.45
		4.00 DU/GA	0.48
		4.50 DU/GA	0.51
		5.00 DU/GA	0.54
		5.50 DU/GA	0.57
		6.00 DU/GA	0.60

* Based on average 2,500 square feet per lot of impervious coverage.

For combinations of land covers listed above, an area-weighted "C_c x A_t" sum should be computed based on the equation $C_c \times A_t = (C_1 \times A_1) + (C_2 \times A_2) + \ldots + (C_n \times A_n)$, where $A_8 = (A_1 + A_2 + \ldots + A_n)$, the total drainage basin area.

TABLE 3.2.1.B COEFFICIENTS FOR THE RATIONAL METHOD "I_R" EQUATION

Design Storm Return Frequency	a_R	b_R
2 years	1.58	0.58
5 years	2.33	0.63
10 years	2.44	0.64
25 years	2.66	0.65
50 years	2.75	0.65
100 years	2.61	0.63

TABLE 3.2.1.C K_R VALUES FOR T_T USING THE RATIONAL METHOD

Land Cover Category	k_R
Forest with heavy ground litter and meadow	2.5
Fallow or minimum tillage cultivation	4.7
Short grass pasture and lawns	7.0
Nearly bare ground	10.1
Grassed waterway	15.0
Paved area (sheet flow) and shallow gutter flow	20.0

FIGURE 3.2.1.A 2-YEAR 24-HOUR ISOPLUVIALS

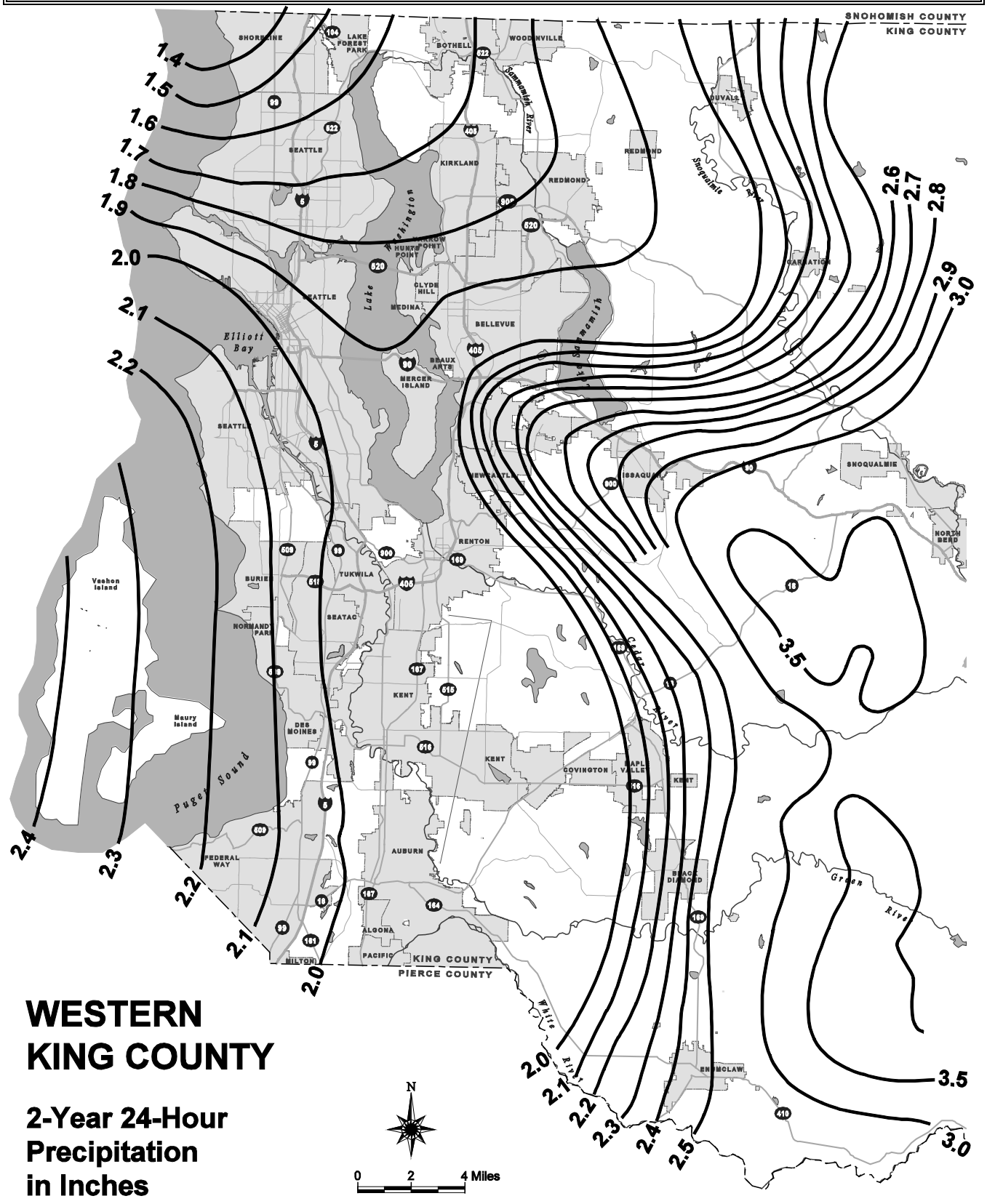


FIGURE 3.2.1.B 10-YEAR 24-HOUR ISOPLUVIALS

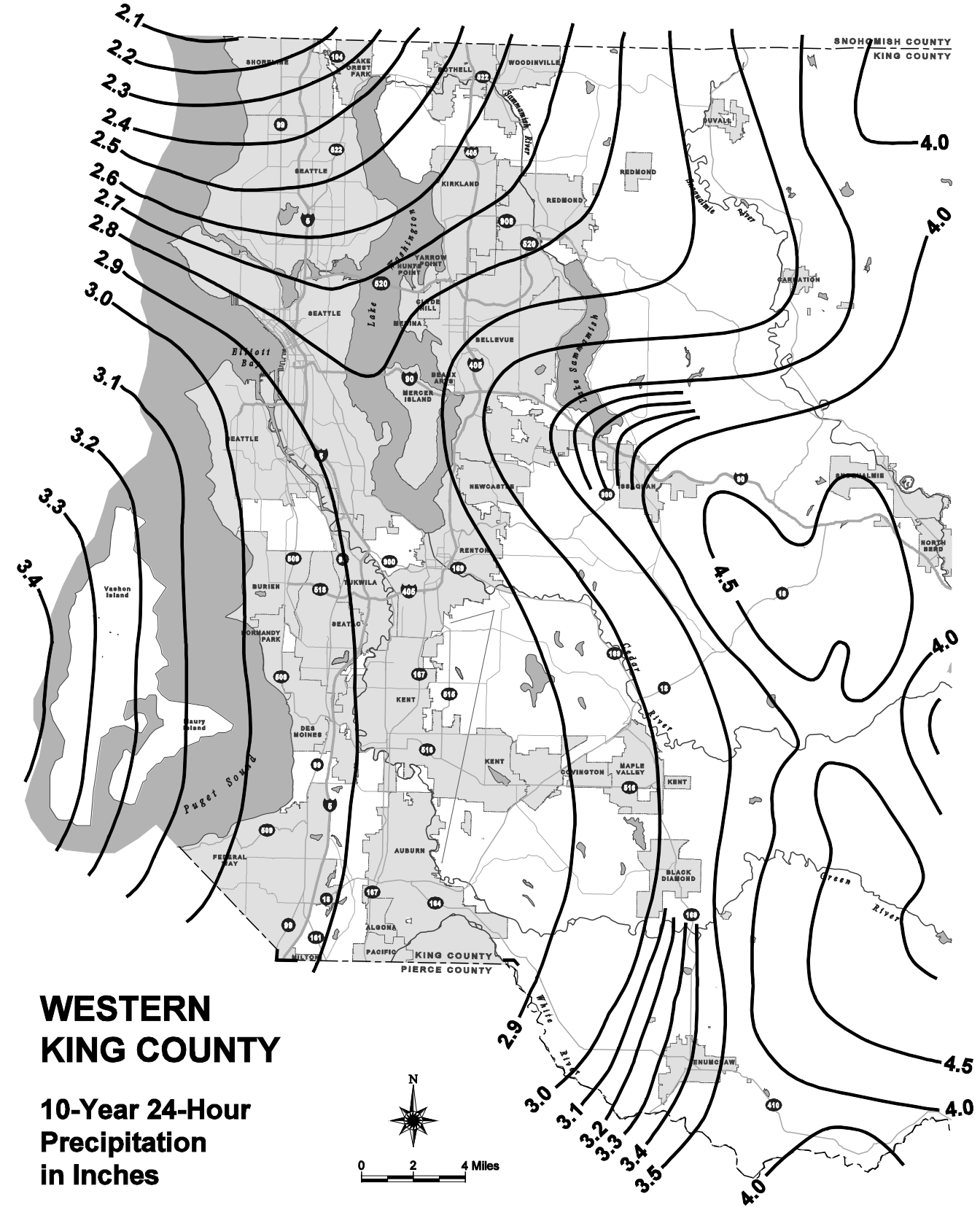


FIGURE 3.2.1.C 25-YEAR 24-HOUR ISOPLUVIALS

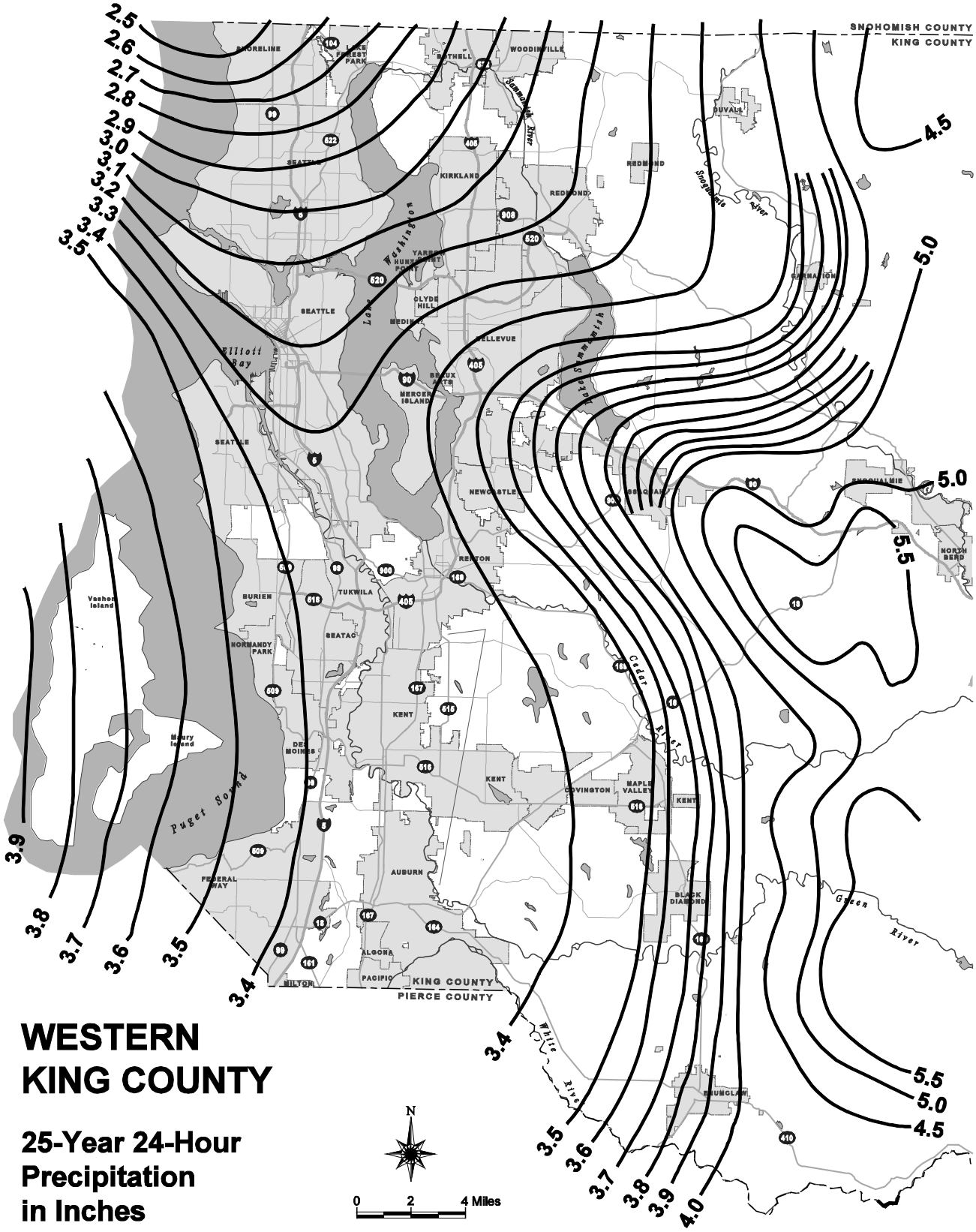
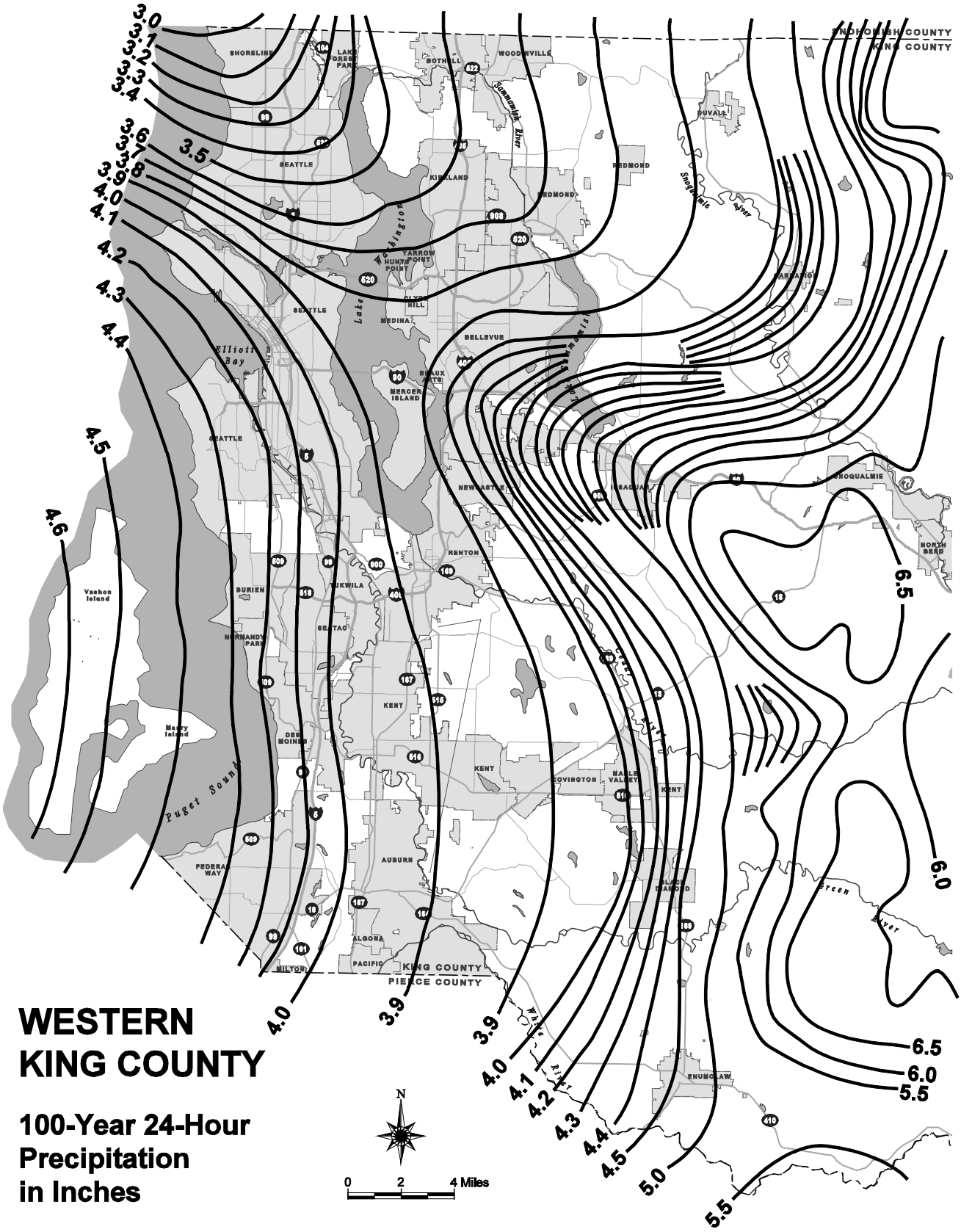


FIGURE 3.2.1.D 100-YEAR 24-HOUR ISOPLUVIALS



❑ RATIONAL METHOD EXAMPLE

Compute the peak flow Q_{25} to size a new roadway cross culvert for a 9.8-acre drainage basin east of Kent, P_{25} = 3.42 inches.

Given: AREAS

A_1 = 4.3 acres of single family residential area at 3.8 DU/GA
A_2 = 2.3 acres of light forest
A_3 = 3.2 acres of pasture
A_t = 9.8 total acres

DESCRIPTION OF FLOWPATH SEGMENTS FOR T_c

L_1	=	300 feet	s_1	=	0.08	forest land cover	k_R = 2.5
L_2	=	200 feet	s_2	=	0.03	meadow	k_R = 2.5
L_3	=	1000 feet	s_3	=	0.015	grassed waterway (ditch)	k_R = 15.0

Compute: COMPOSITE RUNOFF COEFFICIENT C_c

A_1: C_1 = From Table 3.2.1.A, C for 4.00 DU/GA = 0.48, C for 3.50 DU/GA = 0.45. Therefore, C_1 for 3.80 DU/GA = 0.47 by visual interpolation.
A_2: C_2 = 0.15
A_3: C_3 = 0.20
C_c = $[(C_1 \text{ x } A_1) + (C_2 \text{ x } A_2) + (C_3 \text{ x } A_3)]/A_t$
= $[(0.47 \text{ x } 4.3) + (0.15 \text{ x } 2.3) + (0.20 \text{ x } 3.2)]/9.8$ = 0.31

PEAK RAINFALL INTENSITY I_R

First, compute T_c:

$$T_1 = \frac{L_1}{60V_1} = \frac{L_1}{60(k_R\sqrt{s_1})} = \frac{300}{60(2.5\sqrt{0.08})}$$

= 7 minutes

$$T_2 = \frac{L_2}{60V_2} = \frac{L_2}{60(k_R\sqrt{s_2})} = \frac{200}{60(2.5\sqrt{0.03})}$$

= 8 minutes

$$T_3 = \frac{L_3}{60V_3} = \frac{L_3}{60(k_R\sqrt{s_3})} = \frac{1000}{60(15\sqrt{0.015})}$$

= 9 minutes

T_c = $T_1 + T_2 + T_3$ = 7 + 8 + 9 = 24 minutes

Second, compute i_R for R = 25:

$$i_{25} = (a_R)(T_c)^{(-b_R)} = (2.66)(24)^{-(0.65)} = 0.34$$

Third, compute I_R for R = 25:

$$I_{25} = (P_{25})(i_{25}) = (3.42)(0.34) = 1.16$$

PEAK RUNOFF RATE

$Q_{25} = C\, I_{25}\, A = C_c\, I_{25}\, A$ = (0.31)(1.16)(9.8) = 3.5 cfs

3.2.2 CONTINUOUS MODELS AND THE RUNOFF FILES METHOD

The approved continuous model/runoff files implementations of HSPF were developed as tools that have the accuracy and versatility of HSPF but are much simpler to use and provide a framework for efficient design of onsite stormwater detention facilities. This section describes the Runoff Files Method. The term *runoff files* refers to a database of continuous flows presimulated by HSPF. The KCRTS software package has formerly been a tool for using this flow database. Current approved continuous models are listed in Reference Section 6-D (as updated); as of this writing, they include the Western Washington Hydrology Model (WWHM) and MGSFlood[7]. Projects are required to use the same model throughout unless otherwise approved through the adjustment process described in Section 1.4.

The Runoff Files method was developed as a hydrologic modeling tool for western King County to produce results (design flows, detention pond sizing, etc.) comparable to those obtained with the U.S. Environmental Protection Agency's HSPF model but with significantly less effort. This is achieved by providing the user with a set of time series files of unit area land surface runoff ("runoff files") presimulated with HSPF for a range of land cover conditions and soil types within King County. The design flows are estimated and detention facilities are designed by directly accessing and manipulating the runoff file data by means of the continuous modeling software. Typical basic capabilities of the continuous modeling software include:

- Estimating time series of flows for a specified land use and location within King County
- Analyzing flow frequency and duration
- Analyzing water surface frequency and duration
- Plotting analysis results
- Sizing detention facilities.

❑ DEVELOPMENT OF THE RUNOFF FILES

To compile the runoff files, the land surface hydrologic response (represented by a time series of unit area land surface runoff) was generated by HSPF with regional parameters for a variety of land use classifications and for a long-term (over 50-year) rainfall station representing the western lowlands of King County (Sea-Tac Airport). A 158-year extended precipitation timeseries (Puget East) was also developed by MGS Consulting. The City allows the use of either the 50-year Sea-Tac Airport gage data or the 158-year simulated timeseries for sizing. The methods for developing the runoff files are specific to the individual approved models. Consult the program documentation and the software provider's website information for the particular model for background on the development of the runoff files for that model.

Runoff time series were generated with data from these and other stations for the following **eight soil/land cover types:**

- Impervious
- Till forest
- Till pasture
- Till grass
- Outwash forest
- Outwash pasture
- Outwash grass
- Wetland.

HSPF and the approved models simulate surface runoff, interflow, and groundwater flow. Groundwater flow, **induced by surface runoff or occurring naturally**, is usually lost from the system through the

[7] King County no longer provides development, training and maintenance of the KCRTS model, and provides limited support dependent on staff availability.

analysis, but may require consideration in the analysis if it expresses to the surface. Consult the user's guide for application of the interflow and groundwater components of runoff in the approved continuous model.

3.2.2.1 GENERATING TIME SERIES

Most hydrologic analyses will require time series of flows for different land use conditions. For example, to size a flow control detention facility to meet the Peak Rate Flow Control Standard, the 2-, 10-, and 100-year peaks from the facility discharge time series must be compared with 2-, 10-, and 100-year peaks from the predevelopment time series. To generate a flow time series with the approved continuous model, depending on the model used, the program applies the following:

1. As determined by selecting the project's location on a map,
   - The **rainfall region** within which the project lies (i.e., Sea-Tac) and multiplier (a **regional scale factor** applied to the runoff files) to account for variations in rainfall volumes between the ***project site*** and the rainfall station, *or*
   - A calibrated **area-specific rainfall map** developed from the Sea-Tac rainfall data, *or*
   - A long-term (158-year) simulated precipitation timeseries (i.e., Puget East), *or*
   - Site specific calibrated rainfall data. See the approved model's documentation for background on the development of the runoff files for the model.
2. The **time step** to be used in the analysis. As of this manual update, **15-minute time steps are required for all applications including detention sizing and volume analysis**.
3. The **complete historical runoff record** used in the analysis:
4. The **amount of land** (acreage) of each soil/cover group for the subbasin under study, as calculated per model methodology and the methods described in this chapter.
5. If applicable, the **percentage of impervious area** that is effectively connected to the drainage system, typically accounted for by adjusting actual impervious area for the model inputs.

See the user's documentation for the approved model for methodology and guidance for generating a new time series. See Reference Section 6-D for specific guidance to be used with this manual.

❑ SELECTION OF PRECIPITATION RECORD AND REGIONAL SCALE FACTOR

As noted in the previous section, runoff files were developed using rainfall data from Sea-Tac Airport.

The *regional scale factor* is a geographically variable multiplier applied to the flow time series to account for the variations in rainfall amounts, and hence runoff. Whereas previous models (e.g., KCRTS) required determination by mapped values as data input, the scaling effects are determined in the currently approved continuous models (e.g., WWHM and MGS Flood) by selecting the project location within the model. See the approved model user's documentation for background and guidance.

Alternatively, the user can select the 158-year simulated precipitation timeseries (Puget East) for sizing. This precipitation timeseries can be found by selected "Use WS-DOT data" in WWHM or under "Extended Timeseries" in MGS Flood. A scaling factor does not need to be applied to the Puget East precipitation timeseries.

❑ CATEGORIZATION OF SOIL TYPES AND LAND COVER

The Runoff Files method typically supports several land use classifications, including till forest, till pasture, till grass, outwash forest, outwash pasture, outwash grass, wetland, and impervious. These classifications incorporate both the effects of soil type and land cover. In the SCS method, four different hydrologic soil groups are defined (A, B, C, and D) based on soil type as mapped by the SCS. The SCS also defines hydrologic response for about a dozen different land use or cover types. The SCS method

therefore allows the user a considerably greater degree of flexibility in defining land cover and soil types than do continuous models. However, the flexibility and apparent detail available with the SCS method cannot be supported on the basis of the data used to develop that method. The Runoff Files method minimizes the number of land use classifications, thereby simplifying both the analysis and review of development proposals.

Soil Groups for the Continuous Model

The following soil characterization is generally true for continuous models; however, consult the model documentation for specific applicability.

Till Soils

Till soils are underlain at shallow depths by relatively impermeable glacial till. The principal SCS soil group within the City classified as a till soil is the Alderwood series (SCS hydrological soil group C). The hydrologic response of till soils in an undeveloped, forested state is characterized by relatively slight surface runoff, substantial interflow occurring along the interface between the till soil and the underlying glacial till, and slight groundwater seepage into the glacial till.

Bedrock soils, primarily Beausite and Ovall soils in King County, are underlain by either sandstone or andesite bedrock, and a large group of alluvial soils.

Alluvial soils are found in valley bottoms. These are generally fine-grained and often have a high seasonal water table. There has been relatively little experience in calibrating the HSPF model to runoff from these soils, so in the absence of better information, these soils have been grouped as till soils. Most alluvial soils are classified by the SCS in hydrologic soil groups C and D.

Outwash Soils

Outwash soils are formed from highly permeable sands and gravels. The principal SCS soil group classified as an outwash soil is the Everett series. Where outwash soils are underlain at shallow depths (less than 5 feet) by glacial till or where outwash soils are saturated, they may need to be treated as till soils for the purpose of application in the model. Refer to the model documentation for specifics.

Wetland Soils

Wetland soils have a high water content, are poorly drained, and are seasonally saturated. For the purposes of applying continuous modeling in King County, wetland soils can be assumed to coincide with wetlands as defined in the critical areas code (RMC 4-3-050).

The approximate correspondence between SCS soil types and the appropriate soil group for typical continuous modeling is given in Table 3.2.2.A (refer to the model documentation for specific soil group application for the model). If the soils underlying a proposed project have not been mapped, or if existing soils maps are in error or not of sufficient resolution, then a soils analysis and report shall be prepared and stamped by a ***civil engineer*** with expertise in soils to verify underlying soil conditions.

TABLE 3.2.2.A EQUIVALENCE BETWEEN SCS SOIL TYPES AND TYPICAL CONTINUOUS MODELING SOIL TYPES

SCS Soil Type	SCS Hydrologic Soil Group	Soil Group for Continuous Model	Notes
Alderwood (AgB, AgC, AgD)	C	Till	
Arents, Alderwood Material (AmB, AmC)	C	Till	
Arents, Everett Material (An)	B	Outwash	1
Beausite (BeC, BeD, BeF)	C	Till	2
Bellingham (Bh)	D	Till	3
Briscot (Br)	D	Till	3
Buckley (Bu)	D	Till	4
Earlmont (Ea)	D	Till	3
Edgewick (Ed)	C	Till	3
Everett (EvB, EvC, EvD, EwC)	A/B	Outwash	1
Indianola (InC, InA, InD)	A	Outwash	1
Kitsap (KpB, KpC, KpD)	C	Till	
Klaus (KsC)	C	Outwash	1
Neilton (NeC)	A	Outwash	1
Newberg (Ng)	B	Till	3
Nooksack (Nk)	C	Till	3
Norma (No)	D	Till	3
Orcas (Or)	D	Wetland	
Oridia (Os)	D	Till	3
Ovall (OvC, OvD, OvF)	C	Till	2
Pilchuck (Pc)	C	Till	3
Puget (Pu)	D	Till	3
Puyallup (Py)	B	Till	3
Ragnar (RaC, RaD, RaC, RaE)	B	Outwash	1
Renton (Re)	D	Till	3
Salal (Sa)	C	Till	3
Sammamish (Sh)	D	Till	3
Seattle (Sk)	D	Wetland	
Shalcar (Sm)	D	Till	3
Si (Sn)	C	Till	3
Snohomish (So, Sr)	D	Till	3
Sultan (Su)	C	Till	3
Tukwila (Tu)	D	Till	3
Woodinville (Wo)	D	Till	3

Notes:

1. Where outwash soils are saturated or underlain at shallow depth (<5 feet) by glacial till, they should be treated as till soils.
2. These are bedrock soils, but calibration of HSPF by King County shows bedrock soils to have similar hydrologic response to till soils.
3. These are alluvial soils, some of which are underlain by glacial till or have a seasonally high water table. In the absence of detailed study, these soils should be treated as till soils.
4. Buckley soils are formed on the low-permeability Osceola mudflow. Hydrologic response is assumed to be similar to that of till soils.

Land Cover Types in Continuous Modeling

Continuous models support land cover types including forest, pasture, grass, and impervious. These cover types shall be applied in accordance with Core Requirement #3 and as specified in Table 3.2.2.B. Predevelopment land cover types are determined by whether the project is in a Peak Rate Flow Control Standard Area or Flow Control Duration Standard Area and whether the area in question is a target surface, as defined in Section 1.2.3.1. Target surfaces within Peak Rate Flow Control Standard Areas and Flow Control Duration Standard Matching Existing Condition Areas and non-target surfaces are modeled as ***existing site conditions***; for target surfaces in Flow Control Duration Standard Matching Forested Condition Areas, the predeveloped condition is assumed to be forested (***historical) site conditions***.

TABLE 3.2.2.B CONTINUOUS MODEL COVER GROUPS AND AREAS OF APPLICATION

Continuous Model	Application	
Cover Group	**Predevelopment**	**Post-Development**
Forest	All forest/shrub cover, irrespective of age.	All permanent (e.g., protected by covenant or critical area designation) 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 areas, except for lawns in redevelopment areas with predevelopment densities in excess of 4 DU/GA.	Unprotected forest in rural residential development shall be considered half pasture, half grass. Pasture areas to be retained on large rural residential lots (10 acres or greater) may be modeled as half pasture, half grass.
Grass	Lawns in redevelopment areas with predevelopment densities in excess of 4 DU/GA.	All post-development grassland and landscaping and all onsite forested land not protected by covenant or designated as a protection area (wellhead, wetland, or buffer) in RMC 4-3-050 (except in rural areas as noted above). For purposes of runoff modeling, underdrained pervious areas may be modeled explicitly to account for attenuation and infiltration, or may be modeled as 50% impervious/50% grass where either: (a) there is no added liner, (b) where the added liner is a treatment liner, or (c) where the added liner is one that does not restrict infiltration rates below the in situ soil infiltration rate. Other lined underdrained systems must be modeled explicitly or as 100% impervious.
Wetland	All delineated wetland areas.	All delineated wetland areas.
Impervious(1)	All impervious surfaces, including heavily compacted gravel and dirt roads, parking areas, etc., and open water bodies (ponds and lakes).	All impervious surfaces, including compacted gravel and dirt roads, parking areas, etc., and open water bodies, including onsite detention and water quality ponds.(2)

(1) Impervious acreage used in computations should be the **effective impervious area (EIA)**. This is the effective area as determined through layouts of the proposal and on-site BMP credit reductions from Table 1.2.9.A in Chapter 1. Non-effective impervious areas are considered the same as the surrounding pervious land cover.

(2) To avoid iterations in the facility sizing process, the “assumed size” of the facility need only be within 80% of the final facility size when modeling its contribution of runoff from direct rainfall.

The following factors are considered in specifying the above land cover types to be used in hydrologic analysis with continuous modeling:

- Cover types are applied to anticipate ultimate land use conditions. For example, probable clearing of woodland after development is nominally complete suggests that the post-development land use be specified as grassland (either pasture or grass) unless the forest cover is protected by covenant.
- In areas of redevelopment, there are often significant changes between the predevelopment and post-development efficiencies of the drainage system. For example, in conversion of low density residential areas to higher density land use, impervious areas prior to redevelopment may not be efficiently connected to a drainage system (e.g., downspouts draining to splash blocks, ditched instead of piped roadway systems). These problems are addressed by defining an "effective impervious fraction" for existing impervious areas and by generally requiring predevelopment grasslands to be modeled as pasture land.
- All onsite, predevelopment forest/shrub cover and all offsite forest/shrub cover is defined as "forest," irrespective of age. Post-development onsite land use is defined as forested only if forested areas are in a critical area buffer or are otherwise protected and will have a minimum 80% canopy cover within 5 years. In urban areas, unprotected onsite forest cover should be treated as grass in the post-development analysis. In rural areas, unprotected forest cover should be assumed 50% grass, 50% pasture.
- The HSPF grass parameters were developed by the USGS study of regional hydrology and have generally been interpreted as providing the hydrologic response for "urban" grasslands (lawns, etc.), which have relatively low infiltration rates and are drained effectively. The HSPF "pasture" parameters were developed to provide a hydrologic response intermediate to the USGS forest and grass parameters, as might be typified by ungrazed or lightly grazed pasture with good grass cover. Because it is impossible to adequately control grassland management after development, all **post-development grassland should be modeled as "grass"** (with the exception of unprotected forest, and pasture areas on large lots, in rural development as noted above). All **predevelopment grassland should be modeled as "pasture"** except for redevelopment of areas with predevelopment land use densities of 4 DU/GA or greater (which are modeled as grass).

❑ CALCULATION OF IMPERVIOUS AREA

Total Impervious Coverage

Table 3.2.2.C lists percent impervious coverage for use in continuous runoff modeling analysis of **existing** residential, commercial, and industrial areas. The tabulated figures are useful in offsite analysis that includes large developed residential areas, making a detailed survey of impervious coverage impractical.

Impervious coverage for **proposed** residential, commercial, and industrial development must be estimated for each specific proposal. Impervious coverage of streets, sidewalks, hard surface trails, etc., shall be taken from layouts of the proposal. House/driveway or building coverage shall be as follows:

- For urban residential development, the assumed impervious coverage shall not be less than 4,000 square feet per lot or the maximum impervious coverage permitted by Table 3.2.2.C, whichever is less.
- For commercial, multi-family, and industrial development, impervious coverage shall be estimated from layouts of the proposal.

TABLE 3.2.2.C MAXIMUM IMPERVIOUS COVERAGE FOR RESIDENTIAL AREAS	
Zoning Designation	**Maximum Impervious Surface Area**
Resource Conservation (RC)	Lots 5 acres or more: 20% Lots 10,000 sq ft: 55%. For each additional 10,000 sq ft increase in lot size, the impervious coverage shall be decreased by 1.75% to a minimum of 20% for a 5-acre lot Lots 10,000 sq ft or less: 55%
Residential-1 (R-1)	30%
Residential-4 (R-4)	55%
Residential-8 (R-8)	75%
Residential-10 (R-10)	Detached units: 75% Attached units: 65%
Residential-14 (R-14)	85%

Effective Impervious Area

The net hydrologic response of an impervious area depends on whether that area is effectively connected (usually by pipes or a channel) to a storm drainage system. The impervious area that the user inputs to the continuous model is the "Effective Impervious Area" (EIA).

Non-effective impervious area (i.e., total impervious area less EIA) is assumed to have the same hydrologic response as the immediately surrounding pervious area. For example, for existing residential areas with rooftops draining to splash pads on lawns or landscaping, the non-effective portion of the roof areas would be treated as pasture for predevelopment conditions (if DU/GA < 4.0) and grass for post-development conditions. *Note: Credits for infiltration/dispersion of downspouts on individual lots in proposed single family residential subdivisions are applied separately on a site-specific basis. Core Requirement #9 outlines where the use of on-site BMPs may be used to reduce the effective impervious area of the project..*

The effective impervious area can be determined from detailed ***site*** surveys.

3.2.2.2 TIME SERIES STATISTICAL ANALYSIS

When using a continuous runoff model to size flow control, water quality, and conveyance facilities, design flows and durations must be determined through statistical analysis of time series data generated by the software. ***Flow frequency analysis*** is used for determining design peak flows while ***flow duration analysis*** is used for determining durations of flow exceedance.

❑ FLOW FREQUENCY ANALYSIS

Flow frequency is a commonly used but often misunderstood concept. The *frequency of a given flow* is the average return interval for flows equal to or greater than the given flow. The *flow frequency* is actually the inverse of the probability that the flow will be equaled or exceeded in any given year (the *exceedance probability*). For example, if the exceedance probability is 0.01, or 1 in 100, that flow is referred to as the 100-year flow. Assuming no underlying changes in local climate, one would expect to see about 10 peak annual flows equal to or greater than the 100-year flow in a 1,000-year period. Similarly, the 2-year flow is the flow with a probability of 0.5, or 1 in 2, of being equaled or exceeded in any given year. In a 100-year period, one would expect to observe 50 peak annual flows greater than or equal to the 2-year flow. The number of peak annual flows actually equal to the 2-year flow may be zero, since peak annual flows come from a continuous spectrum.

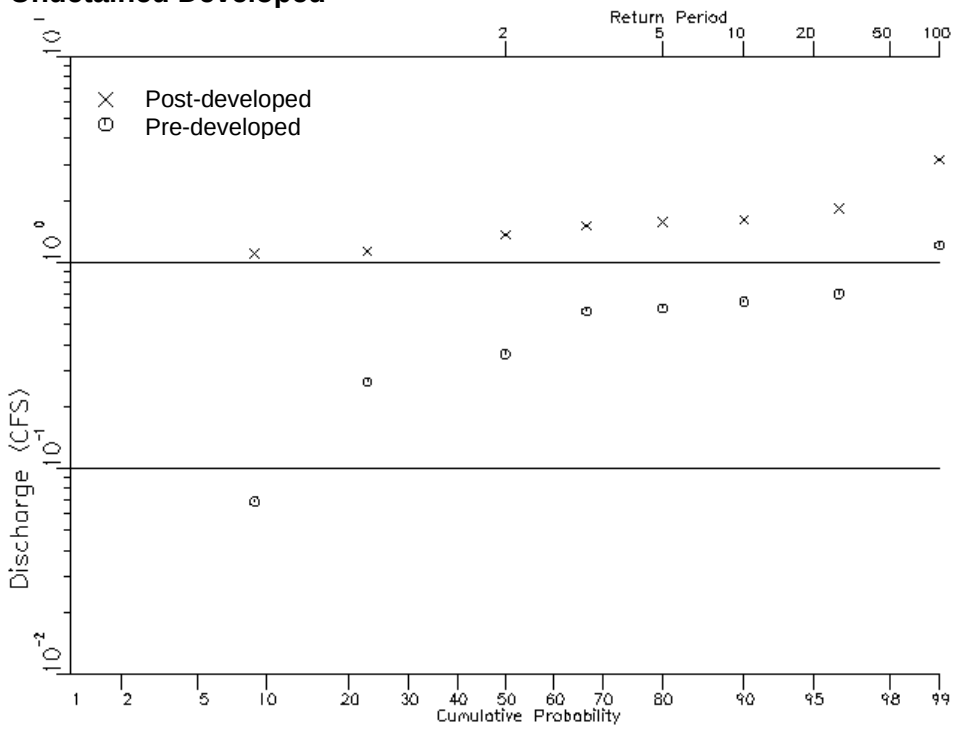
There are many **methods for estimating exceedance probabilities** and therefore flow frequencies. The *USGS Bulletin 17B* methods are commonly used, as are graphical methods using the Gringorten, Cunane,

or Weibull plotting schemes (Maidment, 1993). Graphical methods for flow frequency estimation involve assigning exceedance probabilities, and therefore return intervals, to each annual peak in a series of annual peak observations, and then plotting the peak flows against their assigned return periods. This plot is known as a *flow-frequency curve*, and it is a very useful tool for analyzing flood probabilities. Examples of flow-frequency curves for a small basin under various conditions are shown in Figure 3.2.2.A.

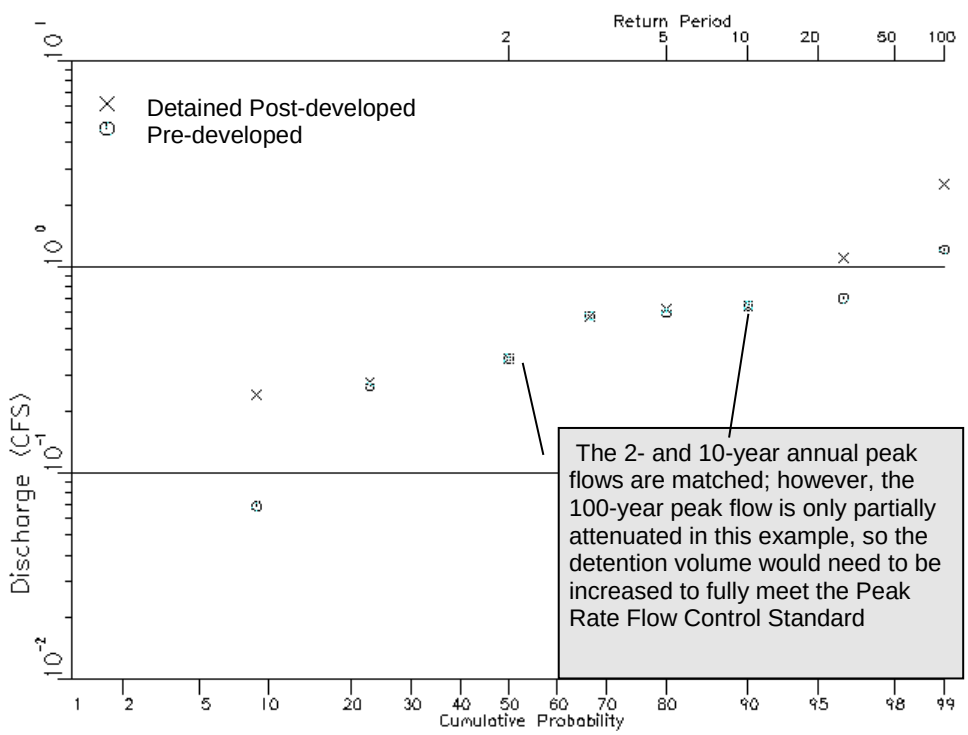
Flow-frequency curves are used in continuous flow simulations to determine the effect of land use change and assess the effectiveness of detention facilities. Using continuous methodology to design detention facilities to control peak flows, the analyst must match (i.e., *not exceed*) the post-development (detained) and predevelopment flow-frequency curves at the frequencies of interest, as shown in Figure 3.2.2.A, rather than match specific design events as when using an event model.

FIGURE 3.2.2.A EXAMPLE FLOW FREQUENCY ANALYSIS

Undetained Developed



Detained Developed



Flow frequency information is derived from the time series flow file by plotting the peak annual events in the runoff file and calculating runoff frequencies using a Log Pearson distribution or other statistical analysis. Typical return periods calculated in continuous models are the 100-year, 50-year, 25-year, 10-year, 5-year, 3-year, 2-year, and lesser storms for low-flow regime, LID and water quality applications.

❑ FLOW DURATION ANALYSIS

Flow duration analysis is important because it identifies the changes in durations of all high flows rather than simply the change in frequency of the peak annual flows. Channel scour and bank erosion rates rise proportionally with increases in flow durations. Flow duration analysis can only be conducted with continuous flow models or from gage records.

A **flow duration curve** is a plot of flow rate against the percentage of time that the flow rate is exceeded. In a continuous flow model, the *percent exceedance* of a given flow is determined by counting the number of time steps during which that flow is equaled or exceeded and dividing that number by the total number of time steps in the simulation period. Flow duration curves are usually plotted with a linear flow scale versus a log scale of percent exceedance. The log scale for exceedance percentage is used because geomorphically significant flows (flows capable of moving sediment) and flows that exceed the 2-year flow typically occur less than one percent of the total time.

❑ DURATIONS AND PEAKS FOR FLOW CONTROL STANDARDS

The Flow Control Duration Standard matching existing site conditions and Flow Control Duration Standard matching forested site conditions per Section 1.2.3.1 requires matching predevelopment and post-development flow duration curves for all flows from 50% of the 2-year flow up to the full 50-year flow.

To simplify design, brief excursions[8] of post development durations above the target predevelopment durations are allowed for matching flows greater than 50% of the predevelopment 2-year peak flow. These excursions shall not increase the duration of discharge by more than 10% at any flow level and must be strictly below the target duration curve at the low end of the range of control from 50% of the 2-year peak flow to the 2-year peak flow. This allows efficient design using only two orifices for most applications, although two-orifice designs may not allow sizing with automatic pond sizing routines; see the software documentation for guidance. An example of a flow duration analysis is shown in Figure 3.2.2.B.

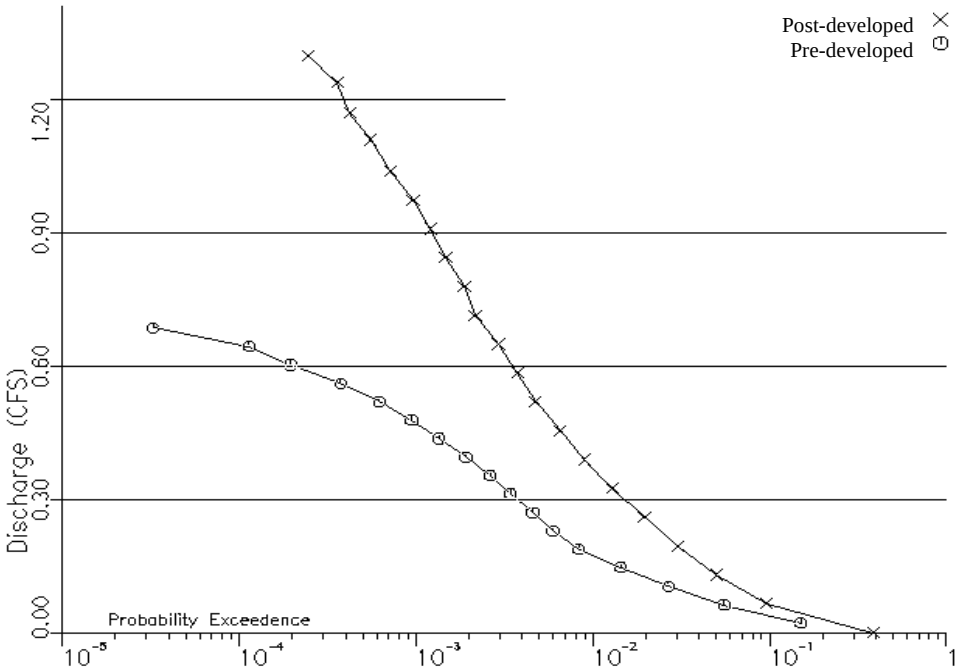
The Flood Problem Flow Control Standard matches predevelopment and post-development flow durations over the same range of predevelopment flows as the Flow Control Duration Standard and requires matching the 100-year post-development peak flow. This standard provides additional storage volume over the Flow Control Duration Standard facility, which substantially mitigates the impacts of increased volumes of surface runoff on downstream, volume-sensitive flooding problems.

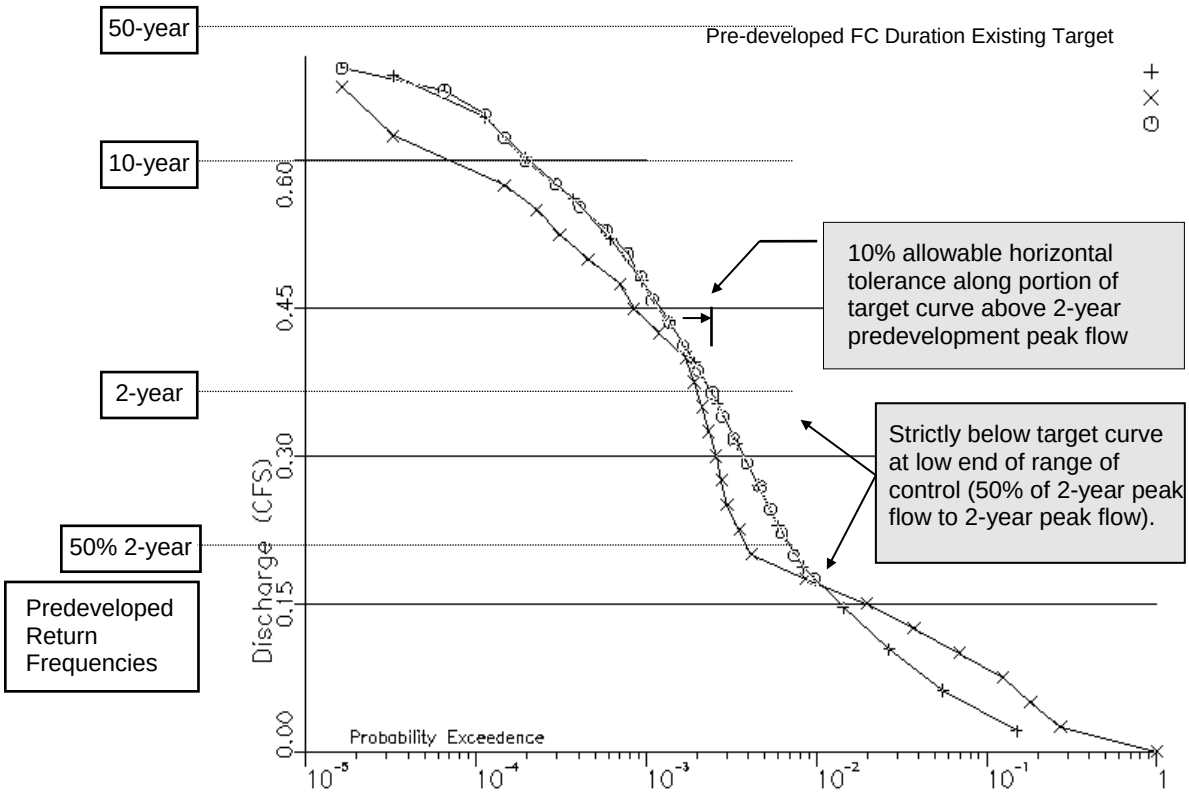
The Peak Rate Flow Control Standard does not require flow duration analysis because it addresses peak flows only (the 2-year, 10-year, and 100-year peaks).

The Low Impact Development (LID) performance standard requires that stormwater discharges shall match (i.e., *not exceed)* developed discharge durations to pre-developed durations for the range of pre-developed discharge rates from 8% of the 2-year peak flow to 50% of the 2-year peak flow. No excursions above the pre-developed durations are allowed.

[8] Brief excursions may not result in more than 50% of the target duration curve being exceeded.

FIGURE 3.2.2.B EXAMPLE FLOW DURATION ANALYSIS





When evaluating impacts to closed depressions, ponding areas and wetlands, or when evaluating for tightlined system requirements in critical areas per Core Requirement #1, frequencies of water levels or determination of *average annual runoff volumes* must be determined through statistical analysis of time series data generated using a continuous runoff model.

❑ ASSESSING WATER LEVEL STATISTICS

Stage frequency analysis consists of estimating and plotting recurrence estimates for water levels within a storage feature in the same manner as flow frequency analysis is conducted for discharges. Stage frequency analysis is required for assessing runoff impacts to offsite closed depressions and ponding areas as required under Core Requirements #2 and #3, and as discussed Section 3.3.6, "Point of Compliance Analysis," or as required for analyses of wetland impacts pursuant to Core Requirement #9.

❑ ASSESSING ANNUAL AVERAGE RUNOFF VOLUMES

To compute the *annual average runoff volume*, the volume of runoff (surface + interflow) of a time series must be computed using the approved model. The analysis is performed using the entire period of record. The total volume is divided by the number of full water years being analyzed to determine the annual average runoff volume.

3.2.3 THE APPROVED MODEL

The continuous hydrologic analysis tools prescribed in this manual are generically described as the "**approved model";** a list of the approved models is found in Reference Section 6-D. At this writing, the approved continuous hydrologic models[9] include the **Western Washington Hydrologic Model (WWHM)** and **MGS Flood**, both of which are variants of the Hydrologic Simulation Program-FORTRAN (HSPF) model. HSPF is also an approved model, but is more complex than other approved models and is typically used for basin planning and master drainage plan analyses.

General instruction and guidance for use of the approved model is found in the user's documentation for the model. Guidance specific to the City for the continuous runoff models approved for use with this manual is contained in Reference Section 6-D. A brief overview of HSPF follows below.

3.2.4 THE HSPF MODEL

HSPF is the parent model from which the other approved model methods are built. It is a very versatile continuous hydrologic/hydraulic model that allows for a complete range of hydrologic analysis. This model has been extensively used in King, Snohomish, and Thurston counties and found to be an accurate tool for representing hydrologic conditions in this area. The USGS has developed **regional parameters** to describe the common soil/cover combinations found in this area. In many cases, these regional parameters can be used to represent rainfall/runoff relationships in lieu of ***site***-specific calibration parameters.

Unfortunately, the HSPF model is very difficult to use. Design engineers using HSPF should study this model in detail and obtain training before using it on a project. For these reasons, the HSPF model is recommended only for large and complex projects where the capabilities of the approved model are too limited.

The strengths of HSPF relative to the approved model are as follows:

1. HSPF can be calibrated to local conditions.
2. HSPF can model, link, and route many separate subbasins.
3. HSPF includes the groundwater component of streamflow.

[9] KCRTS is no longer maintained by King County and is not an approved model for use with the SWDM

4. HSPF can address groundwater connections and perform low-flow analysis.
5. HSPF can handle more complex hydrologic routing (e.g., evaporation, seasonal infiltration, etc.).

The HSPF model is generally recommended for large sites where these additional features are required for comprehensive hydrologic and/or hydraulic analysis.

Anyone planning a project that is large enough to require Large Project Drainage Review and submittal of a Master Drainage Plan (MDP) per Section 1.1.2.5 should meet with CED review staff regarding appropriate hydrologic analysis prior to initiating such analysis. If a project subject to Large Project Drainage Review drains to a wetland, a salmonid stream with low-flow sensitivities, or a ***ground water protection area***, it is likely that the City will require a calibrated HSPF model. If such a project drains to erosion-sensitive streams or has features with complex hydraulics, the City may recommend or require an HSPF model using the USGS regional parameters. Smaller or less sensitive subbasins within a MDP area can be analyzed with the approved model.

Additional data is required to develop an HSPF model. At a minimum, development of an HSPF model requires collection of onsite rainfall data for a period from seven to twelve months. This data is used to determine which regional long-term rainfall record is most appropriate for modeling the ***site*** and for determining transposition factors for the long-term records. If calibration is required, the onsite rainfall data is used. Calibration also requires the installation of flow gages and the collection of flow data against which simulated flows can be compared. HSPF analysis is based on simulations with long-term rainfall records (greater than 30 years). Long-term precipitation records in HSPF format can be obtained from King County for the Sea-Tac rain gage and the Puget East 158-year simulated precipitation timeseries.

Land surface representation with HSPF follows the same procedures and classification as used with the approved model.

Conceptually, the outputs required from an HSPF analysis are consistent with those required from an approved model analysis, including frequency and durational analysis. Flow and/or water level frequencies shall be estimated using the full set of annual peaks from the long-term simulations using the *USGS Bulletin 17B* methods as well as the Gringorten or Cunane graphical methods. Durational analyses can be produced from the HSPF model and the results presented graphically. If a wetland is modeled, water level analyses may be required. Monthly, seasonal, and annual water balance and flow information, if appropriate, can be calculated with the HSPF model.

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3.3 HYDROLOGIC DESIGN PROCEDURES AND CONSIDERATIONS

This section presents the design procedures and considerations for sizing flow control facilities to meet the required hydrologic performance specified in Core Requirement #3, Section 1.2.3. It includes the following procedures and special considerations for proper hydrologic design:

- "General Hydrologic Design Process," Section 3.3.1
- "Flow Control Design Using the Runoff Files Method," Section 3.3.2
- "Conveyance System Design with the Runoff Files Method," Section 3.3.2
- "Safety Factors in Hydrologic Design," Section 3.3.4
- "Design Options for Addressing Downstream Drainage Problems," Section 3.3.5
- "Point of Compliance Analysis," Section 3.3.6
- "Onsite Closed Depressions and Ponding Areas," Section 3.3.7.

3.3.1 GENERAL HYDROLOGIC DESIGN PROCESS

This section presents the general process involved in conducting a hydrologic analysis using the runoff computation and analysis tools described in Section 3.2 to design flow control facilities for a project. The process is described as follows:

1. Review the **core and special requirements** in Chapter 1 to determine all requirements that will apply to the proposed project.
   a) Determine the applicable flow control standard (outflow performance criteria and land cover assumptions).
   b) If downstream drainage problems are identified through offsite analysis per Core Requirement #2, determine if they will necessitate additional onsite flow control or other measures as described in Section 3.3.5.
2. Determine and demonstrate in the Technical Information Report (see Section 2.3) the **predeveloped conditions** per Core Requirement #3, Flow Control (see Section 1.2.3).
3. Identify and delineate the drainage basin for each ***natural discharge location*** from the ***project site***.
   a) Identify **existing drainage features** such as streams, conveyance systems, detention facilities, ponding areas, depressions, wetlands, etc.
   b) Identify existing land uses.
   c) Identify soil types using SCS soil survey or onsite evaluation.
   d) Convert SCS soil types to soil classifications for the approved model.
4. Select and delineate **appropriate subbasins**, including subbasins tributary to major drainage features and important conveyance points, and subbasins for separate computation of onsite flows and offsite flows.
5. Determine **hydrologic parameters** for each subbasin under predeveloped conditions.
   a) Categorize soil types and land cover.
   b) Determine total impervious areas and effective impervious areas within each subbasin.
   c) Determine areas for each soil/cover type in each subbasin.

6. Determine the **runoff time series** for predeveloped conditions at each ***natural discharge location***.
   a) Compute the predeveloped condition runoff time series for each subbasin using 15-minute time steps.
   b) **For subbasins that drain to a drainage feature with significant detention storage** (e.g., existing detention facilities, ponding areas, closed depressions), route the runoff time series through the feature per the storage routing methods in the approved model. This will yield an attenuated flow series, which becomes the effective runoff time series for that subbasin.
   c) Sum the appropriate subbasin runoff time series to obtain the total runoff time series for each ***natural discharge location***.
   d) Determine the 100-year peak flow for each ***natural discharge location***.
7. Repeat Steps 4 through 6 for the proposed post-development condition.
8. Compare the **100-year peak flows** for the appropriate predeveloped and post-development conditions at each ***natural discharge location***.
   a) Check the "**Discharge Requirements**" criteria in Core Requirement #1 to determine the acceptable manner of discharge from the ***project site*** (using **existing conditions**).
   b) Check the **flow control exemptions** in Core Requirement #3 to determine if a flow control facility is required (using ***existing site*** or ***historical site conditions***, as specified in Core Requirement #3).
   c) Check the requirement for **bypass of runoff from non-target surfaces** in Core Requirement #3 to determine if runoff from non-target surfaces must be conveyed around onsite flow control facilities (using **existing conditions**).
9. If flow control facilities are required, determine their location and make any necessary adjustments to the developed condition subbasins.
10. **Design and size each flow control facility** using the methods described in Section 3.2 and the Runoff Files Method design procedure in Section 3.3.2.
    a) Analyze the appropriate predeveloped condition runoff time series to determine target release rates for the proposed facility. *Note: If the target release rates are zero, an infiltration facility will be required.*
    b) Compute the post-development runoff time series for the proposed facility.
    c) Use the post-development runoff time series and an iterative process to size the facility to meet the required level of performance set forth in Core Requirement #3. See the approved model user's documentation for procedures in sizing flow control facilities using continuous flow time series.
11. **Design required onsite conveyance systems** using the appropriate runoff computation method (either the Rational method or the Runoff Files method with 15-minute time steps) as specified in Section 3.2.

3.3.2 FLOW CONTROL DESIGN USING THE RUNOFF FILES METHOD

Flow control facility design using the approved modeling software involves four basic steps:

1. Determining the statistical characteristics (peaks or durations) of predevelopment flows (using 15-minute time steps) which set the targets for the facility release rates,
2. Developing preliminary facility volume and orifice configuration,
3. Routing post-development flow time series through the preliminary facility to check performance, and
4. Iteratively revising the facility and checking performance until the target flow conditions are achieved.

Instead of using individual design rainfall events as in an event model, the design of the facility is based on simulation of the facility's performance using the full historical (over 50-years) time series record of simulated post-development flows, and also on comparison of the outflow record to characteristics of the predevelopment flow record. Final design is achieved when the outflow time series meets the target flow specifications.

Detention facility design with a continuous model is based on aggregate flow statistics, not upon individual storms. *When designing detention facilities with a continuous model, the return period of the peak flow leaving the facility for a particular event may not have the same return period as the peak flow entering the facility during the same event.* Unlike event models, continuous models have natural variability in the ratio of storm peak and volume. This lack of correspondence in the return periods of peak inflows and outflows in continuous models means that facility design using a continuous runoff model is more complicated than with an event method and in general has to be done on an iterative trial-and-error basis to obtain an optimal (i.e., least volume) design.

The effect of detention facilities in controlling peak flows is dependent on both the volume and peak of the inflowing hydrograph. *Generally, it is high volume storms rather than high intensity storms that cause detention facilities to fill and overtop.* The hydrographs produced by a continuous runoff model show considerable variability in the relationships between peak flows and storm volumes. For example, one event produced by high rainfall intensities in a relatively short duration storm may produce high peak flows with a relatively small hydrograph volume. By contrast, a second rainfall event may have relatively low intensities but long duration, producing a runoff hydrograph with large volumes and relatively small peak. Due to this natural variability, the peak annual outflows from a detention facility may not correspond in time to the annual peaks of the inflow record.

Similarly, the predevelopment peak annual flows may not occur during the same storm as the peak annual flows for the post-development flow series. This is because the types of storms that produce high flows from undeveloped land covers are different from those that produce high flows from impervious surfaces. Forests generate high streamflows in response to long-duration, high-volume rainfall events that soak the soil profile, whereas impervious surfaces produce the highest flow rates in response to high precipitation intensity. This is another reason why detention facility design with a continuous runoff model is based on aggregate flow statistics, not upon individual storm hydrographs.

The following is a typical procedure for hydrologic design of detention/infiltration facilities using a continuous runoff model. Specific guidance for conducting hydrologic analysis and design with the approved model is provided in the approved model user's documentation.

1. **Create time series of flows** from the predevelopment area using graphic elements that detail the predevelopment land cover, the post-development area tributary to the facility, any onsite post-development bypass area, and any offsite flow-through areas.
2. **Add any offsite flow-through time series** to the predevelopment flow time series using similar graphic elements to produce a time series of total predevelopment outflows from the ***project site***. Similarly, add the same offsite flow-through time series to the time series of post-development flows tributary to the facility to produce a time series of total post-development inflows to the facility.
3. Generate **peak annual flow estimates, flow duration curves** and **flow frequency curves** for pre- and post-development time series.
4. Enter the **Facility** element for the scenario and specify initial facility specifications for the type of facility proposed. Use of two orifices is usually sufficient for most designs. If designing an infiltration facility, the bottom orifice may be elevated or zero orifices may be specified.
5. **Route** the complete facility inflow time series through the facility. The outflow time series is automatically saved. Use the analysis tools to evaluate facility performance. **When sizing the facility to account for credits from on-site BMPs** per Core Requirement #9 and Appendix C, **note that it is necessary to *turn infiltration off* for on-line on-site BMPs draining to the facility, to avoid**

counting the flow reduction effect twice. For facilities designed using this manual, explicit modeling of infiltrative BMPs for downstream flow control facility sizing is not allowed.

6. **Adjust orifice configuration and facility size, iterate until desired performance is achieved.** Use of the automatic facility sizing routine in the approved model is helpful.
7. **Verify the facility performance** by routing the complete time series of inflows and checking the post-development peak flows and/or durations at the ***project site*** boundary against the target flows and/or durations (see the criteria for "Evaluating Flow Control Performance" provided below). When explicitly modeling BMPs for compliance with the LID Performance standard, **two separate routings are necessary** to evaluate the *flow control credit based facility performance* and the *explicitly modeled BMPs for the LID Performance standard*.

Evaluating Flow Control Performance

Evaluating the performance of facility designs intended to provide **flow frequency control** is comparatively straightforward: the post-development facility annual peak flows should be strictly less than or equal to predevelopment annual peak flows at each of the specified return periods.

Note: Peak flow matching is required per Core Requirement #3. The automatic sizing routines in the approved continuous runoff models are based on duration matching and ***do not evaluate for peak flow compliance****. The user must complete this evaluation as an additional step to verify compliance.*

Evaluating the design performance of detention facilities providing **flow duration control**, however, generally requires several iterations. In fact, considerable time could be spent attempting to match predevelopment and post-development duration curves. Some flexibility in assessing the adequacy of fit is clearly needed to expedite both design and review. Therefore, flow duration designs will be accepted as meeting performance standards when the following conditions are met:

1. The post-development **flow duration curve** lies strictly on or below the predevelopment curve at the **lower limit of the range of flow control** (between 50% of the 2-year and the 2-year).[10]
2. **At any flow value within the upper range of flow control** (from the 2-year to the 50-year), the post-development duration of the flow is no more than 1.1 times the predevelopment flow duration.
3. The **target duration curve** may not be exceeded along more than 50% of the range of control.
4. Where a facility or BMP is used to meet the LID Performance Standard, the post-development **flow duration curve** lies strictly on or below the predevelopment curve for the **range of pre-developed discharge rates for the LID Performance standard** (from 8% of the 2-year peak flow to 50% of the 2-year peak flow).[11]

[10] For small projects, the lower limit of the range of control for flow control duration standard matching existing site conditions is considered met with a minimum diameter (0.25 inches) lower orifice in a low head facility (maximum effective storage depth of 3 feet) where full duration control cannot be achieved at the lower limit. Predeveloped flow durations, within allowed tolerances, must be met for all flows above the best achievable lower limit. The LID Performance standard must also be met; performance results could be influenced by the minimum diameter.

[11] See Core Requirement #9 and Appendix C for application of pre-sized on-site BMPs for mitigating the LID Performance standard in lieu of explicit modeling

3.3.3 CONVEYANCE SYSTEM DESIGN WITH THE RUNOFF FILES METHOD

This section provides guidance for use of the Runoff Files method in determining peak flows for the design and analysis of conveyance elements, overflow structures, and other peak flow sensitive drainage features.

Rainfall events that create the highest rates of runoff from developed areas are typically shorter in duration and are characterized by brief periods of high intensity rainfall. To simulate the runoff from higher intensity, shorter duration rainfall events, **a 15-minute time series is used.**

The following is the typical procedure for hydrologic design and analysis of conveyance facilities using the Runoff Files method:

1. Select and delineate **appropriate subbasins**.
   a) Select separate subbasins for major drainage features and important conveyance points.
   b) Identify existing land covers offsite and post-development land covers onsite.
   c) Identify soil types by using the SCS soil survey or by directly evaluating the ***site***.
   d) Convert SCS soil types to the approved model soil classifications.
2. Determine **hydrologic parameters** for each subbasin.
   a) Within the approved model, locate the project to determine appropriate rainfall region and/or regional scale factor.
   b) Categorize soil types and land cover per Table 3.2.2.A and Table 3.2.2.B.
   c) Determine total impervious areas and effective impervious areas within each subbasin.
   d) Determine areas for each soil/cover type in each subbasin.
3. Determine **peak flows** for the conveyance element being analyzed.
   a) Following the approved model guidance, assemble the post-development scenario including an element for each subbasin and using 15-minute time steps.
   b) Set the point of compliance at the confluence of the post-developed subbasins being routed to the conveyance element. Run the scenario for the developed subbasins and conduct a **flow frequency analysis** on the results of the scenario run. From this analysis the 10-year, 25-year, and 100-year peak flows can be determined. These design flows can then be used to size or assess the capacity of pipe systems, culverts, channels, spillways, and overflow structures.

3.3.4 SAFETY FACTORS IN HYDROLOGIC DESIGN

It is often appropriate to apply safety factors to detention volumes or conveyance design flows. This manual does not require safety factors for detention or conveyance design, but it does recommend the use of safety factors when the designer believes the results of the approved model are not sufficiently conservative given local conditions. *The approved model methodology does not include inherent safety factors as it is meant to account for "average" conditions. On a particular* ***site****, the approved model may overestimate or underestimate flow rates and detention volumes.*

Within any soil/cover group, there is a range of hydrologic response dependent on local soil and geologic conditions for which the approved model methodology does not account. The USGS regional parameters for HSPF that were used to create the runoff files produce "average" runoff time series that overestimate peak flows in some basins and underestimate them in others. Similarly, the detention volumes designed with the approved model for a given conversion type are in the middle of the range of volumes that would be created if exact local hydrologic conditions were known for every project of that type. Therefore, some of the detention facilities designed with the approved model are oversized and some are undersized, depending on variable ***site*** conditions.

Because of the uncertainty in local hydrologic response, the City recommends, but does not require, that a volume safety factor of 10% be applied to all detention facilities. If downstream resources are especially sensitive, or if the designer believes that the approved model significantly overestimates predevelopment flows or underestimates post-development flows, a volume safety factor of up to 20% may be appropriate. If a volume safety factor is applied to a detention facility, the volume should be increased by the given percentage at each one-foot stage increment. Safety factors for conveyance systems should be evaluated with respect to the potential damages and costs of failures due to backwatering, overtopping, etc. Applications of safety factors fall strictly within a professional engineer's judgment and accountability for design. Section 4 of the Technical Information Report should state what safety factor was applied to the design of the flow control facility.

3.3.5 DESIGN OPTIONS FOR ADDRESSING DOWNSTREAM DRAINAGE PROBLEMS

See Chapter 1, Table 1.2.3.A for options for addressing downstream drainage problems.

3.3.6 POINT OF COMPLIANCE ANALYSIS

The *point of compliance* is the location where flow control performance standards are evaluated. In most cases, the point of compliance is the outlet of a proposed detention facility where, for example, 2- and 10-year discharges must match predevelopment 2- and 10-year peak flow rates.

The point of compliance for hydrologic control moves downstream of the detention facility outlet or the property boundary under the following circumstances:

1. The proposed project discharges to an **offsite closed depression with a** ***severe flooding problem*** per Section 1.2.2, and the project adds impervious surface greater than or equal to 10% of the 100-year water surface area of the closed depression (see Table 1.2.3.A). In these cases, the closed depression becomes the point of compliance, and the engineer must ensure that ***project site*** runoff does not aggravate the flooding problem (or create a new flooding problem).
2. The proposed project includes an **onsite runoff bypass,** a small developed area that bypasses the flow control facility (see Section 1.2.3.2). In such cases, runoff from the remainder of the ***project site*** is overdetained so that the sum of the detained and undetained flows meets the required flow control performance standard. The point of compliance for such projects is where the onsite bypass flows join the detained flows.

3. The proposed project **bypasses offsite flows** around an onsite closed depression, ponding area, or wetland (see Section 3.3.7). As with onsite bypasses, the point of compliance in this case is where detained flows converge with the bypassed flows.

The approved model allows multiple points of compliance for evaluating runoff performance within a scenario. The automatic facility sizing routine in the approved model requires a point of compliance to size an individual facility; a separate point of compliance is required for downstream evaluation. See the approved model user's documentation for modeling application of points of compliance to meet the requirements of this manual.

Note: When controlling flow durations at a downstream point of compliance to demonstrate no adverse impact, the 10% tolerance specified for Level 2 performance may ***not*** *be used. Predevelopment condition flow durations should be matched to the extent feasible for all flows above the level of concern. The resultant facility should also be checked to verify that the minimum onsite performance standard (e.g., Level 1, Level 2, or Level 3 per Section 1.2.3.1) has also been met.*

❑ OFFSITE CLOSED DEPRESSIONS

If a project drains to an offsite closed depression with existing or potential flooding problems, then the water surface levels of the closed depression must not be allowed to increase for return frequencies at which flooding occurs, up to and including the 100-year frequency. This section describes the point of compliance analysis necessary to size detention facilities discharging to such a closed depression. *If the closed depression is classified as a wetland, other requirements apply per Section 1.2.2, Core Requirement #2.*

The closed depression is first modeled (using the ***site's*** predevelopment condition) to determine the return frequency at which flooding currently occurs and the water levels associated with return frequencies in excess of this frequency. These flooding levels and their probabilities dictate the detention performance for the proposed development. The proposed detention facility is then iteratively sized such that discharge from the ***site's*** post-development condition does not increase water surface levels for the frequencies at which flooding occurs—that is, after development, water level frequency curves must match for all frequencies equal to or greater than the frequency at which flooding occurs (up to the 100-year water level).

The infiltration rate must be determined in order to accurately model the closed depression. *In the case of a closed depression with an existing flooding problem, the infiltration rate is most realistically depicted by calibrating the model to known flooding events.* This should be done using the full historical runoff files and setting the closed depression outflow (infiltration) such that recorded or anecdotal levels of flooding occur during the same storm events in the historical record.

Where a flooding problem might be created by discharge of post-development flows to a closed depression, and in the absence of information on dates and water surface levels in the closed depression during past runoff events, infiltration rates must be determined through testing as follows:

- For a closed depression without standing water, two or more **test pits** should be dug in the bottom of the closed depression to a depth of 10 feet or to the water table, whichever is reached first. The test pits shall be dug under the supervision of a geotechnical engineer, and a test pit log shall be kept. Evidence of high water table shall be noted.
- If the test pit reveals deep homogeneous permeable material with no evidence of a high water table, then infiltration tests shall be performed in the bottom of the closed depression at locations of similar elevation and on opposite sides of the bottom area (as feasible). **Surface infiltration rates** shall be determined using the methods for assessing measured infiltration rates included in Section 5.2. The measured rates should be used directly, without applying correction factors.
- If the closed depression has standing water or is a defined as a wetland according to RMC 4-3-050, or if test pits show evidence of a high water table or underlying impermeable material, then procedures for determining infiltration rates will be established on a case-by-case basis in coordination with CED.

- In the event that a closed depression with a documented ***severe flooding problem*** is located on private property and all reasonable attempts to gain **access to the closed depression** have been **denied**, the Flood Problem Flow Control Standard shall be applied with a 20% factor of safety on the storage volume.

❑ ONSITE RUNOFF BYPASS

It is sometimes impractical to collect and detain runoff from an entire project area, so provisions are made to allow undetained discharge from onsite bypass areas (see Section 1.2.3.2) while overdetaining the remainder of the runoff to compensate for unmitigated flows. A schematic of an onsite runoff bypass is shown in Figure 3.3.6.A.

For projects employing onsite runoff bypass, flow control performance standards are evaluated at the point of compliance, the point where detained and undetained flows from the ***project site*** are combined.

Point of Compliance Analysis for Onsite Bypass Areas

1. In the approved model, create a **predeveloped condition** element for the entire project area including the predevelopment detained area and the predevelopment bypass area. Route the scenario and apply the analysis tools to determine **flow targets** (either flow frequencies or durations, depending on the applicable design standard) from the predeveloped condition runoff time series.
2. Create and route separate **developed condition elements** for the detained area and the bypass area, producing a separate time series for each area.
3. Ensure that the flow characteristics of the developed runoff element for the bypass area do not exceed the targets determined in Step 1 or the 0.4 cfs threshold in Core Requirement #3. If the bypass area flows exceed the targets or threshold, then the bypass is not feasible.
4. Estimate **allowable release rates** from the detention facility for each return period of interest with the following equation:

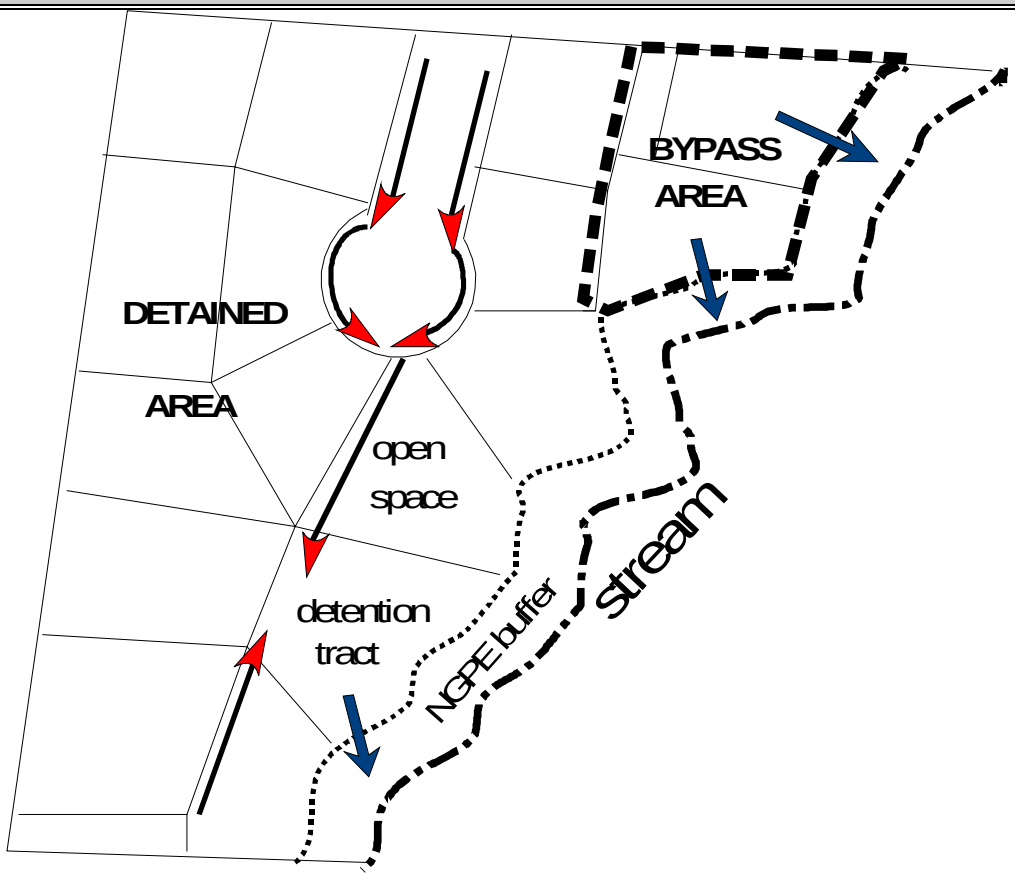
   $$\text{Allowable release} = (\text{Total Project Area Flow})_{\text{predeveloped cond.}} - (\text{Bypass Area Flow})_{\text{developed cond.}}$$

Note: WWHM 2012 and later supports the direct sizing of onsite detention facilities based on the results at a downstream point-of-compliance. See the WWHM user's documentation for further details.

1. Develop a **preliminary design of the flow control facility** based on the estimated release rate(s).
2. **Route** post-development flows from the detained area through the detention facility to create a detention facility outflow time series. Provide a downstream point of compliance and route the bypass area and the facility outflow to the downstream POC.
3. The approved model determines the **total project post-development outflow** by adding the detention facility outflow runoff time series to the post-development runoff time series from the bypass area at the downstream point of compliance. Check characteristics of the total project post-development outflow against the targets determined in Step 1.
4. If compliance is not achieved (e.g., 2- and 10-year post-development flows exceed 2- and 10-year predevelopment flows), revise the facility design (or revise the project design to reduce the bypass area) and repeat Steps 6 through 8.

For WWHM 2012 and later, Steps 6 through 8 have been automated for facility sizing by using the point of compliance option in the facility element of the model. See the WWHM user's documentation for guidance.

FIGURE 3.3.6.A SCHEMATIC OF AN ONSITE RUNOFF BYPASS



3.3.7 ONSITE CLOSED DEPRESSIONS AND PONDING AREAS

Onsite closed depressions, ponding areas, and wetlands require special consideration when determining detention performance targets; if altered, they can shift the point of compliance downstream. However, the critical areas code (RMC 4-3-050) regulates wetlands (*note that most closed depressions and ponding areas are wetlands by definition*) and generally does not permit alteration through either filling or gross hydrologic changes such as bypassing offsite flows. *Note: Post-development discharges to offsite closed depressions, ponding areas, or wetlands (with the exception of those in Flood Problem Flow Control Areas per the Flow Control Applications Map or those discussed in Section 3.3.6) are normally not required to meet special performance standards unless there is a* ***severe flooding problem*** *as defined in Section 1.2.2.*

❑ GENERAL REQUIREMENTS

The following general requirements apply to **onsite closed depressions, ponding areas, and wetlands** (referred to below as "features"):

1. **Flow attenuation** provided by onsite wetlands and ponding areas, and storage provided by onsite closed depressions must be accounted for when computing both existing onsite and offsite flows.

   - **Existing onsite flows** must be routed through onsite wetlands and ponding areas to provide accurate target release rates for the developed ***site***. *Note: Closed depressions will have no outflow for some portions of the* ***site*** *for some events, although overflow may occur during extreme events.*
   - **Existing offsite flows** will increase at the project boundary if the feature is filled or if the offsite flows are bypassed around the feature. To compensate, post-development onsite flows must be

overdetained, and the point of compliance will shift downstream to where the detained flows converge with the bypassed offsite flows.

2. **If the onsite feature is used for detention**, the 100-year floodplain must be delineated considering developed onsite and existing offsite flows to the feature. *Note: Additional storage volume may be necessary within the feature, and the point of compliance is the discharge point from the feature.*

3. **If the detention facility for the proposed project discharges to an onsite wetland, ponding area, or closed depression that is not altered**[12] **by the proposed project, AND Flow Control Duration or Flood Problem Flow Control is provided**, the point of compliance is the discharge point of the detention facility, not the outlet of the onsite feature. If **Peak Rate Flow Control** is being provided, the point of compliance is the outlet of the onsite feature.

❑ FLOODPLAIN DELINEATION FOR LAKES, WETLANDS, CLOSED DEPRESSIONS, AND PONDING AREAS

A minor floodplain analysis is required for onsite or adjacent lakes, wetlands, and closed depressions that do not have an approved floodplain or flood hazard study (see Section 4.4.2; note the exceptions). Minor floodplain studies establish an assumed base flood elevation below which development is not allowed.

The following are guidelines for minor floodplain analysis of volume sensitive water bodies:

1. **Create time series** representing tributary flows to the feature from the entire tributary area. Where the feature is contained entirely onsite and where no offsite flows exist, use the tributary area for the proposed developed condition.

2. Where the feature is only partially onsite, or where there are offsite flows to the feature, **assume the entire tributary area is fully built out under current zoning**, accounting for required open space and protected critical areas in the basin as well as impervious surfaces and grass.

3. **For potential future development, assume detention standards per Section 1.2.3.1. For simplicity the proposed detention may be simulated with a single assumed detention pond just upstream of the feature.** This pond should be sized to the appropriate detention standard and predevelopment condition assumption as noted in Section 1.2.3.1 and will require generating a predevelopment time series for the basin. Large water bodies may provide significant floodwater storage and may also be included in the analysis. Most existing detention in the basin, with exception of that providing duration control, will have little effect on the analysis and should be discounted.

4. Sum all subbasin time series to create a **single composite time series** for the drainage feature.

5. Develop **routing curves** for the feature. As appropriate, consider infiltration as an outflow for closed depressions.

6. **Route the time series** through the storage feature, generate water surface frequency curves, and note the 100-year water surface elevation.

[12] *Not altered* means existing on- and offsite flows to the feature will remain unchanged and the feature will not be excavated or filled.