Stormwater Management Manual for Western Washington

Volume I - Minimum Technical Requirements and Site Planning
Volume II - Construction Stormwater Pollution Prevention
Volume III - Hydrologic Analysis and Flow Control Design/BMPs
Volume IV - Source Control BMPs
Volume V - Runoff Treatment BMPs

Prepared by:

Washington State Department of Ecology
Water Quality Program

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With a publication of this size and complexity there will inevitably be errors that must be corrected and clarifications that are needed. There will also be new information and technological updates. Ecology intends to publish corrections, updates and new technical information on our Stormwater Homepage. This web site will not be used to make revisions in key policy areas – such as the thresholds and minimum requirements in Volume I. We encourage you to check this site periodically and incorporate corrections and updates into your copies of the Manual. You can also visit this web site for updates and additional information about other Ecology stormwater activities.

Foreword

Objective of the Manual

Urban development causes significant changes in patterns of stormwater flow from land into receiving waters. Water quality can be affected when runoff carries sediment or other pollutants into streams, wetlands, lakes, and marine waters or into ground water. Stormwater management can help to reduce these effects. Stormwater management involves careful application of site design principles, construction techniques to prevent sediments and other pollutants from entering surface or ground water, source controls, and treatment of runoff to reduce pollutants and the impact of altered hydrology.

The objective of this manual is to provide a commonly accepted set of technical standards and guidance on stormwater management measures that will control the quantity and quality of stormwater produced by new development and redevelopment. The Department Ecology believes that when the standards and recommendations of this manual are properly applied, stormwater runoff should generally comply with water quality standards and protect beneficial uses of the receiving waters. We recognize that individual circumstances vary greatly, and in some instances compliance with the manual may not ensure compliance with water quality standards.

Development of The Manual

The Ecology stormwater manual was originally developed in 1992 in response to a directive of the Puget Sound Water Quality Management Plan.

In preparing this revision to the1992 Manual, Ecology has relied heavily on contributions from advisory committees. There were five separate advisory committees, with nearly 100 members, representing a broad range of expertise and interests. Their insights and practical knowledge – gained from years of experience in the field – have been particularly valuable.

Two public review drafts were prepared and presented at public workshops. Ecology staff reviewed numerous public comments and in consultation with the advisory committees, incorporated many of those comments into the final document.

What Are Some of the Key Revisions to the Manual?

The manual has been made easier to use. In this publication, we have revised the format and organization of material to make it more "user friendly," we have improved the graphics, and we have included an index.

The geographic scope of the manual has been expanded to include all of Western Washington. New federal regulations under the Clean Water Act and the Safe Drinking Water Act, as well as state regulations under the Growth Management Act, make it necessary to expand the scope of the manual to include regions outside Puget Sound. Ecology is working with Eastern Washington communities and other interested parties to complete a separate manual for Eastern Washington.

Technical material has been updated. Our knowledge of the impacts of stormwater runoff and the methods for controlling it has improved. New research findings, changes in federal stormwater regulations, and proposed and actual listings under the Endangered Species Act (ESA) call for significant changes in the way we manage urban runoff. We have updated the manual to include new information and standards that we believe are more protective. Those changes include:

- Changing thresholds for selection of Best Management Practices (BMPs) to require nearly all projects to apply appropriate flow control and runoff treatment BMPs – including on-site stormwater management techniques.
- Increasing flow control requirements to address both peak flows and duration of high flows, and calling for the use of continuous runoff models when available.
- Adding a requirement for higher levels of treatment (enhanced treatment) for discharges from most industrial, commercial, and multifamily sites and arterials and highways.

Organization of This Manual

The manual is organized into five volumes. For more information on how to use the manual, refer to Volume I, Chapter 1.

- Volume I provides an introduction and overview, establishes Minimum Requirements applicable to new development and redevelopment projects, and provides site planning guidance.
- Volume II covers stormwater pollution prevention at construction sites with a primary focus on erosion and sediment control.
- Volume III covers hydrologic analysis methods for estimating pre- and post-developed runoff quantities and flow rates, and provides details of detention facility design, construction, and maintenance.

- Volume IV addresses control of runoff pollution produced by urban land uses with a primary emphasis on source control BMPs.
- Volume V provides the details of treatment BMP selection, design, construction, and maintenance.

How is the Manual Applied?

The users of this manual will be engineers, planners, environmental scientists, plan reviewers, and inspectors at the local, state, and federal government levels and private industry. Local government officials may adopt and apply the requirements of this manual directly or adopt and apply the requirements of an equivalent manual. Local government staff may use this manual, or their own manual, as a reference for reviewing stormwater site plans, checking BMP designs, and for providing technical advice in general. Private industry may use the manual for information on how to develop and implement stormwater site plans, and as a reference for technical specifications of BMPs.

The manual itself has no independent regulatory authority. The minimum requirements and technical guidance in the manual only become required through:

- Ordinances and rules established by local governments; and,
- Permits and other authorizations issued by local, state, and federal authorities.

Adoption of either Ecology's manual or an equivalent manual is a key element of local stormwater programs called for in the Puget Sound Water Quality Management Plan.

Adoption of either Ecology's manual or an equivalent manual is required for all municipalities currently covered under the National Pollutant Discharge Elimination System (NPDES) Municipal Stormwater Permit. The manual is also referenced in Ecology's construction and industrial stormwater permits.

Under new federal regulations, many additional cities and counties will be required to apply for coverage under an NPDES Permit. In order to satisfy federal regulations, Ecology intends to require that those jurisdictions adopt either Ecology's manual or an equivalent manual.

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Stormwater Management in Western Washington

Volume III
Hydrologic Analysis and Flow Control Design/BMPs

Prepared by:
Washington State Department of Ecology
Water Quality Program

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Credit for Figures

The figures in Chapter 3 are reproduced, with permission, from King County's Surface Water Design Manual.

Chapter 1 – Introduction

1.1 Purpose of this Volume

Best Management Practices (BMPs) are schedules of activities, prohibitions of practices, maintenance procedures, managerial practices, or structural features that prevent or reduce adverse impacts to waters of Washington State. As described in Volume I of this stormwater manual, BMPs for long-term management of stormwater at developed sites can be divided into three main categories:

- BMPs addressing the volume and timing of stormwater flows;
- BMPs addressing prevention of pollution from potential sources; and
- BMPs addressing treatment of runoff to remove sediment and other pollutants.

This volume of the stormwater manual focuses mainly on the first category. It presents techniques of hydrologic analysis, and BMPs related to management of the amount and timing of stormwater flows from developed sites. The purpose of this volume is to provide guidance on the estimation and control of stormwater runoff quantity.

BMPs for preventing pollution of stormwater runoff and for treating contaminated runoff are presented in Volumes IV and V, respectively.

1.2 Content and Organization of this Volume

Volume III of the stormwater manual contains three chapters. Chapter 1 serves as an introduction. Chapter 2 reviews methods of hydrologic analysis, covers the use of hydrograph methods for designing BMPs, and provides an overview of various computerized modeling methods and analysis of closed depressions. Chapter 3 describes flow control BMPs and provides design specifications for roof downspouts and detention facilities. It also provides design considerations of infiltration facilities for flow control.

The three Appendices to this volume contain the isopluvial maps for western Washington, information and assumptions on the western Washington hydrology model, and more detailed information on pilot infiltration testing.

Design considerations for conveyance systems are not included in the stormwater manual, as this topic is adequately covered in standard engineering references.

1.3 How to Use this Volume

Volume I should be consulted to determine Minimum Requirements for flow management (e.g. Minimum Requirements #4, #5 and #7 in Chapter 2 of Volume I). After the Minimum Requirements have been determined, this volume should be consulted to design flow management facilities. These facilities can then be included in Stormwater Site Plans (see Volume I, Chapter 3).

Chapter 2 - Hydrologic Analysis

The broad definition of hydrology is "the science which studies the source, properties, distribution, and laws of water as it moves through its closed cycle on the earth (the hydrologic cycle)." As applied in this manual, however, the term "hydrologic analysis" addresses and quantifies only a small portion of this cycle. That portion is the relatively short-term movement of water over the land resulting directly from precipitation and called surface water or stormwater runoff. Localized and long-term ground water movement must also be of concern, but generally only as this relates to the movement of water on or near the surface, such as stream base flow or infiltration systems.

The purpose of this chapter is to define the minimum computational standards required, to outline how these may be applied, and to reference where more complete details may be found, should they be needed. This chapter also provides details on the hydrologic design process; that is, what are the steps required in conducting a hydrologic analysis, including flow routing.

2.1 Minimum Computational Standards

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

1. For the purpose of designing runoff treatment BMPs, a calibrated continuous simulation hydrologic model based on the EPA's HSPF (Hydrologic Simulation Program-Fortran) should be used to calculate runoff and determine the water quality design flow rates. In the absence of a continuous model, the Soil Conservation Service (SCS) now Natural Resources Conservation Service (NRCS) Unit Hydrograph (SCSUH) method, or equivalent hydrograph techniques such as the Santa Barbara Urban Hydrograph (SBUH) method must be used to calculate runoff and determine the water quality design flow rates.

   For the purpose of designing runoff treatment BMPs that are sized based upon the volume of runoff (wetpool treatment facilities), the NRCS curve number method should be used to determine the water quality design storm. The water quality design storm is the volume of runoff predicted from the 6-month, 24-hour storm.

   For the purpose of designing flow control BMPs, a calibrated continuous simulation hydrologic model, based on the EPA's HSPF, must be used where available. Where a calibrated continuous hydrologic model is not available the use of the SBUH method with the parameters specified in Volume I Minimum

Requirement # 7 “Interim Guideline” is recommended for runoff flow control purposes.

The circumstances under which different methodologies apply are summarized below.

Summary of the application design methodologies		
	BMP designs in western Washington	
Method	Treatment	Flow Control
SCSUH/SBUH	Method applies for BMPs that are sized based on the design 24-hr runoff volume in Volume 5. **Note**: These BMPs don’t require generating a hydrograph.	Modified method applies where an approved continuous runoff model is not available. See Volume I, Minimum Requirement # 7: Flow Control, “Interim Guideline”
Continuous Model	Method applies for BMPs that are sized based on the design runoff flow rates in Volume 5.	Method applies where available

2. If a basin plan is being prepared, then the hydrologic analysis must be performed using a continuous simulation model such as the EPA's HSPF model, the EPA's Stormwater Management Model (SWMM), or an equivalent model as approved by the local government.

Significant progress has been made by the United States Geological Survey (in cooperation with the counties of King, Snohomish, Pierce, and Thurston) with the development of a local version of the HSPF model. This work has involved development of “runoff files” for various land types defined by vegetation, and soil type. These runoff files will describe runoff characteristics of simulated runoff from a watershed with measured runoff. As a result, one will be able to simulate runoff from any other ungauged basin where only the distribution of land types is known. The model will be able to be applied on individual development sites of less than about 200 acres.

A continuous simulation model has a considerable advantage over the single event-based methods such as the SCSUH, SBUH, or the Rational Method. The single event model cannot take into account storm events that may occur just before or just after the single event (the design storm) that is under consideration. In addition, the runoff files generated by the HSPF model are the result of a considerable effort to introduce local parameters and actual rainfall data into the model and are therefore believed to result in better estimation of runoff than the SCSUH, SBUH, or Rational methods.

2.1.1 Discussion of Hydrologic Analysis Methods Used for Designing BMPs

This section provides a discussion of the methodologies to be used for calculating stormwater runoff from a project site. It includes a discussion of estimating stormwater runoff with single event models, such as the SBUH, versus continuous simulation models.

Single Event and Continuous Simulation Model

The use of single event hydrologic models has limitations when designing flow control BMPs and efforts are underway to make improved hydrologic analysis methods more widely available and used. HSPF is a continuous simulation model that is capable of simulating a wider range of hydrologic responses than the single event models such as the SBUH method. Ecology has developed a continuous simulation hydrologic model based on the HSPF for use in western Washington (see Section 2.2). Continuous rainfall records/data files have been obtained and appropriate adjustment factors were developed as input to HSPF. Input algorithms (referred to as IMPLND and PERLND) have been developed for a number of watershed basins in King, Pierce, Snohomish, and Thurston counties. These rainfall files and model algorithms are used in the HSPF in western Washington. Local counties and municipalities will be encouraged to develop a continuous simulation model that is calibrated for their basins. However, until such a model is developed for a specific basin, the input data mentioned above must be used throughout western Washington.

The SBUH model or a calibrated continuous simulation model based on HSPF may be used for designing runoff treatment BMPs. Please note, to meet Minimum Requirement #6 - Runoff Treatment - using the SBUH model, the water quality design storm specified in Volume I must be treated. Where a continuous simulation model is available, the treatment BMPs must be sized using the appropriate design criteria specified for the BMPs in Volume 5. The discussion below will focus on the use of the SBUH method for estimating runoff and developing a runoff hydrograph.

The SBUH method, as recommended in the 1992 Manual, tends to overestimate runoff from predeveloped areas. In Volume I, certain changes to the SBUH parameters are recommended which are intended to result in more accurate estimates of runoff using SBUH. (See Minimum Requirement # 7: Flow Control, "Interim Guideline"). The suggested changes to the SBUH parameters are based on the runoff comparisons between the SBUH model and King County Runoff Time Series (KCRTS), an HSPF-based continuous simulation model.

Concerns with SBUH

A summary of the concerns with SBUH is in order.

- While SBUH may give acceptable estimates of total runoff volumes, it tends to overestimate peak flow rates from pervious areas because it cannot adequately model subsurface flow (which is a dominant flow regime for pre-development conditions in western Washington basins). One reason SBUH overestimates the peak flow rate for pervious areas is that the actual time of concentration is typically greater than what is assumed. Better flow estimates could be made if a longer time of concentration was used. This would change both the peak flow rate (i.e., it would be lower) and the shape of the hydrograph (i.e., peak occurs somewhat later) such that the hydrograph would better reflect actual predeveloped conditions.

Another reason for overestimation of the runoff is the curve numbers (CN) in the 1992 Manual. These curve numbers were developed by US-Natural Resources Conservation Service (NRCS), formerly the Soil Conservation Service (SCS) and published as the Western Washington Supplemental Curve Numbers. These CN values are typically higher than the standard CN values published in Technical Release 55, June 1986. In 1995, the NRCS recalled the use of the western Washington CNs for floodplain management and found that the standard CNs better describe the hydrologic conditions for rainfall events in western Washington. However, based on runoff comparisons with the KCRTS better estimates of runoff are obtained when using the western Washington CNs for the developed areas such as parks, lawns, and other landscaped areas. Accordingly, the CNs in this manual are changed to those in the Technical Release 55 except for the open spaces category for the developed areas which include, lawn, parks, golf courses, cemeteries, and landscaped areas. For these areas, the western Washington CNs are used. These changes are intended to provide better runoff estimates using the SBUH method.

The other major weakness of the current use of SBUH is that it is used to model a 24-hour storm event, which is too short to model longer-term storms in western Washington. The use of a longer-term (e.g. 3- or 7-day storm) is perhaps better suited for western Washington.

The SBUH model may not be adequate for modeling the hydrologic conditions in western Washington and therefore the use of a locally calibrated HSPF is recommended.

Treatment

When designing a runoff treatment BMP, SBUH or a calibrated continuous simulation hydrologic model based on HSPF may be used to develop the inflow hydrograph to the BMP. SBUH tends to underestimate the time of concentration, thus the peak flow rate occurs too early. This would affect the treatment BMPs that are designed to achieve a specified residence time (designs are more conservative). Calculation of the residence time is sensitive to the shape of the inflow hydrograph. The inflow hydrograph is also of fundamental importance when designing an infiltration or filtration BMP as these BMPs are sized based on a routing of the inflow hydrograph through the BMP. The best solution at this time is to try to account for subsurface flow when estimating the time of concentration. For sites with low impervious cover, this will increase the time of concentration, thus reducing the peak flow rate and shifting the peak rate to a somewhat later time. Note that for BMPs which maintain "permanent pools" (e.g., wet ponds) none of the above concerns apply since the permanent pool volume is adequately predicted by SBUH.

Flow Control

Where a continuous runoff model is not available, it is necessary to use a modified SBUH approach described in Volume I, Minimum Requirement # 7: Flow Control, "Interim Guideline". The modified SBUH approach approximates a design intended to achieve the flow duration standard by adjusting the target peak flow standard, restricting other variables, and applying volume correction factor. The volume correction factor in Figure 2.1 is based on the post development impervious cover and is necessary where the predeveloped condition is modeled as pasture. This correction factor is to be applied to the volume of the BMP without changing its

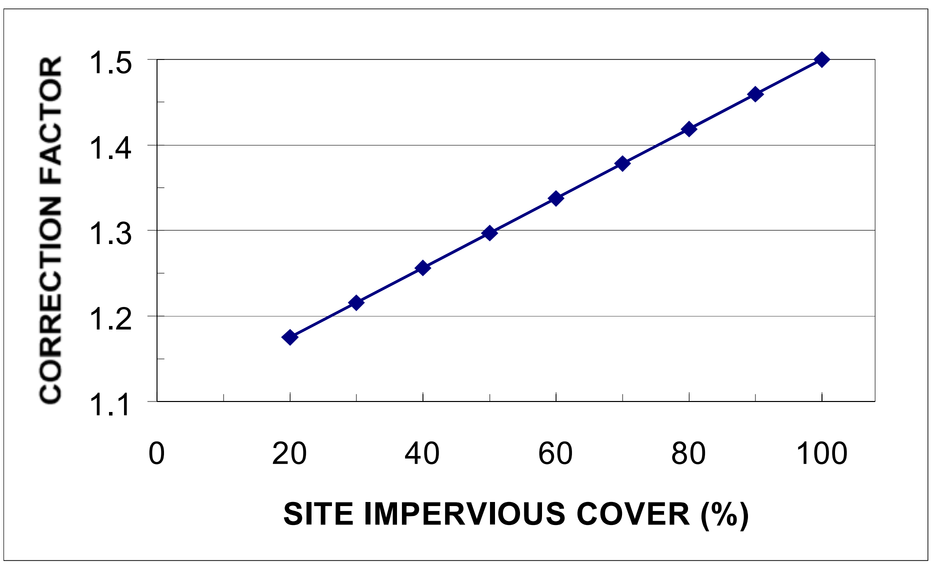


Figure 2.1 Volume Correction Factor to be Applied to Flow Control BMPs Based on Site Impervious Cover

depth or the design of the outlet structure, thus an increase in surface area will result.

Note that it is not necessary to apply the correction factor to the BMP volume for the runoff treatment storm.

Appendix III-A contains isopluvial maps for the 2, 10, and 100-year, 24-hour storm events, which are needed for matching the pre-development and post-development peak runoff associated with these storms.

Other precipitation frequency data may be obtained, for a fee, through Western Regional Climate Center (WRCC) at Tel: (775) 674-7010. WRCC can generate 1-30 day precipitation frequency data for the location of interest using data from 1948 to present (currently August 2000).

2.2 Western Washington Hydrology Model

This section summarizes the assumptions made in creating the western Washington Hydrology Model (WWHM) and discusses limitations of the model. More information on the WWHM and the assumptions can be found in Appendix III-B.

Limitations to the WWHM

The WWHM has been created for the specific purpose of sizing stormwater control facilities for new developments in western Washington. The WWHM can be used for a range of conditions and developments; however, certain limitations are inherent in this software. These limitations are described below.

The WWHM uses the EPA HSPF software program to do all of the rainfall-runoff and routing computations. Therefore, HSPF limitations are included in the WWHM. For example, backwater or tailwater control situations are not explicitly modeled by HSPF. This is also true in the WWHM.

In addition, the WWHM is limited in its routing capabilities. The user is allowed to input a single stormwater control facility and runoff is routed through this facility. If the proposed development site contains multiple facilities in series or involves routing through a natural lake, pond, or wetland in addition to a stormwater control facility then the user should use HSPF to do the routing computations and additional analysis. As of the publication date of this manual, certain model enhancements to the next version of WWHM are being planned that include adding the capability of routing through multiple facilities.

Routing effects become more important as the drainage area increases. For this reason it is recommended that the WWHM not be used for drainage areas greater than one-half square mile (320 acres). The WWHM can be used for small drainage areas less than an acre in size.

Assumptions made in creating the WWHM

Precipitation data.

- The WWHM uses long-term (43-50 years) precipitation data to simulate the potential impacts of land use development in western Washington. A minimum period of 20 years is required to simulate enough peak flow events to produce accurate flow frequency results.
- A total of 17 precipitation stations are used, representing the different rainfall regimes found in western Washington.
- These stations represent rainfall at elevations below 1500 feet - snowfall and snowmelt are not included in the WWHM.
- The primary source for precipitation data is National Weather Service stations.
- The computational time step used in the WWHM is one hour. The one-hour time step was selected to better represent the temporal variability of actual precipitation than daily data.

Precipitation multiplication factors.

- The WWHM uses precipitation multiplication factors to increase or decrease recorded precipitation data to better represent local rainfall conditions.
- The factors are based on the ratio of the 24-hour, 25-year rainfall intensities for the representative precipitation gage and the surrounding area represented by that gage's record.
- The factors have been placed in the WWHM database and linked to each county's map. They will be transparent to the general user, however the advanced user will have the ability to change the coefficient for a specific site. Changes made by the user will be recorded in the WWHM output.

Pan evaporation data.

- The WWHM uses pan evaporation coefficients to compute the actual evapotranspiration potential (AET) for a site, based on the potential evapotranspiration (PET) and available moisture supply. AET accounts for the precipitation that returns to the atmosphere without becoming runoff.
- The pan evaporation coefficients have been placed in the WWHM database and linked to each county's map. They will be transparent to the general user. The advanced user will have the ability to change the coefficient for a specific site. These changes will be recorded in the WWHM output.

Soil data.

- The WWHM uses with three predominate soil type to represent the soils of western Washington: till, outwash, and saturated.
- The user determines actual local soil conditions for the specific development planned and inputs that data into the WWHM. The user inputs the number of acres of outwash (A/B), till (C), and saturated (D) soils for the site conditions.
- Additional soils will be included in the WWHM if appropriate HSPF parameter values are found to represent other major soil groups.

Vegetation data.

- The WWHM will represent the vegetation of western Washington with three predominate vegetation categories: forest, pasture, and lawn (also known as grass).
- The WWHM assumes that predevelopment land conditions are forest (the default condition), although the user has the option of specifying pasture if there is documented evidence that pasture vegetation was native to the predevelopment site.

Development land use data.

- Development land use data are used to represent the type of development planned for the site and are used to determine the appropriate size of the required stormwater mitigation facility.
- For the purposes of the WWHM developed land is divided into two major categories: standard residential and non-standard residential/commercial.
- Standard residential development makes specific assumptions about the amount of impervious area per lot and its division between driveways and rooftops. Streets and sidewalk areas are input separately. Ecology has selected a standard impervious area of 4200 square feet per residential lot, with 1000 square feet of that as driveway, walkways, and patio area, and the remainder as rooftop area.
- The WWHM distinguishes between effective impervious area and non-effective impervious area in calculating total impervious area.
- Credits are given for infiltration and dispersion of roof runoff and for use of porous pavement for driveway areas.
- For non-standard residential/commercial development the user inputs the roof area, landscape area, street, sidewalk, parking areas, and any appropriate non-developed forest and pasture areas.
- Forest and pasture vegetation areas are only appropriate for separate undeveloped parcels dedicated as open space, wetland buffer, or park

within the total area of the development. ***Development areas must only be designated as forest or pasture where legal restrictions can be documented that protect these areas from future disturbances.***

- The WWHM provides options for bypassing a portion of the runoff from the development area around a stormwater detention facility and/or having offsite inflow enter the development area.

Application of WWHM in Re-developments Projects

Redevelopment requirements may allow, for some portions of the redevelopment project area, the predeveloped condition to be modeled as the existing condition rather than forested or pasture condition. For the purposes of modeling using WWHM, project areas where flow mitigation is not required may be modeled as Offsite-Inflow.

Pervious and Impervious Land Categories (PERLND and IMPLND parameter values)

- In WWHM (and HSPF) pervious land categories are represented by PERLNDs; impervious land categories by IMPLNDs
- The WWHM provides 16 unique PERLND parameters that describe various hydrologic factors that influence runoff and 4 parameters to represent IMPLND.
- These values are based on regional parameter values developed by the U.S. Geological Survey for watersheds in western Washington (Dinicola, 1990) plus additional HSPF modeling work conducted by AQUA TERRA Consultants.
- Surface runoff and interflow will be computed based on the PERLND and IMPLND parameter values. Groundwater flow is not computed. It is assumed that very little or no groundwater flow from small catchments reaches the surface to become runoff. This is consistent with King County procedures (King County, 1998).

Guidance for flow control standards.

Flow control standards are used to determine whether or not a proposed stormwater facility will provide a sufficient level of mitigation for the additional runoff from land development.

There are two flow control standards stated in the Ecology Manual: Minimum Requirement #7 - Flow Control and Minimum Requirement #8 - Wetlands Protection (See Volume I). Minimum Requirement #7 specifies specific flow frequency and flow duration ranges for which the postdevelopment runoff cannot exceed predevelopment runoff. Minimum

Requirement #8 specifies that discharges to wetlands must maintain the hydrologic conditions, hydrophytic vegetation, and substrate characteristics necessary to support existing and designated beneficial uses.

Minimum Requirement #7 specifies that stormwater discharges to streams shall match developed discharge durations to predeveloped durations for the range of predeveloped discharge rates from 50% of the 2-year peak flow up to the full 50-year peak flow. In addition, the developed peak discharge rates should not exceed the predeveloped peak discharge rates for 2-, 10-, and 50-year return periods. In general, matching discharge durations between 50% of the 2-year and 50-year will result in matching the peak discharge rates in this range.

- The WWHM computes the predevelopment 2- through 100-year flow frequency values and computes the post-development runoff 2- through 100-year flow frequency values from the outlet of the proposed stormwater facility.

- The model uses pond discharge data to compare the predevelopment and postdevelopment peak flows and durations and determines if the flow control standards have been met.

- There are three criteria by which flow duration values are compared:

  1. If the postdevelopment flow duration values exceed any of the predevelopment flow levels between 50% and 100% of the 2-year predevelopment peak flow values (100 Percent Threshold) then the Standard (1) flow duration requirement has not been met.

  2. If the postdevelopment flow duration values exceed any of the predevelopment flow levels between 100% of the 2-year and 100% of the 50-year predevelopment peak flow values more than 10 percent of the time (110 Percent Threshold) then the Standard (1) flow duration requirement has not been met.

  3. If more than 50 percent of the flow duration levels exceed the 100 percent threshold then the Standard (1) flow duration requirement has not been met.

Minimum Requirement #8 specifies that discharges to wetlands must maintain the hydrologic conditions, hydrophytic vegetation, and substrate characteristics necessary to support existing and designated beneficial uses. Criteria for determining maximum allowed exceedences in alterations to wetland hydroperiods are provided in guidelines cited in Guide Sheet 2B of the Puget Sound Wetland Guidelines (Azous and

Horner, 1997). Because wetland hydroperiod computations are relatively complex and are site specific they will not be included in the WWHM. HSPF is required for wetland hydroperiod analysis.

2.3 Single Event Hydrograph Method

Hydrograph analysis utilizes the standard plot of runoff flow versus time for a given design storm, thereby allowing the key characteristics of runoff such as peak, volume, and phasing to be considered in the design of drainage facilities.

The physical characteristics of the site and the design storm determine the magnitude, volume, and duration of the runoff hydrograph. Other factors such as the conveyance characteristics of channel or pipe, merging tributary flows, branching of channels, and flooding of lowlands can alter the shape and magnitude of the hydrograph. In the following sections, the key elements of hydrograph analysis are presented, namely:

Design storm hyetograph

Runoff parameters

Hydrograph synthesis

Hydrograph routing

Hydrograph summation and phasing

Computer applications

2.3.1 Design Storm Hyetograph

All storm event hydrograph methods require the input of a rainfall distribution or design storm hyetograph. The design storm hyetograph is essentially a plot of rainfall depth versus time for a given design storm frequency and duration. It is usually presented as a dimensionless plot of unit rainfall depth (increment rainfall depth for each time interval divided by the total rainfall depth) versus time.

The hyetographs in Table 2.1 represent the rainfall distributions in Washington State. The hyetograph Type IA is the standard NRCS rainfall distribution as modified by King County and resolved to 10-minute time intervals for greater sensitivity in computing peak rates of runoff in urbanizing basins of western Washington. The hyetograph was interpolated from the NRCS mass distribution by Surface Water Management Division staff from King County. It may differ slightly from the distribution used in other NRCS-based computer models, particularly those that are not resolved to 10-minute time intervals. The hyetograph Type II is the standard NRCS rainfall distribution for eastern Washington. Figure 2.2 shows the 24-hr design storm hyetographs for the Types IA and II rainfall distributions.

The design storm hyetograph is constructed by multiplying the dimensionless hyetograph times the rainfall depth (in inches) for the design storm.

The total depth of rainfall (in tenths of an inch) for storms of 24-hour duration and 2, 5, 10, 25, 50, and 100-year recurrence intervals are published by the National Oceanic and Atmospheric Administration (NOAA). The information is presented in the form of "isopluvial" maps for each state. Isopluvial maps are maps where the contours represent total inches of rainfall for a specific duration. Isopluvial maps for the 2, 5, 10, 25, 50, and 100-year recurrence interval and 24-hour duration storm events can be found in the NOAA Atlas 2, "Precipitation - Frequency Atlas of the Western United States, Volume IX-Washington." Appendix II-A provides the isopluvials for the 2, 10, and 100-year, 24-hour design storms. Other precipitation frequency data may be obtained through Western Regional Climate Center (WRCC) at Tel: (775) 674-7010. WRCC can generate 1-30 day precipitation frequency data for the location of interest using data from 1948 to present (currently August 2000).

For project sites in western Washington with tributary drainage areas above elevation 1000 MSL, an additional total precipitation must be added to the total depth of rainfall, for the 25, 50, and 100-year design storm events, to account for the potential average snowmelt which occurs during major storm events.

This snowmelt factor (M_s) may be computed as follows:

This snowmelt factor (M_s) is

M_s (in inches) = $0.004\,(MB_{el} - 1000)$;
where:
MB_{el} = the mean tributary basin elevation above sea level (in feet).

Example:
Given: Project location at an elevation of MB_{el} = 1837 feet.
Design Storm Event: 100-year P_{100} = 7 inches

Compute: $M_s = 0.004\,(MB_{el} - 1000) = (0.004)\,(1837 - 1000)$
$= 3.35$ inches

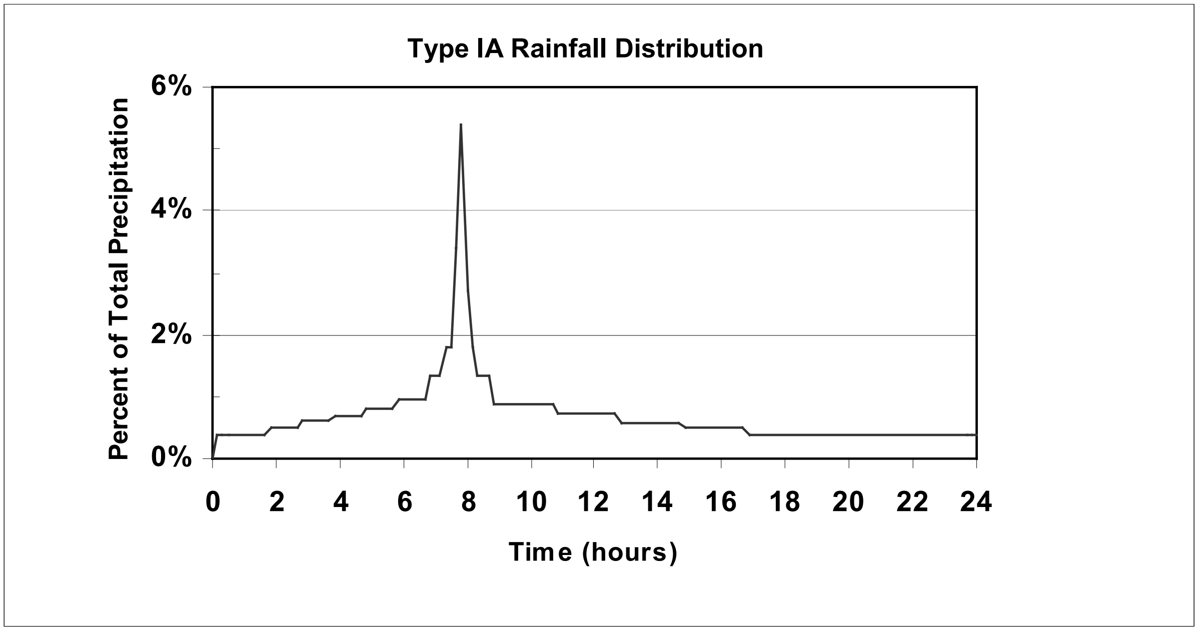
Adjusted P_{100} $= P_{100} + M_s$
= (7 inches) + (3.35 inches)
= 10.35 inches

Table 2.1 24-Hour Design Storm Hyetograph Values - 10 minute Resolution		
Time (hour)	Type IA Rainfall Distribution	Type II Rainfall Distribution
0	0	0
0.166667	0.004	0.0017
0.333333	0.004	0.0016
0.5	0.004	0.0017
0.666667	0.004	0.0017
0.833333	0.004	0.0016
1	0.004	0.0017
1.166667	0.004	0.0017
1.333333	0.004	0.0016
1.5	0.004	0.0017
1.666667	0.004	0.0017
1.833333	0.005	0.0016
2	0.005	0.0017
2.166667	0.005	0.0017
2.333333	0.005	0.0016
2.5	0.005	0.0017
2.666667	0.005	0.0017
2.833333	0.006	0.0016
3	0.006	0.0017
3.166667	0.006	0.0033
3.333333	0.006	0.0034
3.5	0.006	0.0033
3.666667	0.006	0.0033
3.833333	0.007	0.0034
4	0.007	0.0033
4.166667	0.007	0.0025
4.333333	0.007	0.0025
4.5	0.007	0.0025
4.666667	0.007	0.0025
4.833333	0.0082	0.0025
5	0.0082	0.0025
5.166667	0.0082	0.0025
5.333333	0.0082	0.0025
5.5	0.0082	0.0025
5.666667	0.0082	0.0025
5.833333	0.0095	0.0025
6	0.0095	0.0025
6.166667	0.0095	0.0042
6.333333	0.0095	0.0041
6.5	0.0095	0.0042
6.666667	0.0095	0.0042
6.833333	0.0134	0.0041
7	0.0134	0.0042

Table 2.1 (cont.) 24-Hour Design Storm Hyetograph Values - 10 minute Resolution		
Time (hour)	**Type IA Rainfall Distribution**	**Type II Rainfall Distribution**
7.166667	0.0134	0.0033
7.333373	0.018	0.0034
7.5	0.018	0.0033
7.666667	0.034	0.0033
7.833333	0.054	0.0034
8	0.027	0.0033
8.166667	0.018	0.005
8.333333	0.0134	0.005
8.5	0.0134	0.005
8.666667	0.0134	0.005
8.833333	0.0088	0.005
9	0.0088	0.005
9.166667	0.0088	0.005
9.333333	0.0088	0.005
9.5	0.0088	0.005
9.666667	0.0088	0.005
9.833333	0.0088	0.005
10	0.0088	0.005
10.16667	0.0088	0.0083
10.33333	0.0088	0.0084
10.5	0.0088	0.0083
10.66667	0.0088	0.0117
10.83333	0.0072	0.0116
11	0.0072	0.0117
11.16667	0.0072	0.02
11.33333	0.0072	0.02
11.5	0.0072	0.055
11.66667	0.0072	0.1
11.83333	0.0072	0.19
12	0.0072	0.075
12.16667	0.0072	0.03
12.33333	0.0072	0.008
12.5	0.0072	0.008
12.66667	0.0072	0.008
12.83333	0.0057	0.008
13	0.0057	0.008
13.16667	0.0057	0.0083
13.33333	0.0057	0.0084
13.5	0.0057	0.0083
13.66667	0.0057	0.0083
13.83333	0.0057	0.0084
14	0.0057	0.0083

Table 2.1 (cont.) 24-Hour Design Storm Hyetograph Values - 10 minute Resolution		
Time (hour)	**Type IA Rainfall Distribution**	**Type II Rainfall Distribution**
14.16667	0.0057	0.0058
14.33333	0.0057	0.0059
14.5	0.0057	0.0058
14.66667	0.0057	0.0058
14.83333	0.005	0.0059
15	0.005	0.0058
15.16667	0.005	0.0033
15.33333	0.005	0.0034
15.5	0.005	0.0033
15.66667	0.005	0.0033
15.83333	0.005	0.0034
16	0.005	0.0033
16.16667	0.005	0.0042
16.33333	0.005	0.0041
16.5	0.005	0.0042
16.66667	0.005	0.0042
16.83333	0.004	0.0041
17	0.004	0.0042
17.16667	0.004	0.0025
17.33333	0.004	0.0025
17.5	0.004	0.0025
17.66667	0.004	0.0025
17.83333	0.004	0.0025
18	0.004	0.0025
18.16667	0.004	0.0033
18.33333	0.004	0.0034
18.5	0.004	0.0033
18.66667	0.004	0.0033
18.83333	0.004	0.0034
19	0.004	0.0033
19.16667	0.004	0.0025
19.33333	0.004	0.0025
19.5	0.004	0.0025
19.66667	0.004	0.0025
19.83333	0.004	0.0025
20	0.004	0.0025
20.16667	0.004	0.0017
20.33333	0.004	0.0016
20.5	0.004	0.0017
20.66667	0.004	0.0017
20.83333	0.004	0.0016
21	0.004	0.0017
21.16667	0.004	0.0025

Table 2.1 (cont.) 24-Hour Design Storm Hyetograph Values - 10 minute Resolution		
Time (hour)	**Type IA Rainfall Distribution**	**Type II Rainfall Distribution**
21.33333	0.004	0.0025
21.5	0.004	0.0025
21.66667	0.004	0.0025
21.83333	0.004	0.0025
22	0.004	0.0025
22.16667	0.004	0.0017
22.33333	0.004	0.0016
22.5	0.004	0.0017
22.66667	0.004	0.0017
22.83333	0.004	0.0016
23	0.004	0.0017
23.16667	0.004	0.0017
23.33333	0.004	0.0016
23.5	0.004	0.0017
23.66667	0.004	0.0017
23.83333	0.004	0.0016
24	0.004	0.0017



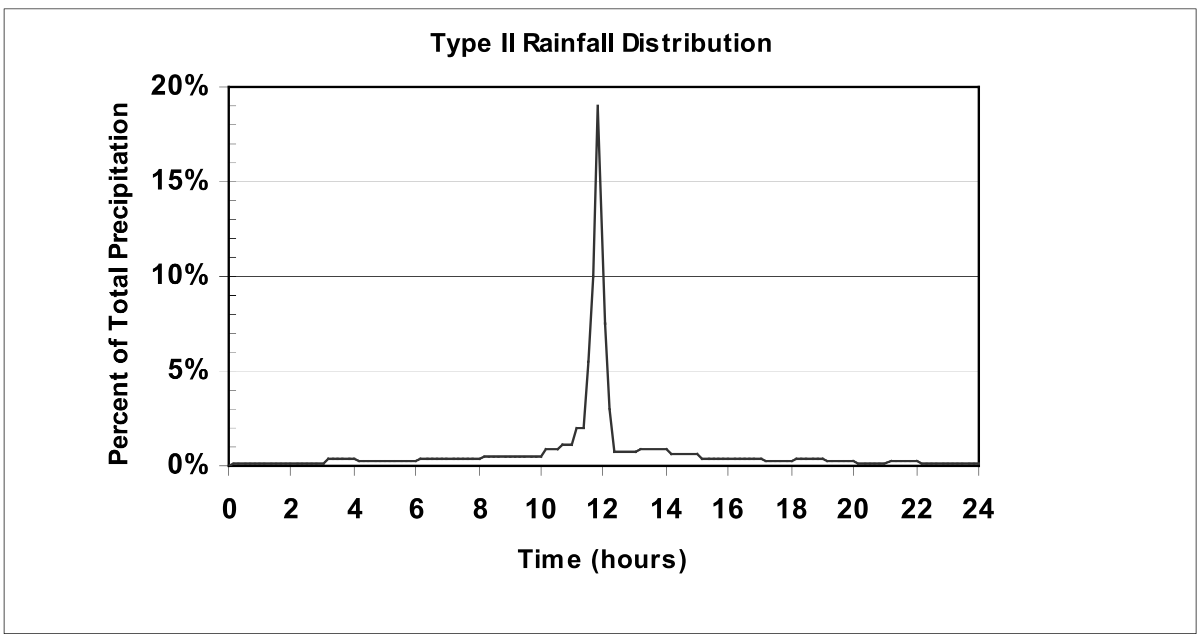


Figure 2.2 24-hour Design Storm Hyetographs

2.3.2 Runoff Parameters

All storm event hydrograph methods require input of parameters that describe physical drainage basin characteristics. These parameters provide the basis from which the runoff hydrograph is developed. This section describes the three key parameters (area, curve number, and time of concentration) used to develop the hydrograph using the method of hydrograph synthesis discussed in Section 2.3.3.

Area

The proper selection of homogeneous basin areas is required to obtain the highest degree of accuracy in hydrograph analysis. Significant differences in land use within a given drainage basin must be addressed by dividing the basin area into subbasin areas of similar land use and/or runoff characteristics. For example, a drainage basin consisting of a concentrated residential area and a large forested area should be divided into two subbasin areas accordingly. Hydrographs should then be computed for each subbasin area and summed to form the total runoff hydrograph for the basin.

To further enhance the accuracy of hydrograph analysis, all pervious and impervious areas within a given basin or subbasin must be analyzed separately, i.e., curve numbers and time of concentrations must be determined separately. This may be done by computing separate hydrographs for each area and combining them to form the total runoff hydrograph. This procedure is explained further in Section 2.3.3 "Hydrograph Synthesis." By analyzing pervious and impervious areas separately, the errors associated with averaging these areas are avoided and the true shape of the runoff hydrograph is better approximated.

Curve Number

The NRCS (formerly SCS) has, for many years, conducted studies of the runoff characteristics for various land types. After gathering and analyzing extensive data, NRCS has developed relationships between land use, soil type, vegetation cover, interception, infiltration, surface storage, and runoff. The relationships have been characterized by a single runoff coefficient called a "curve number." The National Engineering Handbook - Section 4: Hydrology (NEH-4, SCS, August 1972) contains a detailed description of the development and use of the curve number method.

NRCS has developed "curve number" (CN) values based on soil type and land use. They can be found in "Urban Hydrology for Small Watersheds", Technical Release 55 (TR-55), June 1986, published by the NRCS. The combination of these two factors is called the "soil-cover complex." The soil-cover complexes have been assigned to one of four hydrologic soil groups, according to their runoff characteristics. NRCS has classified over 4,000 soil types into these four soil groups. Table 2.2 shows the hydrologic soil group of most soils in the state of Washington and provides a brief description of the four groups. For details on other soil types refer to the NRCS publication mentioned above (TR-55, 1986).

Table 2.2 Hydrologic Soil Series for Selected Soils in Washington State			
Soil Type	**Hydrologic Soil Group**	**Soil Type**	**Hydrologic Soil Group**
Agnew	C	Hoko	C
Ahl	B	Hoodsport	C
Aits	C	Hoogdal	C
Alderwood	C	Hoypus	A
Arents, Alderwood	B	Huel	A
Arents, Everett	B	Indianola	A
Ashoe	B	Jonas	B
Baldhill	B	Jumpe	B
Barneston	C	Kalaloch	C
Baumgard	B	Kapowsin	C/D
Beausite	B	Katula	C
Belfast	C	Kilchis	C
Bellingham	D	Kitsap	C
Bellingham variant	C	Klaus	C
Boistfort	B	Klone	B
Bow	D	Lates	C
Briscot	D	Lebam	B
Buckley	C	Lummi	D
Bunker	B	Lynnwood	A
Cagey	C	Lystair	B
Carlsborg	A	Mal	C
Casey	D	Manley	B
Cassolary	C	Mashel	B
Cathcart	B	Maytown	C
Centralia	B	McKenna	D
Chehalis	B	McMurray	D
Chesaw	A	Melbourne	B
Cinebar	B	Menzel	B
Clallam	C	Mixed Alluvial	variable
Clayton	B	Molson	B
Coastal beaches	variable	Mukilteo	C/D
Colter	C	Naff	B
Custer	D	Nargar	A
Custer, Drained	C	National	B
Dabob	C	Neilton	A
Delphi	D	Newberg	B
Dick	A	Nisqually	B
Dimal	D	Nooksack	C
Dupont	D	Norma	C/D
Earlmont	C	Ogarty	C
Edgewick	C	Olete	C
Eld	B	Olomount	C
Elwell	B	Olympic	B
Esquatzel	B	Orcas	D
Everett	A	Oridia	D
Everson	D	Orting	D
Galvin	D	Oso	C
Getchell	A	Ovall	C
Giles	B	Pastik	C
Godfrey	D	Pheeney	C
Greenwater	A	Phelan	D
Grove	C	Pilchuck	C
Harstine	C	Potchub	C
Hartnit	C	Poulsbo	C
Hoh	B	Prather	C

Table 2.2 Hydrologic Soil Series for Selected Soils in Washington State (cont)			
Soil Type	**Hydrologic Soil Group**	**Soil Type**	**Hydrologic Soil Group**
Puget	D	Solleks	C
Puyallup	B	Spana	D
Queets	B	Spanaway	A/B
Quilcene	C	Springdale	B
Ragnar	B	Sulsavar	B
Rainier	C	Sultan	C
Raught	B	Sultan variant	B
Reed	D	Sumas	C
Reed, Drained or Protected	C	Swantown	D
Renton	D	Tacoma	D
Republic	B	Tanwax	D
Riverwash	variable	Tanwax, Drained	C
Rober	C	Tealwhit	D
Salal	C	Tenino	C
Salkum	B	Tisch	D
Sammamish	D	Tokul	C
San Juan	A	Townsend	C
Scamman	D	Triton	D
Schneider	B	Tukwila	D
Seattle	D	Tukey	C
Sekiu	D	Urbana	C
Semiahmoo	D	Vailton	B
Shalcar	D	Verlot	C
Shano	B	Wapato	D
Shelton	C	Warden	B
Si	C	Whidbey	C
Sinclair	C	Wilkeson	B
Skipopa	D	Winston	A
Skykomish	B	Woodinville	B
Snahopish	B	Yelm	C
Snohomish	D	Zynbar	B
Solduc	B		

Notes:

Hydrologic Soil Group Classifications, as Defined by the Soil Conservation Service:

A = *(Low runoff potential) Soils having low runoff potential and high infiltration rates, even when thoroughly wetted. They consist chiefly of deep, well to excessively drained sands or gravels and have a high rate of water transmission (greater than 0.30 in/hr.).*

B = *(Moderately low runoff potential). Soils having moderate infiltration rates when thoroughly wetted and consist chiefly of moderately deep to deep, moderately well to well drained soils with moderately fine to moderately coarse textures. These soils have a moderate rate of water transmission (0.15-0.3 in/hr.).*

C = *(Moderately high runoff potential). Soils having low infiltration rates when thoroughly wetted and consist chiefly of soils with a layer that impedes downward movement of water and soils with moderately fine to fine textures. These soils have a low rate of water transmission (0.05-0.15 in/hr.).*

D = *(High runoff potential). Soils having high runoff potential. They have very low infiltration rates when thoroughly wetted and consist chiefly of clay soils with a high swelling potential, soils with a permanent high water table, soils with a hardpan or clay layer at or near the surface, and shallow soils over nearly impervious material. These soils have a very low rate of water transmission (0-0.05 in/hr.).*

* = *From SCS, TR-55, Second Edition, June 1986, Exhibit A-1. Revisions made from SCS, Soil Interpretation Record, Form #5, September 1988 and various county soil surveys.*

Table 2.3 shows the CNs, by land use description, for the four hydrologic soil groups. These numbers are for a 24-hour duration storm and typical antecedent soil moisture condition preceding 24-hour storms.

The following are important criteria/considerations for selection of CN values:

Many factors may affect the CN value for a given land use. For example, the movement of heavy equipment over bare ground may compact the soil so that it has a lesser infiltration rate and greater runoff potential than would be indicated by strict application of the CN value to developed site conditions.

CN values can be area weighted when they apply to pervious areas of similar CNs (within 20 CN points). However, high CN areas should not be combined with low CN areas. In this case, separate hydrographs should be generated and summed to form one hydrograph unless the low CN areas are less than 15 percent of the subbasin.

Separate CN values must be selected for the pervious and impervious areas of an urban basin or subbasin. For residential districts the percent impervious area given in Table 2.3 must be used to compute the respective pervious and impervious areas. For proposed commercial areas, planned unit developments, etc., the percent impervious area must be computed from the site plan. For all other land uses the percent impervious area must be estimated from best available aerial topography and/or field reconnaissance. The pervious area CN value must be a weighted average of all the pervious area CNs within the subbasin. The impervious area CN value shall be 98.

For storm duration other than 24 hours, an adjustment must be made to the CN values given in Table 2.3. Based on information obtained from SCS, the following equation shall be used for adjusting these CNs for the seven-day design storm:

$$CN\ (7\ day) = 0.1549\ CN + 0.8451\ [(CN^{2.365}/631.8) + 15)]$$

Example: The following is an example of how CN values are selected for a sample project.

Select CNs for the following development:

Existing Land Use	-	forest (undisturbed)
Future Land Use	-	residential plat (3.6 DU/GA)
Basin Size	-	60 acres
Soil Type	-	80 percent Alderwood, 20 percent Ragnor

Table 2.2 shows that Alderwood soil belongs to the “C” hydrologic soil group and Ragnor soil belongs to the “B” group. Therefore, for the existing condition, CNs of 70 and 55 are read from Table 2.3 and areal weighted to obtain a CN value of 67. For the developed condition with 3.6 DU/GA the percent impervious of 39 percent is interpolated from Table 2.3 and used to compute pervious and impervious areas of 36.6 acres and 23.4 acres, respectively. The 36.6 acres of pervious area is assumed to be in Fair condition (for a conservative design) with residential yards and lawns covering the same proportions of Alderwood and Ragnor soil (80 percent and 20 percent respectively). Therefore, CNs of 90 and 85 are read from Table 2.3 and areal weighted to obtain a pervious area CN value of 89. The impervious area CN value is 98. The result of this example is summarized below:

On-Site Condition	Existing	Developed
Land use	Forest	Residential
Pervious area	60 ac.	36.6 ac.
CN of pervious area	67	89
Impervious area	0 ac.	23.4 ac.
CN of impervious area	--	98

Table 2.3
Runoff Curve Numbers for Selected Agricultural, Suburban, and Urban Areas

(Sources: TR 55, 1986, and Stormwater Management Manual, 1992. See Section 2.1.1 for explanation)

Cover type and hydrologic condition.			CNs for hydrologic soil group A	B	C	D
Curve Numbers for Pre-Development Conditions						
Pasture, grassland, or range-continuous forage for grazing:						
Fair condition (ground cover 50% to 75% and not heavily grazed).			49	69	79	84
Good condition (ground cover >75% and lightly or only occasionally grazed)			39	61	74	80
Woods:						
Fair (Woods are grazed but not burned, and some forest litter covers the soil).			36	60	73	79
Good (Woods are protected from grazing, and litter and brush adequately cover the soil).			30	55	70	77
Curve Numbers for Post-Development Conditions						
Open space (lawns, parks, golf courses, cemeteries, landscaping, etc.)[1]						
Fair condition (grass cover on 50% - 75% of the area).			77	85	90	92
Good condition (grass cover on >75% of the area)			68	80	86	90
Impervious areas:						
Open water bodies: lakes, wetlands, ponds etc.			100	100	100	100
Paved parking lots, roofs[2], driveways, etc. (excluding right-of-way)			98	98	98	98
Porous Pavers and Permeable Interlocking Concrete (assumed as 85% impervious and 15% lawn)						
Fair lawn condition (weighted average CNs).			95	96	97	97
Good lawn condition (weighted average CNs).			94	95	96	97
Paved			98	98	98	98
Gravel (including right-of-way)			76	85	89	91
Dirt (including right-of-way)			72	82	87	89
Pasture, grassland, or range-continuous forage for grazing:						
Poor condition (ground cover <50% or heavily grazed with no mulch).			68	79	86	89
Fair condition (ground cover 50% to 75% and not heavily grazed).			49	69	79	84
Good condition (ground cover >75% and lightly or only occasionally grazed)			39	61	74	80
Woods:						
Poor (Forest litter, small trees, and brush are destroyed by heavy grazing or regular burning).			45	66	77	83
Fair (Woods are grazed but not burned, and some forest litter covers the soil).			36	60	73	79
Good (Woods are protected from grazing, and litter and brush adequately cover the soil).			30	55	70	77
Single family residential[3]: Dwelling Unit/Gross Acre	Should only be used for subdivisions > 50 acres	Average Percent impervious area[3,4]				
1.0 DU/GA		15	Separate curve number			
1.5 DU/GA		20	shall be selected for			
2.0 DU/GA		25	pervious & impervious			
2.5 DU/GA		30	portions of the site or			
3.0 DU/GA		34	basin			
3.5 DU/GA		38				
4.0 DU/GA		42				
4.5 DU/GA		46				
5.0 DU/GA		48				
5.5 DU/GA		50				
6.0 DU/GA		52				
6.5 DU/GA		54				
7.0 DU/GA		56				
7.5 DU/GA		58				
PUD's, condos, apartments, commercial businesses, industrial areas & & subdivisions < 50 acres	%impervious must be computed	Separate curve numbers shall be selected for pervious and impervious portions of the site				
For a more detailed and complete description of land use curve numbers refer to chapter two (2) of the Soil Conservation Service's Technical Release No. 55 , (210-VI-TR-55, Second Ed., June 1986).						

[1] Composite CN's may be computed for other combinations of open space cover type.
[2]Where roof runoff and driveway runoff are infiltrated or dispersed according to the requirements in Chapter 2, the average percent impervious area may be adjusted in accordance with the procedure described under "Flow Credit for Roof Downspout Infiltration" and "Flow Credit for Roof Downspout Dispersion" in Chapter 2.
[3]Assumes roof and driveway runoff is directed into street/storm system.
[4]All the remaining pervious area (lawn) are considered to be in good condition for these curve numbers.

SCS Curve Number Equations for determination of runoff depths and volumes

The rainfall-runoff equations of the SCS curve number method relates a land area's runoff depth (precipitation excess) to the precipitation it receives and to its natural storage capacity, as follows:

$$Q_d = (P - 0.2S)^2 / (P + 0.8S) \qquad \text{for } P \geq 0.2S$$

and $$Q_d = 0 \qquad \text{for } P < 0.2S$$

Where:

Q_d = runoff depth in inches over the area,
P = precipitation depth in inches over the area, and
S = potential maximum natural detention, in inches over the area, due to infiltration, storage, etc.

The area's potential maximum detention, S, is related to its curve number, CN:

$$S = (1000 / CN) - 10$$

The combination of the above equations allows for estimation of the total runoff volume by computing total runoff depth, Q_d, given the total precipitation depth, P. For example, if the curve number of the area is 70, then the value of S is 4.29. With a total precipitation for the design event of 2.0 inches, the total runoff depth would be:

$$Q_d = [2.0 - 0.2\ (4.29)]^2 / [2.0 + 0.8\ (4.29)] = 0.24 \text{ inches}$$

This computed runoff represents inches over the tributary area. Therefore, the total volume of runoff is found by multiplying Q_d by the area (with necessary conversions):

Calculating the design volume for treatment BMPs for which the design criterion is based on the volume of runoff

Total runoff Volume (cu. ft.)	=	3,630 (cu. ft./ac. in.)	x	Q_d (in)	x	A (ac)

If the area is 10 acres, the total runoff volume is:

3,630 cu. ft./ac. in. x 0.24 in. x 10 ac. = 8,712 cu. ft.

This is the design volume for treatment BMPs for which the design criterion is based on the volume of runoff.

When developing the runoff hydrograph, the above equation for Q_d is used to compute the incremental runoff depth for each time interval from the incremental precipitation depth given by the design storm hyetograph. This time distribution of runoff depth is often referred to as the precipitation excess and provides the basis for synthesizing the runoff hydrograph.

Travel Time and Time of Concentration for Use in Hydrograph Analysis

(based on the methods described in SCS TR-55)

Travel time (T_t) is the time it takes water to travel from one location to another in a watershed. T_t is a component of time of concentration (T_c), which is the time it takes for runoff to travel from the hydraulically most distant point of the watershed to the outlet. T_c is computed by summing all the travel times for consecutive components of the drainage conveyance system. T_c influences the shape and peak of the runoff hydrograph. Urbanization usually decreases T_c, thereby increasing peak discharge. T_c can be increased as a result of either ponding behind small or inadequate drainage systems (including storm drain inlets and road culverts) or by reduction of land slope through grading.

Water moves through a watershed as sheet flow, shallow concentrated flow, open channel flow or some combination of these. The type of flow that occurs is best determined by field inspection.

Travel time (T_t) is the ratio of flow length to flow velocity:

$$T_t = \frac{L}{60V}$$

Where:

T_t = travel time (minutes)

L = flow length (feet)
V = average velocity (feet/sec) and
60 = conversion factor from seconds to minutes

Time of concentration (T_c) is the sum of T_t values for the various consecutive flow segments.

$$T_c = T_{t_1} + T_{t_2} + \ldots T_{t_m}$$

Where:

T_c = time of concentration (minutes) and
m = number of flow segments

Sheet Flow

Sheet flow is runoff that flows over the ground surface as a thin, even layer, not concentrated in a channel. It usually occurs in the headwater of streams. With sheet flow, the friction value (n_s) (a modified Manning's effective roughness coefficient that includes the effect of raindrop impact; drag over the plane surface; obstacles such as litter, crop ridges and rocks; and erosion and transportation of sediment) is used. These n_s values are for very shallow flow depths of about 0.1 foot and are only used for travel lengths up to 300 feet. Table 2.4 gives Manning's n_s values for sheet flow for various surface conditions. Table 2.5 gives Manning’s n normal values for various surfaces.

For sheet flow of up to 300 feet, use Manning's kinematic solution to directly compute T_t.

$$T_t = \frac{0.42(n_s L)^{0.8}}{(P_2)^{0.527}(s_o)^{0.4}}$$

Where:

T_t = travel time (min),
n_s = sheet flow Manning's effective roughness coefficient (from Table 2.4).
L = flow length (ft),
P_2 = 2-year, 24-hour rainfall (in), and
s_o = slope of hydraulic grade line (land slope, ft/ft)

Velocity Equation

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

$$V = k\sqrt{s_o}$$

Where:

V = velocity (ft/s)

k = time of concentration velocity factor (ft/s)

s_o = slope of flow path (ft/ft)

"k" is computed for various land covers and channel characteristics with assumptions made for hydraulic radius using the following rearrangement of Manning's equation:

$$k = \frac{1.49(R)^{0.667}}{n}$$

Where:

R = an assumed hydraulic radius
n = Manning's roughness coefficient for open channel flow

Shallow Concentrated Flow: After a maximum of 300 feet, sheet flow is assumed to become shallow concentrated flow. The average velocity for this flow can be calculated using the k_s values from Table 2.4 in which average velocity is a function of watercourse slope and type of channel. After computing the average velocity using the Velocity Equation above, the travel time (T_t) for the shallow concentrated flow segment can be computed using the Travel Time Equation described above.

Open Channel Flow: Open channels are assumed to begin where surveyed cross section information has been obtained, where channels are visible on aerial photographs, or where lines indicating streams appear (in blue) on United States Geological Survey (USGS) quadrangle sheets. The

k_c values from Table 2.4 used in the Velocity Equation above or water surface profile information can be used to estimate average flow velocity. Average flow velocity is usually determined for bankfull conditions. After average velocity is computed the travel time (T_t) for the channel segment can be computed using the Travel Time Equation above.

Lakes or Wetlands: Sometimes it is necessary to estimate the velocity of flow through a lake or wetland at the outlet of a watershed. This travel time is normally very small and can be assumed as zero. Where significant attenuation may occur due to storage effects, the flows should be routed using the "level pool routing" technique described in Section 2.3.4

Limitations: The following limitations apply in estimating travel time (T_t).

Manning's kinematic solution should not be used for sheet flow longer than 300 feet.

In watersheds with storm drains, carefully identify the appropriate hydraulic flow path to estimate T_c. Storm sewers generally handle only a small portion of a large event. The rest of the peak flow travels by streets, lawns, and so on, to the outlet. Consult a standard hydraulics textbook to determine average velocity in pipes for either pressure or non-pressure flow.

A culvert or bridge can act as a reservoir outlet if there is significant storage behind it. A hydrograph should be developed to this point and the "level pool routing" technique described in Section 2.3.4 should be used to determine the outflow rating curve through the culvert or bridge.

Example: The following is an example of travel time and time of concentration calculations.

Given: An existing drainage basin having a selected flow route composed of the following five segments. Note: Drainage basin is in Federal Way and has a P2 = 2.1 inches.

Segment 1: L = 200 ft. Forest with dense brush (sheet flow)
s_o= 0.03 ft/ft, n_s = 0.80
Segment 2: L = 300 ft. Pasture (shallow concentrated flow)
s_o = 0.04 ft/ft, ks = 11
Segment 3:L = 50 ft. Small pond (year around)
s_o = 0.00 ft/ft, kc = 0
Segment 4: L = 300 ft. Grassed waterway (intermittent channel)
s_o = 0.05 ft/ft, kc = 17
Segment 5: L = 500 ft. Grass-lined stream (continuous)
s_o = 0.02 ft/ft, kc = 27

Table 2.4
"n" and "k" Values Used in Time Calculations for Hydrographs

"n_s" Sheet Flow Equation Manning's Values (for the initial 300 ft. of travel)	
Manning values for sheet flow only, from Overton and Meadows 1976 (See TR-55, 1986)	n_s
Smooth surfaces (concrete, asphalt, gravel, or bare hand packed soil)	0.011
Fallow fields or loose soil surface (no residue)	0.05
Cultivated soil with residue cover ≤20%	0.06
Cultivated soil with residue cover >20%	0.17
Short prairie grass and lawns	0.15
Dense grasses	0.24
Bermuda grass	0.41
Range (natural)	0.13
Woods or forest with light underbrush	0.40
Woods or forest with dense underbrush	0.80
(210-VI-TR-55, Second Ed., June 1986)	
"k" Values Used in Travel Time/Time of Concentration Calculations	
Shallow Concentrated Flow (After the initial 300 ft. of sheet flow, R = 0.1)	k_s
1. Forest with heavy ground litter and meadows (n = 0.10)	3
2. Brushy ground with some trees (n= 0.060)	5
3. Fallow or minimum tillage cultivation (n = 0.040)	8
4. High grass (n = 0.035)	9
5. Short grass, pasture and lawns (n = 0.030)	11
6. Nearly bare ground (n = 0.025)	13
7. Paved and gravel areas (n = 0.012)	27
Channel Flow (intermittent) (At the beginning of visible channels R = 0.2)	k_c
1. Forested swale with heavy ground litter (n = 0.10)	5
2. Forested drainage course/ravine with defined channel bed (n = 0.050)	10
3. Rock-lined waterway (n = 0.035)	15
4. Grassed waterway (n = 0.030)	17
5. Earth-lined waterway (n = 0.025)	20
6. CMP pipe, uniform flow (n = 0.024)	21
7. Concrete pipe, uniform flow (0.012)	42
8. Other waterways and pipe	0.508/n
Channel Flow (Continuous stream, R = 0.4)	k_c
9. Meandering stream with some pools (n = 0.040)	20
10. Rock-lined stream (n = 0.035)	23
11. Grass-lined stream (n = 0.030)	27
12. Other streams, man-made channels and pipe	0.807/n

Table 2.5 Values of the Roughness Coefficient, "n"			
Type of Channel and Description	**Manning's "n"* (Normal)**	**Type of Channel and Description**	**Manning's "n"* (Normal)**
A. Constructed Channels		6. Sluggish reaches, weedy	
a. Earth, straight and uniform		deep pools	0.070
1. Clean, recently completed	0.018	7. Very weedy reaches, deep	
2. Gravel, uniform selection,	0.025	pools, or floodways with	
clean		heavy stand of timber and	
3. With short grass, few	0.027	underbrush	0.100
weeds		b. Mountain streams, no vegetation	
b. Earth, winding and sluggish		in channel, banks usually steep,	
1. No vegetation	0.025	trees and brush along banks	
2. Grass, some weeds	0.030	submerged at high stages	
3. Dense weeds or aquatic		1. Bottom: gravel, cobbles and	
plants in deep channels	0.035	few boulders	0.040
4. Earth bottom and rubble		2. Bottom: cobbles with large	
sides	0.030	boulders	0.050
5. Stony bottom and weedy		B-2 Flood plains	
banks	0.035	a. Pasture, no brush	
6. Cobble bottom and clean		1. Short grass	0.030
sides	0.040	2. High grass	0.035
c. Rock lined		b. Cultivated areas	
1. Smooth and uniform	0.035	1. No crop	0.030
2. Jagged and irregular	0.040	2. Mature row crops	0.035
d. Channels not maintained,		3. Mature field crops	0.040
weeds and brush uncut		c. Brush	
1. Dense weeds, high as flow		1. Scattered brush, heavy	
depth	0.080	weeds	0.050
2. Clean bottom, brush on		2. Light brush and trees	0.060
sides	0.050	3. Medium to dense brush	0.070
3. Same, highest stage of		4. Heavy, dense brush	0.100
flow	0.070	d. Trees	
4. Dense brush, high stage	0.100	1. Dense willows, straight	0.150
B. Natural Streams		2. Cleared land with tree	
B-1 Minor streams (top width		stumps, no sprouts	0.040
at flood stage < 100ft.)		3. Same as above, but with	
a. Streams on plain		heavy growth of sprouts	0.060
1. Clean, straight, full stage		4. Heavy stand of timber, a few	
no rifts or deep pools	0.030	down trees, little	
2. Same as above, but more		undergrowth, flood stage	
stones and weeds	0.035	below branches	0.100
3. Clean, winding, some		5. Same as above, but with	
pools and shoals	0.040	flood stage reaching	
4. Same as above, but some		branches	0.120
Weeds	0.040		
5. Same as 4, but more			
Stones	0.050		

*Note, these "n" values are "normal" values for use in analysis of channels. For conservative design for channel capacity the "maximum" values listed in other references should be considered. For channel bank stability the minimum values should be considered.

Calculate travel times ($T_{t's}$) for each reach and then sum them to calculate the drainage basin time of concentration (T_c).

Segment 1: Sheet flow (L <300 feet),

$$T_t = \frac{0.42(n_s L)^{0.8}}{(P_2)^{0.527}(s_o)^{0.4}}$$

$$T_1 = \frac{(0.42)[(0.80)(200)]^{0.8}}{(2.1)^{0.527}(0.03)^{0.4}} = 68 \text{ minutes}$$

Segment 2: Shallow concentrated flow,

$$V = k\sqrt{s_o}$$

$$V_2 = (11)\sqrt{(0.04)} = 2.2\, ft/s$$

$$T_2 = \frac{L}{60V} = \frac{(300)}{60(2.2)} = 2 \text{ minutes}$$

Segment 3: Flat water surface

$T_3 = 0$ minutes

Segment 4: Intermittent channel flow

$$V_4 = (17)\sqrt{(0.05)} = 3.8\, ft/s$$

$$T_4 = \frac{(300)}{60(3.8)} = 1 \text{ minute}$$

Segment 5: Continuous stream

$$V_5 = (27)\sqrt{(0.02)} = 3.8\, ft/s$$

$$T_5 = \frac{(500)}{60(3.8)} = 2 \text{ minutes}$$

T_c = T1 + T2 + T3 + T4 + T5

T_c = 68 + 2 + 0 + 1 + 2 = 73 minutes

It is important to note how the initial sheet flow segment's travel time dominates the time of concentration computation. This will nearly always be the case for relatively small drainage basins and in particular for the existing site conditions. This also illustrates the significant impact urbanization has on the surface runoff portion of the hydrologic process.

2.3.3 Hydrograph Synthesis – Santa Barbara Urban Hydrograph

The Santa Barbara Urban Hydrograph (SBUH) method is described below. It is given here as a guideline only, as it is only one of the many SCS-based hydrograph methods that are available for use.

The SBUH method, like the Soil Conservation Service Unit Hydrograph (SCSUH) method, is based on the curve number (CN) approach, and also uses SCS equations for computing soil absorption and precipitation excess. The SCSUH method works by converting the incremental runoff depths (precipitation excess) for a given basin and design storm into a runoff hydrograph via application of a dimensionless unit hydrograph. The shape of the SCS unit hydrograph (time to peak, time base, and peak) are determined by a single parameter - the basin time of concentration. The SBUH method, on the other hand, converts the incremental runoff depths into instantaneous hydrographs that are then routed through an imaginary reservoir with a time delay equal to the basin time of concentration.

The SBUH method was developed by the Santa Barbara County Flood Control and Water Conservation District, California. The SBUH method directly computes a runoff hydrograph without going through an intermediate process (unit hydrograph) as the SCSUH method does. By comparison, the calculation steps of the SBUH method are much simpler and can be programmed on a calculator or a spreadsheet program.

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

- Step one - computing the instantaneous hydrograph, and
- Step two - computing the runoff hydrograph.

The instantaneous hydrograph, I(t), in cfs, at each time step, dt, is computed as follows:

I_t = $60.5\ R_t\ A/\ d_t$

Where R_t = total runoff depth (both impervious and pervious runoffs) at time increment dt, in inches (also known as precipitation excess)

A = area in acres

d_t = time interval in minutes*

*NOTE: A maximum time interval of 10 minutes should be used for all design storms of 24-hour duration. A maximum time interval of 60 minutes should be used for the 100-year, 7-day design storm.

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

Q_{t+1} $= Q_t + w[I_t + I_{t+1} - 2Q_t]$

Where: $w = d_t/(2T_c + d_t)$

d_t = time interval in minutes

Example: To illustrate the SBUH method, Tables 2.6 and 2.7 show runoff hydrograph values computed by this method for both existing and developed conditions. Figure 2.3 illustrates the hydrographs for existing and developed conditions. Note, this example was prepared using the Excel 5.0 spreadsheet program and illustrates how the method can be used with a personal computer. Copies of this program and a Fortran version are available (with minimal documentation) from King County Surface Water Management Division.

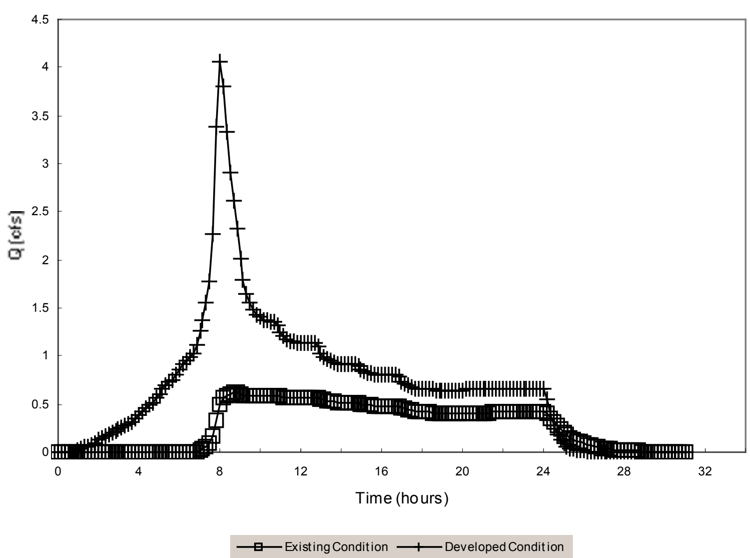


Figure 2.3 SBUH Hydrograph for Existing and Developed Site Conditions

Table 2.6
SBUH Values for Existing Site Condition

Given: Area = 10 acres	P = 2.9 inches (10-yr, 24-hr. event)			dt =10 minutes
PERVIOUS AREA:	Area = 10 acres	CN = 74	S = 3.513514	0.2S = 0.70
IMPERVIOUS AREA:	Area = 0 acres	CN = 98	S = 0.204082	0.2S = 0.04
Tc = 73 minutes	w = 0.064103	where S = potential maximum natural detention (as defined earlier)		

Column (1) = Time Increment
Column (2) = Time (min)
Column (3) = Type IA Storm Distribution
Column (4) = Column (3) * P
Column (5) = Accumulated sum of Column (4)
Column (6) = If (P < 0.2S) = 0, If (P > 0.2S) = (Column (5) - 0.2S)^2/(Column (5) + 0.8S), where the PERVIOUS AREA S value is used
Column (7) = Column (6) of the present step - Column (6) of the previous step
Column (8) = Same as Column (6) except use IMPERVIOUS AREA S value
Column (9) = Column (8) of the present step - Column (8) of the previous step
Column (10) = (PERVIOUS AREA/TOTAL AREA)*Column (7)+(IMPERVIOUS AREA/TOTAL AREA)*Column (9)
Column (11) = (60.5*Column (10)*Total Area)/dt, where dt = 10 or 60 minutes
Column (12) = Column (12) of previous time step + w * [(Column (11) of previous time step + Column (11) of present time step) - (2 * Column (12) of previous time step)] where w = routing constant = dt/(2Tc + dt) = 0.0641

(1) Time Increment	(2) Time (minute)	(3) Rainfall Distrib. (fraction)	(4) Incre. Rainfall (inches)	(5) Accumul. Rainfall (inches)	(6) PERVIOUS Accum. Runoff (inches)	(7) PERVIOUS Incre. Runoff (inches)	(8) IMPERVIOUS Accum. Runoff (inches)	(9) IMPERVIOUS Incre. Runoff (inches)	(10) Total Runoff (inches)	(11) Instant Flowrate (cfs)	(12) Design Flowrate (cfs)
1	0	0	0	0	0	0	0	0	0	0.0	0.0
2	10	0.004	0.012	0.012	0.000	0.000	0.000	0.000	0.000	0.0	0.0
3	20	0.004	0.012	0.023	0.000	0.000	0.000	0.000	0.000	0.0	0.0
4	30	0.004	0.012	0.035	0.000	0.000	0.000	0.000	0.000	0.0	0.0
5	40	0.004	0.012	0.046	0.000	0.000	0.000	0.000	0.000	0.0	0.0
6	50	0.004	0.012	0.058	0.000	0.000	0.001	0.001	0.000	0.0	0.0
7	60	0.004	0.012	0.070	0.000	0.000	0.004	0.002	0.000	0.0	0.0
8	70	0.004	0.012	0.081	0.000	0.000	0.007	0.003	0.000	0.0	0.0
9	80	0.004	0.012	0.093	0.000	0.000	0.011	0.004	0.000	0.0	0.0
10	90	0.004	0.012	0.104	0.000	0.000	0.015	0.005	0.000	0.0	0.0
11	100	0.004	0.012	0.116	0.000	0.000	0.020	0.005	0.000	0.0	0.0
12	110	0.005	0.015	0.131	0.000	0.000	0.027	0.007	0.000	0.0	0.0
13	120	0.005	0.015	0.145	0.000	0.000	0.035	0.008	0.000	0.0	0.0
14	130	0.005	0.015	0.160	0.000	0.000	0.044	0.008	0.000	0.0	0.0
15	140	0.005	0.015	0.174	0.000	0.000	0.053	0.009	0.000	0.0	0.0
16	150	0.005	0.015	0.189	0.000	0.000	0.062	0.009	0.000	0.0	0.0
17	160	0.005	0.015	0.203	0.000	0.000	0.072	0.010	0.000	0.0	0.0
18	170	0.006	0.017	0.220	0.000	0.000	0.084	0.012	0.000	0.0	0.0
19	180	0.006	0.017	0.238	0.000	0.000	0.097	0.013	0.000	0.0	0.0
20	190	0.006	0.017	0.255	0.000	0.000	0.110	0.013	0.000	0.0	0.0
21	200	0.006	0.017	0.273	0.000	0.000	0.123	0.013	0.000	0.0	0.0
22	210	0.006	0.017	0.290	0.000	0.000	0.137	0.014	0.000	0.0	0.0
23	220	0.006	0.017	0.307	0.000	0.000	0.151	0.014	0.000	0.0	0.0
24	230	0.007	0.020	0.328	0.000	0.000	0.168	0.017	0.000	0.0	0.0
25	240	0.007	0.020	0.348	0.000	0.000	0.185	0.017	0.000	0.0	0.0
26	250	0.007	0.020	0.368	0.000	0.000	0.202	0.017	0.000	0.0	0.0
27	260	0.007	0.020	0.389	0.000	0.000	0.219	0.017	0.000	0.0	0.0
28	270	0.007	0.020	0.409	0.000	0.000	0.237	0.018	0.000	0.0	0.0
29	280	0.007	0.020	0.429	0.000	0.000	0.255	0.018	0.000	0.0	0.0
30	290	0.008	0.024	0.453	0.000	0.000	0.276	0.021	0.000	0.0	0.0
31	300	0.008	0.024	0.477	0.000	0.000	0.297	0.021	0.000	0.0	0.0
32	310	0.008	0.024	0.501	0.000	0.000	0.318	0.021	0.000	0.0	0.0
33	320	0.008	0.024	0.524	0.000	0.000	0.340	0.022	0.000	0.0	0.0
34	330	0.008	0.024	0.548	0.000	0.000	0.362	0.022	0.000	0.0	0.0
35	340	0.008	0.024	0.572	0.000	0.000	0.384	0.022	0.000	0.0	0.0
36	350	0.010	0.028	0.599	0.000	0.000	0.409	0.026	0.000	0.0	0.0
37	360	0.010	0.028	0.627	0.000	0.000	0.435	0.026	0.000	0.0	0.0
38	370	0.010	0.028	0.655	0.000	0.000	0.461	0.026	0.000	0.0	0.0

(1) Time Increment	(2) Time (minute)	(3) Rainfall Distrib. (fraction)	(4) Incre. Rainfall (inches)	(5) Accumul. Rainfall (inches)	(6) PERVIOUS Accum. Runoff (inches)	(7) PERVIOUS Incre. Runoff (inches)	(8) IMPERVIOUS Accum. Runoff (inches)	(9) IMPERVIOUS Incre. Runoff (inches)	(10) Total Runoff (inches)	(11) Instant Flowrate (cfs)	(12) Design Flowrate (cfs)
39	380	0.010	0.028	0.682	0.000	0.000	0.486	0.026	0.000	0.0	0.0
40	390	0.010	0.028	0.710	0.000	0.000	0.512	0.026	0.000	0.0	0.0
41	400	0.010	0.028	0.737	0.000	0.000	0.539	0.026	0.000	0.0	0.0
42	410	0.013	0.039	0.776	0.001	0.001	0.575	0.037	0.001	0.1	0.0
43	420	0.013	0.039	0.815	0.003	0.002	0.613	0.037	0.002	0.1	0.0
44	430	0.013	0.039	0.854	0.006	0.003	0.650	0.037	0.003	0.2	0.0
45	440	0.018	0.052	0.906	0.011	0.005	0.700	0.050	0.005	0.3	0.1
46	450	0.018	0.052	0.958	0.017	0.006	0.750	0.050	0.006	0.4	0.1
47	460	0.034	0.099	1.057	0.032	0.015	0.846	0.096	0.015	0.9	0.2
48	470	0.054	0.157	1.213	0.065	0.032	0.999	0.153	0.032	2.0	0.3
49	480	0.027	0.078	1.292	0.085	0.020	1.075	0.077	0.020	1.2	0.5
50	490	0.018	0.052	1.344	0.099	0.014	1.127	0.051	0.014	0.9	0.6
51	500	0.013	0.039	1.383	0.110	0.011	1.165	0.038	0.011	0.7	0.6
52	510	0.013	0.039	1.422	0.122	0.012	1.203	0.038	0.012	0.7	0.6
53	520	0.013	0.039	1.460	0.134	0.012	1.241	0.038	0.012	0.7	0.6
54	530	0.009	0.026	1.486	0.143	0.008	1.266	0.025	0.008	0.5	0.6
55	540	0.009	0.026	1.511	0.151	0.009	1.291	0.025	0.009	0.5	0.6
56	550	0.009	0.026	1.537	0.160	0.009	1.317	0.025	0.009	0.5	0.6
57	560	0.009	0.026	1.563	0.169	0.009	1.342	0.025	0.009	0.5	0.6
58	570	0.009	0.026	1.588	0.178	0.009	1.367	0.025	0.009	0.6	0.6
59	580	0.009	0.026	1.614	0.188	0.009	1.392	0.025	0.009	0.6	0.6
60	590	0.009	0.026	1.639	0.197	0.010	1.417	0.025	0.010	0.6	0.6
61	600	0.009	0.026	1.665	0.207	0.010	1.442	0.025	0.010	0.6	0.6
62	610	0.009	0.026	1.690	0.217	0.010	1.468	0.025	0.010	0.6	0.6
63	620	0.009	0.026	1.716	0.227	0.010	1.493	0.025	0.010	0.6	0.6
64	630	0.009	0.026	1.741	0.237	0.010	1.518	0.025	0.010	0.6	0.6
65	640	0.009	0.026	1.767	0.247	0.010	1.543	0.025	0.010	0.6	0.6
66	650	0.007	0.021	1.788	0.256	0.009	1.564	0.021	0.009	0.5	0.6
67	660	0.007	0.021	1.808	0.265	0.009	1.585	0.021	0.009	0.5	0.6
68	670	0.007	0.021	1.829	0.274	0.009	1.605	0.021	0.009	0.5	0.6
69	680	0.007	0.021	1.850	0.283	0.009	1.626	0.021	0.009	0.5	0.6
70	690	0.007	0.021	1.871	0.292	0.009	1.647	0.021	0.009	0.5	0.6
71	700	0.007	0.021	1.892	0.301	0.009	1.667	0.021	0.009	0.6	0.6
72	710	0.007	0.021	1.913	0.310	0.009	1.688	0.021	0.009	0.6	0.6
73	720	0.007	0.021	1.934	0.319	0.009	1.709	0.021	0.009	0.6	0.6
74	730	0.007	0.021	1.955	0.329	0.009	1.729	0.021	0.009	0.6	0.6
75	740	0.007	0.021	1.975	0.338	0.010	1.750	0.021	0.010	0.6	0.6
76	750	0.007	0.021	1.996	0.348	0.010	1.771	0.021	0.010	0.6	0.6
77	760	0.007	0.021	2.017	0.358	0.010	1.791	0.021	0.010	0.6	0.6
78	770	0.006	0.017	2.034	0.366	0.008	1.808	0.016	0.008	0.5	0.6
79	780	0.006	0.017	2.050	0.374	0.008	1.824	0.016	0.008	0.5	0.6
80	790	0.006	0.017	2.067	0.382	0.008	1.841	0.016	0.008	0.5	0.5
81	800	0.006	0.017	2.083	0.389	0.008	1.857	0.016	0.008	0.5	0.5
82	810	0.006	0.017	2.100	0.398	0.008	1.873	0.016	0.008	0.5	0.5
83	820	0.006	0.017	2.116	0.406	0.008	1.890	0.016	0.008	0.5	0.5
84	830	0.006	0.017	2.133	0.414	0.008	1.906	0.016	0.008	0.5	0.5
85	840	0.006	0.017	2.149	0.422	0.008	1.923	0.016	0.008	0.5	0.5
86	850	0.006	0.017	2.166	0.430	0.008	1.939	0.016	0.008	0.5	0.5
87	860	0.006	0.017	2.183	0.439	0.008	1.955	0.016	0.008	0.5	0.5
88	870	0.006	0.017	2.199	0.447	0.008	1.972	0.016	0.008	0.5	0.5
89	880	0.006	0.017	2.216	0.455	0.008	1.988	0.016	0.008	0.5	0.5
90	890	0.005	0.015	2.230	0.463	0.007	2.003	0.014	0.007	0.4	0.5
91	900	0.005	0.015	2.245	0.470	0.007	2.017	0.014	0.007	0.5	0.5
92	910	0.005	0.015	2.259	0.478	0.008	2.031	0.014	0.008	0.5	0.5
93	920	0.005	0.015	2.274	0.485	0.008	2.046	0.014	0.008	0.5	0.5
94	930	0.005	0.015	2.288	0.493	0.008	2.060	0.014	0.008	0.5	0.5
95	940	0.005	0.015	2.303	0.501	0.008	2.075	0.014	0.008	0.5	0.5

(1) Time Increment	(2) Time (minute)	(3) Rainfall Distrib. (fraction)	(4) Incre. Rainfall (inches)	(5) Accumul. Rainfall (inches)	(6) PERVIOUS Accum. Runoff (inches)	(7) PERVIOUS Incre. Runoff (inches)	(8) IMPERVIOUS Accum. Runoff (inches)	(9) IMPERVIOUS Incre. Runoff (inches)	(10) Total Runoff (inches)	(11) Instant Flowrate (cfs)	(12) Design Flowrate (cfs)
96	950	0.005	0.015	2.317	0.508	0.008	2.089	0.014	0.008	0.5	0.5
97	960	0.005	0.015	2.332	0.516	0.008	2.103	0.014	0.008	0.5	0.5
98	970	0.005	0.015	2.346	0.524	0.008	2.118	0.014	0.008	0.5	0.5
99	980	0.005	0.015	2.361	0.532	0.008	2.132	0.014	0.008	0.5	0.5
100	990	0.005	0.015	2.375	0.539	0.008	2.147	0.014	0.008	0.5	0.5
101	1000	0.005	0.015	2.390	0.547	0.008	2.161	0.014	0.008	0.5	0.5
102	1010	0.004	0.012	2.401	0.554	0.006	2.173	0.012	0.006	0.4	0.5
103	1020	0.004	0.012	2.413	0.560	0.006	2.184	0.012	0.006	0.4	0.5
104	1030	0.004	0.012	2.424	0.566	0.006	2.196	0.012	0.006	0.4	0.4
105	1040	0.004	0.012	2.436	0.573	0.006	2.207	0.012	0.006	0.4	0.4
106	1050	0.004	0.012	2.448	0.579	0.006	2.219	0.012	0.006	0.4	0.4
107	1060	0.004	0.012	2.459	0.585	0.006	2.230	0.012	0.006	0.4	0.4
108	1070	0.004	0.012	2.471	0.592	0.006	2.242	0.012	0.006	0.4	0.4
109	1080	0.004	0.012	2.482	0.598	0.006	2.253	0.012	0.006	0.4	0.4
110	1090	0.004	0.012	2.494	0.605	0.007	2.265	0.012	0.007	0.4	0.4
111	1100	0.004	0.012	2.506	0.611	0.007	2.276	0.012	0.007	0.4	0.4
112	1110	0.004	0.012	2.517	0.618	0.007	2.288	0.012	0.007	0.4	0.4
113	1120	0.004	0.012	2.529	0.625	0.007	2.299	0.012	0.007	0.4	0.4
114	1130	0.004	0.012	2.540	0.631	0.007	2.311	0.012	0.007	0.4	0.4
115	1140	0.004	0.012	2.552	0.638	0.007	2.322	0.012	0.007	0.4	0.4
116	1150	0.004	0.012	2.564	0.644	0.007	2.334	0.012	0.007	0.4	0.4
117	1160	0.004	0.012	2.575	0.651	0.007	2.346	0.012	0.007	0.4	0.4
118	1170	0.004	0.012	2.587	0.658	0.007	2.357	0.012	0.007	0.4	0.4
119	1180	0.004	0.012	2.598	0.664	0.007	2.369	0.012	0.007	0.4	0.4
120	1190	0.004	0.012	2.610	0.671	0.007	2.380	0.012	0.007	0.4	0.4
121	1200	0.004	0.012	2.622	0.678	0.007	2.392	0.012	0.007	0.4	0.4
122	1210	0.004	0.012	2.633	0.685	0.007	2.403	0.012	0.007	0.4	0.4
123	1220	0.004	0.012	2.645	0.691	0.007	2.415	0.012	0.007	0.4	0.4
124	1230	0.004	0.012	2.656	0.698	0.007	2.426	0.012	0.007	0.4	0.4
125	1240	0.004	0.012	2.668	0.705	0.007	2.438	0.012	0.007	0.4	0.4
126	1250	0.004	0.012	2.680	0.712	0.007	2.449	0.012	0.007	0.4	0.4
127	1260	0.004	0.012	2.691	0.719	0.007	2.461	0.012	0.007	0.4	0.4
128	1270	0.004	0.012	2.703	0.726	0.007	2.472	0.012	0.007	0.4	0.4
129	1280	0.004	0.012	2.714	0.732	0.007	2.484	0.012	0.007	0.4	0.4
130	1290	0.004	0.012	2.726	0.739	0.007	2.496	0.012	0.007	0.4	0.4
131	1300	0.004	0.012	2.738	0.746	0.007	2.507	0.012	0.007	0.4	0.4
132	1310	0.004	0.012	2.749	0.753	0.007	2.519	0.012	0.007	0.4	0.4
133	1320	0.004	0.012	2.761	0.760	0.007	2.530	0.012	0.007	0.4	0.4
134	1330	0.004	0.012	2.772	0.767	0.007	2.542	0.012	0.007	0.4	0.4
135	1340	0.004	0.012	2.784	0.774	0.007	2.553	0.012	0.007	0.4	0.4
136	1350	0.004	0.012	2.796	0.781	0.007	2.565	0.012	0.007	0.4	0.4
137	1360	0.004	0.012	2.807	0.788	0.007	2.576	0.012	0.007	0.4	0.4
138	1370	0.004	0.012	2.819	0.795	0.007	2.588	0.012	0.007	0.4	0.4
139	1380	0.004	0.012	2.830	0.803	0.007	2.599	0.012	0.007	0.4	0.4
140	1390	0.004	0.012	2.842	0.810	0.007	2.611	0.012	0.007	0.4	0.4
141	1400	0.004	0.012	2.854	0.817	0.007	2.623	0.012	0.007	0.4	0.4
142	1410	0.004	0.012	2.865	0.824	0.007	2.634	0.012	0.007	0.4	0.4
143	1420	0.004	0.012	2.877	0.831	0.007	2.646	0.012	0.007	0.4	0.4
144	1430	0.004	0.012	2.888	0.838	0.007	2.657	0.012	0.007	0.4	0.4
145	1440	0.004	0.012	2.900	0.845	0.007	2.669	0.012	0.007	0.4	0.4

Table 2.7
SBUH Values for Developed Site Condition

Given: Area = 10 acres P = 2.9 inches (10-yr., 24-hr. event) dt = 10 minutes
PERVIOUS AREA: Area = 6.1 acres CN = 89 S = 1.235955 0.2S = 0.25
IMPERVIOUS AREA: Area = 3.9 acres CN = 98 S = 0.204082 0.2S = 0.04
Tc = 28 minutes w = 0.151515 where S = potential maximum natural detention (as defined earlier)

Column (1) = Time Increment Column (2) = Time (min)
Column (3) = Type IA Storm Distribution
Column (4) = Column (3) * P
Column (5) = Accumulated sum of Column (4)
Column (6) = If (P < 0.2S) = 0, If (P > 0.2S) = (Column (5) - 0.2S)^2/(Column (5) + 0.8S), where the PERVIOUS AREA S value is used
Column (7) = Column (6) of the present step - Column (6) of the previous step
Column (8) = Same as Column (6) except use IMPERVIOUS AREA S value
Column (9) = Column (8) of the present step - Column (8) of the previous step
Column (10) = (PERVIOUS AREA/TOTAL AREA)*Column (7)+(IMPERVIOUS AREA/TOTAL AREA)*Column (9)
Column (11) = (60.5*Column (10)*Total Area)/dt, where dt = 10 or 60 minutes
Column (12) = Column (12) of previous time step + w * [(Column (11) of previous time step + Column (11) of present time step) - (2 * Column (12) of previous time step)] where w = routing constant = dt/(2Tc + dt) = 0.0641

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
					PERVIOUS		IMPERVIOUS				
Time Increment	Time (minute)	Rainfall Distrib. (fraction)	Incre. Rainfall (inches)	Accumul. Rainfall (inches)	Accum. Runoff (inches)	Incre. Runoff (inches)	Accum. Runoff (inches)	Incre. Runoff (inches)	Total Runoff (inches)	Instant Flowrate (cfs)	Design Flowrate (cfs)
1	0	0	0	0	0	0	0	0	0	0.0	0.0
2	10	0.004	0.012	0.012	0.000	0.000	0.000	0.000	0.000	0.0	0.0
3	20	0.004	0.012	0.023	0.000	0.000	0.000	0.000	0.000	0.0	0.0
4	30	0.004	0.012	0.035	0.000	0.000	0.000	0.000	0.000	0.0	0.0
5	40	0.004	0.012	0.046	0.000	0.000	0.000	0.000	0.000	0.0	0.0
6	50	0.004	0.012	0.058	0.000	0.000	0.001	0.001	0.000	0.0	0.0
7	60	0.004	0.012	0.070	0.000	0.000	0.004	0.002	0.001	0.1	0.0
8	70	0.004	0.012	0.081	0.000	0.000	0.007	0.003	0.001	0.1	0.0
9	80	0.004	0.012	0.093	0.000	0.000	0.011	0.004	0.002	0.1	0.0
10	90	0.004	0.012	0.104	0.000	0.000	0.015	0.005	0.002	0.1	0.1
11	100	0.004	0.012	0.116	0.000	0.000	0.020	0.005	0.002	0.1	0.1
12	110	0.005	0.015	0.131	0.000	0.000	0.027	0.007	0.003	0.2	0.1
13	120	0.005	0.015	0.145	0.000	0.000	0.035	0.008	0.003	0.2	0.1
14	130	0.005	0.015	0.160	0.000	0.000	0.044	0.008	0.003	0.2	0.1
15	140	0.005	0.015	0.174	0.000	0.000	0.053	0.009	0.003	0.2	0.2
16	150	0.005	0.015	0.189	0.000	0.000	0.062	0.009	0.004	0.2	0.2
17	160	0.005	0.015	0.203	0.000	0.000	0.072	0.010	0.004	0.2	0.2
18	170	0.006	0.017	0.220	0.000	0.000	0.084	0.012	0.005	0.3	0.2
19	180	0.006	0.017	0.238	0.000	0.000	0.097	0.013	0.005	0.3	0.2
20	190	0.006	0.017	0.255	0.000	0.000	0.110	0.013	0.005	0.3	0.3
21	200	0.006	0.017	0.273	0.001	0.000	0.123	0.013	0.006	0.3	0.3
22	210	0.006	0.017	0.290	0.001	0.001	0.137	0.014	0.006	0.4	0.3
23	220	0.006	0.017	0.307	0.003	0.001	0.151	0.014	0.006	0.4	0.3
24	230	0.007	0.020	0.328	0.005	0.002	0.168	0.017	0.008	0.5	0.4
25	240	0.007	0.020	0.348	0.008	0.003	0.185	0.017	0.008	0.5	0.4
26	250	0.007	0.020	0.368	0.011	0.003	0.202	0.017	0.009	0.5	0.4
27	260	0.007	0.020	0.389	0.015	0.004	0.219	0.017	0.009	0.5	0.5
28	270	0.007	0.020	0.409	0.019	0.004	0.237	0.018	0.009	0.6	0.5
29	280	0.007	0.020	0.429	0.023	0.005	0.255	0.018	0.010	0.6	0.5
30	290	0.008	0.024	0.453	0.029	0.006	0.276	0.021	0.012	0.7	0.6
31	300	0.008	0.024	0.477	0.036	0.007	0.297	0.021	0.012	0.7	0.6
32	310	0.008	0.024	0.501	0.043	0.007	0.318	0.021	0.013	0.8	0.7
33	320	0.008	0.024	0.524	0.051	0.008	0.340	0.022	0.013	0.8	0.7
34	330	0.008	0.024	0.548	0.059	0.008	0.362	0.022	0.013	0.8	0.7
35	340	0.008	0.024	0.572	0.068	0.009	0.384	0.022	0.014	0.8	0.8
36	350	0.010	0.028	0.599	0.078	0.011	0.409	0.026	0.016	1.0	0.8
37	360	0.010	0.028	0.627	0.089	0.011	0.435	0.026	0.017	1.0	0.9
38	370	0.010	0.028	0.655	0.101	0.012	0.461	0.026	0.017	1.0	0.9

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
					PERVIOUS		IMPERVIOUS				
Time Increment	Time (minute)	Rainfall Distrib. (fraction)	Incre. Rainfall (inches)	Accumul. Rainfall (inches)	Accum. Runoff (inches)	Incre. Runoff (inches)	Accum. Runoff (inches)	Incre. Runoff (inches)	Total Runoff (inches)	Instant Flowrate (cfs)	Design Flowrate (cfs)
39	380	0.010	0.028	0.682	0.113	0.012	0.486	0.026	0.018	1.1	1.0
40	390	0.010	0.028	0.710	0.126	0.013	0.512	0.026	0.018	1.1	1.0
41	400	0.010	0.028	0.737	0.139	0.013	0.539	0.026	0.018	1.1	1.0
42	410	0.013	0.039	0.776	0.158	0.019	0.575	0.037	0.026	1.6	1.1
43	420	0.013	0.039	0.815	0.179	0.020	0.613	0.037	0.027	1.6	1.3
44	430	0.013	0.039	0.854	0.200	0.021	0.650	0.037	0.027	1.7	1.4
45	440	0.018	0.052	0.906	0.229	0.029	0.700	0.050	0.037	2.3	1.6
46	450	0.018	0.052	0.958	0.260	0.031	0.750	0.050	0.038	2.3	1.8
47	460	0.034	0.099	1.057	0.320	0.061	0.846	0.096	0.074	4.5	2.3
48	470	0.054	0.157	1.213	0.424	0.103	0.999	0.153	0.123	7.4	3.4
49	480	0.027	0.078	1.292	0.478	0.054	1.075	0.077	0.063	3.8	4.1
50	490	0.018	0.052	1.344	0.516	0.037	1.127	0.051	0.043	2.6	3.8
51	500	0.013	0.039	1.383	0.544	0.028	1.165	0.038	0.032	1.9	3.3
52	510	0.013	0.039	1.422	0.572	0.028	1.203	0.038	0.032	2.0	2.9
53	520	0.013	0.039	1.460	0.601	0.029	1.241	0.038	0.032	2.0	2.6
54	530	0.009	0.026	1.486	0.620	0.019	1.266	0.025	0.021	1.3	2.3
55	540	0.009	0.026	1.511	0.639	0.019	1.291	0.025	0.022	1.3	2.0
56	550	0.009	0.026	1.537	0.659	0.019	1.317	0.025	0.022	1.3	1.8
57	560	0.009	0.026	1.563	0.678	0.019	1.342	0.025	0.022	1.3	1.7
58	570	0.009	0.026	1.588	0.698	0.020	1.367	0.025	0.022	1.3	1.5
59	580	0.009	0.026	1.614	0.717	0.020	1.392	0.025	0.022	1.3	1.5
60	590	0.009	0.026	1.639	0.737	0.020	1.417	0.025	0.022	1.3	1.4
61	600	0.009	0.026	1.665	0.757	0.020	1.442	0.025	0.022	1.3	1.4
62	610	0.009	0.026	1.690	0.777	0.020	1.468	0.025	0.022	1.3	1.4
63	620	0.009	0.026	1.716	0.797	0.020	1.493	0.025	0.022	1.3	1.4
64	630	0.009	0.026	1.741	0.818	0.020	1.518	0.025	0.022	1.3	1.4
65	640	0.009	0.026	1.767	0.838	0.020	1.543	0.025	0.022	1.3	1.4
66	650	0.007	0.021	1.788	0.855	0.017	1.564	0.021	0.018	1.1	1.3
67	660	0.007	0.021	1.808	0.871	0.017	1.585	0.021	0.018	1.1	1.3
68	670	0.007	0.021	1.829	0.888	0.017	1.605	0.021	0.018	1.1	1.2
69	680	0.007	0.021	1.850	0.905	0.017	1.626	0.021	0.018	1.1	1.2
70	690	0.007	0.021	1.871	0.922	0.017	1.647	0.021	0.018	1.1	1.2
71	700	0.007	0.021	1.892	0.939	0.017	1.667	0.021	0.018	1.1	1.1
72	710	0.007	0.021	1.913	0.956	0.017	1.688	0.021	0.018	1.1	1.1
73	720	0.007	0.021	1.934	0.973	0.017	1.709	0.021	0.019	1.1	1.1
74	730	0.007	0.021	1.955	0.990	0.017	1.729	0.021	0.019	1.1	1.1
75	740	0.007	0.021	1.975	1.008	0.017	1.750	0.021	0.019	1.1	1.1
76	750	0.007	0.021	1.996	1.025	0.017	1.771	0.021	0.019	1.1	1.1
77	760	0.007	0.021	2.017	1.042	0.017	1.791	0.021	0.019	1.1	1.1
78	770	0.006	0.017	2.034	1.056	0.014	1.808	0.016	0.015	0.9	1.1
79	780	0.006	0.017	2.050	1.070	0.014	1.824	0.016	0.015	0.9	1.0
80	790	0.006	0.017	2.067	1.084	0.014	1.841	0.016	0.015	0.9	1.0
81	800	0.006	0.017	2.083	1.097	0.014	1.857	0.016	0.015	0.9	1.0
82	810	0.006	0.017	2.100	1.111	0.014	1.873	0.016	0.015	0.9	0.9
83	820	0.006	0.017	2.116	1.125	0.014	1.890	0.016	0.015	0.9	0.9
84	830	0.006	0.017	2.133	1.139	0.014	1.906	0.016	0.015	0.9	0.9
85	840	0.006	0.017	2.149	1.153	0.014	1.923	0.016	0.015	0.9	0.9
86	850	0.006	0.017	2.166	1.167	0.014	1.939	0.016	0.015	0.9	0.9
87	860	0.006	0.017	2.183	1.181	0.014	1.955	0.016	0.015	0.9	0.9
88	870	0.006	0.017	2.199	1.195	0.014	1.972	0.016	0.015	0.9	0.9
89	880	0.006	0.017	2.216	1.209	0.014	1.988	0.016	0.015	0.9	0.9
90	890	0.005	0.015	2.230	1.222	0.012	2.003	0.014	0.013	0.8	0.9
91	900	0.005	0.015	2.245	1.234	0.012	2.017	0.014	0.013	0.8	0.9
92	910	0.005	0.015	2.259	1.246	0.012	2.031	0.014	0.013	0.8	0.8
93	920	0.005	0.015	2.274	1.259	0.012	2.046	0.014	0.013	0.8	0.8
94	930	0.005	0.015	2.288	1.271	0.012	2.060	0.014	0.013	0.8	0.8
95	940	0.005	0.015	2.303	1.284	0.012	2.075	0.014	0.013	0.8	0.8

(1) Time Increment	(2) Time (minute)	(3) Rainfall Distrib. (fraction)	(4) Incre. Rainfall (inches)	(5) Accumul. Rainfall (inches)	(6) PERVIOUS Accum. Runoff (inches)	(7) PERVIOUS Incre. Runoff (inches)	(8) IMPERVIOUS Accum. Runoff (inches)	(9) IMPERVIOUS Incre. Runoff (inches)	(10) Total Runoff (inches)	(11) Instant Flowrate (cfs)	(12) Design Flowrate (cfs)
96	950	0.005	0.015	2.317	1.296	0.012	2.089	0.014	0.013	0.8	0.8
97	960	0.005	0.015	2.332	1.309	0.012	2.103	0.014	0.013	0.8	0.8
98	970	0.005	0.015	2.346	1.321	0.012	2.118	0.014	0.013	0.8	0.8
99	980	0.005	0.015	2.361	1.334	0.013	2.132	0.014	0.013	0.8	0.8
100	990	0.005	0.015	2.375	1.346	0.013	2.147	0.014	0.013	0.8	0.8
101	1000	0.005	0.015	2.390	1.359	0.013	2.161	0.014	0.013	0.8	0.8
102	1010	0.004	0.012	2.401	1.369	0.010	2.173	0.012	0.011	0.6	0.8
103	1020	0.004	0.012	2.413	1.379	0.010	2.184	0.012	0.011	0.6	0.7
104	1030	0.004	0.012	2.424	1.389	0.010	2.196	0.012	0.011	0.6	0.7
105	1040	0.004	0.012	2.436	1.399	0.010	2.207	0.012	0.011	0.6	0.7
106	1050	0.004	0.012	2.448	1.409	0.010	2.219	0.012	0.011	0.6	0.7
107	1060	0.004	0.012	2.459	1.419	0.010	2.230	0.012	0.011	0.6	0.7
108	1070	0.004	0.012	2.471	1.429	0.010	2.242	0.012	0.011	0.6	0.7
109	1080	0.004	0.012	2.482	1.439	0.010	2.253	0.012	0.011	0.6	0.7
110	1090	0.004	0.012	2.494	1.449	0.010	2.265	0.012	0.011	0.6	0.7
111	1100	0.004	0.012	2.506	1.460	0.010	2.276	0.012	0.011	0.6	0.7
112	1110	0.004	0.012	2.517	1.470	0.010	2.288	0.012	0.011	0.6	0.6
113	1120	0.004	0.012	2.529	1.480	0.010	2.299	0.012	0.011	0.6	0.6
114	1130	0.004	0.012	2.540	1.490	0.010	2.311	0.012	0.011	0.6	0.6
115	1140	0.004	0.012	2.552	1.500	0.010	2.322	0.012	0.011	0.6	0.6
116	1150	0.004	0.012	2.564	1.510	0.010	2.334	0.012	0.011	0.6	0.6
117	1160	0.004	0.012	2.575	1.521	0.010	2.346	0.012	0.011	0.6	0.6
118	1170	0.004	0.012	2.587	1.531	0.010	2.357	0.012	0.011	0.6	0.6
119	1180	0.004	0.012	2.598	1.541	0.010	2.369	0.012	0.011	0.6	0.6
120	1190	0.004	0.012	2.610	1.551	0.010	2.380	0.012	0.011	0.6	0.6
121	1200	0.004	0.012	2.622	1.562	0.010	2.392	0.012	0.011	0.6	0.6
122	1210	0.004	0.012	2.633	1.572	0.010	2.403	0.012	0.011	0.7	0.6
123	1220	0.004	0.012	2.645	1.582	0.010	2.415	0.012	0.011	0.7	0.6
124	1230	0.004	0.012	2.656	1.592	0.010	2.426	0.012	0.011	0.7	0.7
125	1240	0.004	0.012	2.668	1.603	0.010	2.438	0.012	0.011	0.7	0.7
126	1250	0.004	0.012	2.680	1.613	0.010	2.449	0.012	0.011	0.7	0.7
127	1260	0.004	0.012	2.691	1.623	0.010	2.461	0.012	0.011	0.7	0.7
128	1270	0.004	0.012	2.703	1.633	0.010	2.472	0.012	0.011	0.7	0.7
129	1280	0.004	0.012	2.714	1.644	0.010	2.484	0.012	0.011	0.7	0.7
130	1290	0.004	0.012	2.726	1.654	0.010	2.496	0.012	0.011	0.7	0.7
131	1300	0.004	0.012	2.738	1.664	0.010	2.507	0.012	0.011	0.7	0.7
132	1310	0.004	0.012	2.749	1.675	0.010	2.519	0.012	0.011	0.7	0.7
133	1320	0.004	0.012	2.761	1.685	0.010	2.530	0.012	0.011	0.7	0.7
134	1330	0.004	0.012	2.772	1.695	0.010	2.542	0.012	0.011	0.7	0.7
135	1340	0.004	0.012	2.784	1.706	0.010	2.553	0.012	0.011	0.7	0.7
136	1350	0.004	0.012	2.796	1.716	0.010	2.565	0.012	0.011	0.7	0.7
137	1360	0.004	0.012	2.807	1.726	0.010	2.576	0.012	0.011	0.7	0.7
138	1370	0.004	0.012	2.819	1.737	0.010	2.588	0.012	0.011	0.7	0.7
139	1380	0.004	0.012	2.830	1.747	0.010	2.599	0.012	0.011	0.7	0.7
140	1390	0.004	0.012	2.842	1.758	0.010	2.611	0.012	0.011	0.7	0.7
141	1400	0.004	0.012	2.854	1.768	0.010	2.623	0.012	0.011	0.7	0.7
142	1410	0.004	0.012	2.865	1.778	0.010	2.634	0.012	0.011	0.7	0.7
143	1420	0.004	0.012	2.877	1.789	0.010	2.646	0.012	0.011	0.7	0.7
144	1430	0.004	0.012	2.888	1.799	0.010	2.657	0.012	0.011	0.7	0.7
145	1440	0.004	0.012	2.900	1.810	0.010	2.669	0.012	0.011	0.7	0.7

2.3.4 Hydrograph Routing (Sizing Detention Facilities)

A methodology is presented here for routing a hydrograph through an existing retention/detention facility or closed depression, and for sizing a new retention/detention facility using hydrograph analysis.

Storage Routing Technique: The "level pool routing" technique presented here is one of the simplest and most commonly used hydrograph routing methods. This method is described in "Handbook of Applied Hydrology," Chow, V. Te, 1964, and elsewhere, and is based on the continuity equation:

Inflow - Outflow = Change in Storage

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

Where I = Inflow at time 1 and time 2
O = Outflow at time 1 and time 2
S = Storage at time 1 and time 2
Δt = Time interval, 2-1

The time interval, Δt, must be consistent with the time interval used in developing the inflow hydrograph. The time interval used for a 24-hour storm is 10 minutes while the time interval used for a 7-day storm is 60 minutes. The Δt variable can be eliminated by dividing it into the storage variables to obtain the following rearranged equation:

$I_1 + I_2 + 2S_1 - O_1 = O_2 + 2S_2$

If the time interval, Δt, is in minutes and the units of storage (S) are in cubic feet (cf), this can be converted to cubic feet per second (cfs) by dividing by 60.

The terms I_1, I_2, O_1, and S_1 are known from the inflow hydrograph and from the storage and outflow values of the previous time step. The unknowns O_2 and S_2 can be solved interactively from the given stage-storage and stage-discharge curves.

The following section gives the specific hydrograph routing steps:

1. Develop stage-storage relationship

Develop Stage – Storage Relationship

For retention/detention facilities with vertical sides (vaults), the stored volume is simply the bottom area times the height.

For trapezoidal shaped ponds with equal side slopes the stored volume in terms of an aspect ratio R_A, and bottom width b_W, can be calculated from the following equation:

$$\text{Volume } S(h) = \tfrac{4}{3} h^3 \tan^2\theta + h^2 \tan\theta \,(R_A b_W + b_W) + h R_A b_W^2$$

Where h = stage height (ft) or water depth above pond bottom

$\tan\theta$ = horizontal component of the side slope

(i.e., $\tan\theta = \frac{3}{1}$ or 3 for a side slope of 3:1)

b_L = length of the pond bottom

b_w = width of the pond bottom

S(h) = storage (cu. ft.) at stage height, h.

R_A = Length/Width

Trapezoidal Pond Dimensions

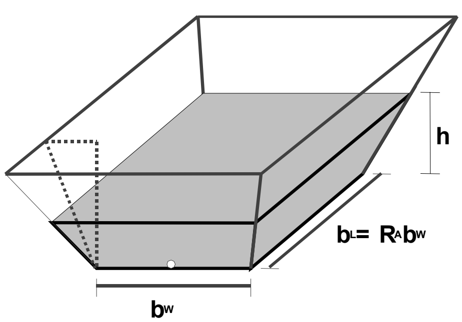
SS = Horizontal component of the side slope (SS:H to 1:V)

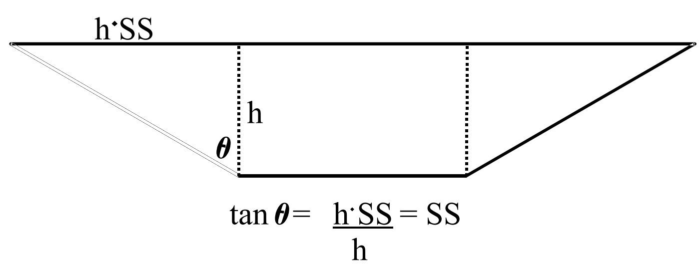
b_L = Bottom length of the pond

b_W = Bottom width of the pond

R_A = Bottom area aspect ratio (length to width ratio)

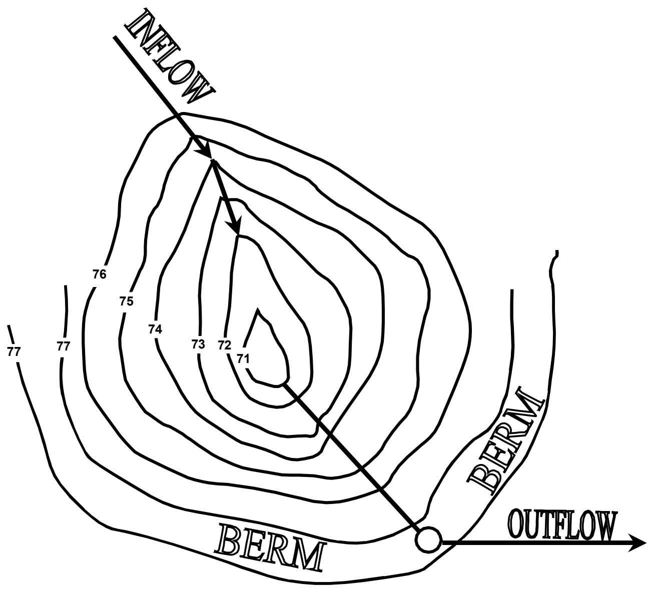
Bottom length b_L in terms of bottom width b_W; $b_L = R_A \cdot b_W$





For irregularly shaped areas the stage-storage curve may be developed as follows:

a. Obtain topographic contours of an existing or proposed reservoir/basin facility site and planimeter (or otherwise compute) the area enclosed by each contour. For example, see Figure 2.4 in which each contour represents a one-foot interval. Contour 71 is the lowest portion of the site and represents zero storage. Contour 76 represents a potential stage of 5 feet above the bottom of the facility.



Area within each contour	
71	600 sq. ft.
72	4,400 sq. ft.
73	8,700 sq. ft.
74	11,400 sq. ft.
75	14,500 sq. ft.
76	19,000 sq. ft.

Figure 2.4 Proposed Retention/Detention Facility Site Contours At 1-Foot Intervals

b. Calculate the average area between each contour. For the example given above, the average area between contours 71 and 72 would be:

$$\frac{600+4{,}400}{2} = \underline{2{,}500\ sq.ft.}$$

c. Calculate the volume between contours by multiplying the average area between contours by the difference in elevation. To illustrate, the volume between contours 71 and 72 would be:

(2,500)(1 foot) = <u>2,500 cu. ft.</u>

Similarly,

Area 72-73	-	6,550 cu. ft.
Area 73-74	-	10,050 cu. ft.
Area 74-75	-	12,950 cu. ft.
Area 75-76	-	16,750 cu. ft.

d. Define the total storage below each contour. This is just the sum of the volumes computed in the previous step for the contour in question. For example, there is no storage below contour 71; 2,500 cu. ft. below Contour 72, and (6,550 + 2,500 = 9,050 cu. ft. below Contour 73).

In summary,

Contours	Stage	Sum of Volumes	Total Volume
Contours 71-72	1	0 + 2,500	= 2,500 cu. ft.
Contours 72-73	2	2,500 + 6,500	= 9,050 cu. ft.
Contours 73-74	3	9,050 + 10,050	= 19,100 cu. ft.
Contours 74-75	4	19,100 + 12,950	= 32,050 cu. ft.
Contours 75-76	5	32,050 + 16,750	= 48,800 cu. ft.

The stage-storage relationship for this examples is shown in Figure 2.5

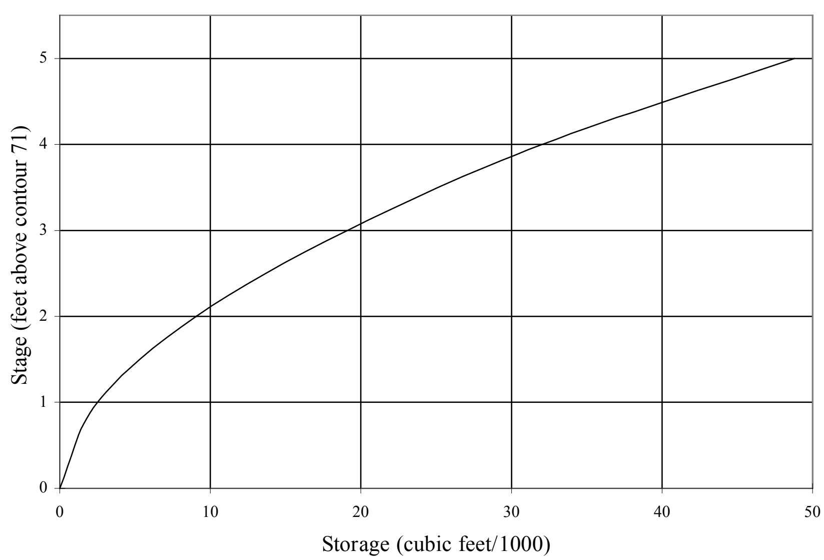


Figure 2.5 Stage Storage Relationship

2. Develop a routing curve

Develop a Routing Curve

A routing curve is simply a plot of outflow for a given stage versus a term, O + 2S, for the same stage. This curve may be easily plotted by setting up a table like Table 2.8. The units for the expression of outflow, O, are cubic-feet per second for the time period of interest. For this example, the time period, Δt, of 60 minutes will be used for illustrative purposes. (Usually Δt will be 10 minutes to correspond to the time steps used in preparing the hydrographs.) Therefore, all variables in the rearranged continuity equation must have the units of cfs. This means that the storage which was plotted in cubic feet (cf) must be converted from cf to cfs by dividing it by the time interval, Δt. From the above example, for the storage below Contour 72 this would be:

$$\frac{S}{\Delta t} = \frac{2{,}500\,ft^3}{(60\min)} \times \frac{1\min}{60\sec} = \underline{0.694\ cfs}$$

Table 2.8 Tabulation of Data for Routing Curve					
Elevation Feet	Stage Feet	Outflow* O cfs	Storage S cfs**	2S cfs	O+2S cfs
71	0	0.00	0.000	0.000	0.00
72	1	1.74	0.694	1.389	3.13
73	2	2.46	2.514	5.028	7.48
74	3	3.01	5.306	10.611	13.62
75	4	3.47	8.903	17.806	21.28
76	5	3.88	13.556	27.111	30.99

* *from 8" orifice, stage-discharge relationship [Q = (orifice area)(0.62)* $\sqrt{2gh}$ *, where h = stage]*

** *from stage-storage curve, Figure2.5, storage volume converted to cfs for 60-minute time intervals.*

For this example, the maximum allowable outflow was arbitrarily selected to be 4.0 cfs (this value is normally the pre-developed runoff rate from the site) and the maximum stage in the pond of 5 feet.

The "Outflow" column in Table 2.8 is the stage-discharge relationship for an 8-inch orifice outlet pipe. The 8-inch pipe size was chosen because the maximum allowable outflow is approximately 4.0 cfs at the maximum desired storage depth of 5 feet. From Table 2.8 the routing curve is plotted as shown in Figure 2.6.

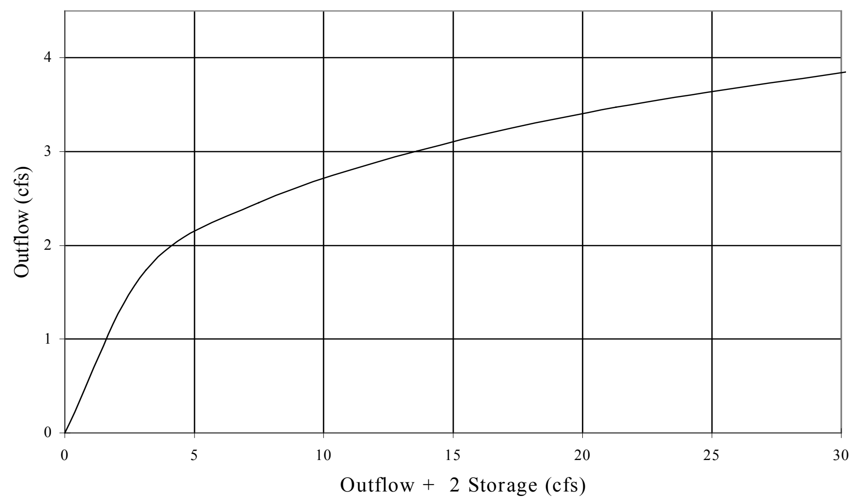


Figure 2.6 Routing Curve for Δt= 60 minutes

3. Route inflow hydrograph through proposed storage facility

Route Inflow Hydrograph Through Facility

The final step is to route the inflow hydrograph through the proposed storage facility by completing successive columns of Table 2.9 for each time period. For illustrative purposes a simple triangular shaped inflow hydrograph will be assumed as shown in Figure 2.7.

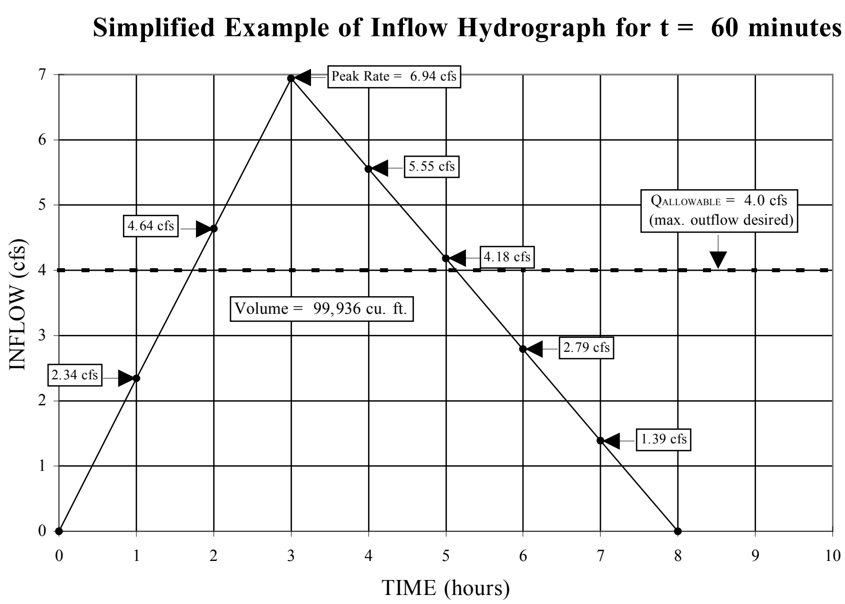


Figure 2.7 Simplified Example of Inflow Hydrograph

The routing table is completed by the following steps, using the routing equation:

$I_n + I_{n+1} + 2S_n - O_n = O_{n+1} + 2S_{n+1}$ (For each time period, Δt)

Where subscripts n and n+1 are used to indicate the beginnings of the time periods n and n+1 respectively.

Table 2.9
Tabular Calculation of Outflow Using Level Pool Routing
ROUTING TABLE
Time Periods (Δt = 60 min, usually Δt = 10min.)

Time Periods	(1) I_n (cfs)	(2) I_{n+1} (cfs)	(3) $2S_n$ (cfs)	(4) $I_n+I_{n+1}+2S_n$ (cfs)	(5) O_n (cfs)	(6) $O_{n+1}+2S_{n+1}$ (cfs)	(7) Stage (ft)	(8) Elevation (ft)
1	0.00	2.34	0.00	2.34	0.00	2.34	0.00	71.00
2	2.34	4.64	1.04	8.02	1.30	6.72	0.75	71.75
3	4.64	6.94	4.39	15.97	2.33	13.64	1.82	72.82
4	6.94	5.55	10.63	23.12	3.01	20.11	3.00	74.00
5	5.55	4.18	16.71	26.44	3.40	23.04	3.85	74.85
6	4.18	2.79	19.49	26.46	3.55	22.91	4.18	75.18*
7	2.79	1.39	19.37	23.55	3.54	20.01	4.17	75.17
8	1.39	0.00	16.61	18.00	3.40	14.60	3.83	74.83
9	0.00	0.00	11.54	11.54	3.07	8.47	3.13	74.13
10	0.00	0.00	5.92	5.92	2.54	3.38	2.16	73.16
11	0.00	0.00	1.60	1.60	1.78	0.00	1.06	72.06
12	0.00	0.00	0.00	0.00	0.00	0.00	0.00	71.00

*Maximum floodplain elevation

Steps for Calculating Outflow Using Level Pool Routing

1. Inflow, I_n, is the inflow at the beginning of each time period and is read from the inflow hydrograph for each time period. For period 1, I_n in this example is zero, and this is entered in Row (1) and Column (1) of the routing table. For period 2, I_n = 2.34 cfs and is entered in Row (2) Column (1) of the routing table.
2. I_{n+1}, inflow at the end of each time period, is read from the inflow hydrograph, and entered in Column (2) for each time period. This same value is the initial inflow, I_n, for the next time period. For our example, I_{n+1}, for time period 1 is 2.34 cfs; this is also entered in Column (1) of time period 2 as I_n.
3. Two times the initial storage, $2S_n$, is entered in Column (3). For the example, initial storage for time period 1 is zero. Subsequent values entered as $2S_n$, are calculated after the remaining values for the preceding time period are filled in. This is explained below.
4. Enter in Column (4) the sum of Columns (1), (2), and (3), for the appropriate time period. In this case, the sum for time period 1 is 2.34 cfs.

5. O_n, outflow initially for the time period, is entered in Column (5). For time period 1, O_n, is zero in this case. Subsequent values are read from the routing curve after Column (6) has been calculated. This is explained below.

6. From the routing equation, the entry in Column (6), $O_{n+1} + 2S_{n+1}$, is the difference between Column (4), $I_n + I_{n+1} + 2S_n$, and Column (5), O_n. For time period 1, this value is 2.34 cfs. This is entered and the process repeated.

7. The value of O_n for any time period is obtained by taking the previous time period value of $O_{n+1} + 2S_{n+1}$, and then finding the corresponding value of Outflow O in Table 2.8. For example, O_n for time period 2 is found by taking the value for $O_{n+1} + 2S_{n+1}$, for time period 1 or 2.34 cfs, then by interpolating Table 2.8 a value of 1.30 cfs is obtained for O_n for time period 2 and is entered in Column (5).

8. The value of $2S_n$ for the time period 2 is the difference between the value of $(O_{n+1} + 2S_{n+1})$ for time period 1 and the value of O_n for time period 2. For this example,

   $O_{n+1} + 2S_{n+1} = 2.34$ (for time period 1)
   $O_n = 1.30$ (for time period 2)
   $2S_n = (O_{n+1} + 2S_{n+1}) - O_n = 1.04$ (for time period 2)

   Therefore, enter 1.04 in Column (3) for time period 2.

9. Find the sum of Columns (1), (2), and (3) and enter value in Column (4).

10. Subtract value in Column (5) from Column (4) and enter result in Column (6).

11. Refer to Table 2.8 for next value of O_n, corresponding to result of Step 10.

Continue this process until the outflow hydrograph, as represented by the tabulated values of O_n, returns to zero.

Column (7) of entries in the Table 2.9 (stage at the beginning of each time period) is obtained by dividing each value of $2S_n$ by 2, and referring to the stage-storage curve, Figure 2.5. For example, for time period 3, $2S_n$ equals 4.39 cfs-min, half of which is 2.19 cfs-min. Referring to stage-storage curve for storage 2.19 cfs-min. (or $2.19 \times 3600 = 7{,}901$ cu. ft.), a stage of 1.82 feet is obtained by interpolating the data for *Figure2.5*. Adding these stages to the elevation of the first contour (71) allows floodplain or maximum water surface (Column (8)) for each time period to be computed.

Finally, plot the values of O_n for each time period to plot the complete outflow hydrograph, as shown in Figure 2.8. The volume that must be stored is represented by the dark shaded area, and may be obtained through graphical techniques. The volume may also be closely estimated from the largest tabulated value of $2S_n$, divided by 2, and converted to cubic feet ($19.49/2 \times 3{,}600 = 35{,}082$ ft^3). This will exactly coincide with the true peak only if two hydrographs cross exactly at the end of a time interval. However, this inaccuracy in volume would be very small, and for practical purposes may be neglected.

In summary, the characteristics of the sample detention facility and the selected eight-inch orifice outlet are such that the peak runoff rate will be reduced below the required 4.0 cfs. Furthermore, the full 5 feet of available storage is not used and the maximum floodplain elevation generated in the pond is 75.18 feet. This indicates that additional trials or iterations could be performed to optimize the size and outlet control of this sample detention pond.

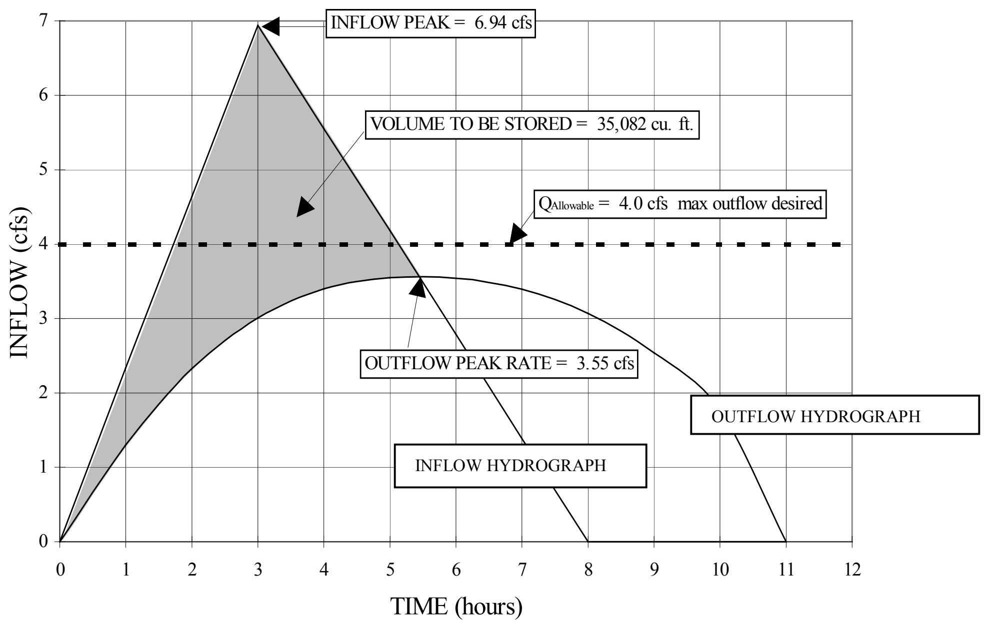


Figure 2.8 Simplified Example of Inflow and Outflow Hydrograph

Sizing a Detention Facility for Multiple Design Storm Events

To design a storage facility to meet given performance requirements, for example, to match the pre-development and post-development runoff for the 2, 10, and 100-year storms, it is usually necessary to perform many iterative routings to arrive at a minimum facility size with the proper outlet (orifice) control. Each iterative routing requires that the facility size (stage-storage curve) and/or outlet configuration (stage-discharge curve) be adjusted and tested for performance. Such iteration can be cumbersome, even with the use of a computer. To minimize the number of iterations, a graphical evaluation of the developed inflow hydrographs is useful in approximating the storage volume and outlet configuration of a hypothetical detention pond that meets the performance requirements, prior to beginning the iteration process to finalize the design of a detention facility.

The following example presents a graphical approach to approximating storage volume and outlet configuration as displayed in Figure 2.9.

1. Assume the following performance requirements (allowable release rates) and developed peak inflow rates have been noted from hydrographs generated for the purposes of sizing a standard on-site detention pond:

Design Storm	Allowable Release	Developed Inflows
2-year, 24-hour	P_2Q_{rel} = 0.23 cfs	P_2Q_{dev} = 1.65 cfs
10-year, 24-hour	$P_{10}Q_{rel}$ = 1.40 cfs	$P_{10}Q_{dev}$ = 3.46 cfs
100-year, 24-hour	$P_{100}Q_{rel}$ = 3.47 cfs	$P_{100}Q_{dev}$ = 5.89 cfs

Note: This example illustrates detaining the peak flows for the 2, 10, and 100-year, 24-hour duration design storms.

The required performance for the 100-year design storm may in some cases be more than the pre-developed flow rate depending on downstream conditions.

2. Plots of the developed inflow hydrographs are used to graphically approximate the detention storage required to achieve the performance.
3. Starting with the 2-year hydrograph, the 2-year allowable release rate (P_2Q_{rel}) (which must not be exceeded) is plotted as a horizontal line extending from time zero to the point where it intercepts the falling limb of the hydrograph. A line is drawn from the beginning of the inflow hydrograph to this point. This line approximates the outflow rating curve of a control structure of a hypothetical detention facility that would restrict outflow to not exceed $P2Q_{rel}$ and thus approximates the rising limb of a hypothetical outflow hydrograph.
4. As in standard inflow-outflow hydrograph analysis, the area under the inflow hydrograph, less the area under the rising limb of the hypothetical outflow hydrograph, graphically approximates the amount of inflow which must be stored, detained and released once the inflow hydrograph falls below the allowable release rate. This volume of storage for the 2-year storm (as shaded) is termed S_2 and can thus be approximated by measuring the shaded area with a planimeter. In this example, the area under the inflow hydrograph equals 29,648 cu. ft. and the area under the rising limb of the hypothetical outflow hydrograph equals 9,658 cu. ft. $\therefore$ S_2 = 29,648 - 9,658 = 19,990 cu. ft..

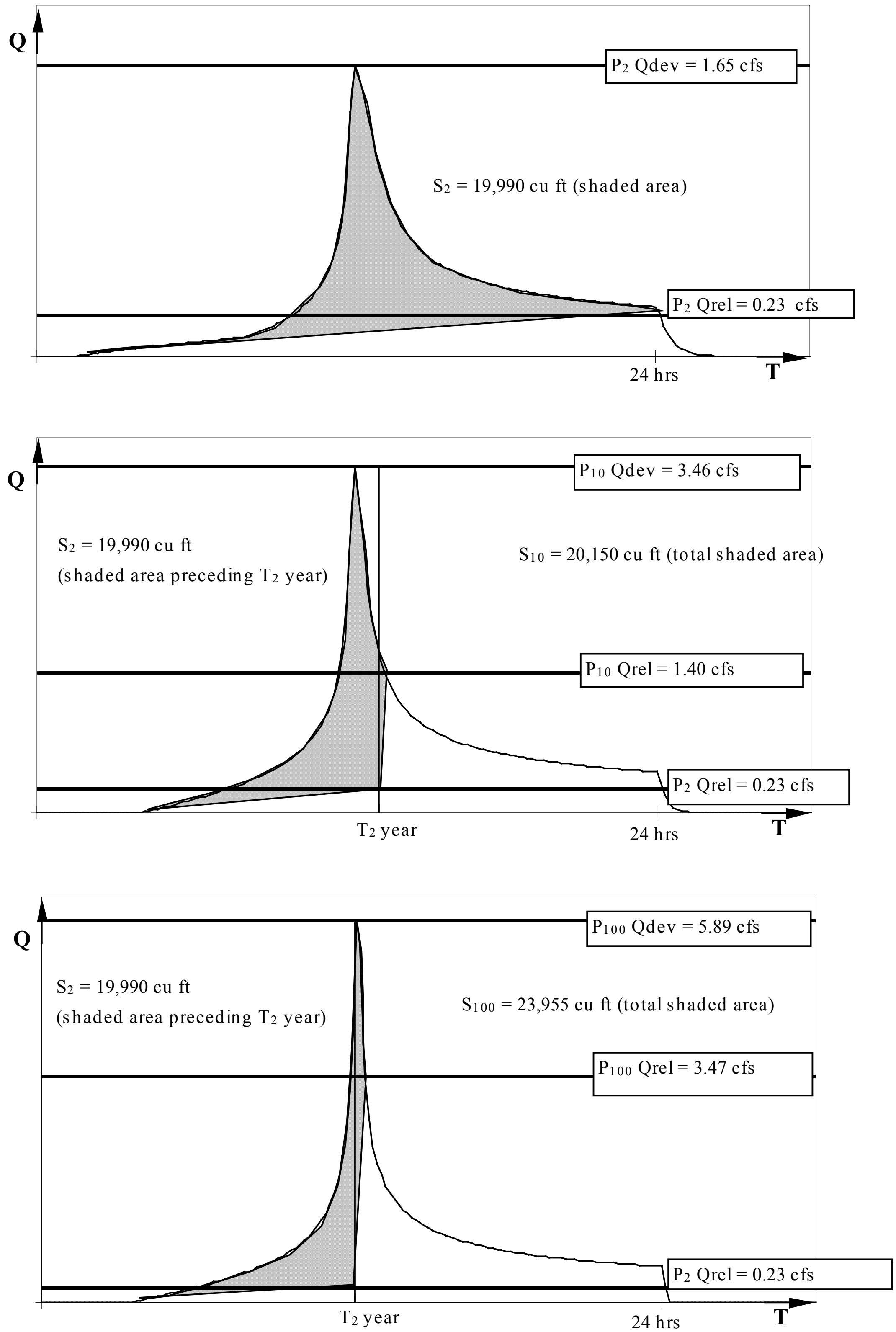


Figure 2.9 Plots of the Developed Condition Inflow Hydrographs for the 2, 10, and 100-Year Design Storms

5. The 10 and 100-year developed inflow hydrographs must now each be examined to determine which will require the most storage volume in addition to the 19,990 cu. ft. approximated for the 2-year storm. Note, the amount of storage volume needed to control the 10-year storm may exceed that of the 100-year when using this method. This occurs because the peak flows for the 10- and 100-year inflow hydrographs are similar in magnitude, and the difference between 10-year allowable release and developed peak rates can be substantially greater than for the 100-year. The interception point with the $P_{10}Q_{rel}$ thus occurs further down the falling limb than for the 100-year, resulting in a larger storage volume required.

   The 10 or 100-year allowable release rate ($P_{10}Q_{rel}$ or $P_{100}Q_{rel}$) (which must not be exceeded) is plotted as a horizontal line extending from time zero to the point where it intercepts the falling limb of the corresponding hydrograph.

   By trial and error, the time (T_2-year) at which the S_2 volume occurs, while maintaining P_2Q_{rel}, is determined by planimeter. From this point, a line is drawn to connect to the $P_{10}Q_{rel}$ or $P_{100}Q_{rel}$ point on the falling limb. The area from time T_2-year under the inflow hydrograph to this point, less the area under the rising limb of the hypothetical outflow hydrograph (shown as the slender shaded triangle(s)), represents the additional storage volume needed to meet the required performance. The total storage volume S_{10} or S_{100} can then be computed by adding the additional storage volume to S_2.

6. From the storage volumes computed above, choose the largest of the three volumes for the initial pond sizing. In this case the 100-year volume, $S_{100.}$, is the largest. Therefore, call it S_d.

   S_d = 23,955 cu. ft.

7. Using the volume equation for trapezoidal shaped ponds and the design volume S_d from above, estimate the detention pond's dimensions by first determining the pond's bottom width, b_W.

   Assuming a 3:1 side slope, a design depth (h_d) of 4 feet, and a square bottom (bottom area aspect ratio or length to width ratio, $R_A = 1$).

   $\tan\theta = 3$

   R_A = Bottom Area Aspect Ratio = 1

   Bottom Length $b_L = R_A b_W$

   Volume $S(h) = \frac{4}{3}h^3\tan^2\theta + h^2\tan\theta\,(b_L + b_W) + hb_Lb_W$

   S(h) in terms of the aspect ratio and

   $b_w = \frac{4}{3}h^3\tan^2\theta + h^2\tan\theta\,(R_Ab_W + b_W) + hR_Ab_W^{\,2}$

Rearranging the volume equation to solve for b_w:

$$b_W^2 h R_A + b_W h^2 \tan\theta\,(1+R_A) + \tfrac{4}{3} h^3 \tan^2\theta - S(h) = 0$$

Solve the quadratic for bottom width b_W:

$$b_W = \frac{-(1+R_A)h^2\tan\theta + \sqrt{[(1+R_A)h^2\tan\theta]^2 - 4R_A h(\tfrac{4}{3}h^3\tan^2\theta - S(h))}}{2R_A h}$$

$$b_W = \frac{-(1+1)(4^2)(3) + \sqrt{[(1+1)(4^2)(3)]^2 - (4)(1)(4)(\tfrac{4}{3})(4^3)(3^2) - 23{,}955)}}{(2)(1)(4)} =$$

65.08 ft.

8. Compute the stage-storage curve using the bottom width calculated above and the following equation for S(h) in terms of the aspect ratio and b_w for trapezoidal shaped ponds. Remembering that for the square bottomed pond in our example the bottom area aspect ratio or length to width ratio, $R_A = 1$, and the bottom length $b_L = R_A b_W$.

Volume $S(h) = \tfrac{4}{3} h^3 \tan^2\theta + h^2 \tan\theta\,(R_A b_W + b_W) + h R_A b_W^2$

Table 2.10
Stage – Storage Table

Stage, h*	Storage, S(h)
0.0	0.0
0.5	2,217
1.0	4,637
1.5	7,271
2.0	10,128
2.5	13,215
3.0	16,543
3.5	20,120
4.0	23,955

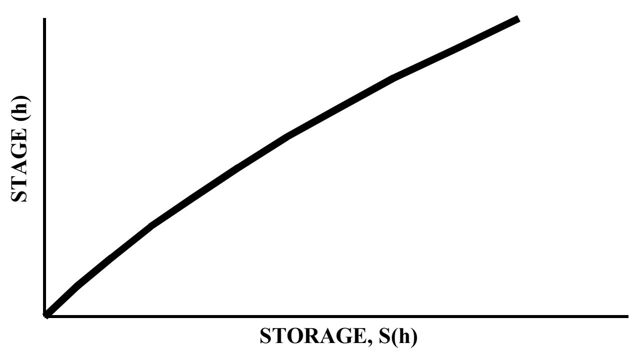


Figure 2.10 Stage-Storage Curve

****Note*:* *Stage heights, h, should be adjusted so that they measure from the outlet invert rather than the pond bottom. In this example, the outlet invert is assumed to be at the same elevation as the pond bottom. Therefore, no adjustment is required.*

9. From the stage-storage curve shown in Figure 2.10, determine the depth, h, required for the 2-year storage volume.

   For S_2 = 19,990 cu. ft. and interpolating Table 2.10: h_2 = <u>3.48 feet</u>

Special Note: *It has been found through experience that usually only two orifices are necessary to meet 2, 10 and 100-year performance requirements. The bottom orifice is therefore sized to meet the 2-year performance requirement, while the top orifice is located above the 2-year water surface and is sized and situated such that both the 10-year and 100-year performance requirements are met. This is further illustrated as the example continues.*

10. Size the bottom orifice for the 2-year allowable release, P_2Q_{rel} = 0.23 cfs, using the following derivation of the orifice equation:

    $Q = CA\sqrt{2gh}$ Standard orifice equation

    Where:

    C = entrance loss coefficient = 0.62 (typical)
    A = area of orifice = $\pi d^2/4$, where d = diameter of orifice
    g = acceleration of gravity = 32.2 ft/s²
    h = head on orifice

    For 2-year allowable release:

    P_2Q_{rel} = 0.23 cfs; h_2 = 3.48 feet

    $P_2Q_{rel} = CA_b\sqrt{2gh_2}$

    rearranging to solve for A_b:

    $A_b = P_2Q_{rel}/ C\sqrt{2gh_2}$

    substituting for diameter:

    $$\pi d_b^2/4 = P_2Q_{rel}/ C\sqrt{2gh_2}$$
    $$d_b^2 = 4P_2Q_{rel}/\pi C\sqrt{2gh_2}$$
    $$d_b = \sqrt{4P_2Q_{rel} / \pi C\sqrt{2gh_2}}$$
    $$d_b = \sqrt{4(0.23) / \pi (0.62)\sqrt{2(32.2)(3.48)}}$$
    $$d_b = 0.18 \text{ feet } (= 2.16 \text{ inches})$$

11. Sketch and consider the following flow control restrictor schematic as shown in Figure 2.11:

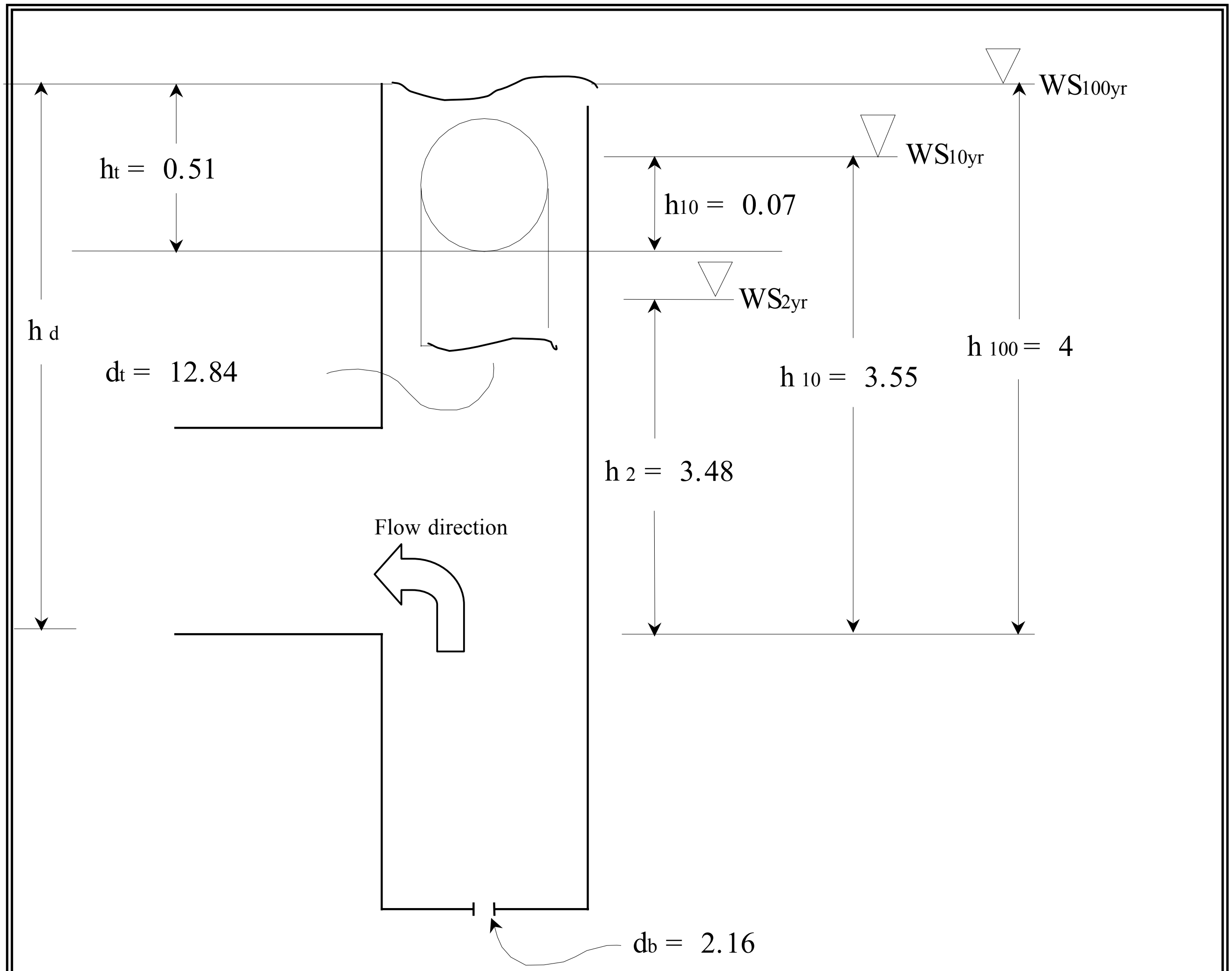


Figure 2.11 Flow Restrictor Schematic

12. Size and situate the top orifice at or above the 2-year water surface ($WS_{2yr.}$). This may require some trial and error. The release rate for this orifice is:

$P_{100}Q_{rel}$ = $Q_t + Q_b$, where $P_{100}Q_{rel}$ = 3.47 cfs (total allowable flow at maximum head)

$Q_b = CA_b\sqrt{2gh}$ where $A_b = \pi d^2/4$

d_b = 0.18 feet, thus $A_b = \pi(0.18)^2/4 = \underline{0.03 \text{ ft}^2}$

h = 4 feet

$$\therefore Q_b = (0.62)(0.025)\sqrt{2(32.2)4} = \underline{0.30 \text{ cfs}}$$

Q_t = 3.47 cfs - 0.30 cfs = $\underline{3.17 \text{ cfs}}$

The size of this orifice will depend on its vertical location as specified by its head, h_t. h_t in this example, must be less than the design depth of 4 ft. minus the two-year depth of 3.48 ft. or 0.52 ft. so that the top orifice is above the 2-year water surface. Therefore, try a h_t = 0.51 ft:

Top Orifice Diameter, $d_t = \sqrt{\dfrac{4Q_t}{\pi C\sqrt{2gh_t}}}$

$$d_t = \sqrt{\frac{4(3.17)}{\pi(0.62)\sqrt{2(32.2)(0.51)}}} = \underline{1.07 \text{ feet}} \text{ (= 12.8 inches)}$$

13. Check for 10-year volume (S_{10}) by:
   a. Computing h_{10} (see Figure 2.11)

      $Q_{10} = Q_t - Q_2$; or in this case $P_{10}Q_{rel} = Q_t - P_2Q_{rel}$

      $P_{10}Q_{rel} = CA_t\sqrt{2gh_{10}} + P_2Q_{rel}$

      Solving for h_{10}:

      $$h_{10} = \left[\frac{P_{10}Q_{rel} - P_2Q_{rel}}{CA_t}\right]^2 \times (2g)^{-1}$$

      $A_t = \pi d^2/4$ where d= 1.07 ft.; $A_t = 0.90 \text{ ft}^2$

      $$h_{10} = \left[\frac{1.40 - 0.23}{0.62(0..90)}\right]^2 \times (2(32.2))^{-1} = 0.07 \text{ ft.}$$

   b. Computing h_{10} = 3.48 feet + 0.07 feet = 3.55 feet

   c. Check the stage-storage curve to see if there is sufficient volume at h_{10} = 3.55 feet:

      For h_{10} = 3.55 feet; S(h) = 20,503 cu. ft.

      Recalling S_{10} = 20,150 cu. ft.

      <u>20,503 cu. ft. > 20,150 cu. ft.</u> <u>Okay</u>

Note: *Since the top-orifice size of 12.84 inches is too large to feasibly install in the upper 0.51 feet of the riser pipe, a notch weir must be substituted. Also, the 100-year water surface in this example is shown at the same elevation as the top of the riser. This will not necessarily be the case. In fact, it may be <u>slightly</u> above the riser and still meet performance.*

The stage/storage/discharge information for a hypothetical detention facility developed above can then be used as a guide to design the actual detention facility. The design is checked using the "level pool routing" technique described at the beginning of this section. The stage/storage/discharge data from the actual facility design is used with each of the developed inflow hydrographs routed through the facility in order to demonstrate that the required performance is met.

The practical minimum orifice sizing may be between ½ inch and 1 inch in diameter. This could restrict the ability of small sites to satisfy some detention requirements. Local governments should pursue alternative detention requirements, such as the use of regional facilities (for a group of small lots, for example), for small sites.

This is the point in the process where a number of iterations may be required in order to calibrate and optimize the actual facility design to meet performance with the minimal amount of storage volume. With experience and over time, techniques and methods will be developed that may assist the design engineer in this process. Ecology and local governments will inform the engineering community of these techniques and methods as they become available.

2.3.5 Hydrograph Summation and Phasing

One of the key advantages of hydrograph analysis is the ability to accurately describe the cumulative effect of runoff from several basins and/or sub-basins having different runoff characteristics and travel times. This cumulative effect is best characterized by a single hydrograph, which is obtained by summing the individual hydrographs from tributary basins at a particular "discharge point of interest." The general procedure for performing a hydrograph summation is described below:

Hydrograph Summation

1. 1.Select the "discharge point of interest" at which the hydrographs will be summed.

2. Estimate the time required for each hydrograph to travel from its point discharge to the "discharge point of interest." This travel time can be estimated using the methods presented in Section 2.3.2. under "Travel Time and Time of Concentration."
3. Shift each hydrograph according to its travel time to the "discharge point of interest" as shown below in Figure 2.12.

4. Sum the shifted hydrographs by adding the ordinate flow values at each time interval as shown below in Figure 2.13.

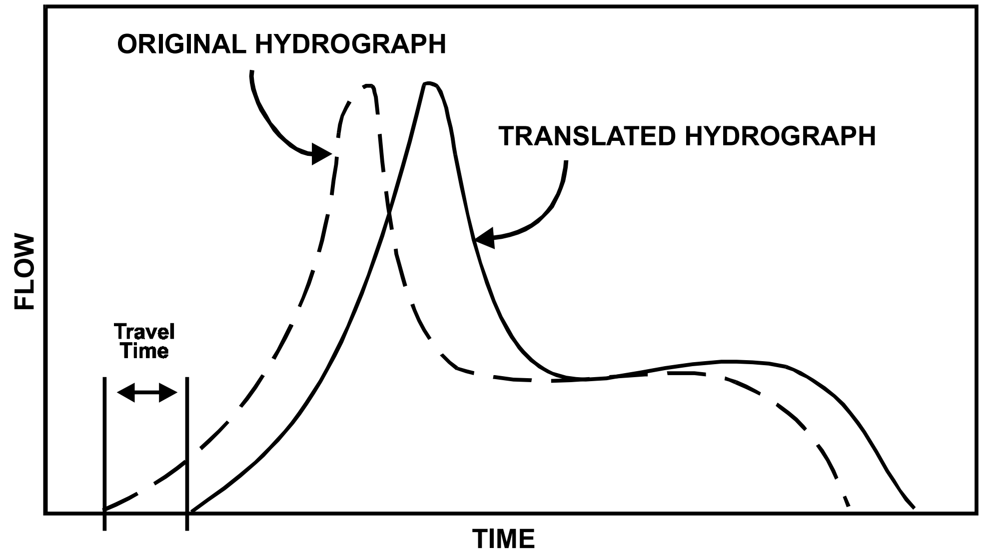


Figure 2.12 Shifting the Hydrographs Using Travel Time

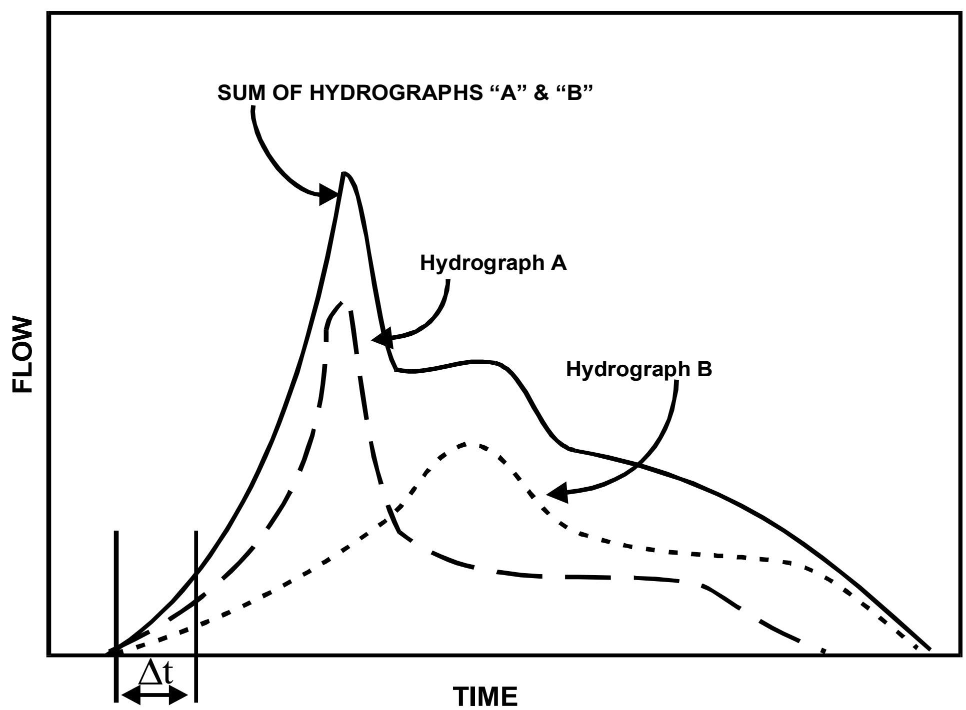


Figure 2.13 Summing the Shifted Hydrographs

Note: Δt has been previously defined as 10 minutes or less for a 24-hour duration storm and 60 minutes for a 7-day duration storm.

Hydrograph Phasing Analysis

The ability to characterize cumulative effects through the summation of hydrographs provides a valuable tool for analyzing the interaction of on-site and off-site hydrographs both before and after development. This interaction of hydrographs is generally referred to as "hydrograph phasing" due to the similarity with compound wave-shapes. This hydrograph phasing analysis is required in order to determine the effect of the compound hydrograph shape on the downstream system.

The general procedure for performing a hydrograph phasing analysis is as follows:

Select the "discharge point of interest" at which the on-site and off-site runoff hydrographs will be summed and compute travel times as explained under "Hydrograph Summation."

Compute the pre-developed on-site hydrograph and the existing off-site hydrograph for the design storm of interest. Shift and sum these hydrographs as explained under "Hydrograph Summation."

Compute the post-developed on-site hydrograph. If on-site detention is provided, this hydrograph will be the outflow hydrograph from the facility. Shift and sum this hydrograph with the existing off-site hydrograph.

Plot the above two summations as shown in Figure 2.14 below to obtain a comparison of cumulative effects:

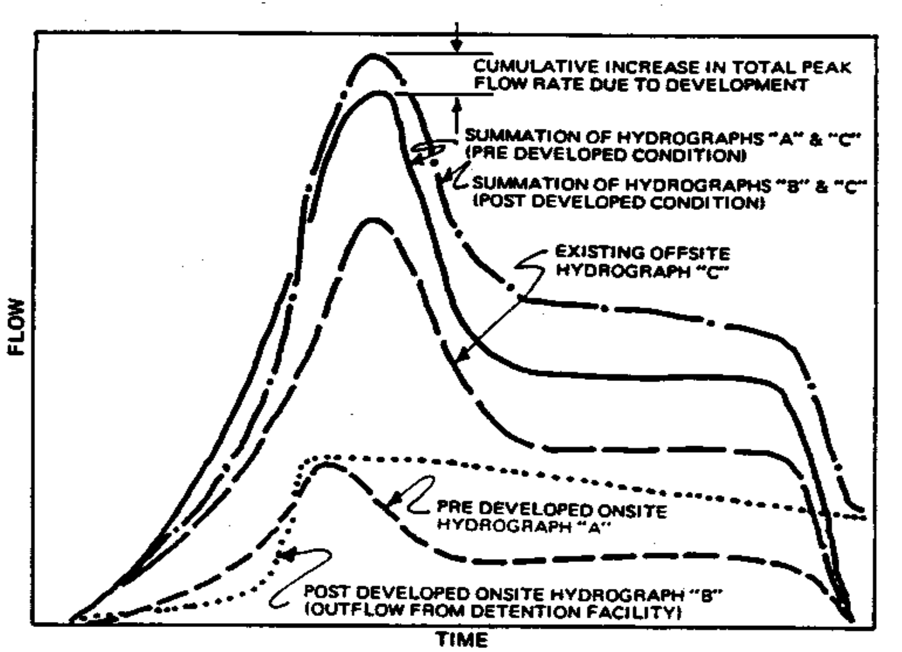


Figure 2.14 Hydrograph Phasing Analysis

Figure 2.14 illustrates how a development with standard on-site detention can cause an increase in peak flow at some point downstream. If this is the case, the local government shall require that this condition be addressed by reducing the release rate from the detention facility such that the cumulative effect downstream is negligible as shown in Figure 2.15.

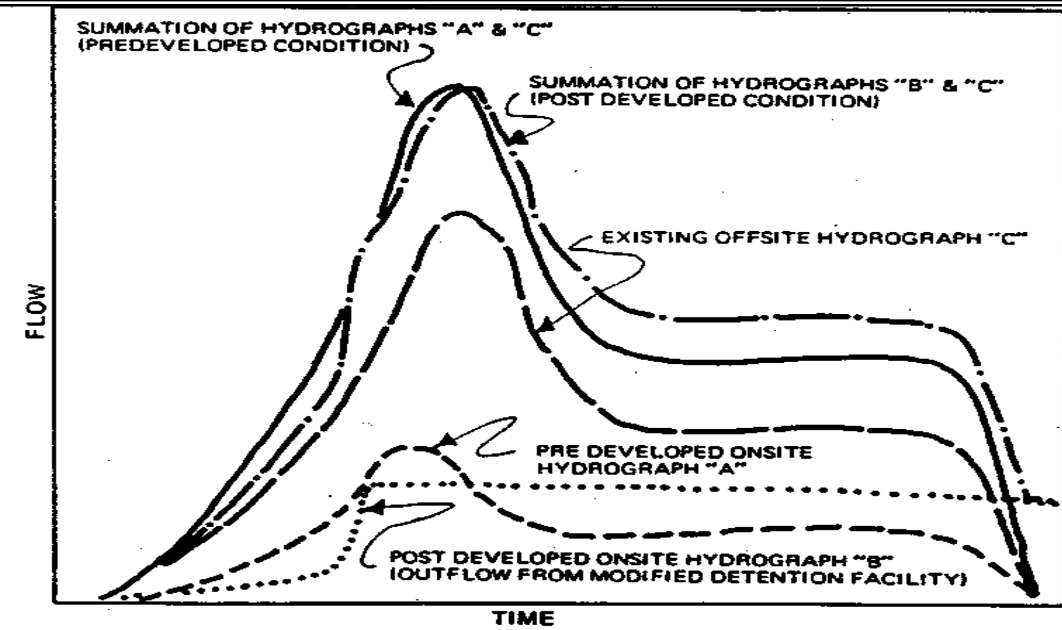


Figure 2.15 Reducing the Release Rate to Decrease Downstream Effects

2.3.6 Computer Applications

SBUH Method and Level Pool Routing: The computations required to generate the runoff hydrographs and perform the level pool routing techniques presented in this chapter can be performed manually. However, due to the number of computations and repetitive nature, a programmable calculator and/or a personal computer will perform these computations much quicker and with a likely higher degree of accuracy. There are also commercial programs available that perform these calculations.

Computer Models: Local governments may make available programs and application templates developed in-house. These will likely be available on a "make-your-own-copy basis" and will be provided with minimal documentation and no formal support. Software developers have prepared programs that they market and support. Local governments may maintain a list of these programs as they approve them for use.

2.4 Closed Depression Analysis

The analysis of closed depressions requires careful assessment of the existing hydrologic performance in order to evaluate the impacts a proposed project will have. The applicable requirements, (see Minimum Requirement #7) and the local government's Sensitive Areas Ordinance and Rules (if applicable) should be thoroughly reviewed prior to proceeding with the analysis. A calibrated continuous simulation hydrologic model must be used for closed depression analysis and design of mitigation facilities. Where an adequately calibrated continuous simulation model is not available, the procedures below may be followed.

Analysis and Design Criteria: The infiltration rates used in the analysis of closed depressions must be determined according to the procedures in Section 3.3. For closed depressions containing standing water, soil texture tests must be performed on dry land adjacent to, and on opposite sides of the standing water (as is feasible). The elevation of the testing surface at the bottom of the test pit must be one foot above the standing water elevation. A minimum of four tests must be performed to prepare an average surface infiltration rate.

Projects proposing to modify or compensate for replacement storage in a closed depression must meet the design criteria for detention ponds as described in this volume.

Method of Analysis: Closed depressions are analyzed using hydrographs routed as described in Section 2.3.4 “Hydrograph Routing.” Infiltration must be addressed where appropriate. In assessing the impacts of a proposed project on the performance of a closed depression there are three cases that dictate different approaches to meeting Minimum Requirement #7 and applicable local requirements. Note that where there is a flooding potential, concern about rising ground water levels, or there are local sensitive area ordinances and rules, this analysis may not be sufficient and the local governments may require more stringent analysis.

***Case 1*:** The 100-year, 7-day duration design storm flow from the drainage basin tributary to the closed depression is routed into the closed depression using only infiltration as outflow. If predevelopment runoff does not overflow the closed depression, then no runoff may leave the closed depression for the 100-year, 7-day duration design storm following development of a proposed project. This may be accomplished by excavating additional storage volume in the closed depression (subject to all applicable requirements, for example providing a defined overflow system). See Table 2.11 for a summary of Case 1.

Case 2: The 100-year, 7-day duration design storm flow from the drainage basin tributary to the closed depression is routed into the closed depression using only infiltration as outflow. If runoff does overflow the closed depression, then the 100-year, 24-hour duration design storm flow from the drainage basin tributary to the closed depression is routed into the closed depression using only infiltration as outflow. If this does not cause overflow, then the allowable release rate is that which occurred for the 100-year, 7-day duration design storm. This performance objective can be met by excavating additional storage volume in the closed depression (subject to all applicable requirements, for example providing a defined overflow system). See Table 2.11 for a summary of Case 2.

Case 3**:** The 100-year, 7-day duration design storm flow and the 100-year, 24-hour duration design storm flow from the drainage basin tributary to the closed depression are routed into the closed depression using only infiltration as outflow, and both cause overflow to occur. Then the closed depression must be analyzed as a detention/infiltration pond. The required performance, therefore, is to meet the runoff duration standard in Volume I using an adequately calibrated continuous simulation model. This will require that a control structure, emergency overflow spillway, access road, and other design criteria are met and, if it is to be maintained by the local government, the closed depression placed in a tract dedicated to the local government. If it is to be privately maintained, it must be located in a drainage easement dedicated to the public. See Table 2.11 for a summary of Case 3.

Table 2.11
Closed Depression Summary Table

Case No.	Check for Pre-development Overflow: Condition	Check for Pre-development Overflow: Overflow	Post-development Overflow Requirement
1	100-year, 7-day	None	None
2	100-year, 7 day AND 100-year, 24-hour	Some None	Match the pre-development overflow
3	100-year, 7 day AND 100-year, 24-hour	Some Some	Meet the runoff duration standard in Volume I

Chapter 3 - Flow Control Design

Note: Figures in Chapter 3 courtesy of King County, except as noted

This chapter presents methods, criteria, and details for hydraulic analysis and design of flow control facilities and roof downspout controls. *Flow control facilities* are detention or infiltration facilities engineered to meet the flow control standards specified in Volume I. *Roof downspout controls* are infiltration or dispersion systems for use in individual lots, proposed plats, and short plats. Roof downspouts may be used in conjunction with, and in addition to, any flow control facilities that may be necessary. Implementation of roof downspout controls may reduce the total effective impervious area and result in less runoff from these surfaces. Ecology's Hydrology Model incorporates flow credits for implementing two types of roof downspout controls. These are:

- If roof runoff is ***infiltrated*** according to the requirements of this section, the roof area may be discounted from the total project area used for sizing the flow control facility as required in Volume I.
- If roof runoff is ***dispersed*** using a dispersion trench designed according to the requirements of this section on single-family lots greater than 22,000 square feet, and the *vegetative flow*[*] path of the roof runoff is 50 feet or larger, the roof area may be modeled as grassed surface

This chapter also provides a description of the use of infiltration facilities for flow control. Additional design considerations and general limitations of the infiltration facilities and small site BMPs are covered in Volume V.

Roof downspout controls and small site BMPs should be applied to individual commercial lot developments when the percent impervious area and pollutant characteristics are comparable to those from residential lots.

3.1 Roof Downspout Controls

This section presents the criteria for design and implementation of roof downspout controls. *Roof downspout controls* are simple pre-engineered designs for infiltrating and/or dispersing runoff from roof areas for the purposes of increasing opportunities for groundwater recharge and reduction of runoff volumes from new developments.

Selection of Roof Downspout Controls

Large lots in rural areas (5 acres or greater) typically have enough area to disperse or infiltrate roof runoff. Lots created in urban areas will typically be smaller (about 8,000 square feet) and have a limited amount of area in which to site infiltration or dispersion trenches.

[*] *Vegetative flow* path is measured from the downspout or dispersion system discharge point to the downstream property line, stream, wetland, or other impervious surface.

Downspout infiltration should be used in those soils that readily infiltrate (coarse sands and cobbles to medium sands). Dispersion BMPs should be used for urban lots located in less permeable soils, where if infiltration is not feasible. Where dispersion is not feasible because of very small lot size, or where there is a potential for creating drainage problems on adjacent lots, downspouts should be connected to the street storm drain system, which directs the runoff to a regional facility.

Where roof downspout controls are planned, the following three types must be considered in descending order of preference:

- Downspout infiltration systems (Section 3.1.1)
- Downspout dispersion systems (Section 3.1.2)
- Downspout perforated stub-out connections (Section 3.1.3)

Figure 3.1 illustrates, in general, how roof downspout controls are selected and applied in single-family subdivision projects. However, local jurisdictions may adopt approaches that are more specific to their locality. Where supported by appropriate soil infiltration tests, downspout infiltration in finer soils may be practical using a larger infiltration system.

Note: Other innovative downspout control BMPs such as rain barrels, ornamental ponds, downspout cisterns, or other downspout water storage devices may also be used if approved by the reviewing authority.

Roof Downspout Controls in Potential Landslide Hazard Areas

If or where local governments have identified "geologically hazardous areas" (WAC 365-195-410), we recommend that lots immediately adjacent to the hazard area collect roof runoff in a tightline system which conveys the runoff to the base of the slope.

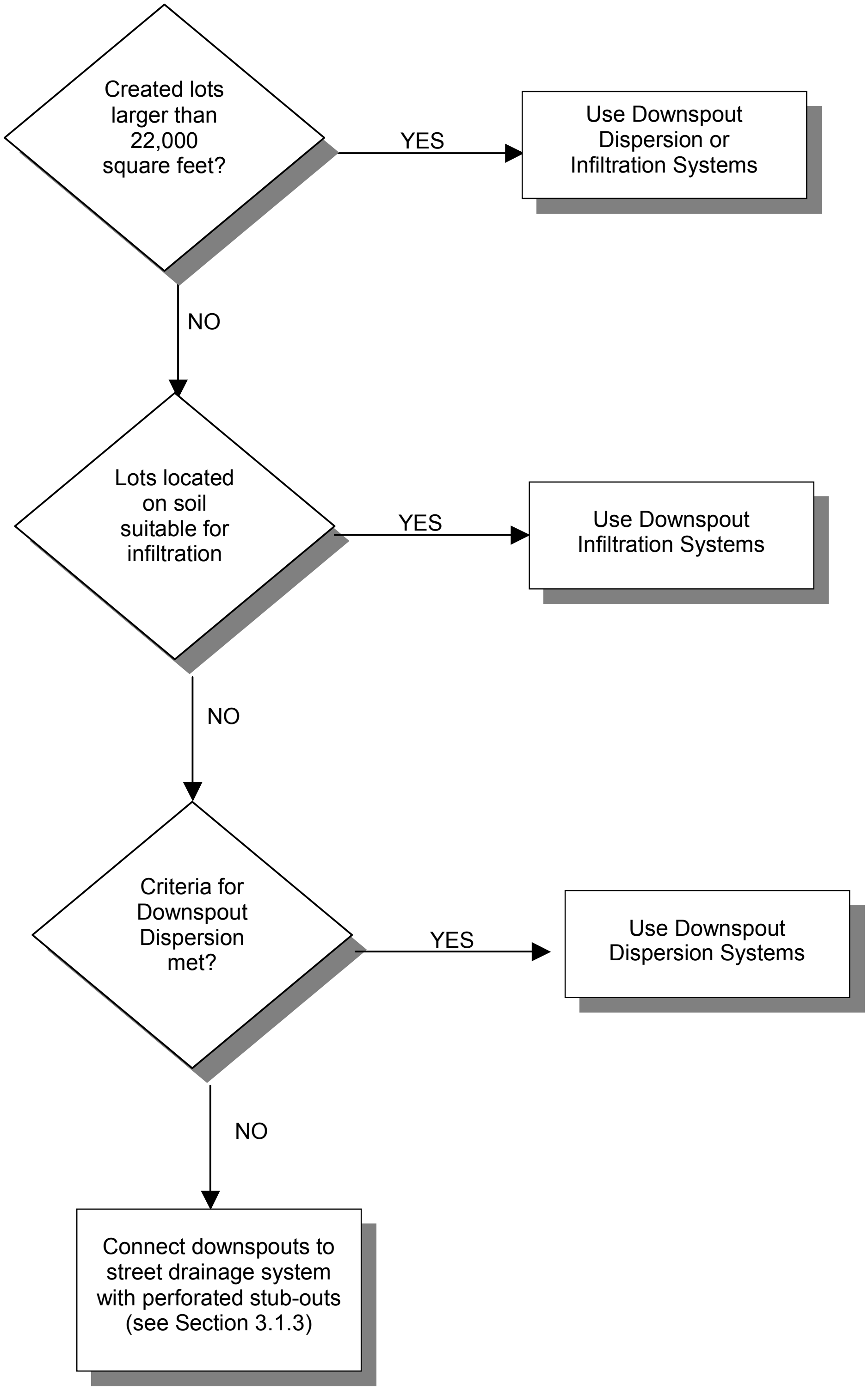


Figure 3.1 Flow Diagram Showing Selection of Roof Downspout Controls

3.1.1 Downspout Infiltration Systems

Downspout infiltration systems are trench or drywell designs intended only for use in infiltrating runoff from roof downspout drains. They are not designed to directly infiltrate runoff from pollutant-generating impervious surfaces.

Application

The following apply to parcels as described in Volume I:

1. Single family subdivision projects subject to Minimum Requirement #7 for flow control (Volume I) must provide for individual downspout infiltration systems on all lots smaller than 22,000 square feet if feasible. Local governments may specify a different lot size that is more appropriate - based on local soil and slope conditions and rainfall. Concentrated flows may not be directed to adjoining lots. They must be dispersed and retained on the building lot to the maximum extent possible.
2. The feasibility or applicability of downspout infiltration must be evaluated for all subdivision single-family lots smaller than 22,000 square feet. The evaluation procedure detailed below must be used to determine if downspout infiltration is feasible or whether downspout dispersion can be used in lieu of infiltration.
3. For subdivision single-family lots greater than or equal to 22,000 square feet, downspout infiltration is optional, and the evaluation procedure detailed below may be used if downspout infiltration is being proposed voluntarily.
4. If site-specific tests indicate less than 3 feet of permeable soil from the proposed final grade to the seasonal high groundwater table, then a downspout dispersion system per Section 3.1.2 may be used in lieu of infiltration.
5. On lots or sites with more than 3 feet of permeable soil from the proposed final grade to the seasonal high groundwater table, downspout infiltration is considered feasible if the soils are outwash type soils and the infiltration trench can be designed to meet the minimum design criteria specified below.

Note: If downspout infiltration is not provided on these lots, then a downspout dispersion system must be provided per Section 3.1.2.

Flow Credit for Roof Downspout Infiltration

If roof runoff is infiltrated according to the requirements of this section, the roof area may be discounted from the project area used for sizing the flow control facility as required in Volume I, Minimum Requirement #7.

Procedure for Evaluating Feasibility

1. A soils report must be prepared by a locally licensed onsite sewage designer or by other suitably trained persons working under the supervision of a professional engineer registered in the State of Washington to determine if soils suitable for infiltration are present on the site. The report must reference a sufficient number of soils logs to establish the type and limits of soils on the project site. The report should at a minimum identify the limits of any *outwash type soils* (i.e., those meeting USDA soil texture classes ranging from coarse sand and cobbles to medium sand) versus other soil types and include an inventory of topsoil depth.
2. On lots or sites with no outwash type soils, a downspout dispersion system per Section 3.1.2 may be used in lieu of infiltration.
3. On lots or sites containing outwash type soils (coarse sand and cobbles to medium sand), additional site-specific testing must be done. Individual lot or site tests must consist of at least one soils log at the location of the infiltration system, a minimum of 4 feet in depth (from proposed grade), identifying the SCS series of the soil and the USDA textural class of the soil horizon through the depth of the log, and noting any evidence of high groundwater level, such as mottling.

   Note: This testing must also be carried out on lots or sites where downspout infiltration is being proposed in soils other than outwash.
4. If site-specific tests indicate less than 3 feet of permeable soil from the proposed final grade to the seasonal high groundwater table, then a downspout dispersion system per Section 3.1.2 may be used in lieu of infiltration.
5. On lots or sites with more than 3 feet of permeable soil from the proposed final grade to the seasonal high groundwater table, downspout infiltration is considered feasible if the soils are outwash type soils and the infiltration trench can be designed to meet the minimum design criteria specified below.

Design Criteria for Infiltration Trenches

Figure 3.2 shows a typical downspout infiltration trench system, and Figure 3.3 presents an alternative infiltration trench system for sites with coarse sand and cobble soils. These systems are designed as specified below.

General

1. The following minimum lengths (linear feet) per 1,000 square feet of roof area based on soil type may be used for sizing downspout infiltration trenches.

Coarse sands and cobbles	20 LF
Medium sand	30 LF
Fine sand, loamy sand	75 LF

Sandy loam	125 LF
Loam	190 LF

2. Maximum length of trench must not exceed 100 feet from the inlet sump.
3. Minimum spacing between trench centerlines must be 6 feet.
4. Filter fabric must be placed over the drain rock as shown on Figure 3.2 prior to backfilling.
5. Infiltration trenches may be placed in fill material if the fill is placed and compacted under the direct supervision of a geotechnical engineer or professional civil engineer with geotechnical expertise, and if the measured infiltration rate is at least 8 inches per hour. Trench length in fill must be 60 linear feet per 1,000 square feet of roof area. Infiltration rates can be tested using the methods described in Section 3.3.
6. Infiltration trenches should not be built on slopes steeper than 25 percent (4:1). A geotechnical analysis and report may be required on slopes over 15 percent or if located within 200 feet of the top of steep slope or landslide hazard area.
7. Trenches may be located under pavement if a small yard drain or catch basin with grate cover is placed at the end of the trench pipe such that overflow would occur out of the catch basin at an elevation at least one foot below that of the pavement, and in a location which can accommodate the overflow without creating a significant adverse impact to downhill properties or drainage systems. This is intended to prevent saturation of the pavement in the event of system failure.

Design Criteria for Infiltration Drywells

Figure 3.4 shows a typical downspout infiltration drywell system. These systems are designed as specified below.

General

1. Drywell bottoms must be a minimum of 1 foot above seasonal high groundwater level or impermeable soil layers.
2. If using drywells, each drywell may serve up to 1000 square feet of impervious surface for either medium sands or coarse sands.
3. Typically drywells are 48 inches in diameter (minimum) and have a depth of 5 feet (4 feet of gravel and 1 foot of suitable cover material).
4. Filter fabric (geotextile) must be placed on top of the drain rock and on trench or drywell sides prior to backfilling.
5. Spacing between drywells must be a minimum of 4 feet.
6. Downspout infiltration drywells must not be built on slopes greater than 25% (4:1). Drywells may not be placed on or above a landslide hazard area or slopes greater than 15% without evaluation by a

professional engineer with geotechnical expertise or qualified geologist and jurisdiction approval.

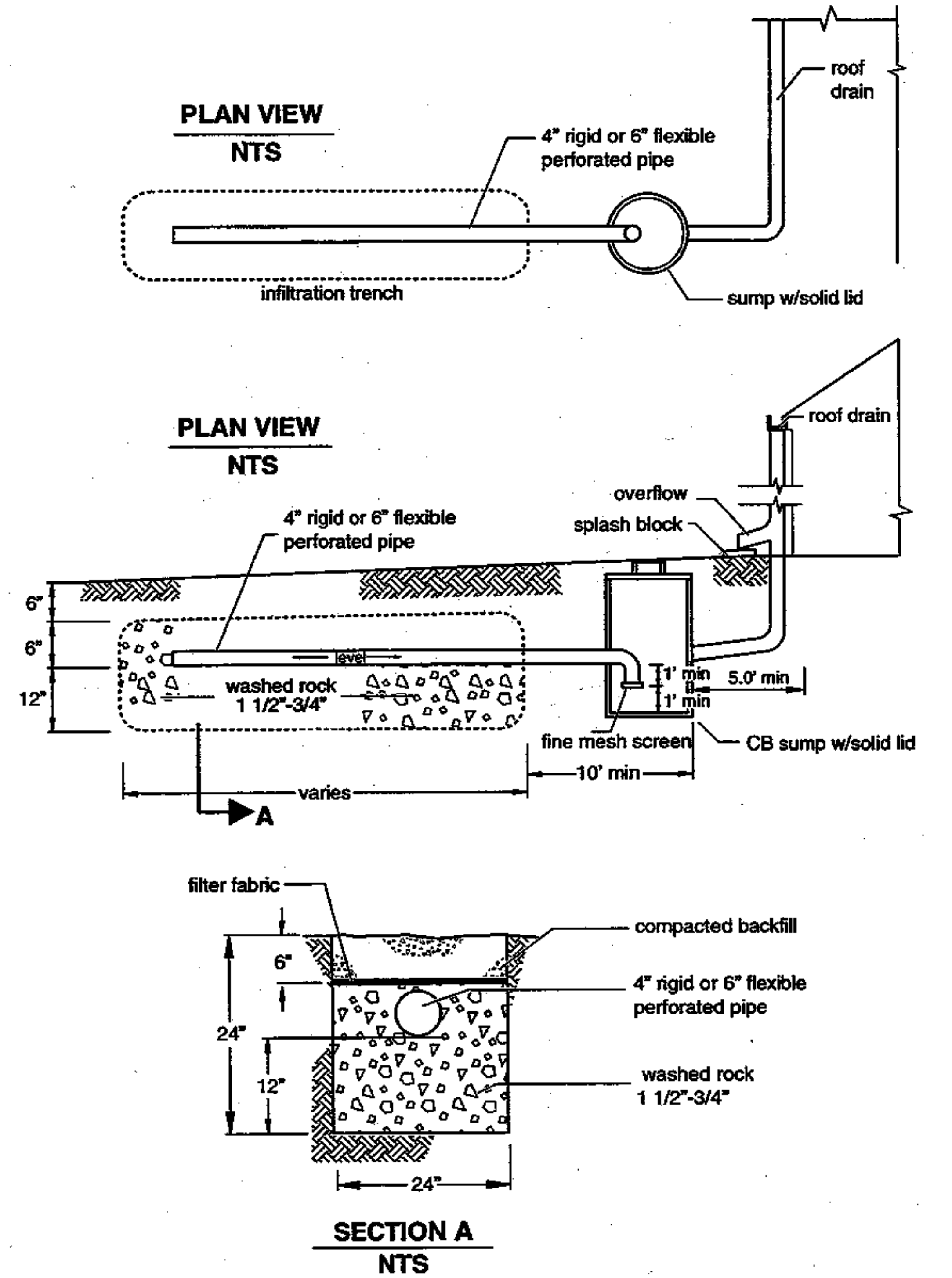


Figure 3.2 Typical Downspout Infiltration Trench

Source: King County

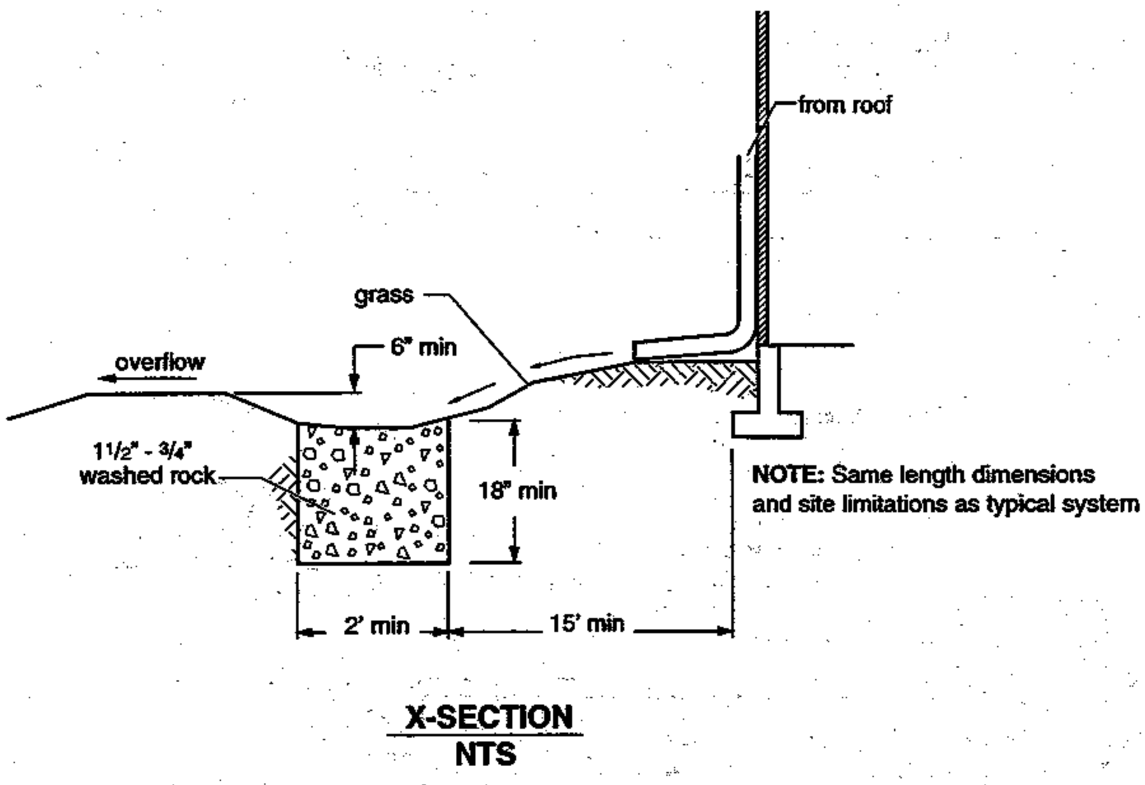


Figure 3.3 Alternative Downspout Infiltration Trench System For Coarse Sand And Gravel

Source: King County

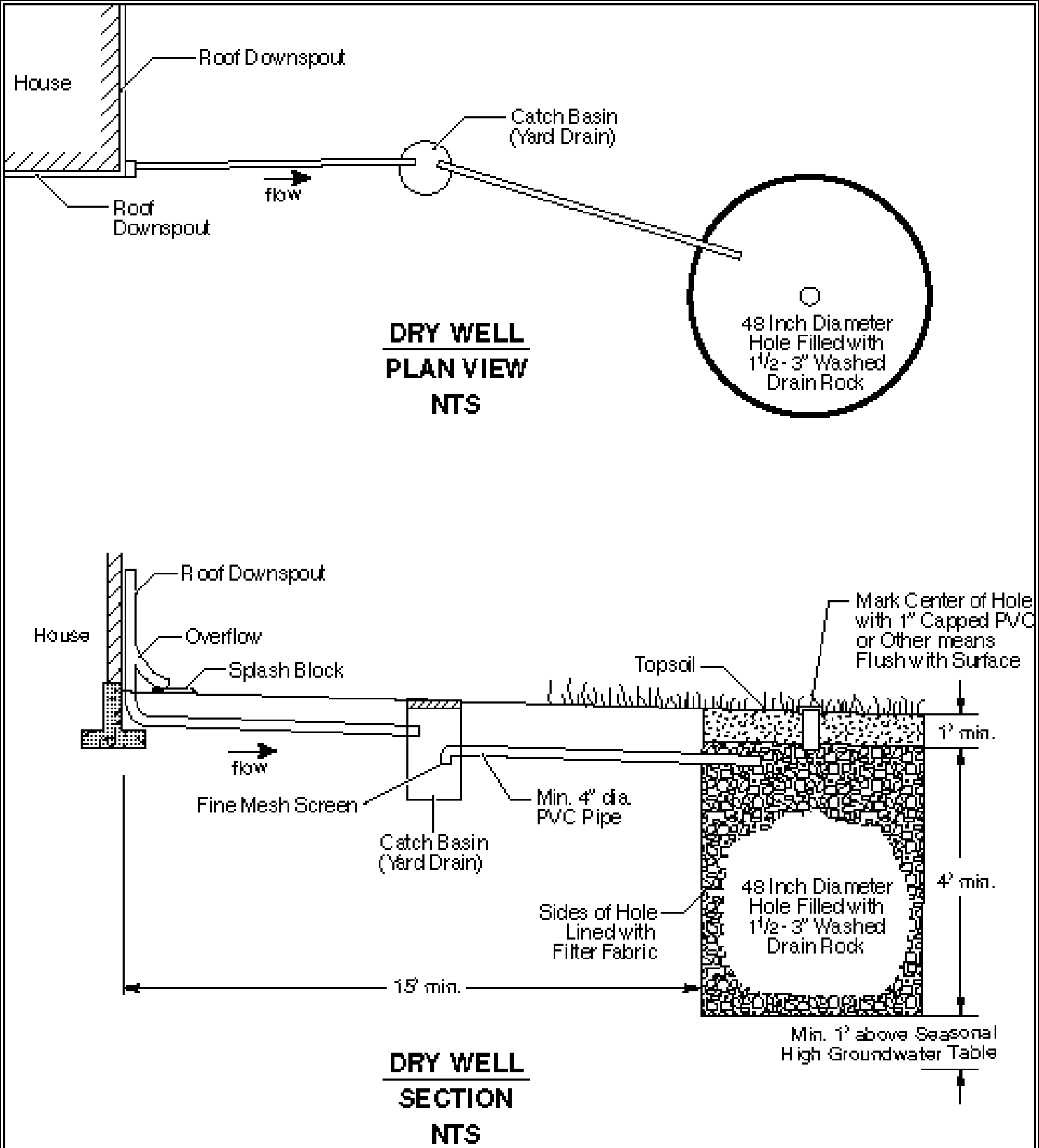


Figure 3.4 Typical Downspout Infiltration Drywell

Setbacks

Local governments may require specific setbacks in sites with steep slopes, land slide areas, open water features, springs, wells, and septic tank drain fields. Adequate room for maintenance access and equipment should also be considered. Examples of setbacks commonly used include the following:

1. All infiltration systems should be at least 10 feet from any structure, property line, or sensitive area (except steep slopes).
2. All infiltration systems must be at least 50 feet from the top of any sensitive area steep slope. This setback may be reduced to 15 feet based on a geotechnical evaluation, but in no instances may it be less than the buffer width.
3. For sites with septic systems, infiltration systems must be downgradient of the drainfield unless the site topography clearly prohibits subsurface flows from intersecting the drainfield.

3.1.2 Downspout Dispersion Systems

Downspout dispersion systems are splash blocks or gravel-filled trenches, which serve to spread roof runoff over vegetated pervious areas. Dispersion attenuates peak flows by slowing entry of the runoff into the conveyance system, allows for some infiltration, and provides some water quality benefits.

Application

Downspout dispersion must be used in all subdivision single-family lots, which meet one of the following criteria:

1. Lots greater than or equal to 22,000 square feet where downspout infiltration is not being provided according to the requirements in Section 3.1.1.
2. Lots smaller than 22,000 square feet where soils are not suitable for downspout infiltration (as determined in Section 3.1.1) and where the design criteria below can be met.

Flow Credit for Roof Downspout Dispersion

If roof runoff is dispersed using a dispersion trench designed according to the requirements of this section on single-family lots greater than 22,000 square feet, and the *vegetative flow*[*] path of the roof runoff is 50 feet or larger, the roof area may be modeled as grassed surface - rather than impervious surface - when sizing the flow control facility as required in Volume I, Minimum Requirement #7.

Design Criteria

1. Downspout trenches designed as shown in Figure 3.5 should be used for all downspout dispersion applications except where splash blocks are allowed below.

[*] *Vegetative flow* path is measured from the downspout or dispersion system discharge point to the downstream property line, stream, wetland, or other impervious surface.

2. Splash blocks shown in Figure 3.7 may be used for downspouts discharging to a *vegetated flowpath* at least 50 feet in length as measured from the downspout to the downstream property line, structure, steep slope, stream, wetland, or other impervious surface. Sensitive area buffers may count toward flowpath lengths.

3. If the vegetated flowpath (measured as defined above) is less than 25 feet on a subdivision single family lot, a perforated stub-out connection per Section 3.1.3 may be used in lieu of downspout dispersion. A perforated stub-out may also be used where implementation of downspout dispersion might cause erosion or flooding problems, either on site or on adjacent lots. This provision might be appropriate, for example, for lots constructed on steep hills where downspout discharge could be cumulative and might pose a potential hazard for lower lying lots, or where dispersed flows could create problems for adjacent offsite lots. Perforated stub-outs are not appropriate when seasonal water table is <1 foot below trench bottom.

4. For sites with septic systems, the discharge point of all dispersion systems must be downgradient of the drainfield. This requirement may be waived if site topography clearly prohibits flows from intersecting the drainfield.

Design Criteria for Dispersion Trenches

1. A vegetated flowpath of at least 25 feet in length must be maintained between the outlet of the trench and any property line, structure, stream, wetland, or impervious surface. A vegetated flowpath of at least 50 feet in length must be maintained between the outlet of the trench and any steep slope. Sensitive area buffers may count towards flowpath lengths.

2. Trenches serving up to 700 square feet of roof area may be simple 10-foot-long by 2-foot wide gravel filled trenches as shown in Figure 3.5. For roof areas larger than 700 square feet, a dispersion trench with notched grade board as shown in Figure 3.6 may be used as approved by the local jurisdiction. The total length of this design must not exceed 50 feet and must provide at least 10 feet of trench per 700 square feet of roof area.

3. A setback of at least 5 feet should be maintained between any edge of the trench and any structure or property line.

4. No erosion or flooding of downstream properties may result.

5. Runoff discharged towards landslide hazard areas must be evaluated by a geotechnical engineer or qualified geologist. The discharge point may not be placed on or above slopes greater than 20% or above erosion hazard areas without evaluation by a geotechnical engineer or qualified geologist and jurisdiction approval.

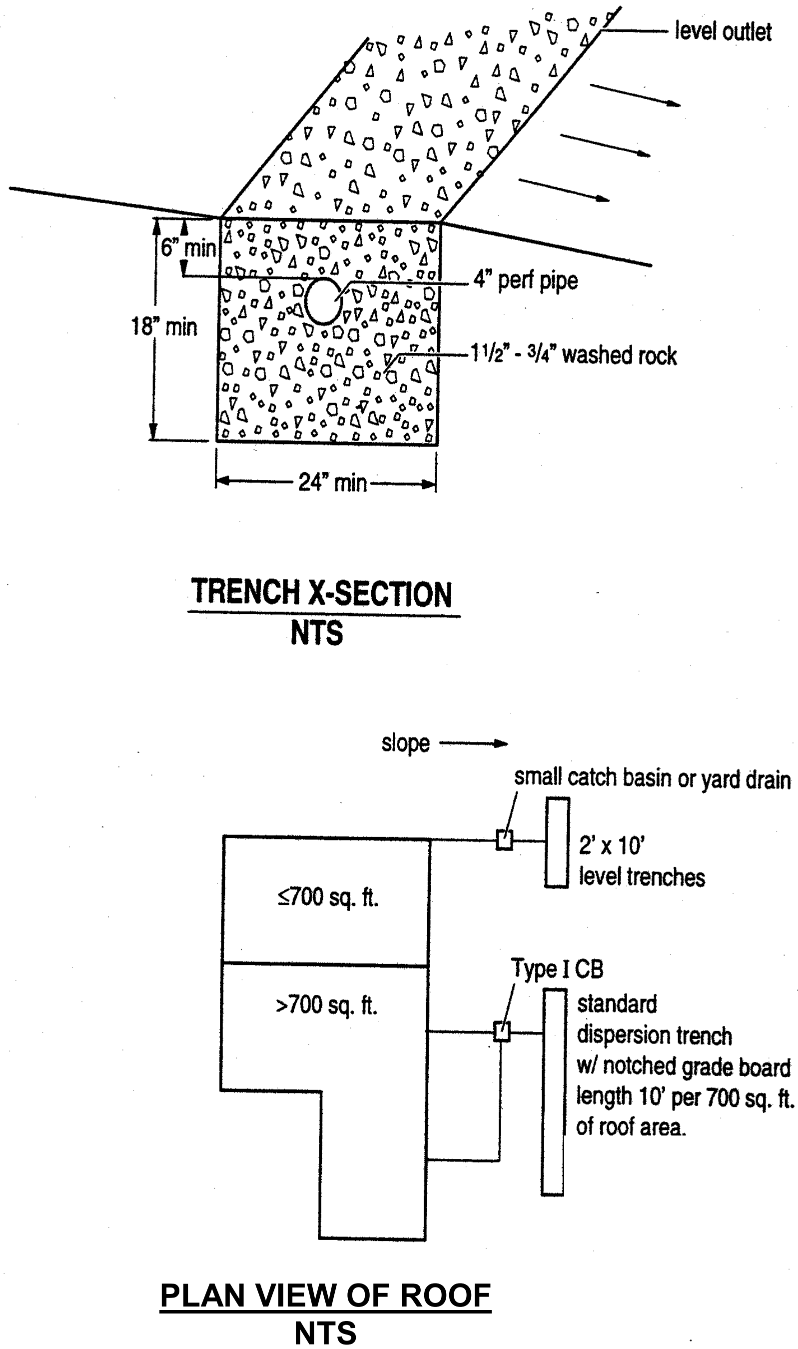
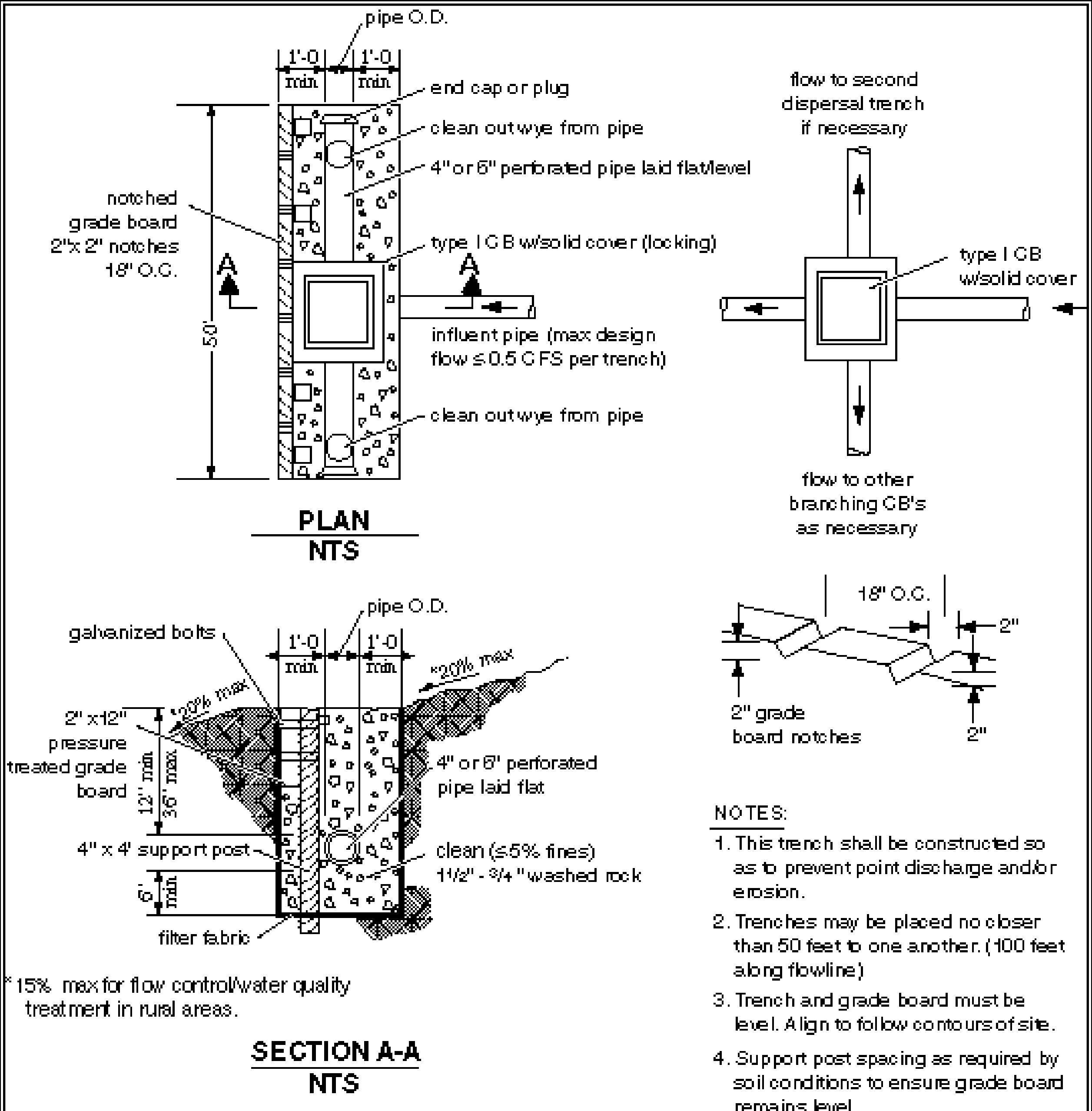


Figure 3.5 Typical Downspout Dispersion Trench

Source: King County



NOTES:

1. This trench shall be constructed so as to prevent point discharge and/or erosion.
2. Trenches may be placed no closer than 50 feet to one another. (100 feet along flowline)
3. Trench and grade board must be level. Align to follow contours of site.
4. Support post spacing as required by soil conditions to ensure grade board remains level.

Figure 3.6 Standard Dispersion Trench with Notched Grade Board

Design Criteria for Splashblocks

A typical downspout splashblock is shown in Figure 3.7. In general, if the ground is sloped away from the foundation and there is adequate vegetation and area for effective dispersion, splashblocks will adequately disperse storm runoff. If the ground is fairly level, if the structure includes a basement, or if foundation drains are proposed, splashblocks with downspout extensions may be a better choice because the discharge point is moved away from the foundation. Downspout extensions can include piping to a splashblock/discharge point a considerable distance from the downspout, as long as the runoff can travel through a well-vegetated area as described below.

The following apply to the use of splashblocks:

1. A vegetated flowpath of at least 50 feet should be maintained between the discharge point and any property line, structure, steep slope, stream, wetland, lake, or other impervious surface. Sensitive area buffers may count toward flowpath lengths.
2. A maximum of 700 square feet of roof area may drain to each splashblock.
3. A splashblock or a pad of crushed rock (2 feet wide by 3 feet long by 6 inches deep) should be placed at each downspout discharge point.
4. No erosion or flooding of downstream properties may result.
5. Runoff discharged towards landslide hazard areas must be evaluated by a professional engineer with geotechnical expertise or a qualified geologist. Splashblocks may not be placed on or above slopes greater than 20% or above erosion hazard areas without evaluation by a professional engineer with geotechnical expertise or qualified geologist and jurisdiction approval.
6. For sites with septic systems, the discharge point must be downslope of the primary and reserve drainfield areas. This requirement may be waived if site topography clearly prohibits flows from intersecting the drainfield or where site conditions (soil permeability, distance between systems, etc) indicate that this is unnecessary.

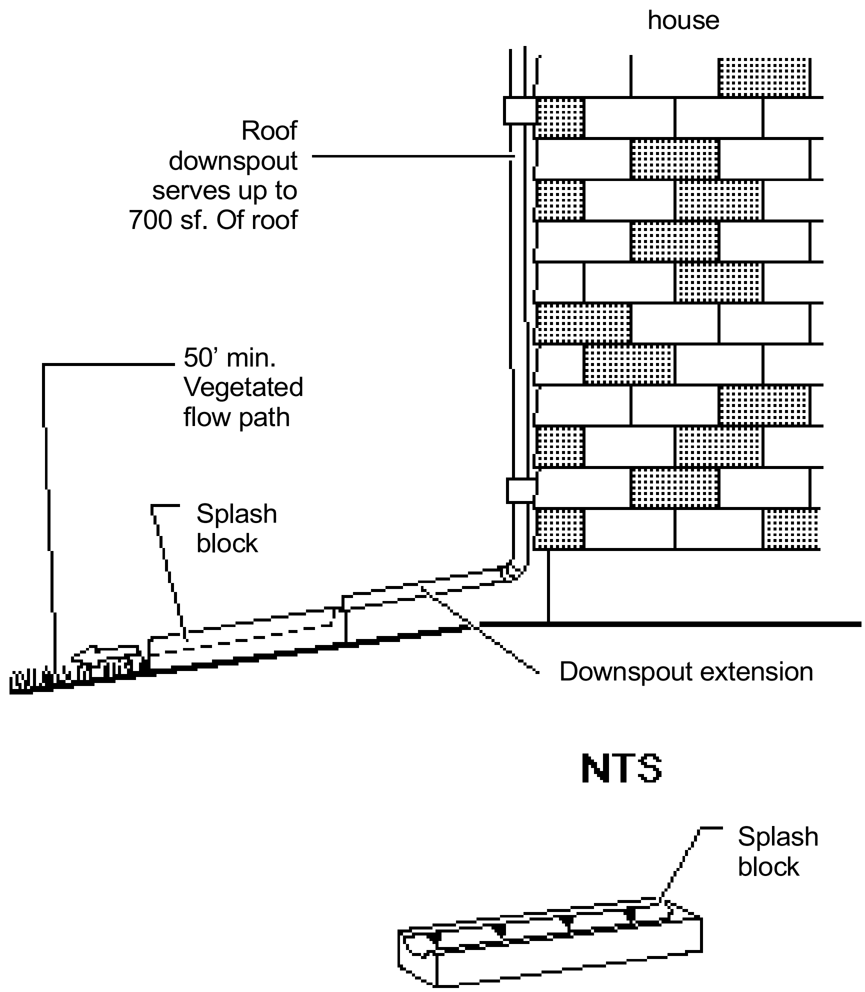


Figure 3.7 Typical Downspout Splashblock Dispersion

3.1.3 Perforated Stub-Out Connections

A perforated stub-out connection is a length of perforated pipe within a gravel-filled trench that is placed between roof downspouts and a stub-out to the local drainage system. Figure 3.8 illustrates a perforated stub-out connection. These systems are intended to provide some infiltration during drier months. During the wet winter months, they may provide little or no flow control. Perforated stub-outs are not appropriate when seasonal water table is < 1 foot below trench bottom.

In single-family subdivision projects subject to Minimum Requirement #7 for flow control (see Volume I), perforated stub-out connections may be used only when downspout infiltration or dispersion is not feasible per the criteria in Sections 3.1.1 and 3.1.2.

Location of the connection should be selected to allow a maximum amount of runoff to infiltrate into the ground (ideally a dry location on the site that is relatively well drained). To facilitate maintenance, the perforated pipe portion of the system should not be located under impervious or heavily compacted (e.g., driveways and parking areas) surfaces.

Perforated stub-out connections should consist of at least 10 feet of perforated pipe per 5,000 square feet of roof area laid in a level, 2-foot wide trench backfilled with washed drain rock. The drain rock should extend to a depth of at least 8 inches below the bottom of the pipe and should cover the pipe. The pipe should be laid level and the rock trench covered with filter fabric and 6 inches of fill (see Figure 3.8).

Setbacks are the same as for infiltration trenches.

Potential runoff discharge towards a landslide hazard area must be evaluated by a professional engineer with geotechnical expertise or a qualified geologist**.** The perforated portion of the pipe may not be placed on or above slopes greater than 20% or above erosion hazard areas without evaluation by a professional engineer with geotechnical expertise or qualified geologist and jurisdiction approval.

For sites with septic systems, the perforated portion of the pipe must be downgradient of the drainfield primary and reserve areas. This requirement can be waived if site topography will clearly prohibit flows from intersecting the drainfield or where site conditions (soil permeability, distance between systems, etc) indicate that this is unnecessary.

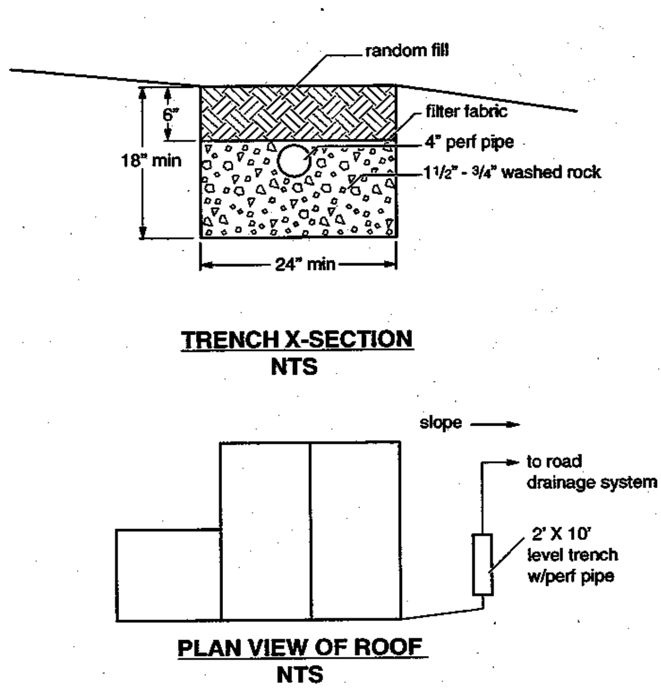


Figure 3.8 Perforated Stub-Out Connection

Source: King County

3.2 Detention Facilities

This section presents the methods, criteria, and details for design and analysis of detention facilities. These facilities provide for the temporary storage of increased surface water runoff resulting from development pursuant to the performance standards set forth in Minimum Requirement #7 for flow control (Volume I).

There are three primary types of detention facilities described in this section: detention ponds, tanks, and vaults.

3.2.1 Detention Ponds

The design criteria in this section are for detention ponds. However, many of the criteria also apply to infiltration ponds (Section 3.3 and Volume V), and water quality wetponds and combined detention/wetponds (Volume V).

Dam Safety for Detention BMPs

Stormwater detention facilities that can impound 10 acre-feet (435,600 cubic feet; 3.26 million gallons) or more with the water level <u>at the embankment crest</u> are subject to the state's dam safety requirements, even if water storage is intermittent and infrequent (WAC 173-175-020(1)). The principal safety concern is for the downstream population at risk if the dam should breach and allow an uncontrolled release of the pond contents. Peak flows from dam failures are typically much larger than the 100-year flows which these ponds are typically designed to accommodate.

The Dam Safety Office of the Department of Ecology uses consequence dependent design levels for critical project elements. There are eight design levels with storm recurrence intervals ranging from 1 in 500 for design step, 1 to 1 in 1,000,000 for design step 8. The specific design step for a particular project depends on the downstream population and other resources that would be at risk from a failure of the dam. Precipitation events more extreme than the 100-year event may be rare at any one location, but have historically occurred somewhere within Washington State every few years on average.

With regard to the engineering design of stormwater detention facilities, the primary effect of the state's dam safety requirements is in sizing the emergency spillway to accommodate the runoff from the dam safety design storm without overtopping the dam. The hydrologic computation procedures are the same as for the original pond design, except that the computations must use more extreme precipitation values and the appropriate dam safety design storm hyetographs. This information is described in detail within guidance documents developed by and available from the Dam Safety Office. In addition to the other design requirements for stormwater detention BMPs described elsewhere in this manual, dam safety requirements should be an integral part of planning and design for stormwater detention ponds. It is most cost-effective to consider these requirements right from the beginning of the project.

In addition to the hydrologic and hydraulic issues related to precipitation and runoff, other dam safety requirements include geotechnical issues, construction inspection and documentation, dam breach analysis, inundation mapping, emergency action planning, and periodic inspections by project owners and by Dam Safety engineers. All of these requirements, plus procedural requirements for plan review and approval and payment of construction permit fees are described in detail in guidance documents developed by and available from the Dam Safety Office.

In addition to the written guidance documents, Dam Safety engineers are available to provide technical assistance to project owners and design engineers in understanding and addressing the dam safety requirements for their specific project. In the interest of providing a smooth integration of dam safety requirements into the stormwater detention project and streamlining Dam Safety's engineering review and issuance of the construction permit, it is recommended and requested that Dam Safety be contacted early in the facilities planning process. The Dam Safety Office is located in the Ecology headquarters building in Lacey. Electronic versions of the guidance documents in PDF format are available on the Department of Ecology Web site at http://www.ecy.wa.gov/programs/wr/dams/dss.html.

Design Criteria

Standard details for detention ponds are shown in Figure 3.9 through Figure 3.11. Control structure details are provided in Section 3.2.4.

General

1. Ponds must be designed as flow-through systems (however, parking lot storage may be utilized through a back-up system; see Section 3.2.5). Developed flows must enter through a conveyance system separate from the control structure and outflow conveyance system. Maximizing distance between the inlet and outlet is encouraged to promote sedimentation.
2. Pond bottoms should be level and be located a minimum of 0.5 foot (preferably 1 foot) below the inlet and outlet to provide sediment storage.
3. Design guidelines for outflow control structures are specified in Section 3.2.4.
4. A geotechnical analysis and report must be prepared for steep slopes (i.e., slopes over 15%), or if located within 200 feet of the top of a steep slope or landslide hazard area. The scope of the geotechnical report should include the assessment of impoundment seepage on the stability of the natural slope where the facility will be located within the setback limits set forth in this section.

Side Slopes

1. Interior side slopes up to the emergency overflow water surface should not be steeper than 3H:1V unless a fence is provided (see "Fencing").
2. Exterior side slopes must not be steeper than 2H:1V unless analyzed for stability by a geotechnical engineer.
3. Pond walls may be vertical retaining walls, provided: (a) they are constructed of reinforced concrete per Section 3.2.3, Material; (b) a fence is provided along the top of the wall; (c) the entire pond perimeter may be retaining walls, however, it is recommended that at least 25 percent of the pond perimeter be a vegetated soil slope not steeper than 3H:1V; and (d) the design is stamped by a licensed civil engineer with structural expertise. Other retaining walls such as rockeries, concrete,

masonry unit walls, and keystone type wall may be used if designed by a geotechnical engineer or a civil engineer with structural expertise. If the entire pond perimeter is to be retaining walls, ladders should be provided on the walls for safety reasons.

Embankments

1. Pond berm embankments higher than 6 feet must be designed by a professional engineer with geotechnical expertise.
2. For berm embankments 6 feet or less, the minimum top width should be 6 feet or as recommended by a geotechnical engineer.
3. Pond berm embankments must be constructed on native consolidated soil (or adequately compacted and stable fill soils analyzed by a geotechnical engineer) free of loose surface soil materials, roots, and other organic debris.
4. Pond berm embankments greater than 4 feet in height must be constructed by excavating a key equal to 50 percent of the berm embankment cross-sectional height and width unless specified otherwise by a geotechnical engineer.
5. Embankment compaction should be accomplished in such a manner as to produce a dense, low permeability engineered fill that can tolerate post-construction settlements with a minimum of cracking. The embankment fill should be placed on a stable subgrade and compacted to a minimum of 95% of the Standard Proctor Maximum Density, ASTM Procedure D698. Placement moisture content should lie within 1% dry to 3% wet of the optimum moisture content. The referenced compaction standard may have to be increased to comply with local regulations.

   The berm embankment should be constructed of soils with the following characteristics per the United States Department of Agriculture's Textural Triangle: a minimum of 20% silt and clay, a maximum of 60% sand, a maximum of 60% silt, with nominal gravel and cobble content. Soils outside this specified range can be used, provided the design satisfactorily addresses the engineering concerns posed by these soils. The paramount concerns with these soils are their susceptibility to internal erosion or piping and to surface erosion from wave action and runoff on the upstream and downstream slopes, respectively. *Note: In general, excavated glacial till is well suited for berm embankment material*.
6. Anti-seepage filter-drain diaphragms must be placed on outflow pipes in berm embankments impounding water with depths greater than 8 feet at the design water surface. See Dam Safety Guidelines, Part IV, Section 3.3.B on pages 3-27 to 3-30. An electronic version of the Dam Safety Guidelines is available in PDF format at www.ecy.wa.gov/programs/wr/dams/dss.html.

Overflow

1. In all ponds, tanks, and vaults, a primary overflow (usually a riser pipe within the control structure; see Section 3.2.4) must be provided to bypass the 100-year developed peak flow over or around the restrictor system. This assumes the facility will be full due to plugged orifices or high inflows; the primary overflow is intended to protect against breaching of a pond embankment (or overflows of the upstream conveyance system in the case of a detention tank or vault). The design must provide controlled discharge directly into the downstream conveyance system or another acceptable discharge point.
2. A secondary inlet to the control structure must be provided in ponds as additional protection against overtopping should the inlet pipe to the control structure become plugged. A grated opening ("jailhouse window") in the control structure manhole functions as a weir (see Figure 3.10) when used as a secondary inlet.

   Note: The maximum circumferential length of this opening must not exceed one-half the control structure circumference. The "birdcage" overflow structure as shown in Figure 3.11 may also be used as a secondary inlet.

Emergency Overflow Spillway

1. In addition to the above overflow provisions, ponds must have an emergency overflow spillway. For impoundments of 10 acre-feet or greater, the emergency overflow spillway must meet the state's dam safety requirements (see above). For impoundments under 10 acre-feet, ponds must have an emergency overflow spillway that is sized to pass the 100-year developed peak flow in the event of total control structure failure (e.g., blockage of the control structure outlet pipe) or extreme inflows. Emergency overflow spillways are intended to control the location of pond overtopping and direct overflows back into the downstream conveyance system or other acceptable discharge point.

2. Emergency overflow spillways must be provided for ponds with constructed berms over 2 feet in height, or for ponds located on grades in excess of 5 percent. As an option for ponds with berms less than 2 feet in height and located at grades less than 5 percent, emergency overflow may be provided by an emergency overflow structure, such as a Type II manhole fitted with a birdcage as shown in Figure 3.11. The emergency overflow structure must be designed to pass the 100-year developed peak flow, with a minimum 6 inches of freeboard, directly to the downstream conveyance system or another acceptable discharge point. Where an emergency overflow spillway would discharge to a steep slope, consideration should be given to providing an emergency overflow structure *in addition to* the spillway.

3. The emergency overflow spillway must be armored with riprap in conformance with the "Outlet Protection" BMP in Volume II. The spillway must be armored full width, beginning at a point midway across the berm embankment and extending downstream to where emergency overflows re-enter the conveyance system (see Figure 3.10).

4. Emergency overflow spillway designs must be analyzed as broad-crested trapezoidal weirs as described in Methods of Analysis at the end of this section (Section 3.2.1). Either one of the weir sections shown in Figure 3.10 may be used.

Access

The following guidelines for access may be used.

1. Maintenance access road(s) should be provided to the control structure and other drainage structures associated with the pond (e.g., inlet or bypass structures). It is recommended that manhole and catch basin lids be in or at the edge of the access road and at least three feet from a property line.
2. An access ramp is needed for removal of sediment with a trackhoe and truck. The ramp must extend to the pond bottom if the pond bottom is greater than 1,500 square feet (measured without the ramp) and it may end at an elevation 4 feet above the pond bottom, if the pond bottom is less than 1,500 square feet (measured without the ramp).

   On large, deep ponds, truck access to the pond bottom via an access ramp is necessary so loading can be done in the pond bottom. On small deep ponds, the truck can remain on the ramp for loading. On small shallow ponds, a ramp to the bottom may not be required if the trackhoe can load a truck parked at the pond edge or on the internal berm of a wetpond or combined pond (trackhoes can negotiate interior pond side slopes).
3. The internal berm of a wetpond or combined detention and wetpond may be used for access if it is no more than 4 feet above the first wetpool cell, if the first wetpool cell is less than 1,500 square feet (measured without the ramp), and if it is designed to support a loaded truck, considering the berm is normally submerged and saturated.
4. Access ramps must meet the requirements for design and construction of access roads specified below.
5. If a fence is required, access should be limited by a double-posted gate or by bollards – that is, two fixed bollards on each side of the access road and two removable bollards equally located between the fixed bollards.

Design of Access Roads

The design guidelines for access road are given below.

1. Maximum grade should be 15 percent.
2. Outside turning radius should be a minimum of 40 feet.
3. Fence gates should be located only on straight sections of road.
4. Access roads should be 15 feet in width on curves and 12 feet on straight sections.
5. A paved apron must be provided where access roads connect to paved public roadways.

Construction of Access Roads

Access roads may be constructed with an asphalt or gravel surface, or modular grid pavement. All surfaces must conform to the jurisdictional standards and manufacturer's specifications.

Fencing

1. A fence is needed at the emergency overflow water surface elevation, or higher, where a pond interior side slope is steeper than 3H:1V, or where the impoundment is a wall greater than 24 inches in height. The fence need only be constructed for those slopes steeper than 3H:1V. Note, however, that other regulations such as the Uniform Building Code may require fencing of vertical walls. If more than 10 percent of slopes are steeper 3H:1V, it is recommended that the entire pond be fenced.

   Also note that detention ponds on school sites will need to comply with safety standards developed by the Department of Health (DOH) and the Superintendent for Public Instruction (SPI). These standards include what is called a 'non-climbable fence.' One example of a non-climbable fence is a chain-link fence with a tighter mesh, so children cannot get a foot-hold for climbing. For school sites, and possibly for parks and playgrounds, the designer should consult the DOH's Office of Environmental Programs.

   A fence is needed to discourage access to portions of a pond where steep side slopes (steeper than 3:1) increase the potential for slipping into the pond. Fences also serve to guide those who have fallen into a pond to side slopes that are flat enough (flatter than 3:1 and unfenced) to allow for easy escape.

2. It is recommended that fences be 6 feet in height. For example designs, see WSDOT Standard Plan L-2, Type 1 or Type 3 chain link fence. The fence may be a minimum of 4 feet in height if the depth of the impoundment (measured from the lowest elevation in the bottom of the

impoundment, directly adjacent to the bottom of the fenced slope, up to the emergency overflow water surface) is 5 feet or less. For example designs, see WSDOT Standard Plan L-2, Type 4 or Type 6 chain link fence.

3. Access road gates may be 16 feet in width consisting of two swinging sections 8 feet in width. Additional vehicular access gates may be needed to facilitate maintenance access.
4. Pedestrian access gates (if needed) should be 4 feet in width.
5. Vertical metal balusters or 9 gauge galvanized steel fabric with bonded vinyl coating can be used as fence material. For steel fabric fences, the following aesthetic features may be considered:
   a) Vinyl coating that is compatible with the surrounding environment (e.g., green in open, grassy areas and black or brown in wooded areas). All posts, cross bars, and gates may be painted or coated the same color as the vinyl clad fence fabric.
   b) Fence posts and rails that conform to WSDOT Standard Plan L-2 for Types 1, 3, or 4 chain link fence.
6. For metal baluster fences, Uniform Building Code standards apply.
7. Wood fences may be used in subdivisions where the fence will be maintained by homeowners associations or adjacent lot owners.
8. Wood fences should have pressure treated posts (ground contact rated) either set in 24-inch deep concrete footings or attached to footings by galvanized brackets. Rails and fence boards may be cedar, pressure-treated fir, or hemlock.
9. Where only short stretches of the pond perimeter (< 10 percent) have side slopes steeper than 3:1, split rail fences (3-foot minimum height) or densely planted thorned hedges (e.g., barberry, holly, etc.) may be used in place of a standard fence.

Signage

Detention ponds, infiltration ponds, wetponds, and combined ponds should have a sign placed for maximum visibility from adjacent streets, sidewalks, and paths. An example of sign specifications for a permanent surface water control pond is illustrated in Figure 3.12.

Right-of-Way

Right-of-way may be needed for detention pond maintenance. It is recommended that any tract not abutting public right-of-way have 15-20 foot wide extension of the tract to an acceptable access location.

Setbacks

It is recommended that facilities be a minimum of 20 feet from any structure, property line, and any vegetative buffer required by the local government. The detention pond water surface at the pond outlet invert elevation must be set back 100 feet from proposed or existing septic system drainfields. However, the setback requirements are generally specified by the local government, uniform building code, or other statewide regulation and may be different from those mentioned above.

All facilities must be a minimum of 50 feet from the top of any steep (greater than 15 percent) slope. A geotechnical analysis and report must be prepared addressing the potential impact of the facility on a steep slope.

Seeps and Springs

Intermittent seeps along cut slopes are typically fed by a shallow groundwater source (interflow) flowing along a relatively impermeable soil stratum. These flows are storm driven and should discontinue after a few weeks of dry weather. However, more continuous seeps and springs, which extend through longer dry periods, are likely from a deeper groundwater source. When continuous flows are intercepted and directed through flow control facilities, adjustments to the facility design may have to be made to account for the additional base flow (unless already considered in design).

Planting Requirements

Exposed earth on the pond bottom and interior side slopes should be sodded or seeded with an appropriate seed mixture. All remaining areas of the tract should be planted with grass or be landscaped and mulched with a 4-inch cover of hog fuel or shredded wood mulch. Shredded wood mulch is made from shredded tree trimmings, usually from trees cleared on site. The mulch should be free of garbage and weeds and should not contain excessive resin, tannin, or other material detrimental to plant growth.

Landscaping

Landscaping is encouraged for most stormwater tract areas (see below for areas not to be landscaped). However, if provided, landscaping should adhere to the criteria that follow so as not to hinder maintenance operations. Landscaped stormwater tracts may, in some instances, provide a recreational space. In other instances, "naturalistic" stormwater facilities may be placed in open space tracts.

The following guidelines should be followed if landscaping is proposed for facilities.

1. No trees or shrubs may be planted within 10 feet of inlet or outlet pipes or manmade drainage structures such as spillways or flow spreaders. Species with roots that seek water, such as willow or poplar, should be avoided within 50 feet of pipes or manmade structures.

2. Planting should be restricted on berms that impound water either permanently or temporarily during storms. This restriction does not apply to cut slopes that form pond banks, only to berms.

   a) Trees or shrubs may not be planted on portions of water-impounding berms taller than four feet high. Only grasses may be planted on berms taller than four feet.

      Grasses allow unobstructed visibility of berm slopes for detecting potential dam safety problems such as animal burrows, slumping, or fractures in the berm.

   b) Trees planted on portions of water-impounding berms less than 4 feet high must be small, not higher than 20 feet mature height, and have a fibrous root system. Table 3.1 gives some examples of trees with these characteristics developed for the central Puget Sound.

      These trees reduce the likelihood of blow-down trees, or the possibility of channeling or piping of water through the root system, which may contribute to dam failure on berms that retain water.

Note: The internal berm in a wetpond is not subject to this planting restriction since the failure of an internal berm would be unlikely to create a safety problem.

3. All landscape material, including grass, should be planted in good topsoil. Native underlying soils may be made suitable for planting if amended with 4 inches of well-aged compost tilled into the subgrade. Compost used should meet specifications for Grade A compost quality as described in Ecology publication 94-38.

4. Soil in which trees or shrubs are planted may need additional enrichment or additional compost top-dressing. Consult a nurseryman, landscape professional, or arborist for site-specific recommendations.

5. For a naturalistic effect as well as ease of maintenance, trees or shrubs should be planted in clumps to form *"landscape islands"* rather than evenly spaced.

6. The landscaped islands should be a minimum of six feet apart, and if set back from fences or other barriers, the setback distance should also be a minimum of 6 feet. Where tree foliage extends low to the ground, the six feet setback should be counted from the outer drip line of the trees (estimated at maturity).

   This setback allows a 6-foot wide mower to pass around and between clumps.

7. Evergreen trees and trees which produce relatively little leaf-fall (such as Oregon ash, mimosa, or locust) are preferred in areas draining to the pond.

8. Trees should be set back so that branches do not extend over the pond (to prevent leaf-drop into the water).
9. Drought tolerant species are recommended.

Table 3.1
Small Trees and Shrubs with Fibrous Roots

Small Trees / High Shrubs	**Low Shrubs**
*Red twig dogwood *(Cornus stolonifera)*	*Snowberry *(Symporicarpus albus)*
*Serviceberry *(Amelanchier alnifolia)*	*Salmonberry *(Rubus spectabilis)*
*Filbert (*Corylus cornuta*, others)	Rosa rugosa (avoid spreading varieties)
Highbush cranberry *(Vaccinium opulus)*	Rock rose *(Cistus spp.)*
Blueberry *(Vaccinium spp.)*	Ceanothus spp. choose hardier varieties)
Fruit trees on dwarf rootstock	New Zealand flax *(Phormium penax)*
Rhododendron (native and ornamental varieties)	Ornamental grasses (e.g., *Miscanthis, Pennisetum*)
*Native species	

Guidelines for Naturalistic Planting. Stormwater facilities may sometimes be located within open space tracts if "natural appearing." Two generic kinds of naturalistic planting are outlined below, but other options are also possible. Native vegetation is preferred in naturalistic plantings.

Open Woodland. In addition to the general landscaping guidelines above, the following are recommended.

1. Landscaped islands (when mature) should cover a minimum of 30 percent or more of the tract, exclusive of the pond area.
2. Tree clumps should be underplanted with shade-tolerant shrubs and groundcover plants. The goal is to provide a dense understory that need not be weeded or mowed.
3. Landscaped islands should be placed at several elevations rather than "ring" the pond, and the size of clumps should vary from small to large to create variety.
4. Not all islands need to have trees. Shrub or groundcover clumps are acceptable, but lack of shade should be considered in selecting vegetation.

Note: Landscaped islands are best combined with the use of wood-based mulch (hog fuel) or chipped onsite vegetation for erosion control (only for slopes above the flow control water surface). *It is often difficult to sustain a low-maintenance understory if the site was previously hydroseeded. Compost or composted mulch (typically used for constructed wetland soil)* *can be used below the flow control water surface (materials that are resistant to and preclude flotation). The method of construction of soil landscape systems can also cause natural selection of specific plant species. Consult a soil restoration or wetland soil scientist for site-specific recommendations.*

Northwest Savannah or Meadow. In addition to the general landscape guidelines above, the following are recommended.

1. Landscape islands (when mature) should cover 10 percent or more of the site, exclusive of the pond area.
2. Planting groundcovers and understory shrubs is encouraged to eliminate the need for mowing under the trees when they are young.
3. Landscape islands should be placed at several elevations rather than "ring" the pond.

The remaining site area should be planted with an appropriate grass seed mix, which may include meadow or wildflower species. Native or dwarf grass mixes are preferred. Table 3.2 below gives an example of dwarf grass mix developed for central Puget Sound. Grass seed should be applied at 2.5 to 3 pounds per 1,000 square feet.

Note: Amended soil or good topsoil is required for all plantings.

Creation of areas of emergent vegetation in shallow areas of the pond is recommended. Native wetland plants, such as sedges (Carex sp.), bulrush (*Scirpus sp.*), water plantain (*Alisma sp.*), and burreed (*Sparganium sp.*) are recommended. If the pond does not hold standing water, a clump of wet-tolerant, non-invasive shrubs, such as salmonberry or snowberry, is recommended below the detention design water surface.

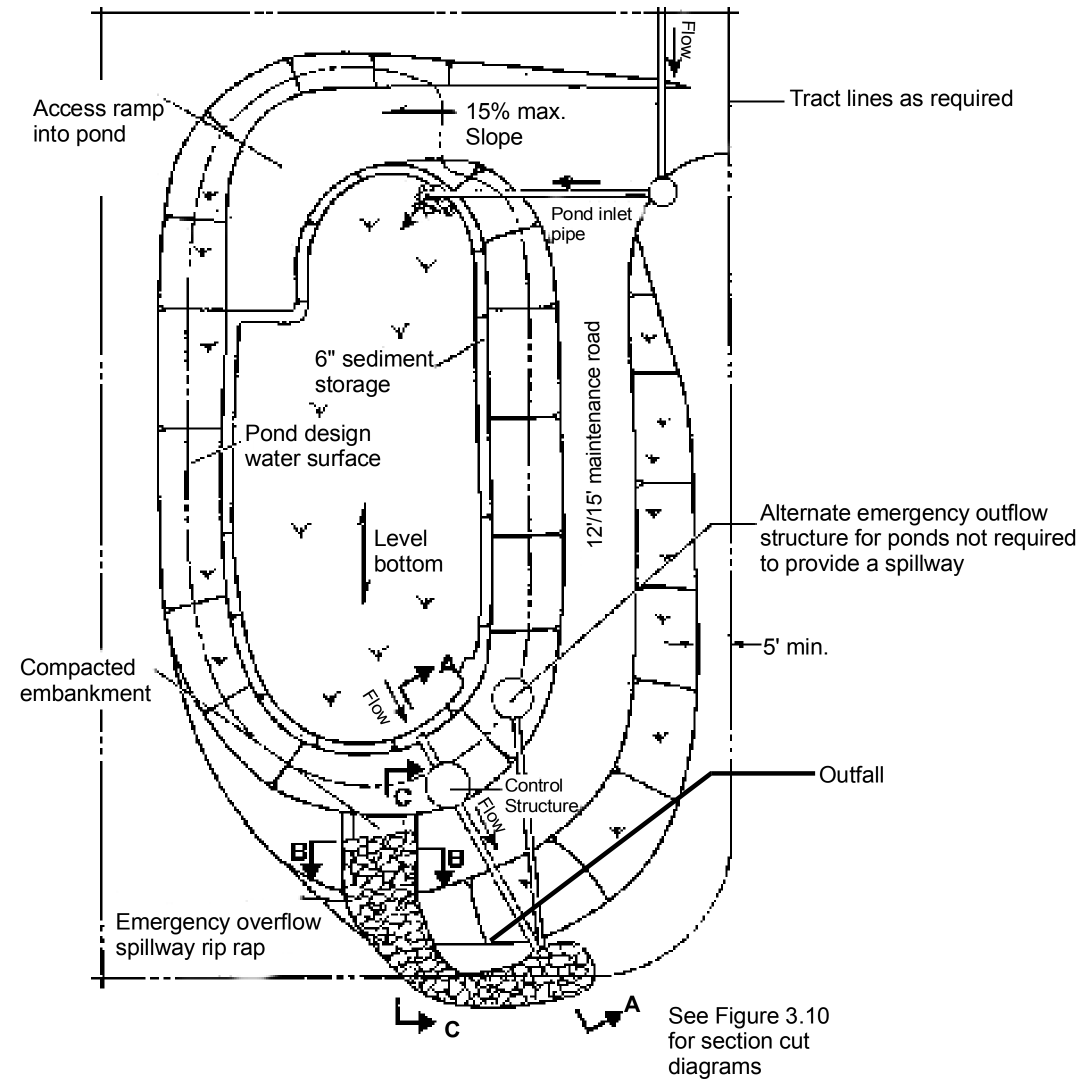
Note: This landscape style is best combined with the use of grass or sod for site stabilization and erosion control.

Seed Mixes. The seed mixes listed below were developed for central Puget Sound.

Table 3.2
Stormwater Tract "Low Grow" Seed Mix

Seed Name	Percentage of Mix
Dwarf tall fescue	40%
Dwarf perennial rye "Barclay"*	30%
Red fescue	25%
Colonial bentgrass	5%

* If wildflowers are used and sowing is done before Labor Day, the amount of dwarf perennial rye can be reduced proportionately to the amount of wildflower seed used.



Note:
This detail is a schematic representation only. Actual configuration will vary depending on specific site constraints and applicable design criteria.

Figure 3.9 Typical Detention Pond

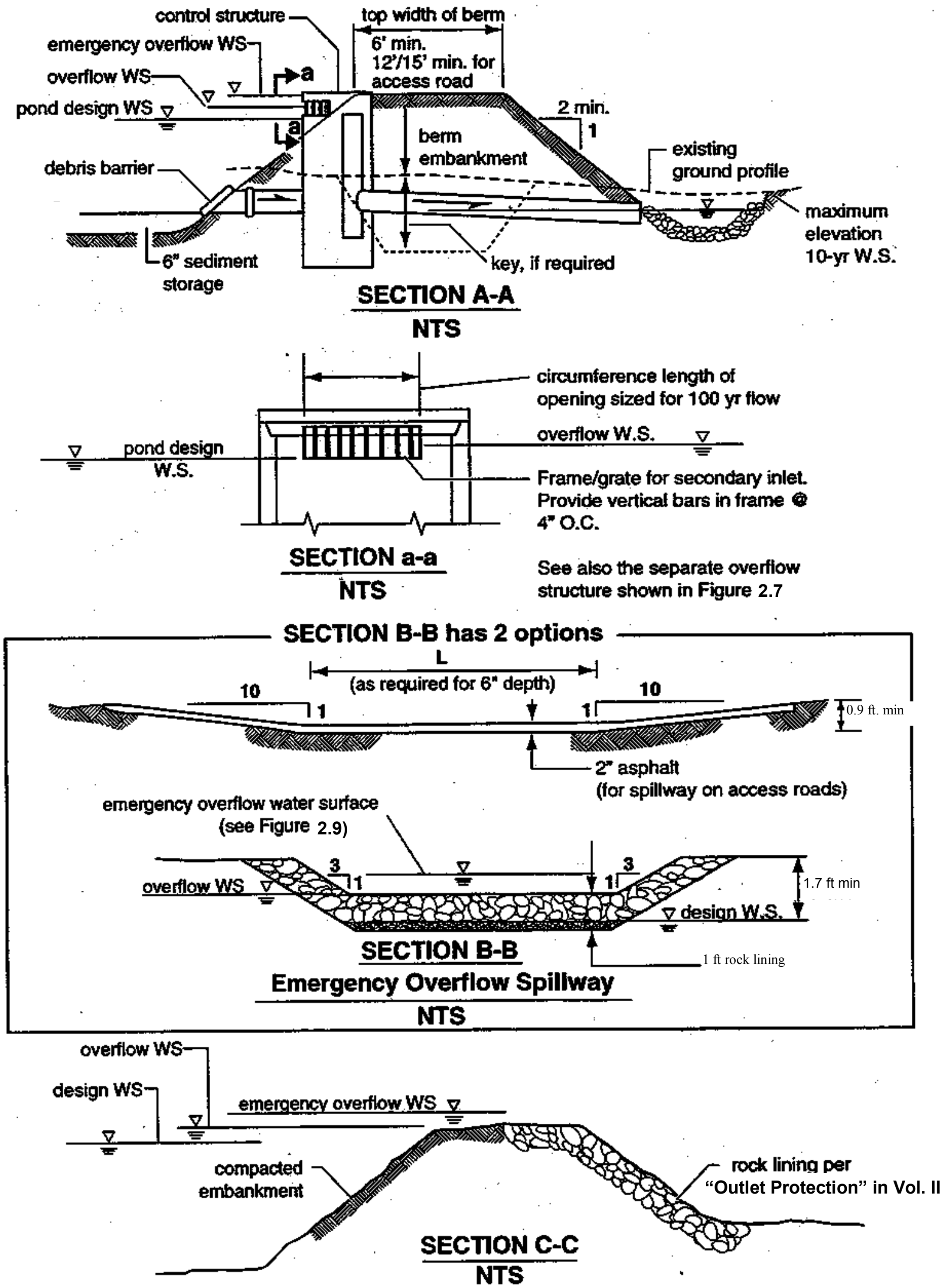
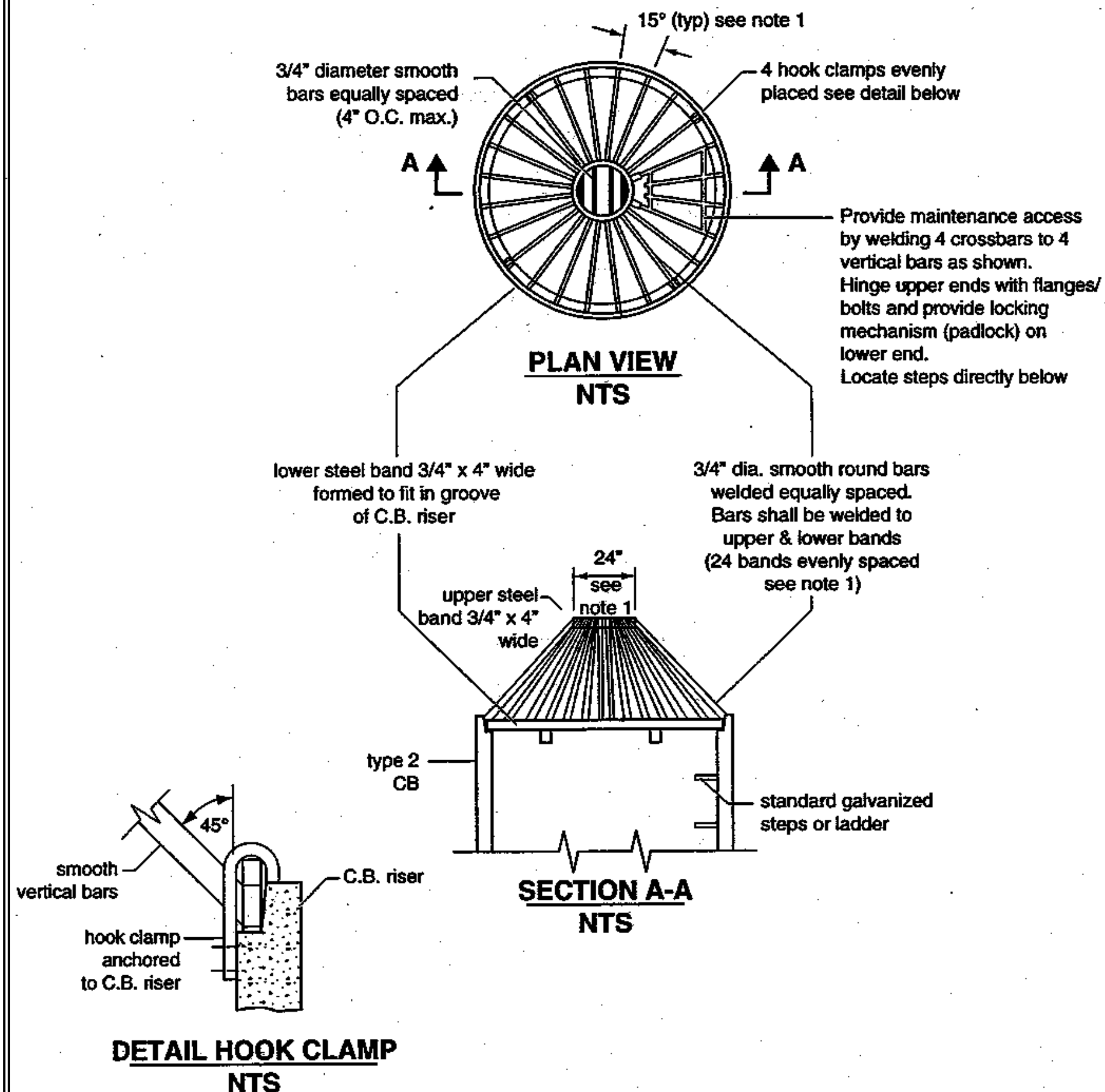


Figure 3.10 Typical Detention Pond Sections



NOTES:

1. Dimensions are for illustration on 54" diameter CB For different diameter CB's adjust to maintain 45° angle on "vertical" bars and 7" o.c. maximum spacing of bars around lower steel band.
2. Metal parts must be corrosion resistant; steel bars must be galvanized.
3. This debris barrier is also recommended for use on the inlet to roadway cross-culverts with high potential for debris collection (except on type 2 streams)

Figure 3.11 Overflow Structure

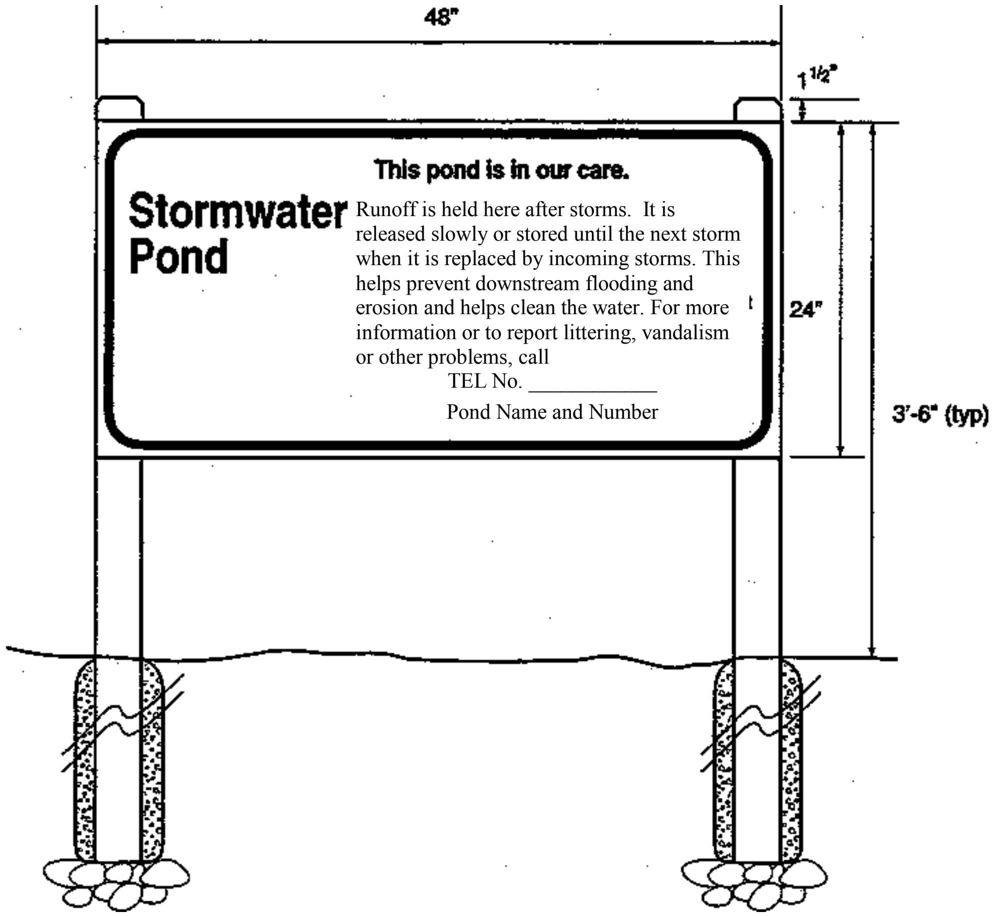


Figure 3.12 Example of Permanent Surface Water Control Pond Sign

Sample Specifications:

Size:	48 inches by 24 inches
Material:	0.125-gauge aluminum
Face:	Non-reflective vinyl or 3 coats outdoor enamel (sprayed).
Lettering:	Silk screen enamel where possible, or vinyl letters.
Colors:	Beige background, teal letters.
Type face:	Helvetica condensed. Title: 3 inch; Sub-Title: 1½ inch; Text: 1 inch; Outer
border:	1/8 inch border distance from edge: 1/4 inch; all text 1¾ inch from border.
Posts:	Pressure treated, beveled tops, 1½ inch higher than sign.
Installation:	Secure to chain link fence if available. Otherwise install on two 4"x4" posts, pressure treated, mounted atop gravel bed, installed in 30-inch concrete filled post holes (8-inch minimum diameter). Top of sign no higher than 42 inches from ground surface.
Placement:	Face sign in direction of primary visual or physical access. Do not block any access road. Do not place within 6 feet of structural facilities (e.g. manholes, spillways, pipe inlets).
Special Notes:	This facility is lined to protect groundwater (if a liner that restricts infiltration of stormwater exists).

Maintenance

General. Maintenance is of primary importance if detention ponds are to continue to function as originally designed. A local government, a designated group such as a homeowners' association, or some individual must accept the responsibility for maintaining the structures and the impoundment area. A specific maintenance plan must be formulated outlining the schedule and scope of maintenance operations. Debris removal in detention basins can be achieved through the use of trash racks or other screening devices.

Design with maintenance in mind. Good maintenance will be crucial to successful use of the impoundment. Hence, provisions to facilitate maintenance operations must be built into the project when it is installed. Maintenance must be a basic consideration in design and in determination of first cost. See Table 3.3 for specific maintenance requirements.

Any standing water removed during the maintenance operation must be disposed of to a sanitary sewer at an approved discharge location *Pretreatment may be necessary.* Residuals must be disposed in accordance with state and local solid waste regulations (See Minimum Functional Standards For Solid Waste Handling, Chapter 173-304 WAC).

Vegetation. If a shallow marsh is established, then periodic removal of dead vegetation may be necessary. Since decomposing vegetation can release pollutants captured in the wet pond, especially nutrients, it may be necessary to harvest dead vegetation annually prior to the winter wet season. Otherwise the decaying vegetation can export pollutants out of the pond and also can cause nuisance conditions to occur. If harvesting is to be done in the wetland, a written harvesting procedure should be prepared by a wetland scientist and submitted with the drainage design to the local government.

Sediment. Maintenance of sediment forebays and attention to sediment accumulation within the pond is extremely important. Sediment deposition should be continually monitored in the basin. Owners, operators, and maintenance authorities should be aware that significant concentrations of metals (e.g., lead, zinc, and cadmium) as well as some organics such as pesticides, may be expected to accumulate at the bottom of these treatment facilities. Testing of sediment, especially near points of inflow, should be conducted regularly to determine the leaching potential and level of accumulation of potentially hazardous material before disposal.

Table 3.3 Specific Maintenance Requirements for Detention Ponds			
Maintenance Component	**Defect**	**Conditions When Maintenance Is Needed**	**Results Expected When Maintenance Is Performed**
General	Trash & Debris	Any trash and debris which exceed 5 cubic feet per 1,000 square feet (this is about equal to the amount of trash it would take to fill up one standard size garbage can). In general, there should be no visual evidence of dumping. If less than threshold all trash and debris will be removed as part of next scheduled maintenance.	Trash and debris cleared from site.
	Poisonous Vegetation and noxious weeds	Any poisonous or nuisance vegetation which may constitute a hazard to maintenance personnel or the public. Any evidence of noxious weeds as defined by State or local regulations. (Apply requirements of adopted Integrated Pest Management (IPM) policies for the use of herbicides).	No danger of poisonous vegetation where maintenance personnel or the public might normally be. (Coordinate with local health department) Complete eradication of noxious weeds may not be possible. Compliance with State or local eradication policies required
	Contaminants and Pollution	Any evidence of oil, gasoline, contaminants or other pollutants (Coordinate removal/cleanup with local water quality response agency).	No contaminants or pollutants present.
	Rodent Holes	Any evidence of rodent holes if facility is acting as a dam or berm, or any evidence of water piping through dam or berm via rodent holes.	Rodents destroyed and dam or berm repaired. (Coordinate with local health department and Ecology Dam Safety Office if pone exceeds 10 acre feet)
	Beaver Dams	Dam results in change or function of the facility.	Facility is returned to design function. (Coordinate trapping of beavers and removal of dams with appropriate permitting agencies)
	Insects	When insects such as wasps and hornets interfere with maintenance activities.	Insects destroyed or removed from site. Apply insecticides in compliance with adopted IPM policies
	Tree Growth and Hazard Trees	Tree growth does not allow maintenance access or interferes with maintenance activity (i.e., slope mowing, silt removal, vactoring, or equipment movements). If trees are not interfering with access or maintenance, do not remove If dead, diseased, or dying trees are identified (Use a certified Arborist to determine health of tree or removal requirements)	Trees do not hinder maintenance activities. Harvested trees should be recycled into mulch or other beneficial uses (e.g., alders for firewood). Remove hazard trees

Table 3.3 Specific Maintenance Requirements for Detention Ponds			
Maintenance Component	**Defect**	**Conditions When Maintenance Is Needed**	**Results Expected When Maintenance Is Performed**
Side Slopes of Pond	Erosion	Eroded damage over 2 inches deep where cause of damage is still present or where there is potential for continued erosion. Any erosion observed on a compacted berm embankment.	Slopes should be stabilized using appropriate erosion control measure(s); e.g., rock reinforcement, planting of grass, compaction. If erosion is occurring on compacted berms a licensed civil engineer should be consulted to resolve source of erosion.
Storage Area	Sediment	Accumulated sediment that exceeds 10% of the designed pond depth unless otherwise specified or affects inletting or outletting condition of the facility.	Sediment cleaned out to designed pond shape and depth; pond reseeded if necessary to control erosion.
	Liner (If Applicable)	Liner is visible and has more than three 1/4-inch holes in it.	Liner repaired or replaced. Liner is fully covered.
Pond Berms (Dikes)	Settlements	Any part of berm which has settled 4 inches lower than the design elevation. If settlement is apparent measure berm to determine amount of settlement. Settling can be an indication of more severe problems with the berm or outlet works. A licensed civil engineer should be consulted to determine the source of the settlement.	Dike is built back to the design elevation.
	Piping	Discernable water flow through pond berm. Ongoing erosion with potential for erosion to continue. (Recommend a Goethechnical engineer be called in to inspect and evaluate condition and recommend repair of condition.	Piping eliminated. Erosion potential resolved.
Emergency Overflow/Spillway and Berms over 4 feet in height.	Tree Growth	Tree growth on emergency spillways create blockage problems and may cause failure of the berm due to uncontrolled overtopping. Tree growth on berms over 4 feet in height may lead to piping through the berm which could lead to failure of the berm.	Trees should be removed. If root system is small (base less than 4 inches) the root system may be left in place. Otherwise the roots should be removed and the berm restored. A licensed civil engineer should be consulted for proper berm/spillway restoration.
	Piping	Discernable water flow through pond berm. Ongoing erosion with potential for erosion to continue. (Recommend a Goethechnical engineer be called in to inspect and evaluate condition and recommend repair of condition.	Piping eliminated. Erosion potential resolved.
Emergency Overflow/Spillway	Emergency Overflow/ Spillway	Only one layer of rock exists above native soil in area five square feet or larger, or any exposure of native soil at the top of out flow path of spillway. (Rip-rap on inside slopes need not be replaced.)	Rocks and pad depth are restored to design standards.
	Erosion	See "Side slopes of Pond"	

Methods of Analysis

Detention Volume and Outflow. The volume and outflow design for detention ponds must be in accordance with Minimum Requirements #7 in Volume I and the hydrologic analysis and design methods in Chapter 1 of this Volume. Design guidelines for restrictor orifice structures are given in Section 3.2.4.

Note: The design water surface elevation is the highest elevation which occurs in order to meet the required outflow performance for the pond.

Detention Ponds in Infiltrative Soils. Detention ponds may occasionally be sited on till soils that are sufficiently permeable for a properly functioning infiltration system (see Section 3.3). These detention ponds have a surface discharge and may also utilize infiltration as a second pond outflow. Detention ponds sized with infiltration as a second outflow must meet all the requirements of Section 3.3 for infiltration ponds, including a soils report, testing, groundwater protection, pre-settling, and construction techniques.

Emergency Overflow Spillway Capacity. For impoundments under 10-acre-feet, the emergency overflow spillway weir section must be designed to pass the 100-year runoff event for developed conditions assuming a broad-crested weir. The **broad-crested weir equation** for the spillway section in Figure 3.13, for example, would be:

$$Q_{100} = C\,(2g)^{1/2}\left[\frac{2}{3}LH^{3/2} + \frac{8}{15}\,(\text{Tan}\,\theta)\,H^{5/2}\right] \qquad \text{(equation 1)}$$

Where Q_{100} = peak flow for the 100-year runoff event (cfs)

- C = discharge coefficient (0.6)
- g = gravity (32.2 ft/sec^2)
- L = length of weir (ft)
- H = height of water over weir (ft)
- θ = angle of side slopes

Assuming C = 0.6 and Tan θ = 3 (for 3:1 slopes), the equation becomes:

$$Q_{100} = 3.21[LH^{3/2} + 2.4\,H^{5/2}] \qquad \text{(equation 2)}$$

To find width L for the weir section, the equation is rearranged to use the computed Q_{100} and trial values of H (0.2 feet minimum):

$$L = [Q_{100}/(3.21H^{3/2})] - 2.4\,H \quad \text{or} \quad 6 \text{ feet minimum} \qquad \text{(equation 3)}$$

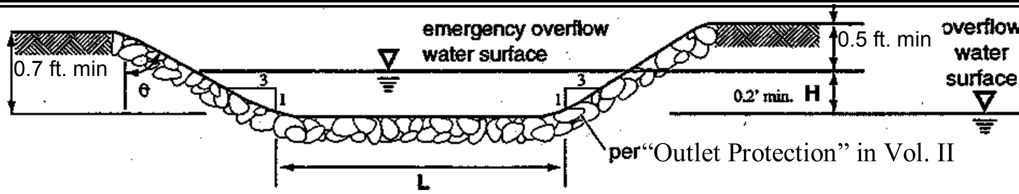


Figure 3.13 Weir Section for Emergency Overflow Spillway

3.2.2 Detention Tanks

Detention tanks are underground storage facilities typically constructed with large diameter corrugated metal pipe. Standard detention tank details are shown in Figure 3.14 and Figure 3.15. Control structure details are shown in Section 3.2.4.

Design Criteria

General. Typical design guidelines are as follows:

1. Tanks may be designed as flow-through systems with manholes in line (see Figure 3.14) to promote sediment removal and facilitate maintenance. Tanks may be designed as back-up systems if preceded by water quality facilities, since little sediment should reach the inlet/control structure and low head losses can be expected because of the proximity of the inlet/control structure to the tank
2. The detention tank bottom should be located 0.5 feet below the inlet and outlet to provide dead storage for sediment.
3. The minimum pipe diameter for a detention tank is 36 inches.
4. Tanks larger than 36 inches may be connected to each adjoining structure with a short section (2-foot maximum length) of 36-inch minimum diameter pipe.
5. Details of outflow control structures are given in Section 3.2.4.

Note: Control and access manholes should have additional ladder rungs to allow ready access to all tank access pipes when the catch basin sump is filled with water (see Figure 3.17, plan view).

Materials. Galvanized metals leach zinc into the environment, especially in standing water situations. This can result in zinc concentrations that can be toxic to aquatic life. Therefore, use of galvanized materials in stormwater facilities and conveyance systems is discouraged. Where other metals, such as aluminum or stainless steel, or plastics are available, they should be used.

Pipe material, joints, and protective treatment for tanks should be in accordance with Section 9.05 of the *WSDOT/APWA Standard Specification*.

Structural Stability. Tanks must meet structural requirements for overburden support and traffic loading if appropriate. H-20 live loads must be accommodated for tanks lying under parking areas and access roads. Metal tank end plates must be designed for structural stability at maximum hydrostatic loading conditions. Flat end plates generally require thicker gage material than the pipe and/or require reinforcing ribs. Tanks must be placed on stable, well consolidated native material with a suitable bedding. Tanks must not be placed in fill slopes, unless analyzed in a geotechnical report for stability and constructability.

Buoyancy. In moderately pervious soils where seasonal groundwater may induce flotation, buoyancy tendencies must be balanced either by ballasting with backfill or concrete backfill, providing concrete anchors, increasing the total weight, or providing subsurface drains to permanently lower the groundwater table. Calculations that demonstrate stability must be documented.

Access. The following guidelines for access may be used.

1. The maximum depth from finished grade to tank invert should be 20 feet.
2. Access openings should be positioned a maximum of 50 feet from any location within the tank.
3. All tank access openings may have round, solid locking lids (usually 1/2 to 5/8-inch diameter Allen-head cap screws).
4. Thirty-six-inch minimum diameter CMP riser-type manholes (Figure 3.15) of the same gage as the tank material may be used for access along the length of the tank and at the upstream terminus of the tank in a backup system. The top slab is separated (1-inch minimum gap) from the top of the riser to allow for deflections from vehicle loadings without damaging the riser tank.
5. All tank access openings must be readily accessible by maintenance vehicles.
6. Tanks must comply with the OSHA confined space requirements, which includes clearly marking entrances to confined space areas. This may be accomplished by hanging a removable sign in the access riser(s), just under the access lid.

Access Roads. Access roads are needed to all detention tank control structures and risers. The access roads must be designed and constructed as specified for detention ponds in Section 3.2.1.

Right-of-Way. Right-of-way may be needed for detention tank maintenance. It is recommended that any tract not abutting public right-of-way have a 15 to 20-foot wide extension of the tract to accommodate an access road to the facility.

Setbacks. It is recommended that facilities be a minimum of 20 feet from any structure, property line, and any vegetative buffer required by the local government and from any septic drainfield. However, the setback requirements are generally specified by the local government, uniform building code, or other statewide regulation and may be different from those mentioned above.

All facilities must be a minimum of 50 feet from the top of any steep (greater than 15 percent) slope. A geotechnical analysis and report must be prepared addressing the potential impact of the facility on a steep slope.

Maintenance. Provisions to facilitate maintenance operations must be built into the project when it is installed. Maintenance must be a basic consideration in design and in determination of first cost. See Table 3.4 for specific maintenance requirements.

Table 3.4 Specific Maintenance Requirements for Detention Vaults/Tanks			
Maintenance Component	**Defect**	**Conditions When Maintenance is Needed**	**Results Expected When Maintenance is Performed**
Storage Area	Plugged Air Vents	One-half of the cross section of a vent is blocked at any point or the vent is damaged.	Vents open and functioning.
	Debris and Sediment	Accumulated sediment depth exceeds 10% of the diameter of the storage area for 1/2 length of storage vault or any point depth exceeds 15% of diameter. (Example: 72-inch storage tank would require cleaning when sediment reaches depth of 7 inches for more than 1/2 length of tank.)	All sediment and debris removed from storage area.
	Joints Between Tank/Pipe Section	Any openings or voids allowing material to be transported into facility. (Will require engineering analysis to determine structural stability).	All joint between tank/pipe sections are sealed.
	Tank Pipe Bent Out of Shape	Any part of tank/pipe is bent out of shape more than 10% of its design shape. (Review required by engineer to determine structural stability).	Tank/pipe repaired or replaced to design.
	Vault Structure Includes Cracks in Wall, Bottom, Damage to Frame and/or Top Slab	Cracks wider than 1/2-inch and any evidence of soil particles entering the structure through the cracks, or maintenance/inspection personnel determines that the vault is not structurally sound. Cracks wider than 1/2-inch at the joint of any inlet/outlet pipe or any evidence of soil particles entering the vault through the walls.	Vault replaced or repaired to design specifications and is structurally sound. No cracks more than 1/4-inch wide at the joint of the inlet/outlet pipe.
Manhole	Cover Not in Place	Cover is missing or only partially in place. Any open manhole requires maintenance.	Manhole is closed.
	Locking Mechanism Not Working	Mechanism cannot be opened by one maintenance person with proper tools. Bolts into frame have less than 1/2 inch of thread (may not apply to self-locking lids).	Mechanism opens with proper tools.
	Cover Difficult to Remove	One maintenance person cannot remove lid after applying normal lifting pressure. Intent is to keep cover from sealing off access to maintenance.	Cover can be removed and reinstalled by one maintenance person.
	Ladder Rungs Unsafe	Ladder is unsafe due to missing rungs, misalignment, not securely attached to structure wall, rust, or cracks.	Ladder meets design standards. Allows maintenance person safe access.

Methods of Analysis **Detention Volume and Outflow**

The volume and outflow design for detention tanks must be in accordance with Minimum Requirement #7 in Volume I and the hydrologic analysis and design methods in Chapter 2. Restrictor and orifice design are given in Section 3.2.4.

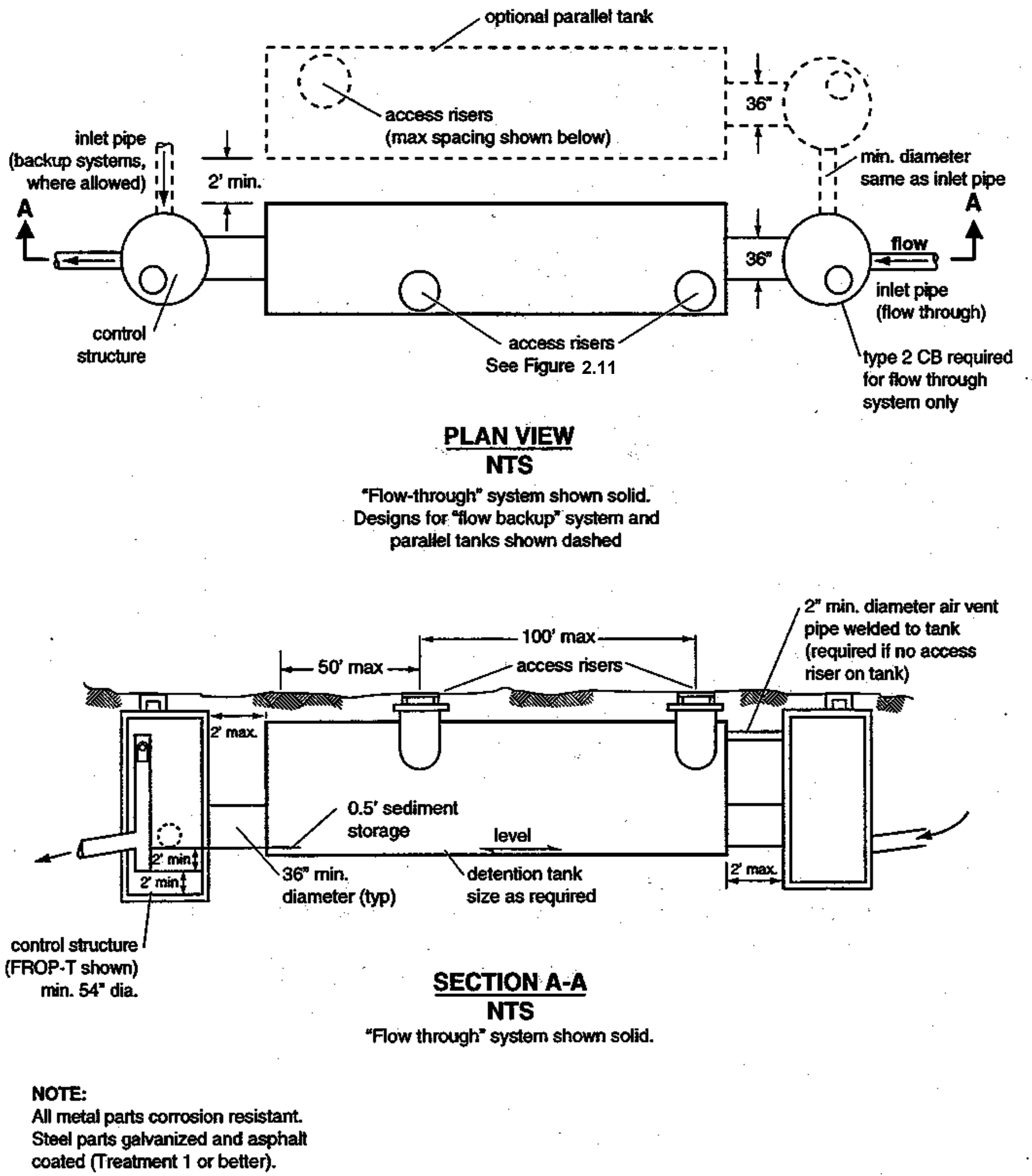


Figure 3.14 Typical Detention Tank

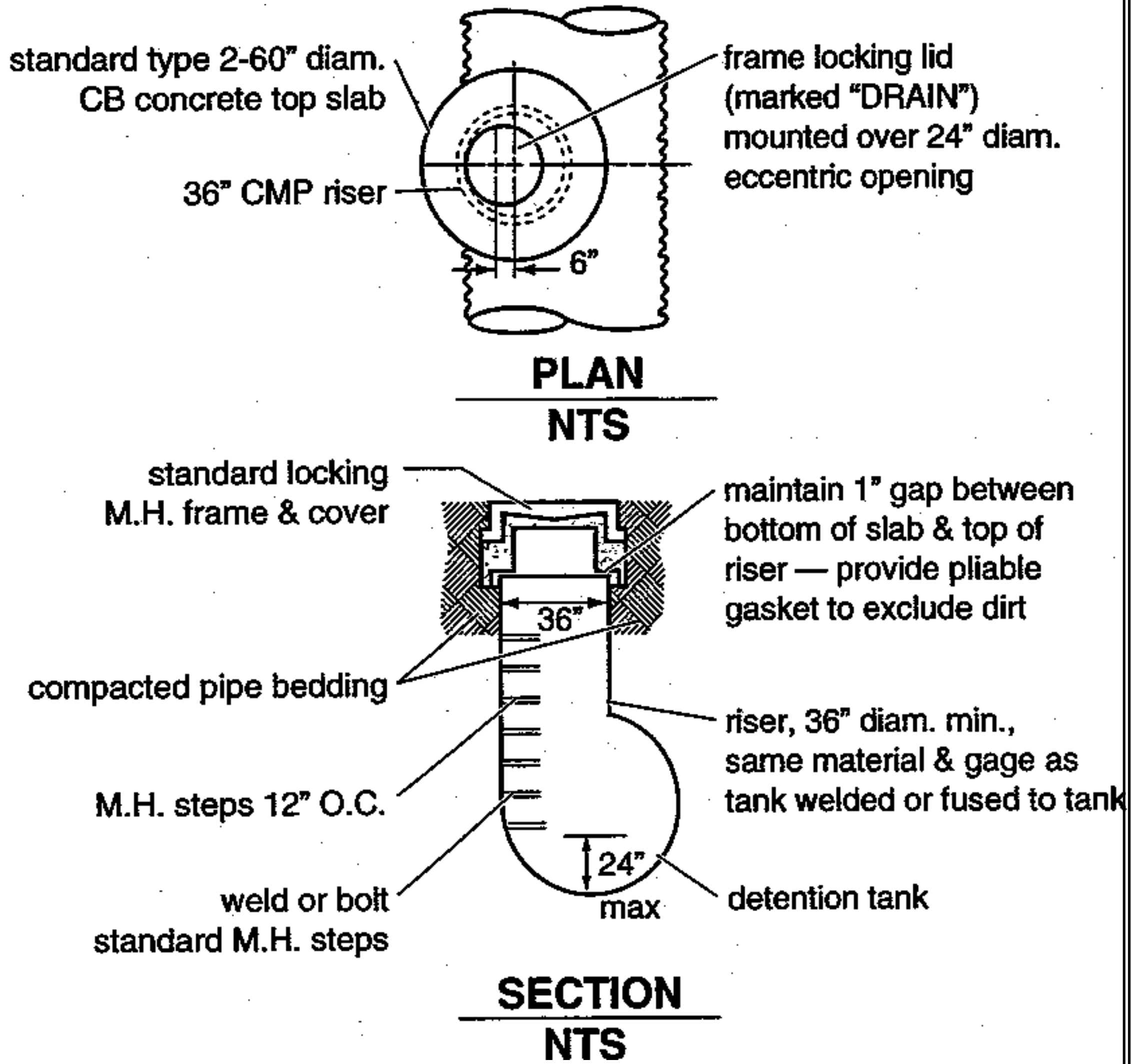


Figure 3.15 Detention Tank Access Detail

Notes:

1. Use adjusting blocks as required to bring frame to grade.
2. All materials to be aluminum or galvanized and asphalt coated (Treatment 1 or better).
3. Must be located for access by maintenance vehicles.
4. May substitute WSDOT special Type IV manhole (RCP only).

3.2.3 Detention Vaults

Detention vaults are box-shaped underground storage facilities typically constructed with reinforced concrete. A standard detention vault detail is shown in Figure 3.16. Control structure details are shown in Section 3.2.4.

Design Criteria

General. Typical design guidelines are as follows:

1. Detention vaults may be designed as flow-through systems with bottoms level (longitudinally) or sloped toward the inlet to facilitate sediment removal. Distance between the inlet and outlet should be maximized (as feasible).
2. The detention vault bottom may slope at least 5 percent from each side towards the center, forming a broad "v" to facilitate sediment removal. More than one "v" may be used to minimize vault depth. However, the vault bottom may be flat with 0.5-1 foot of sediment storage if removable panels are provided over the entire vault. It is recommended that the removable panels be at grade, have stainless steel lifting eyes, and weigh no more than 5 tons per panel.
3. The invert elevation of the outlet should be elevated above the bottom of the vault to provide an average 6 inches of sediment storage over the entire bottom. The outlet should also be elevated a minimum of 2 feet above the orifice to retain oil within the vault.
4. Details of outflow control structures are given in Section 3.2.4.

Materials. Minimum 3,000 psi structural reinforced concrete may be used for detention vaults. All construction joints must be provided with water stops.

Structural Stability. All vaults must meet structural requirements for overburden support and H-20 traffic loading (See Standard Specifications for Highway Bridges, 1998 Interim Revisions, American Association of State Highway and Transportation Officials). Vaults located under roadways must meet any live load requirements of the local government. Cast-in-place wall sections must be designed as retaining walls. Structural designs for cast-in-place vaults must be stamped by a licensed civil engineer with structural expertise. Vaults must be placed on stable, well-consolidated native material with suitable bedding. Vaults must not be placed in fill slopes, unless analyzed in a geotechnical report for stability and constructability.

Access. Access must be provided over the inlet pipe and outlet structure. The following guidelines for access may be used.

1. Access openings should be positioned a maximum of 50 feet from any location within the tank. Additional access points may be needed on large vaults. If more than one "v" is provided in the vault floor, access to each "v" must be provided.
2. For vaults with greater than 1,250 square feet of floor area, a 5' by 10' removable panel should be provided over the inlet pipe (instead of a standard frame, grate and solid cover). Alternatively, a separate access vault may be provided as shown in Figure 3.16.
3. For vaults under roadways, the removable panel must be located outside the travel lanes. Alternatively, multiple standard locking manhole covers may be provided. Ladders and hand-holds need only be provided at the outlet pipe and inlet pipe, and as needed to meet OSHA confined space requirements. Vaults providing manhole access at 12-foot spacing need not provide corner ventilation pipes as specified in Item 10 below.
4. All access openings, except those covered by removable panels, may have round, solid locking lids, or 3-foot square, locking diamond plate covers.
5. Vaults with widths 10 feet or less must have removable lids.
6. The maximum depth from finished grade to the vault invert should be 20 feet.
7. Internal structural walls of large vaults should be provided with openings sufficient for maintenance access between cells. The openings should be sized and situated to allow access to the maintenance "v" in the vault floor.
8. The minimum internal height should be 7 feet from the highest point of the vault floor (not sump), and the minimum width should be 4 feet. However, concrete vaults may be a minimum 3 feet in height and width if used as tanks with access manholes at each end, and if the width is no larger than the height. Also the minimum internal height requirement may not be needed for any areas covered by removable panels.
9. Vaults must comply with the OSHA confined space requirements, which includes clearly marking entrances to confined space areas. This may be accomplished by hanging a removable sign in the access riser(s), just under the access lid.
10. Ventilation pipes (minimum 12-inch diameter or equivalent) should be provided in all four corners of vaults to allow for artificial ventilation prior to entry of maintenance personnel into the vault. Alternatively removable panels over the entire vault may be provided.

Access Roads. Access roads are needed to the access panel (if applicable), the control structure, and at least one access point per cell, and they may be designed and constructed as specified for detention ponds in Section 3.2.1.

Right-of-Way. Right-of-way is needed for detention vaults maintenance. It is recommended that any tract not abutting public right-of-way should have a 15 to 20-foot wide extension of the tract to accommodate an access road to the facility.

Setbacks. It is recommended that facilities be a minimum of 20 feet from any structure, property line, and any vegetative buffer required by the local government and from any septic drainfield. However, the setback requirements are generally specified by the local government, uniform building code, or other statewide regulation and may be different from those mentioned above.

All facilities must be a minimum of 50 feet from the top of any steep (greater than 15 percent) slope. A geotechnical analysis and report must be prepared addressing the potential impact of the facility on a steep slope.

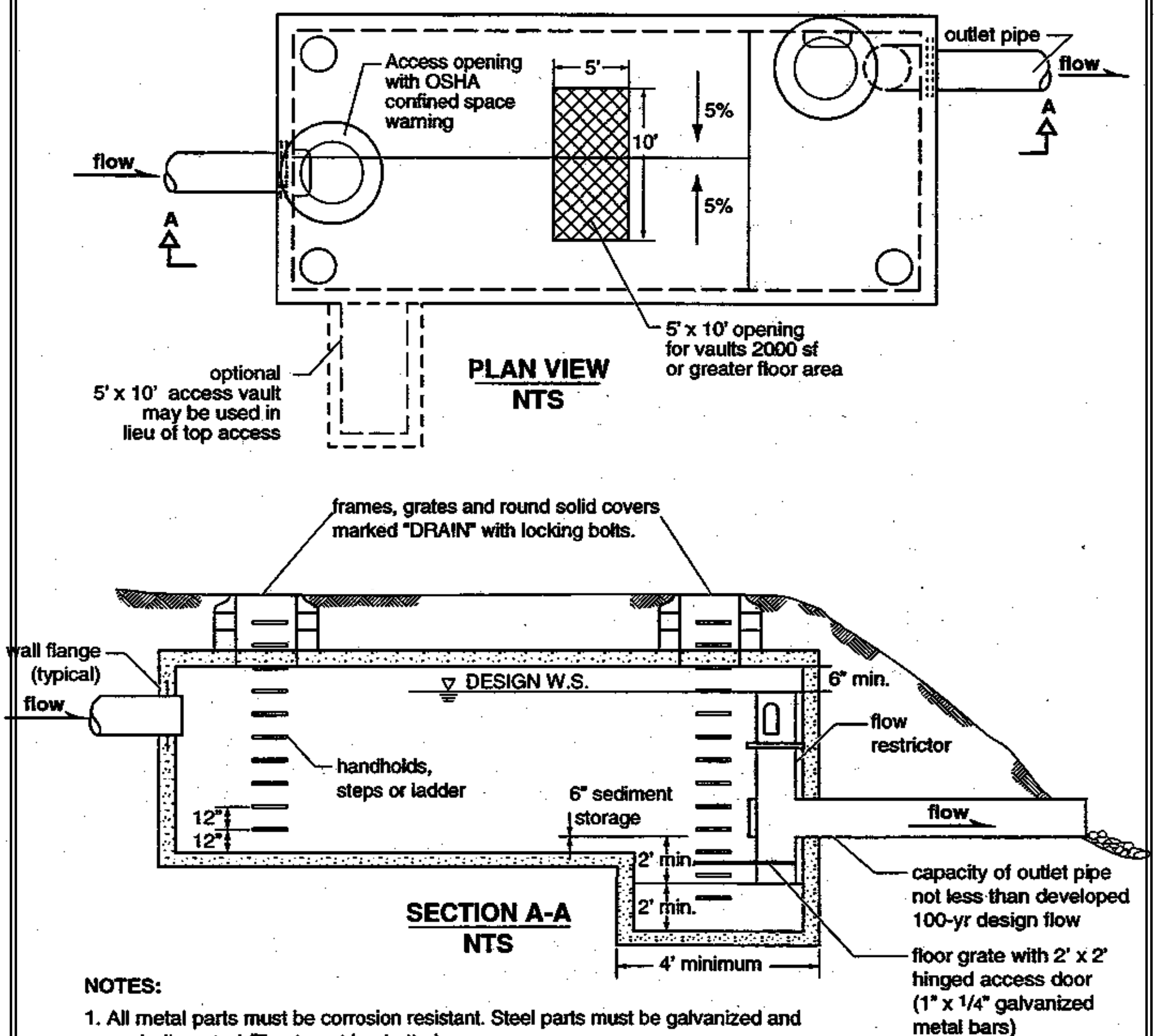
Maintenance. Provisions to facilitate maintenance operations must be built into the project when it is installed. Maintenance must be a basic consideration in design and in determination of first cost. See Table 3.4 for specific maintenance requirements.

Methods of Analysis

Detention Volume and Outflow

The volume and outflow design for detention vaults must be in accordance with Minimum Requirement #7 in Volume I and the hydrologic analysis and design methods in Chapter 1. Restrictor and orifice design are given in Section 3.2.4.

NOTE: All vault areas must be within 50' of an access point



NOTES:

1. All metal parts must be corrosion resistant. Steel parts must be galvanized and asphalt coated (Treatment I or better).
2. Provide water stop at all cast-in-place construction joints. Precast vaults shall have approved rubber gasket system.
3. Vaults ≤ 10' wide must use removable lids.
4. Prefabricated vault sections may require structural modifications to support 5' x 10' opening over main vault. Alternatively, access can be provided via a side vestibule as shown.

Figure 3.16 Typical Detention Vault

3.2.4 Control Structures

Control structures are catch basins or manholes with a restrictor device for controlling outflow from a facility to meet the desired performance. Riser type restrictor devices (“tees” or “FROP-Ts”) also provide some incidental oil/water separation to temporarily detain oil or other floatable pollutants in runoff due to accidental spill or illegal dumping.

The restrictor device usually consists of two or more orifices and/or a weir section sized to meet performance requirements.

Standard control structure details are shown in Figure 3.17 through Figure 3.19.

Design Criteria

Multiple Orifice Restrictor

In most cases, control structures need only two orifices: one at the bottom and one near the top of the riser, although additional orifices may best utilize detention storage volume. Several orifices may be located at the same elevation if necessary to meet performance requirements.

1. Minimum orifice diameter is 0.5 inches. Note: In some instances, a 0.5-inch bottom orifice will be too large to meet target release rates, even with minimal head. In these cases, the live storage depth need not be reduced to less than 3 feet in an attempt to meet the performance standards. Also, under such circumstances, flow-throttling devices may be a feasible option. These devices will throttle flows while maintaining a plug-resistant opening.
2. Orifices may be constructed on a tee section as shown in Figure 3.17 or on a baffle as shown in Figure 3.18.
3. In some cases, performance requirements may require the top orifice/elbow to be located too high on the riser to be physically constructed (e.g., a 13-inch diameter orifice positioned 0.5 feet from the top of the riser). In these cases, a notch weir in the riser pipe may be used to meet performance requirements (see Figure 3.21).
4. Consideration must be given to the backwater effect of water surface elevations in the downstream conveyance system. High tailwater elevations may affect performance of the restrictor system and reduce live storage volumes.

Riser and Weir Restrictor

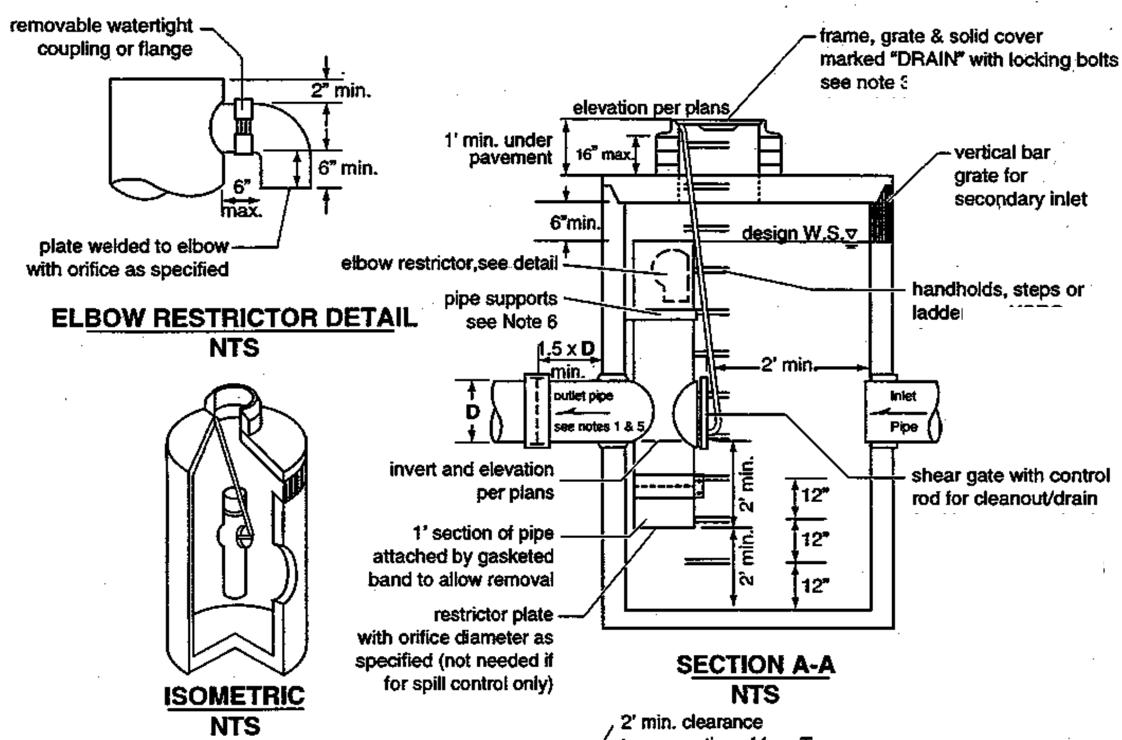
1. Properly designed weirs may be used as flow restrictors (see Figure 3.19 and Figure 3.21 through Figure 3.23). However, they must be designed to provide for primary overflow of the developed 100-year peak flow discharging to the detention facility.
2. The combined orifice and riser (or weir) overflow may be used to meet performance requirements; however, the design must still provide for primary overflow of the developed 100-year peak flow assuming all orifices are plugged. Figure 3.24 can be used to calculate the head in feet above a riser of given diameter and flow.

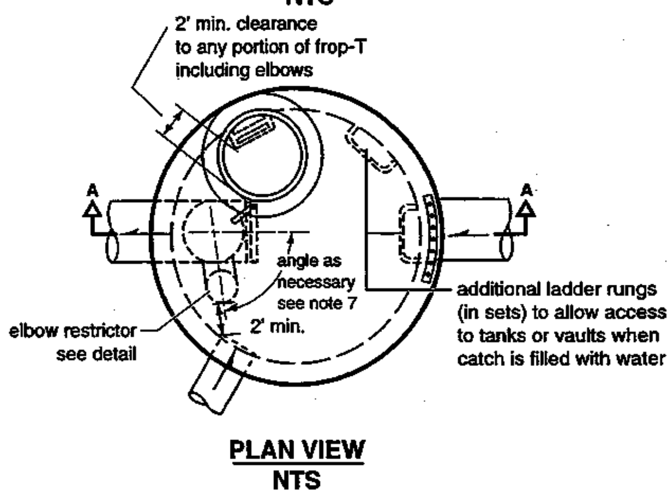
Access. The following guidelines for access may be used.

1. An access road to the control structure is needed for inspection and maintenance, and must be designed and constructed as specified for detention ponds in Section 3.3.1.
2. Manhole and catch basin lids for control structures must be locking, and rim elevations must match proposed finish grade.
3. Manholes and catch-basins must meet the OSRA confined space requirements, which include clearly marking entrances to confined space areas. This may be accomplished by hanging a removable sign in the access riser, just under the access lid.

Information Plate. It is recommended that a brass or stainless steel plate be permanently attached inside each control structure with the following information engraved on the plate:

- Name and file number of project
- Name and company of (1) developer, (2) engineer, and (3) contractor
- Date constructed
- Date of manual used for design
- Outflow performance criteria
- Release mechanism size, type, and invert elevation
- List of stage, discharge, and volume at one-foot increments
- Elevation of overflow
- Recommended frequency of maintenance.





NOTES:

1. Use a minimum of a 54" diameter type 2 catch basin.
2. Outlet Capacity: 100-Year developed peak flow.
3. Metal Parts: Corrosion resistant. Non-Galvanized parts perferred. Galvanized pipe parts to have asphalt treatment 1.
4. Frame and ladder or steps offset so:
   A. Cleanout gate is visible from top.
   B. Climb-down space is clear of riser and cleanout gate.
   C. Frame is clear of curb.
5. If metal outlet pipe connects to cement concrete pipe: outlet pipe to have smooth O.D. equal to concrete pipe I.D. less 1/4".
6. Provide at least one 3" X .090 inches support bracket anchored to concrete wall. (maximum 3'-0" vertical spacing)
7. Locate elbow restrictor(s) as necessary to provide minimum clearance as shown.
8. Locate additional ladder rungs in structures used as access to tanks or vaults to allow access when catch basin is filled with water.

Figure 3.17 Flow Restrictor (TEE)

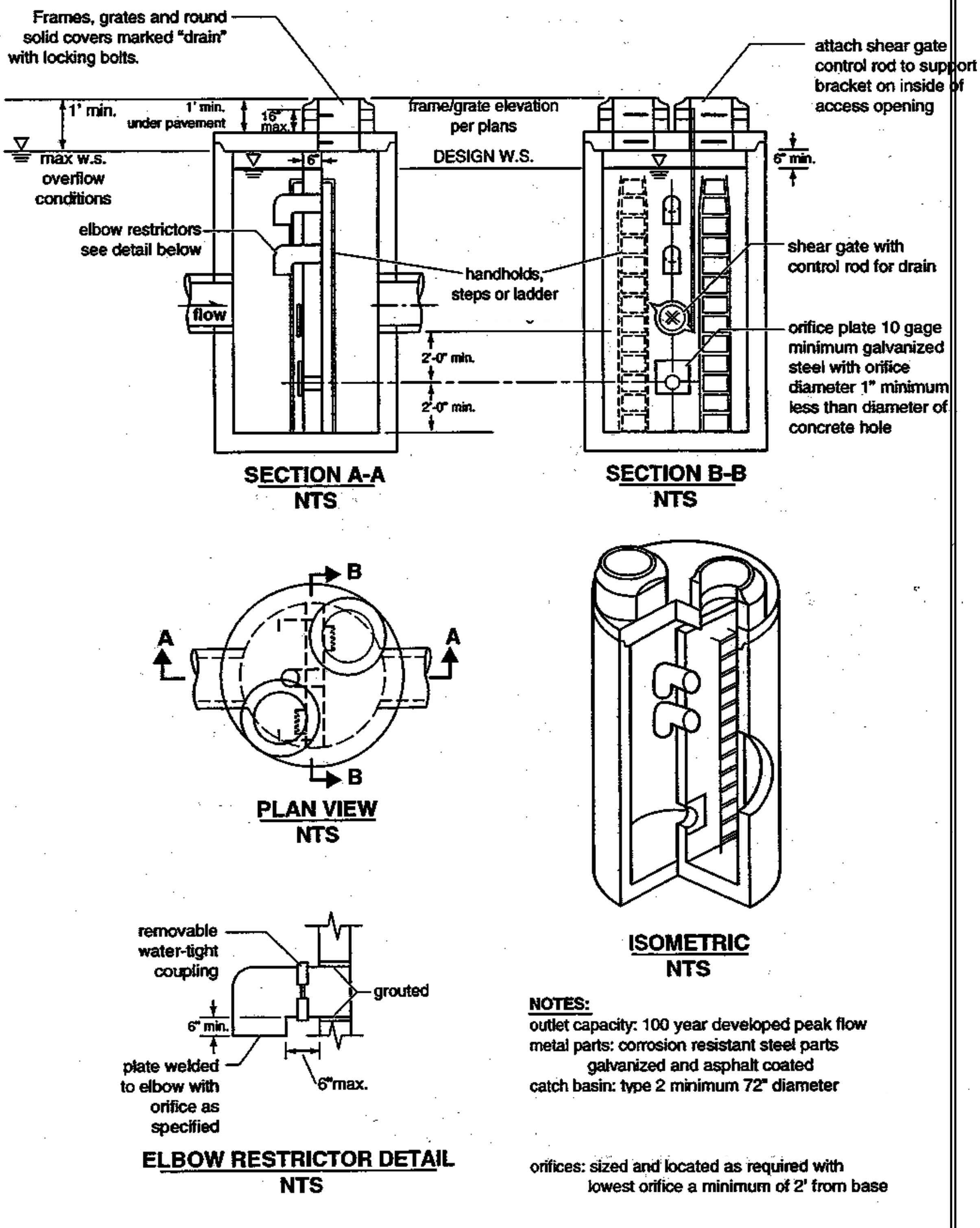
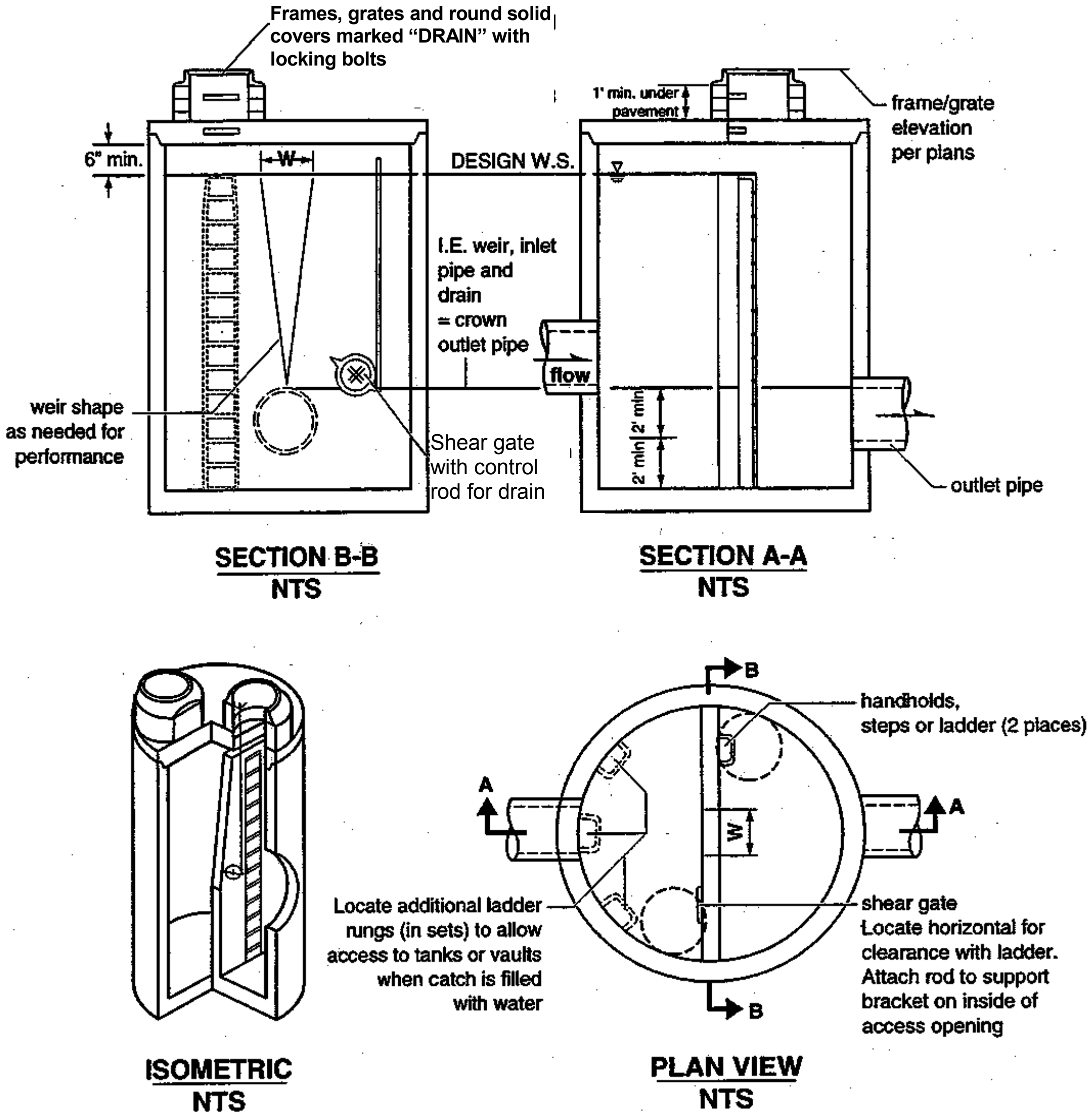


Figure 3.18 Flow Restrictor (Baffle)



NOTES:

Outlet Capacity: 100-year developed peak flow.
Metal Parts: corrosion resistant steel parts galvanized and asphalt coated.
Catch Basin: type 2 Min. 72" diameter

Baffle Wall: to be designed with concrete reinforcing as required.
Spill containment must be provided to temporarily detain oil or floatable pollutants in runoff due to accidental spill

Figure 3.19 Flow Restrictor (Weir)

Maintenance. Control structures and catch basins have a history of maintenance-related problems and it is imperative that a good maintenance program be established for their proper functioning. A typical problem is that sediment builds up inside the structure which blocks or restricts flow to the inlet. To prevent this problem these structures should be routinely cleaned out at least twice per year. Regular inspections of control structures should be conducted to detect the need for non-routine cleanout, especially if construction or land-disturbing activities are occurring in the contributing drainage area.

A 15-foot wide access road to the control structure should be installed for inspection and maintenance.

Table 3.5 provides maintenance recommendations for control structures and catch basins.

Table 3.5
Maintenance of Control Structures and Catchbasins

Maintenance Component	Defect	Condition When Maintenance is Needed	Results Expected When Maintenance is Performed
General	Trash and Debris (Includes Sediment)	Material exceeds 25% of sump depth or 1 foot below orifice plate.	Control structure orifice is not blocked. All trash and debris removed.
	Structural Damage	Structure is not securely attached to manhole wall.	Structure securely attached to wall and outlet pipe.
		Structure is not in upright position (allow up to 10% from plumb).	Structure in correct position.
		Connections to outlet pipe are not watertight and show signs of rust.	Connections to outlet pipe are water tight; structure repaired or replaced and works as designed.
		Any holes--other than designed holes--in the structure.	Structure has no holes other than designed holes.
Cleanout Gate	Damaged or Missing	Cleanout gate is not watertight or is missing.	Gate is watertight and works as designed.
		Gate cannot be moved up and down by one maintenance person.	Gate moves up and down easily and is watertight.
		Chain/rod leading to gate is missing or damaged.	Chain is in place and works as designed.
		Gate is rusted over 50% of its surface area.	Gate is repaired or replaced to meet design standards.
Orifice Plate	Damaged or Missing	Control device is not working properly due to missing, out of place, or bent orifice plate.	Plate is in place and works as designed.
	Obstructions	Any trash, debris, sediment, or vegetation blocking the plate.	Plate is free of all obstructions and works as designed.
Overflow Pipe	Obstructions	Any trash or debris blocking (or having the potential of blocking) the overflow pipe.	Pipe is free of all obstructions and works as designed.
Manhole	See Table 3.4	See Table 3..4	See Table 3.4
Catch Basin	See “Catch Basins”	See “Catch Basins”	See “Catch Basins”
CATCH BASINS			
General	Trash & Debris	Trash or debris which is located immediately in front of the catch basin opening or is blocking inletting capacity of the basin by more than 10%.	No Trash or debris located immediately in front of catch basin or on grate opening.

Table 3.5
Maintenance of Control Structures and Catchbasins

Maintenance Component	Defect	Condition When Maintenance is Needed	Results Expected When Maintenance is Performed
		Trash or debris (in the basin) that exceeds 60 percent of the sump depth as measured from the bottom of basin to invert of the lowest pipe into or out of the basin, but in no case less than a minimum of six inches clearance from the debris surface to the invert of the lowest pipe.	No trash or debris in the catch basin.
		Trash or debris in any inlet or outlet pipe blocking more than 1/3 of its height.	Inlet and outlet pipes free of trash or debris.
		Dead animals or vegetation that could generate odors that could cause complaints or dangerous gases (e.g., methane).	No dead animals or vegetation present within the catch basin.
	Sediment	Sediment (in the basin) that exceeds 60 percent of the sump depth as measured from the bottom of basin to invert of the lowest pipe into or out of the basin, but in no case less than a minimum of 6 inches clearance from the sediment surface to the invert of the lowest pipe. Measured from the bottom of basin to invert of the lowest pipe into or out of the basin.	No sediment in the catch basin
	Structure Damage to Frame and/or Top Slab	Top slab has holes larger than 2 square inches or cracks wider than 1/4 inch (Intent is to make sure no material is running into basin).	Top slab is free of holes and cracks.
		Frame not sitting flush on top slab, i.e., separation of more than 3/4 inch of the frame from the top slab. Frame not securely attached	Frame is sitting flush on the riser rings or top slab and firmly attached.
	Fractures or Cracks in Basin Walls/ Bottom	Maintenance person judges that structure is unsound.	Basin replaced or repaired to design standards.
		Grout fillet has separated or cracked wider than 1/2 inch and longer than 1 foot at the joint of any inlet/outlet pipe or any evidence of soil particles entering catch basin through cracks.	Pipe is regrouted and secure at basin wall.
	Settlement/ Misalignment	If failure of basin has created a safety, function, or design problem.	Basin replaced or repaired to design standards.
	Vegetation	Vegetation growing across and blocking more than 10% of the basin opening.	No vegetation blocking opening to basin.
		Vegetation growing in inlet/outlet pipe joints that is more than six inches tall and less than six inches apart.	No vegetation or root growth present.
	Contamination and Pollution	See "Detention Ponds"	No pollution present.
Catch Basin Cover	Cover Not in Place	Cover is missing or only partially in place. Any open catch basin requires maintenance.	Catch basin cover is closed
	Locking Mechanism Not Working	Mechanism cannot be opened by one maintenance person with proper tools. Bolts into frame have less than 1/2 inch of thread.	Mechanism opens with proper tools.
	Cover Difficult to Remove	One maintenance person cannot remove lid after applying normal lifting pressure. (Intent is keep cover from sealing off access to maintenance.)	Cover can be removed by one maintenance person.
Ladder	Ladder Rungs Unsafe	Ladder is unsafe due to missing rungs, not securely attached to basin wall, misalignment, rust, cracks, or sharp edges.	Ladder meets design standards and allows maintenance person safe access.
Metal Grates (If Applicable)	Grate opening Unsafe	Grate with opening wider than 7/8 inch.	Grate opening meets design standards.

Table 3.5 Maintenance of Control Structures and Catchbasins			
Maintenance Component	**Defect**	**Condition When Maintenance is Needed**	**Results Expected When Maintenance is Performed**
	Trash and Debris	Trash and debris that is blocking more than 20% of grate surface inletting capacity.	Grate free of trash and debris.
	Damaged or Missing.	Grate missing or broken member(s) of the grate.	Grate is in place and meets design standards.

Methods of Analysis

This section presents the methods and equations for design of **control structure restrictor devices.** Included are details for the design of orifices, rectangular sharp-crested weirs, v-notch weirs, sutro weirs, and overflow risers.

Orifices. Flow-through orifice plates in the standard tee section or turn-down elbow may be approximated by the general equation:

$$Q = C\,A\,\sqrt{2gh} \qquad \text{(equation 4)}$$

where Q = flow (cfs)
C = coefficient of discharge (0.62 for plate orifice)
A = area of orifice (ft^2)
h = hydraulic head (ft)
g = gravity (32.2 ft/sec^2)

Figure 3.20 illustrates this simplified application of the orifice equation.

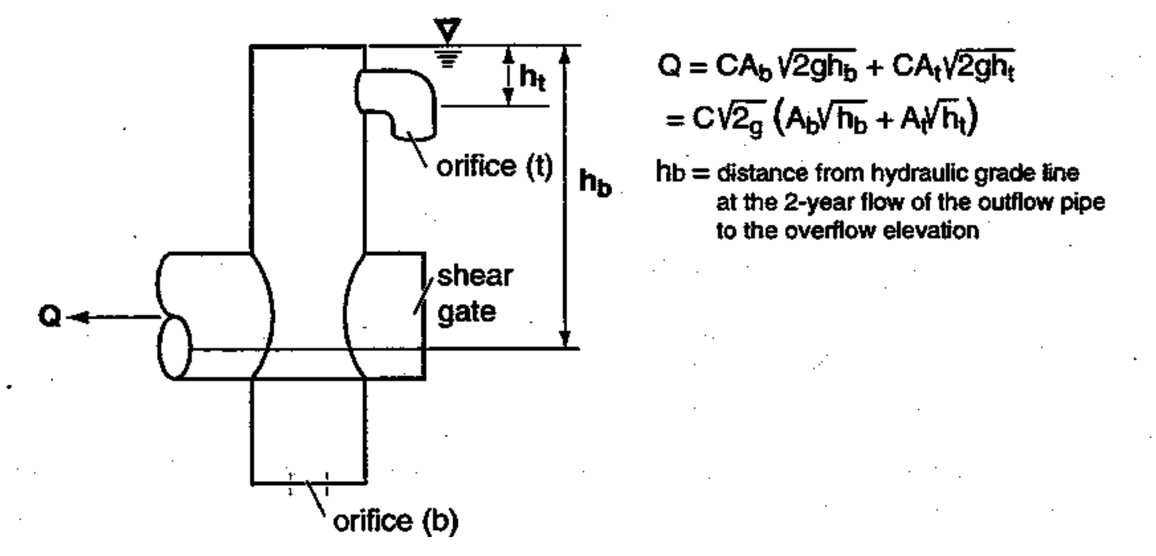


Figure 3.20 Simple Orifice

The diameter of the orifice is calculated from the flow. The orifice equation is often useful when expressed as the orifice diameter in inches:

$$d = \sqrt{\frac{36.88Q}{\sqrt{h}}} \qquad \text{(equation 5)}$$

where d = orifice diameter (inches)
Q = flow (cfs)
h = hydraulic head (ft)

Rectangular Sharp-Crested Weir. The rectangular sharp-crested weir design shown in Figure 3.21 may be analyzed using standard weir equations for the fully contracted condition.

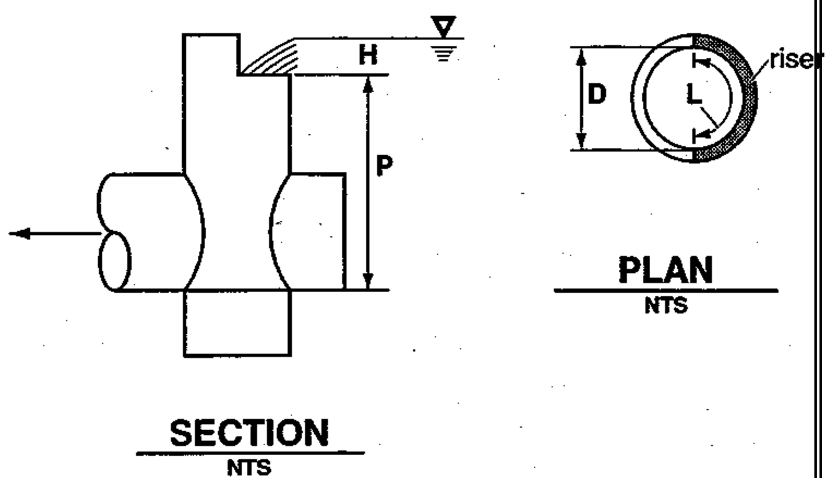


Figure 3.21 Rectangular, Sharp-Crested Weir

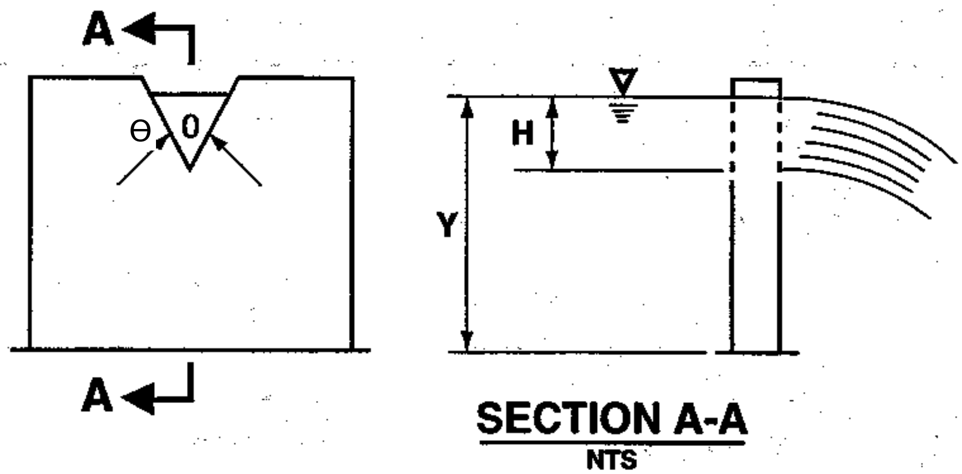
$$Q = C\,(L - 0.2H)H^{3/2} \qquad \text{(equation 6)}$$

where Q = flow (cfs)
C = 3.27 + 0.40 H/P (ft)
H, P are as shown above
L = length (ft) of the portion of the riser circumference as necessary not to exceed 50 percent of the circumference
D = inside riser diameter (ft)

Note that this equation accounts for side contractions by subtracting 0.1H from L for each side of the notch weir.

V-Notch Sharp - Crested Weir

V-notch weirs as shown in Figure 3.22 may be analyzed using standard equations for the fully contracted condition.



$$Q = C_d(\tan \Theta/2)H^{5/2}\text{, in cfs}$$

Where values of C_d may be taken from the following chart:

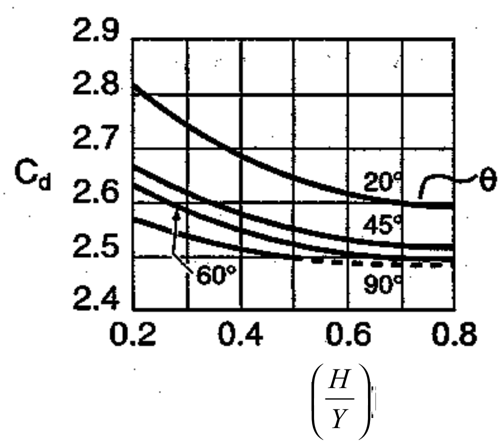


Figure 3.22 V-Notch, Sharp-Crested Weir

Proportional or Sutro Weir. Sutro weirs are designed so that the discharge is proportional to the total head. This design may be useful in some cases to meet performance requirements.

The sutro weir consists of a rectangular section joined to a curved portion that provides proportionality for all heads above the line A-B (see Figure 3.23). The weir may be symmetrical or non-symmetrical.

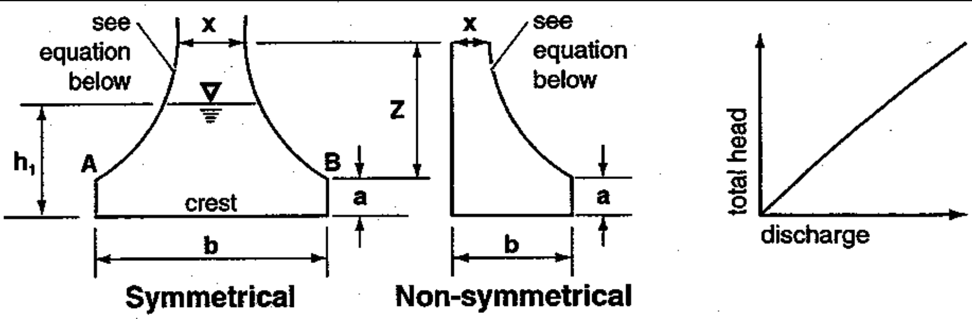


Figure 3.23 Sutro Weir

For this type of weir, the curved portion is defined by the following equation (calculated in radians):

$$\frac{x}{b} = 1 - \frac{2}{\pi} Tan^{-1}\sqrt{\frac{Z}{a}} \qquad \text{(equation 7)}$$

where a, b, x and Z are as shown in Figure 3.23. The head-discharge relationship is:

$$Q = \mathrm{C_d}\,\mathrm{b}\sqrt{2ga(h_1 - \frac{a}{3})} \qquad \text{(equation 8)}$$

Values of *Cd* for both symmetrical and non-symmetrical sutro weirs are summarized in Table 3.6.

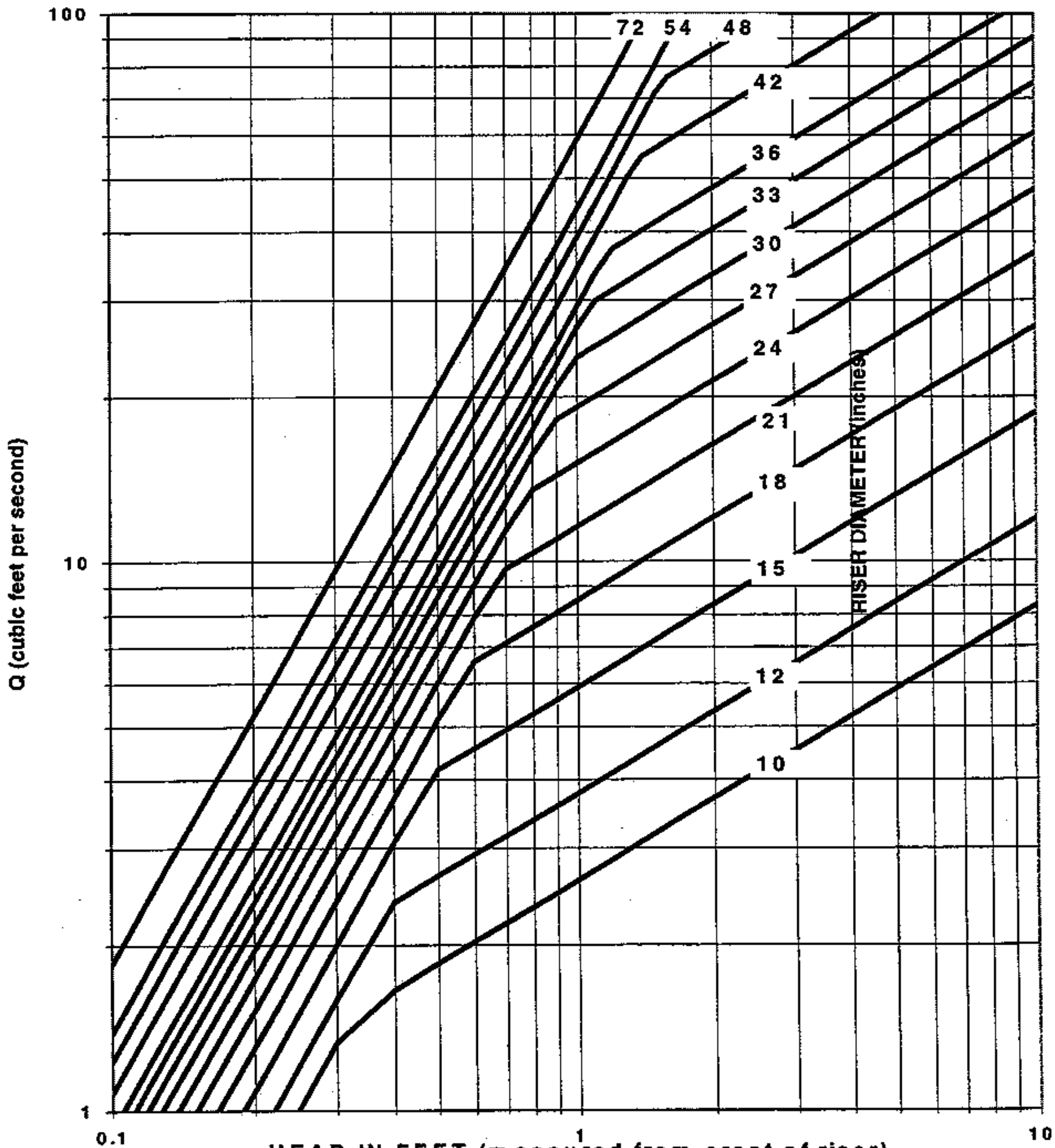
Note: When b > 1.50 or a > 0.30, use Cd=0.6.

Table 3.6
Values of C_d for Sutro Weirs

Cd *Values, Symmetrical*					
			b (ft)		
a (ft)	**0.50**	**0.75**	**1.0**	**1.25**	**1.50**
0.02	0.608	0.613	0.617	0.6185	0.619
0.05	0.606	0.611	0.615	0.617	0.6175
0.10	0.603	0.608	0.612	0.6135	0.614
0.15	0.601	0.6055	0.610	0.6115	0.612
0.20	0.599	0.604	0.608	0.6095	0.610
0.25	0.598	0.6025	0.6065	0.608	0.6085
0.30	0.597	0.602	0.606	0.6075	0.608

***Cd* Values, Non-Symmetrical**					
			***b* (ft)**		
a (ft)	**0.50**	**0.75**	**1.0**	**1.25**	**1.50**
0.02	0.614	0.619	0.623	0.6245	0.625
0.05	0.612	0.617	0.621	0.623	0.6235
0.10	0.609	0.614	0.618	0.6195	0.620
0.15	0.607	0.6115	0.616	0.6175	0.618
0.20	0.605	0.610	0.614	0.6155	0.616
0.25	0.604	0.6085	0.6125	0.614	0.6145
0.30	0.603	0.608	0.612	0.6135	0.614

Riser Overflow. The nomograph in Figure 3.24 can be used to determine the head (in feet) above a riser of given diameter and for a given flow (usually the 100-year peak flow for developed conditions).



$Q_{weir} = 9.739\ DH^{3/2}$

$Q_{orifice} = 3.782\ D^2H^{1/2}$

Q in cfs, D and H in feet

Slope change occurs at weir-orifice transition

Figure 3.24 Riser Inflow Curves

3.2.5 Other Detention Options

This section presents other design options for detaining flows to meet flow control facility requirements.

Use of Parking Lots for Additional Detention. Private parking lots may be used to provide additional detention volume for runoff events greater than the 2-year runoff event provided all of the following are met:

1. The depth of water detained does not exceed 1 foot at any location in the parking lot for runoff events up to and including the 100-year event.
2. The gradient of the parking lot area subject to ponding is 1 percent or greater.
3. The emergency overflow path is identified and noted on the engineering plan. The overflow must not create a significant adverse impact to downhill properties or drainage system.
4. Fire lanes used for emergency equipment are free of ponding water for all runoff events up to and including the 100-year event.

Use of Roofs for Detention

Detention ponding on roofs of structures may be used to meet flow control requirements provided all of the following are met:

1. The roof support structure is analyzed by a structural engineer to address the weight of ponded water.
2. The roof area subject to ponding is sufficiently waterproofed to achieve a minimum service life of 30 years.
3. The minimum pitch of the roof area subject to ponding is 1/4-inch per foot.
4. An overflow system is included in the design to safely convey the 100-year peak flow from the roof
5. A mechanism is included in the design to allow the ponding area to be drained for maintenance purposes or in the event the restrictor device is plugged.

3.3 Infiltration Stormwater Quantity and Flow Control

3.3.1 Purpose

To provide infiltration capacity for stormwater runoff quantity and flow control.

3.3.2 Description

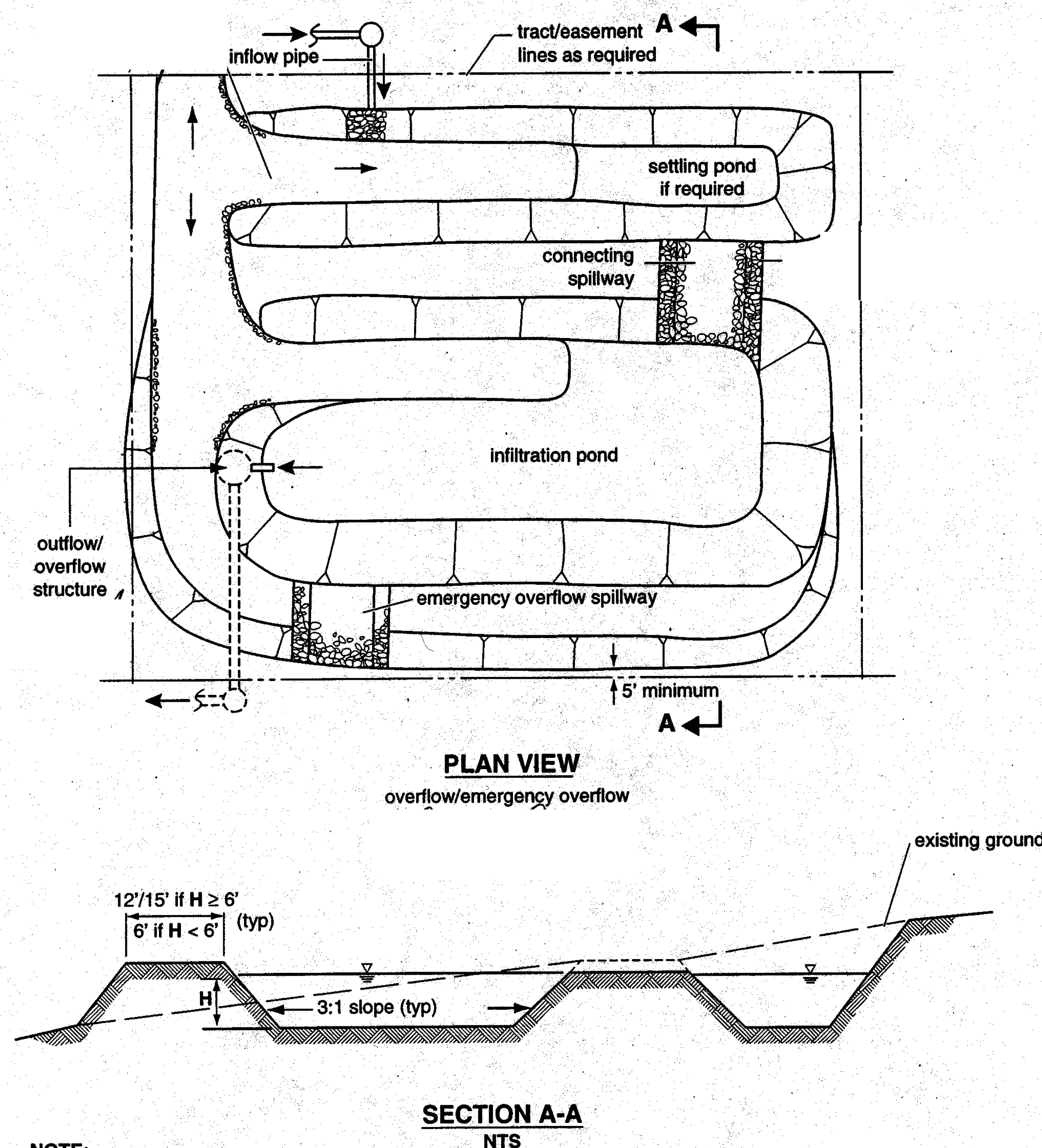
An infiltration BMP is typically an open basin (pond), trench, or buried perforated pipe used for distributing the stormwater runoff into the underlying soil (See Figure 3.25). Stormwater dry-wells receiving uncontaminated or properly treated stormwater can also be considered as infiltration facilities. (See Underground Injection Control Program, Chapter 173-218 WAC).

Coarser more permeable soils can be used for quantity control provided that the stormwater discharge does not cause a violation of ground water quality criteria. Typically, treatment for removal of TSS, oil, and/or soluble pollutants is necessary prior to conveyance to an infiltration BMP. The hydraulic design goal should be to mimic the natural hydrologic balance between surface and ground water, as needed to protect water uses.

3.3.3 Applications

Infiltration facilities are used to convey stormwater runoff from new development or redevelopment to the ground and ground water after appropriate treatment. Runoff, in excess of the infiltration capacity, must be detained and released in compliance with the flow control requirement in Volume I.

- Ground water recharge
- Discharge of uncontaminated or properly treated stormwater to dry-wells in compliance with Ecology's UIC regulations (Chapter 173-218 WAC)
- Retrofits in limited land areas: Infiltration trenches can be considered for residential lots, commercial areas, parking lots, and open space areas.
- Flood control
- Streambank erosion control



NOTE:
Detail is a schematic representation only. Actual configuration will vary depending on specific site constraints and applicable design criteria.

Figure 3.25 Typical Infiltration Pond/Basin

3.3.4 Site Characterization Criteria

One of the first steps in siting and designing infiltration facilities is to conduct a characterization study that includes the following:

Note: Information gathered during initial geotechnical investigations can be used for the site characterization.

Surface Features Characterization:

1. Topography within 500 feet of the proposed facility.
2. Anticipated site use (street/highway, residential, commercial, high-use site).
3. Location of water supply wells within 500 feet of proposed facility.
4. Location of ground water protection areas and/or 1, 5 and 10 year time of travel zones for municipal well protection areas.
5. A description of local site geology, including soil or rock units likely to be encountered, the groundwater regime, and geologic history of the site.

Subsurface Characterization:

1. Subsurface explorations (test holes or test pits) to a depth below the base of the infiltration facility of at least 5 times the maximum design depth of ponded water proposed for the infiltration facility,
2. Continuous sampling (representative samples from each soil type and/or unit within the infiltration receptor) to a depth below the base of the infiltration facility of 2.5 times the maximum design ponded water depth, but not less than 6 feet.
   - For basins, at least one test pit or test hole per 5,000 ft^2 of basin infiltrating surface (in no case less than two per basin).
   - For trenches, at least one test pit or test hole per 50 feet of trench length (in no case less than two per trench).

Note: The depth and number of test holes or test pits, and samples should be increased, if in the judgment of a licensed engineer with geotechnical expertise (P.E.), or other licensed professional acceptable to the local jurisdiction, the conditions are highly variable and such increases are necessary to accurately estimate the performance of the infiltration system. The exploration program may also be decreased if, in the opinion of the licensed engineer or other professional, the conditions are relatively uniform and the borings/test pits omitted will not influence the design or successful operation of the facility. In high water table sites, the subsurface exploration sampling need not be conducted lower than two (2) feet below the ground water table.

3. Prepare detailed logs for each test pit or test hole and a map showing the location of the test pits or test holes. Logs must include at a minimum, depth of pit or hole, soil descriptions, depth to water, presence of stratification *(note: Logs must substantiate whether stratification does or does not exist. The licensed professional may consider additional methods of analysis to substantiate the presence of stratification that will significantly impact the design of the infiltration facility).*

Infiltration Rate Determination:

Determine the representative infiltration rate of the unsaturated vadose zone based on infiltration tests and/or grain-size distribution/texture (see next section). Determine site infiltration rates using the Pilot Infiltration Test (PIT) described in Appendix V-B, if practicable. Such site testing should be considered to verify infiltration rate estimates based on soil size distribution and textural analysis. Infiltration rates may also be estimated based on soil grain-size distributions from test pits or test hole samples (particularly where a sufficient source of water does not exist to conduct a pilot infiltration test). As a minimum, one soil grain-size analysis per soil stratum in each test hole shall be performed within 2.5 times the maximum design water depth, but not less than 6 feet.

Soil Testing:

Soil characterization for each soil unit (soils of the same texture, color, density, compaction, consolidation and permeability) encountered should include:

- Grain-size distribution (ASTM D422 or equivalent AASHTO specification)
- Textural class (USDA) (See Figure 6.1)
- Percent clay content (include type of clay, if known)
- Color/mottling
- Variations and nature of stratification

Infiltration Receptor:

Infiltration receptor (unsaturated and saturated soil receiving the stormwater) characterization should include:

1. Installation of ground water monitoring wells (at least three per infiltration facility, or three hydraulically connected surface and ground water features that will establish a three-dimensional relationship for the ground water table, unless the highest ground water level is known to be at least 50 feet below the proposed infiltration facility) to:

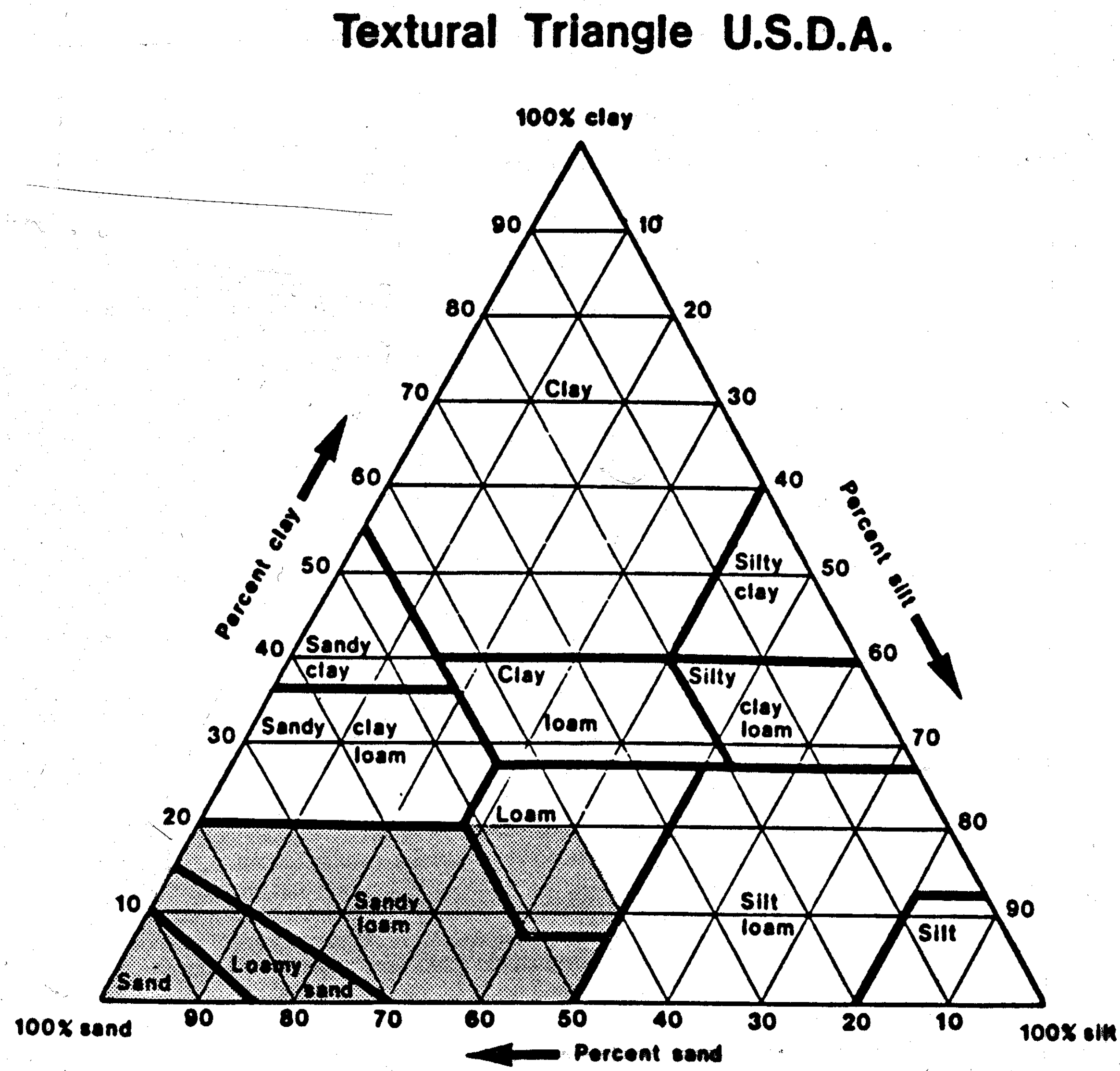
- monitor the seasonal ground water levels at the site during at least one wet season, and,
- consider the potential for both unconfined and confined aquifers, or confining units, at the site that may influence the proposed infiltration facility as well as the groundwater gradient. Other approaches to determine ground water levels at the proposed site could be considered if pre-approved by the local government jurisdiction, and,
- determine the ambient ground water quality, if that is a concern.

2. An estimate of the volumetric water holding capacity of the infiltration receptor soil. This is the soil layer below the infiltration facility and above the seasonal high-water mark, bedrock, hardpan, or other low permeability layer. This analysis should be conducted at a conservatively high infiltration rate based on vadose zone porosity, and the water quality runoff volume to be infiltrated. This, along with an analysis of ground water movement, will be useful in determining if there are volumetric limitations that would adversely affect drawdown.

3. Determination of:

   - Depth to ground water table and to bedrock/impermeable layers
   - Seasonal variation of ground water table based on well water levels and observed mottling
   - Existing ground water flow direction and gradient
   - Lateral extent of infiltration receptor
   - Horizontal hydraulic conductivity of the saturated zone to assess the aquifer's ability to laterally transport the infiltrated water.
   - Impact of the infiltration rate and volume at the project site on ground water mounding, flow direction, and water table; and the discharge point or area of the infiltrating water. A ground water mounding analysis should be conducted at all sites where the depth to seasonal ground water table or low permeability stratum is less than 15 feet and the runoff to the infiltration facility is from more than one acre. *(The site professional can consider conducting an aquifer test, or slug test and the type of ground water mounding analysis necessary at the site)*

Note: A detailed soils and hydrogeologic investigation should be conducted if potential pollutant impacts to ground water are a concern, or if the applicant is proposing to infiltrate in areas underlain by till or other impermeable layers. (Suggested references: "Implementation Guidance for the Ground Water Quality Standards", Department of Ecology, publication 96-2, 1996, and, "Washington State Water Quality Guide," Natural Resources Conservation Service, W. 316 Boone Ave, Spokane WA 99201-2348).



Shaded area is applicable for design of infiltration BMPs

Figure 3.26 USDA Textural Triangle

Source: U.S. Department of Agriculture

3.3.5 Design Infiltration Rate Determination - Guidelines and Criteria

The representative site infiltration rate must be determined from soil test results, the stratification identified during the site characterization, and/or in-situ field measurements.

Historically, infiltration rates have been estimated from soil grain size distribution (gradation) data using the United States Department of Agriculture (USDA) textural analysis approach. To use the USDA textural analysis approach, the grain size distribution test must be conducted in accordance with the USDA test procedure (SOIL SURVEY MANUAL, U.S. Department of Agriculture, October 1993, page 136). This manual only considers soil passing the #10 sieve (2 mm) (U.S. Standard) to determine percentages of sand, silt, and clay for use in Figure 3.26 (USDA Textural Triangle). However, many soil test laboratories use the ASTM soil size distribution test procedure (ASTM D422), which considers the full range of soil particle sizes, to develop soil size distribution curves. The ASTM soil gradation procedure must not be used with Figure 3.26 to perform USDA soil textural analyses.

Three Methods for Determining Long-term Infiltration Rates for Sizing Infiltration Facilities

For designing the infiltration facility the site professional should select one of the three methods described below that will best represent the long-term infiltration rate at the site. The long-term infiltration rate should be used for routing and sizing the basin/trench for the maximum drawdown time of 24 hours. If the pilot infiltration test (table 3.9) or hindcast approach (table 3.8) is selected corroboration with a textural based infiltration rate (table 3.7) is also desirable. Appropriate correction factors must be applied as specified. Verification testing of the completed facility is strongly encouraged. (See Site Suitability Criterion # 7-Verification Testing)

<u>1. USDA Soil Textural Classification</u>

Table 3.7 provides the correlation between USDA soil texture and infiltration rates for estimating infiltration rates for homogeneous soils based on gradations from soil samples and textural analysis. The USDA soil texture – infiltration rate correlation in Table 3.7 is based on the correlation developed by Rawls, et. al. (1982), but with minor changes in the infiltration rates based on WEF/ASCE (1998). The infiltration rates provided through this correlation represent short-term conservative rates for homogeneous soils. These rates not consider the effects of site variability and long-term clogging due to siltation and biomass buildup in the infiltration facility.

Table 3.7 -- Recommended Infiltration Rates based on USDA Soil Textural Classification.			
	*Short-Term Infiltration Rate (in./hr)	Correction Factor, CF	Estimated Long-Term (Design) Infiltration Rate (in./hr)
Clean sandy gravels and gravelly sands (i.e., 90% of the total soil sample is retained in the #10 sieve)	20	2	10
Sand	8	4	2
Loamy Sand	2	4	0.5
Sandy Loam	1	4	0.25
Loam	0.5	4	0.13

*From WEF/ASCE, 1998.

Based on experience with long-term full-scale infiltration pond performance, Ecology's Technical Advisory Committee (TAC) recommends that the short-term infiltration rates be reduced as shown in Table 3.7, dividing by a correction factor of 2 to 4, depending on the soil textural classification. The correction factors provided in Table 3.7 represent an average degree of long-term facility maintenance, TSS reduction through pretreatment, and site variability in the subsurface conditions. These conditions might include deposits of ancient landslide debris, buried stream channels, lateral grain size variability, and other factors that affect homogeneity).

These correction factors could be reduced, subject to the approval of the local jurisdiction, under the following conditions:

- For sites with little soil variability,
- Where there will be a high degree of long-term facility maintenance,
- Where specific, reliable pretreatment is employed to reduce TSS entering the infiltration facility

In no case shall a correction factor less than 2.0 be used.

Correction factors higher than those provided in Table 3.7 should be considered for situations where long-term maintenance will be difficult to implement, where little or no pretreatment is anticipated, or where site conditions are highly variable or uncertain. These situations require the use of best professional judgment by the site engineer and the approval of the local jurisdiction. An Operation and Maintenance plan and a financial bonding plan may be required by the local jurisdiction.

2. ASTM Gradation Testing at Full Scale Infiltration Facilities

As an alternative to Table 3.7, recent studies by Massmann and Butchart (2000) were used to develop the correlation provided in Table 3.8. These studies compare infiltration measurements from full-scale infiltration facilities to soil gradation data developed using the ASTM procedure (ASTM D422). The primary source of the data used by Massmann and Butchart was from Wiltsie (1998), who included limited infiltration studies only on Thurston County sites. However, Massmann and Butchart also included limited data from King and Clark County sites in their analysis. This table provides recommended long-term infiltration rates that have been correlated to soil gradation parameters using the ASTM soil gradation procedure.

Table 3.8 can be used to estimate long-term design infiltration rates directly from soil gradation data, subject to the approval of the local jurisdiction. As is true of Table 3.7, the long-term rates provided in Table 3.8 represent average conditions regarding site variability, the degree of long-term maintenance and pretreatment for TSS control. The long-term infiltration rates in Table 3.8 may need to be decreased if the site is highly variable, or if maintenance and influent characteristics are not well controlled. The data that forms the basis for Table 3.8 was from soils that would be classified as sands or sandy gravels. No data was available for finer soils. Therefore, Table 3.8 should not be used for soils with a d_{10} size (10% passing the size listed) less than 0.05 mm (U.S. Standard Sieve).

Table 3.8 -- Alternative Recommended Infiltration Rates based on ASTM Gradation Testing.

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

The infiltration rates provided in Tables 3.7 and 3.8 represent rates for homogeneous soil conditions. If more than one soil unit is encountered within 6 feet of the base of the facility or 2.5 times the proposed maximum water design depth, use the lowest infiltration rate determined from each of the soil units as the representative site infiltration rate.

If soil mottling, fine silt or clay layers, which cannot be fully represented in the soil gradation tests, are present below the bottom of the infiltration pond, the infiltration rates provided in the tables will be too high and should be reduced. Based on limited full-scale infiltration data (Massmann and Butchart, 2000; Wiltsie, 1998), it appears that the presence of mottling indicates soil conditions that reduce the infiltration rate for homogeneous conditions by a factor of 3 to 4.

3. In-situ Infiltration Measurements

Where feasible, Ecology encourages in-situ infiltration measurements, using a procedure such as the Pilot Infiltration Test (PIT) described in Appendix V-B. Small-scale infiltration tests such as the EPA Falling Head or double ring infiltrometer test (ASTM D3385-88) are not recommended unless modified versions are determined to be acceptable by Ecology or the local jurisdiction. These small-scale infiltration tests tend to seriously overestimate infiltration rates and, based on recent TAC experience, are considered unreliable.

As in the previous methods, the infiltration rate obtained from the test shall be considered to be a short-term rate. This short-term rate must be reduced through correction factors to account for site variability and number of tests conducted, degree of long-term maintenance and influent pretreatment/control, and potential for long-term clogging due to siltation and bio-buildup.

The typical range of correction factors to account for these issues, based on TAC experience, is summarized in Table 3.9. The range of correction factors is for general guidance only. The specific correction factors used shall be determined based on the professional judgment of the licensed engineer or other site professional considering all issues which may affect the long-term infiltration rate, subject to the approval of the local jurisdictional authority**.**

Table 3.9 -- Correction Factors to be Used With In-Situ Infiltration Measurements to Estimate Long-Term Design Infiltration Rates.

Issue	Partial Correction Factor
Site variability and number of locations tested	CF_v = 1.5 to 6
Degree of long-term maintenance to prevent siltation and bio-buildup	CF_m = 2 to 6
Degree of influent control to prevent siltation and bio-buildup	CF_i = 2 to 6

$CF = CF_v + CF_m + CF_i$

The following discussions are to provide assistance in determining the partial correction factors to apply in Table 3.9.

Site variability and number of locations tested - The number of locations tested must be capable of producing a picture of the subsurface conditions that fully represents the conditions throughout the facility site. The partial correction factor used for this issue depends on the level of uncertainty that adverse subsurface conditions may occur. If the range of uncertainty is low - for example, conditions are known to be uniform through previous exploration and site geological factors - one pilot infiltration test may be adequate to justify a partial correction factor at the low end of the range. If the level of uncertainty is high, a partial correction factor near the high end of the range may be appropriate. This might be the case where the site conditions are highly variable due to a deposit of ancient landslide debris, or buried stream channels. In these cases, even with many explorations and several pilot infiltration tests, the level of uncertainty may still be high. A partial correction factor near the high end of the range could be assigned where conditions have a more typical variability, but few explorations and only one pilot infiltration test is conducted. That is, the number of explorations and tests conducted do not match the degree of site variability anticipated.

Degree of long-term maintenance to prevent siltation and bio-buildup The standard of comparison here is the long-term maintenance requirements provided in Volume V, Chapter 4, and any additional requirements by local jurisdictional authorities. Full compliance with these requirements would be justification to use a partial correction factor at the low end of the range. If there is a high degree of uncertainty that long-term maintenance will be carried out consistently, or if the maintenance plan is poorly defined, a partial correction factor near the high end of the range may be justified.

Degree of influent control to prevent siltation and bio-buildup - A partial correction factor near the high end of the range may be justified under the following circumstances:

1. If the infiltration facility is located in a shady area where moss buildup or litter fall buildup from the surrounding vegetation is likely and cannot be easily controlled through long-term maintenance

2. If there is minimal pre-treatment, and the influent is likely to contain moderately high TSS levels.

If influent into the facility can be well controlled such that the planned long-term maintenance can easily control siltation and biomass buildup, then a partial correction factor near the low end of the range may be justified.

The determination of long-term design infiltration rates from in-situ infiltration test data involves a considerable amount of engineering judgment. Therefore, when reviewing or determining the final long-term design infiltration rate, the local jurisdictional authority should consider the results of both textural analyses and in-situ infiltration tests results when available.

3.3.6 Site Suitability Criteria (SSC)

This section provides criteria that must be considered for siting infiltration systems. When a site investigation reveals that any of the seven applicable criteria cannot be met appropriate mitigation measures must be implemented so that the infiltration facility will not pose a threat to safety, health, and the environment.

For site selection and design decisions a geotechnical and hydrogeologic report should be prepared by a qualified engineer with geotechnical and hydrogeologic experience, or an equivalent professional acceptable to the local jurisdiction, under the seal of a registered Professional Engineer. The design engineer may utilize a team of certified or registered professionals in soil science, hydrogeology, geology, and other related fields.

SSC-1 Setback Criteria

Setback requirements are generally required by local regulations, uniform building code requirements, or other state regulations.

These Setback Criteria are provided as guidance.

- Stormwater infiltration facilities should be set back at least 100 feet from drinking water wells, septic tanks or drainfields, and springs used for public drinking water supplies. Infiltration facilities upgradient of drinking water supplies and within 1, 5, and 10-year time of travel zones must comply with Health Dept. requirements (Washington Wellhead Protection Program, DOH, 12/93).
- Additional setbacks must be considered if roadway deicers or herbicides are likely to be present in the influent to the infiltration system
- From building foundations; ≥ 20 feet downslope and ≥100 feet upslope
- From a Native Growth Protection Easement (NGPE); ≥20 feet
- From the top of slopes >15%; ≥ 50 feet.
- Evaluate on-site and off-site structural stability due to extended subgrade saturation and/or head loading of the permeable layer, including the potential impacts to downgradient properties, especially on hills with known side-hill seeps.

SSC-2 Ground Water Protection Areas

A site is not suitable if the infiltration facility will cause a violation of Ecology's Ground Water Quality Standards (See SSC-7 for verification testing guidance). Local jurisdictions should be consulted for applicable pollutant removal requirements upstream of the infiltration facility, and to determine whether the site is located in an aquifer sensitive area, sole source aquifer, or a wellhead protection zone.

SSC-3 High Vehicle Traffic Areas

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

High Vehicle Traffic Areas are:

- Commercial or industrial sites subject to an expected average daily traffic count (ADT) ≥100 vehicles/1,000 ft² gross building area (trip generation), and
- Road intersections with an ADT of ≥ 25,000 on the main roadway, or ≥ 15,000 on any intersecting roadway.

SSC-4 Soil Infiltration Rate/Drawdown Time

Design to completely drain ponded runoff within 24 hours from 10-year, 24-hour recurrence frequency runoff and within 48 hours of the 100-year, 24-hour recurrence frequency runoff.

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

The base of all infiltration basins or trench systems shall be ≥ 5 feet above the seasonal high-water mark, bedrock (or hardpan) or other low permeability layer. A separation down to 3 feet may be considered if the ground water mounding analysis, volumetric receptor capacity, and the design of the overflow and/or bypass structures are judged by the site professional to be adequate to prevent overtopping and meet the site suitability criteria specified in this section.

SSC-6 Cold Climate and Impact of Roadway deicers

- For cold climate design criteria (snowmelt/ice impacts) refer to D. Caraco and R. Claytor reference.
- Potential impact of roadway deicers on potable water wells must be considered in the siting determination. Mitigation measures must be implemented if infiltration of roadway deicers can cause a violation of ground water quality standards.

SSC 7-Verification Testing of the Completed Facility

Verification testing of the completed full-scale infiltration facility is recommended to confirm that the design infiltration parameters are adequate. The site professional should determine the duration and frequency of the verification testing program including the monitoring program for the potentially impacted ground water. The ground water monitoring wells installed during site characterization (See Section 3.3.4) may be used for this purpose. Long-term (more than two years) in-situ drawdown and confirmatory monitoring of the infiltration facility would be preferable (See King County reference).

3.3.7 General Design, Maintenance, and Construction Criteria for Infiltration Facilities

This section covers design, construction and maintenance criteria that apply to infiltration basins and trenches.

Design Criteria – Sizing Facilities

The size of the infiltration facility can be determined by routing the appropriate stormwater runoff through it. To prevent the onset of anaerobic conditions, the infiltration facility must be designed to drain completely 24 hours after the flow to it has stopped.

In general, an infiltration facility would have 2 discharge modes. The primary mode of discharge from an infiltration facility is infiltration into the ground. However, when the infiltration capacity of the facility is reached, additional runoff to the facility will cause the facility to overflow. Overflows from an infiltration facility must comply with the Minimum Requirement #7 for flow control in Volume I.

In order to determine compliance with the flow control requirements, Western Washington Hydrology Model (WWHM), or an appropriately calibrated continuous simulation model based on HSPF, must be used.

When using WWHM for simulating flow through an infiltrating facility, a spreadsheet may be used to calculate infiltration rates as a function of the infiltrating surface area of the facility. A stage-area-storage-discharge table must be generated that shows the facility’s storage and infiltration as a function of the stage. The table must also show the facility’s overflow discharge as a function of stage. This table can be imported to the WWHM as an electronic text file, or, the table can be typed directly into the WWHM. WWHM can route the historic runoff hydrograph for the developed condition through the infiltration pond and determine if the overflow from the facility complies with flow control requirement #7.

Additional Design Criteria

- Slope of the base of the infiltration facility should be <3 percent.
- Spillways/Overflow structures- A nonerodible outlet or spillway with a firmly established elevation must be constructed to discharge overflow. Ponding depth, drawdown time, and storage volume are calculated from that reference point. Overflow Structure-Refer to Chapter 2 for design details

Construction Criteria

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

Maintenance Criteria

Provision should be made for regular and perpetual maintenance of the infiltration basin/trench, with adequate access. Maintenance should be conducted when water remains in the basin or trench for more than 24 hours. An Operation and Maintenance Plan, approved by the local jurisdiction, should ensure maintaining the desired infiltration rate.

Debris/sediment accumulation- Removal of accumulated debris/sediment in the basin/trench should be conducted every 6 months or as needed to prevent clogging, or when water remains in the pond for greater than 24 hours at or less than design storm conditions.

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

For more detailed information on maintenance, see Volume V, Section 4.6 – Maintenance Standards for Drainage Facilities.

Verification of Performance

During the first 1-2 years of operation verification testing (specified in SSC-7) is strongly recommended, along with a maintenance program that results in achieving expected performance levels. Operating and maintaining ground water monitoring wells (specified in Section 3.3.6 - Site Suitability Criteria) is also strongly encouraged.

3.3.8 Infiltration Basins

This section covers design and maintenance criteria specific for infiltration basins. (See schematic in Figure 3.25)

Description:

Infiltration basins are earthen impoundments used for the collection, temporary storage and infiltration of incoming stormwater runoff.

Design Criteria specific for Basins

- Access should be provided for vehicles to easily maintain the forebay (presettling basin) area and not disturb vegetation, or resuspend sediment any more than is absolutely necessary.
- A minimum of one foot of freeboard is recommended when establishing the design ponded water depth. Freeboard is measured from the rim of the infiltration facility to the maximum ponding level or from the rim down to the overflow point if overflow or a spillway is included.
- Lining Material - Basins can be open or covered with a 6 to 12-inch layer of filter material such as coarse sand, or a suitable filter fabric to help prevent the buildup of impervious deposits on the soil surface. A nonwoven geotextile should be selected that will function sufficiently without plugging (see geotextile specifications in Appendix V-C of Volume V). The filter layer can be replaced or cleaned when/if it becomes clogged.
- Vegetation - The embankment, emergency spillways, spoil and borrow areas, and other disturbed areas should be stabilized and planted, preferably with grass, in accordance with Stormwater Site Plan (See Minimum Requirement #1 of Volume I). Without healthy vegetation the surface soil pores would quickly plug.

Maintenance Criteria for Basins

- Maintain basin floor and side slopes to promote dense turf with extensive root growth. This enhances infiltration, prevents erosion and consequent sedimentation of the basin floor, and prevents invasive weed growth. Bare spots are to be immediately stabilized and revegetated.
- Vegetation growth should not be allowed to exceed 18 inches in height. Mow the slopes periodically and check for clogging, and erosion.
- Seed mixtures should be the same as those recommended in Table 3.2. The use of slow-growing, stoloniferous grasses will permit long

intervals between mowing. Mowing twice a year is generally satisfactory. Fertilizers should be applied only as necessary and in limited amounts to avoid contributing to ground water pollution. Consult the local extension agency for appropriate fertilizer types, including slow release fertilizers, and application rates.

3.3.9 Infiltration Trenches

This section covers design, construction, and maintenance criteria specific for infiltration trenches.

Description:

Infiltration trenches are generally at least 24 inches wide, and are backfilled with a coarse stone aggregate, allowing for temporary storage of stormwater runoff in the voids of the aggregate material. Stored runoff then gradually infiltrates into the surrounding soil. The surface of the trench can be covered with grating and/or consist of stone, gabion, sand, or a grassed covered area with a surface inlet. Perforated rigid pipe of at least 8-inch diameter can also be used to distribute the stormwater in a stone trench.

See Figures 3.27 for schematic of an infiltration trench. See Figures 3.28, 3.29, 3.30, and 3.31 for examples of trench designs.

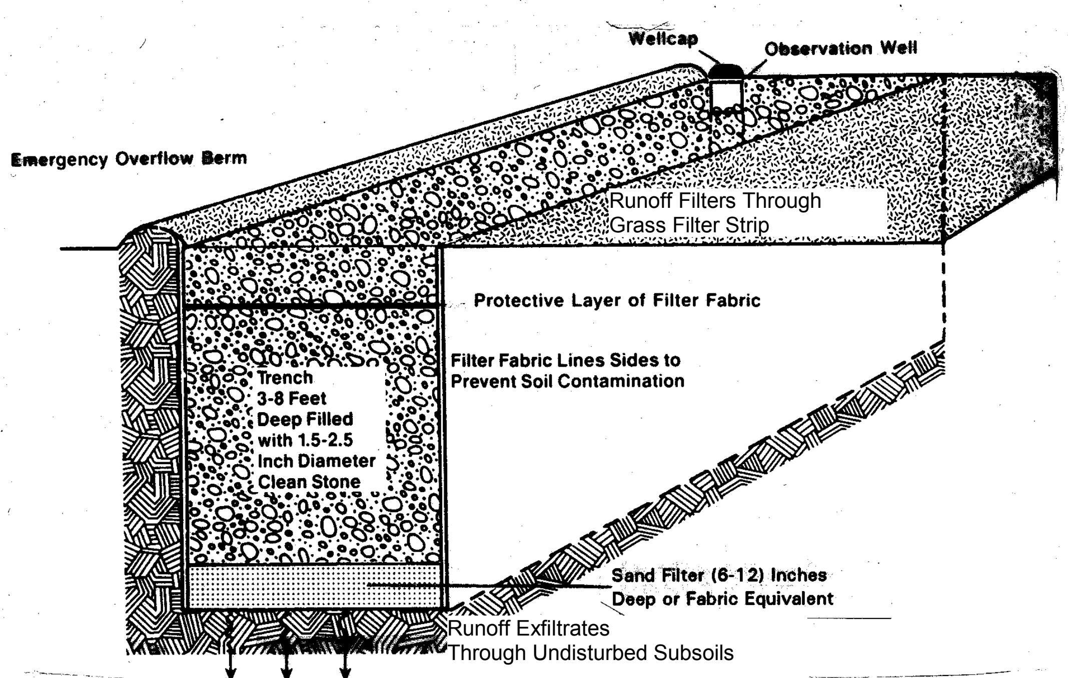


Figure 3.27 Schematic of an Infiltration Trench

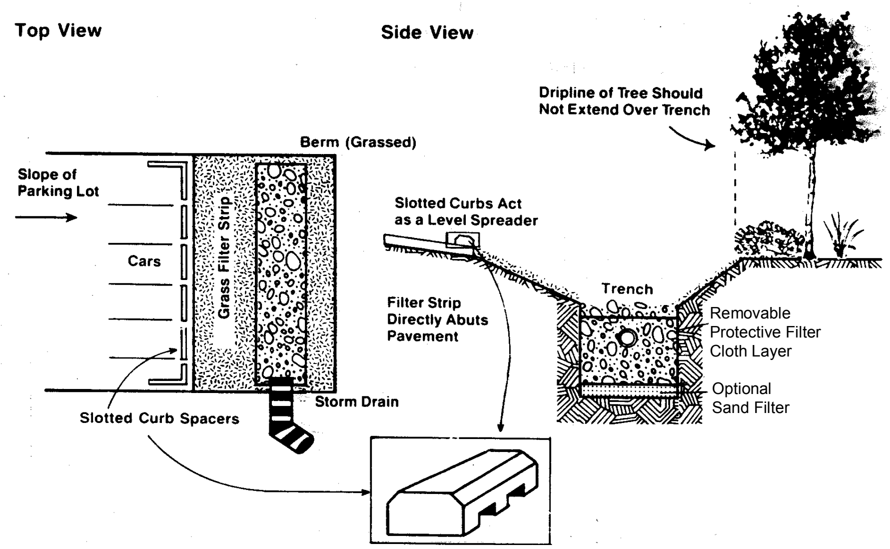


Figure 3.28 Parking Lot Perimeter Trench Design

Source: Schueler (reproduced with Permission

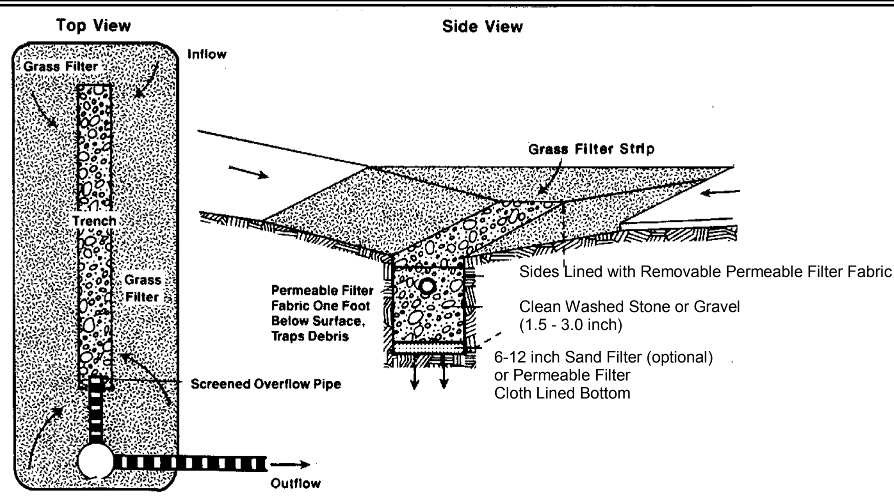


Figure 3.29 Median Strip Trench Design

Source: Schueler (reproduced with permission)

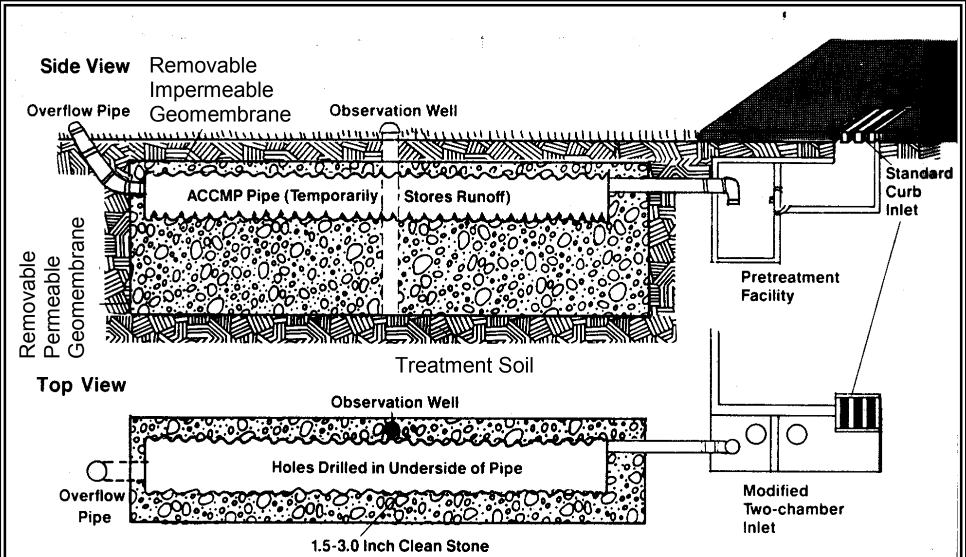


Figure 3.30 Oversized Pipe Trench Design

ACCMP – Asphalt Coated Corrugated Metal Pipe

Source: Schueler (reproduced with permission)

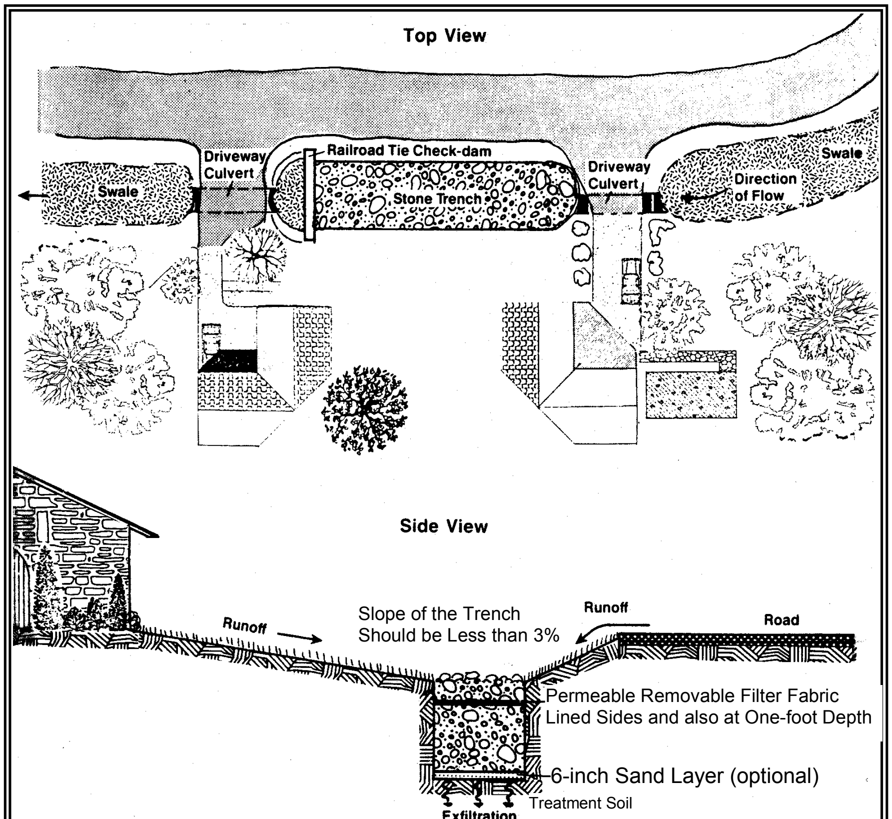


Figure 3.31 Swale/Trench Design

Source: Schueler (reproduced with permission)

Design Criteria

- Due to accessibility and maintenance limitations infiltration trenches must be carefully designed and constructed. The local jurisdiction should be contacted for additional specifications.
- Consider including an access port or open or grated top for accessibility to conduct inspections and maintenance.
- Backfill Material - The aggregate material for the infiltration trench should consist of a clean aggregate with a maximum diameter of 3 inches and a minimum diameter of 1.5 inches. Void space for these aggregates should be in the range of 30 to 40 percent.
- Geotextile fabric liner - The aggregate fill material shall be completely encased in an engineering geotextile material. In the case of an aggregate surface, geotextile should surround all of the aggregate fill material except for the top one-foot, which is placed over the geotextile. Geotextile fabric with acceptable properties must be carefully selected to avoid plugging (see Appendix V-C of Volume V).
- The bottom sand or geotextile fabric as shown in the attached figures is optional.

Refer to the Federal Highway Administration Manual "Geosynthetic Design and Construction Guidelines," Publication No. FHWA HI-95-038, May 1995 for design guidance on geotextiles in drainage applications. Refer to the NCHRP Report 367, "Long-Term Performance of Geosynthetics in Drainage Applications," 1994, for long-term performance data and background on the potential for geotextiles to clog, blind, or to allow piping to occur and how to design for these issues.

- Overflow Channel - Because an infiltration trench is generally used for small drainage areas, an emergency spillway is not necessary. However, a non-erosive overflow channel leading to a stabilized watercourse should be provided at the one-day drawdown level.
- Surface Cover-A stone filled trench can be placed under a porous or impervious surface cover to conserve space.
- Observation Well - An observation well should be installed at the lower end of the infiltration trench to check water levels, drawdown time, sediment accumulation, and conduct water quality monitoring. Figure 3.32 illustrates observation well details. It should consist of a perforated PVC pipe which is 4 to 6 inches in diameter and it should be constructed flush with the ground elevation. For larger trenches a 12-36 inch diameter well can be installed to facilitate maintenance operations such as pumping out the sediment. The top of the well should be capped to discourage vandalism and tampering.

Construction Criteria

- Trench Preparation -Excavated materials must be placed away from the trench sides to enhance trench wall stability. Care should also be taken to keep this material away from slopes, neighboring property, sidewalks and streets. It is recommended that this material be covered with plastic. (see Erosion/sediment control Criteria in Volume II).
- Stone Aggregate Placement and Compaction - The stone aggregate should be placed in lifts and compacted using plate compactors. As a rule of thumb, a maximum loose lift thickness of 12 inches is recommended. The compaction process ensures geotextile conformity to the excavation sides, thereby reducing potential piping and geotextile clogging, and settlement problems.
- Potential Contamination - Prevent natural or fill soils from intermixing with the stone aggregate. All contaminated stone aggregate must be removed and replaced with uncontaminated stone aggregate.
- Overlapping and Covering-Following the stone aggregate placement, the geotextile must be folded over the stone aggregate to form a 12 inch minimum longitudinal overlap. When overlaps are required between rolls, the upstream roll should overlap a minimum of 2 feet over the downstream roll in order to provide a shingled effect.
- Voids behind Geotextile - Voids between the geotextile and excavation sides must be avoided. Removing boulders or other obstacles from the trench walls is one source of such voids. Natural soils should be placed in these voids at the most convenient time during construction to ensure geotextile conformity to the excavation sides. Soil piping, geotextile clogging, and possible surface subsidence will be avoided by this remedial process.
- Unstable Excavation Sites - Vertically excavated walls may be difficult to maintain in areas where the soil moisture is high or where soft or cohesionless soils predominate. Trapezoidal, rather than rectangular, cross-sections may be needed.

Maintenance Criteria

- Sediment buildup in the top foot of stone aggregate or the surface inlet should be monitored on the same schedule as the observation well.

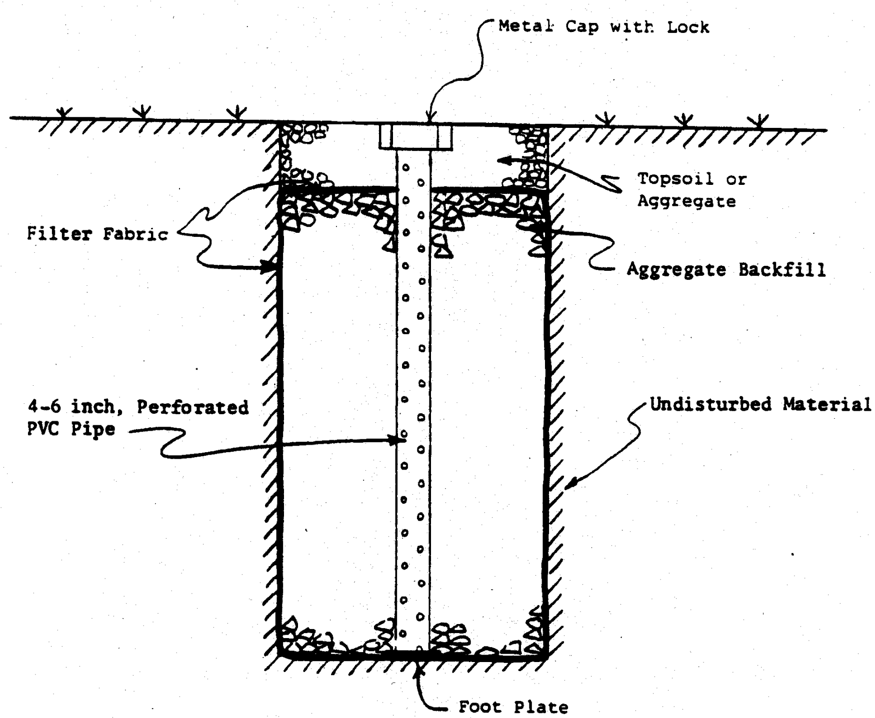


Figure 3.32 Observation Well Details

Volume III References

Chin, D.A., Water Resources Engineering, Prentice Hall, New Jersey, 2000

Chow, V.T., Handbook of Applied Hydrology, McGraw Hill Book Co., New York, 1964.

Department of Ecology, Stormwater Management Manual for the Puget Sound Basin, February, 1992.

Dinacola, R.S., Characterization and Simulation of Rainfall-Runoff Relations for Headwater Basins in Western King and Snohomish Counties, Washington, USGS Water Resources Investigations Report 89-4052, 1989.

Ferguson, Bruce K., Stormwater Infiltration, Lewis Publishers, 1994.

Horner, Richard Fundamentals of Urban Runoff Management-Technical and Institutional Issues, 1994

Huber, Wayne, and Robert Dickinson, Stormwater Management Model Version 4 Part A: User's Manual, Environmental Research Laboratory, Athens, GA, 1988.

King County Runoff Time Series (KCRTS), King County Department of Natural Resources, Personal Communication, 1999.

Massmann, Joel & Carolyn Butchart, U. of Washington Infiltration Characteristics, Performance, and Design of Storm Water Facilities, March 2000

NOAA Atlas 2, Precipitation Frequency Atlas of the Western United States, Volume IX-Washington.

Rawls, W. J., Brakensiek, D. L. and Saxton, K. E. Estimation of Soil Properties. Transactions of the American Society of Agricultural Engineers, Vol. 25, No. 5, pp. 1316-1320, 1982.

Stubdaer, J.M., The Santa Barbara Urban Hydrograph Method, National Symposium on Urban Hydrology and Sediment Control, University of Kentucky, Lexington, KY, 1975.

USDA-SCS, Technical Release No. 20 (TR-20) Model Project Formulation, 1982.

USDA-SCS, Technical Release No. 55: Urban Hydrology for Small Watersheds, 1986.

USEPA, Hydrological Simulation Program - Fortran HSPF Users Manual for Release 9., EPA 600/3-84-066, Environmental Research Laboratory, Athens, GA, June 1984.

Wiltsie, Edward, Stormwater Facilities Performance Study, Infiltration Pond Testing and Data Evaluation, August 10, 1998

Resource Materials (not specifically referenced in text)

Barfield, B. J., and Warner, R. C., and Haan, C. T. Applied Hydrology and Sedimentology for Disturbed Areas. Oklahoma Technical Press, Stillwater, Oklahoma, 1983.

Bentall, R., Methods of Collecting and Interpreting Ground Water Data, U.S. G. S. Water Supply Paper 1544-H., 1963, 97 p.

Bianchi, W.C. and D.C. Muckel, Ground Water Recharge Hydrology, ARS 41-161, USDA, 1970. 62 p.

Bouwer, Herman., Groundwater Hydrology, McGraw-Hill Book Company, Inc., N.Y., 1978.

Camp Dresser & McKee, Larry Walker Associates, Uribe and Associates, and Resource Planning Associates. California Storm Water Best Management Practice Handbooks. March 1993

Caraco, D., Claytor, R., Stormwater BMP Design Supplement for Cold Climates USEPA, December 1997

Davis, S.N. and R.J. DeWiest, Hydrogeology. John Wiley and Sons, N.Y., 1966.

Ferguson, Bruce K., Stormwater Infiltration, Lewis Publishers, 1994.

Ferris, J.G., D.B. Knowles, R.H. Brown, and R.W. Stallman, Theory of Aquifer Tests, USGS, Water Supply Paper No. 1536-E.

Gaus, Jennifer J., Soils of Stormwater Infiltration Basins in the Puget Sound Region: Trace Metal Form and Concentration and Comparison to Washington State Department of Ecology Guidelines, Master of Science, U. of Washington, 1993.

Hannon, J. B., Underground Disposal of Storm Water Runoff, Design Guidelines Manual, California Department of Transportation, U.S. Department of Transportation, Washington, DC (FHWA-TS-80-218), February 1980.

Harrington, Bruce W., Design and Construction of Infiltration Trenches, Seminar-Urban Watershed Management, How to Design Urban Stormwater Best Management Practices, ASCE, July, 1994.

Hilding, Karen, A Survey of Infiltration Basins in the Puget Sound Regions, Masters Project, U. of California, 1993.

Jacobson, Michael A., Summary and Conclusions from Applied Research on Infiltration Basins and Recommendations for Modifying the Washington Department of Ecology Stormwater Management Manual for the Puget Sound Basin. University of Washington, Center for Urban Water Resources Management. May 1993.

King County, Washington, Surface Water Design Manual, September 1, 1998.

Klochak, John R., An Investigation of the Effectiveness of Infiltration Systems in Treating Urban Runoff, Master of Science, U. of Washington, 1992.

Konrad, C. P., Jensen, B. W., Burges, S. J, Reinelt, L. E. On-Site Residential Stormwater Management Alternatives. Center for Urban Water Resources, University of Washington. September 1995.

Livingston, E. H., Infiltration Practices: The Good, the Bad, and the Ugly, National Conference on Urban Runoff Management, Chicago, Ill. 1993.

Moore, John, Seepage: A System for Early Evaluation of the Pollution Potential of Agriculture Groundwater Environments, SCS, 1988.

Pettyjohn, W.A., Introduction to Artificial Ground Water Recharge, USEPA, Ada, Oklahoma, National Waterwell Association, Worthington, Ohio, 1981, 44 p.

Rawls, W. J., D. L. Brakensiek, and K. E. Saxton, Estimation of Soil Properties. Transactions of the American Society of Agricultural Engineers, Vol. 25, No. 5, pp. 1316-1320, 1982.

Schueler, Thomas, et. al., A Current Assessment of Urban Best Management Practices, March, 1992

Soil Conservation Service, SCS National Engineering Handbook Section 8, Engineering Geology , USDA., 1978.

Soil Conservation Service, USDA, SCS National Engineering Handbook, Section 18 Ground Water, 1968.

Soil Conservation Service, USDA, SCS Technical Release No. 36, Ground Water Recharge, Engineering Division, 1967, 22 p.

Todd, D.K., Ground Water Hydrology, John Wiley and Sons, Inc., N.Y., 1959.

Urbonas and Stahre, “Stormwater Best Management Practices”, Prentiss-Hall, 1993

WEF Manual of Practice #23 Urban Runoff Quality Management, Water Environment Federation & ASCE 1998.

Wenzel, L.K., Methods of Determining Permeability of Water Bearing Materials, USGS Water Supply Paper 887, 1942.

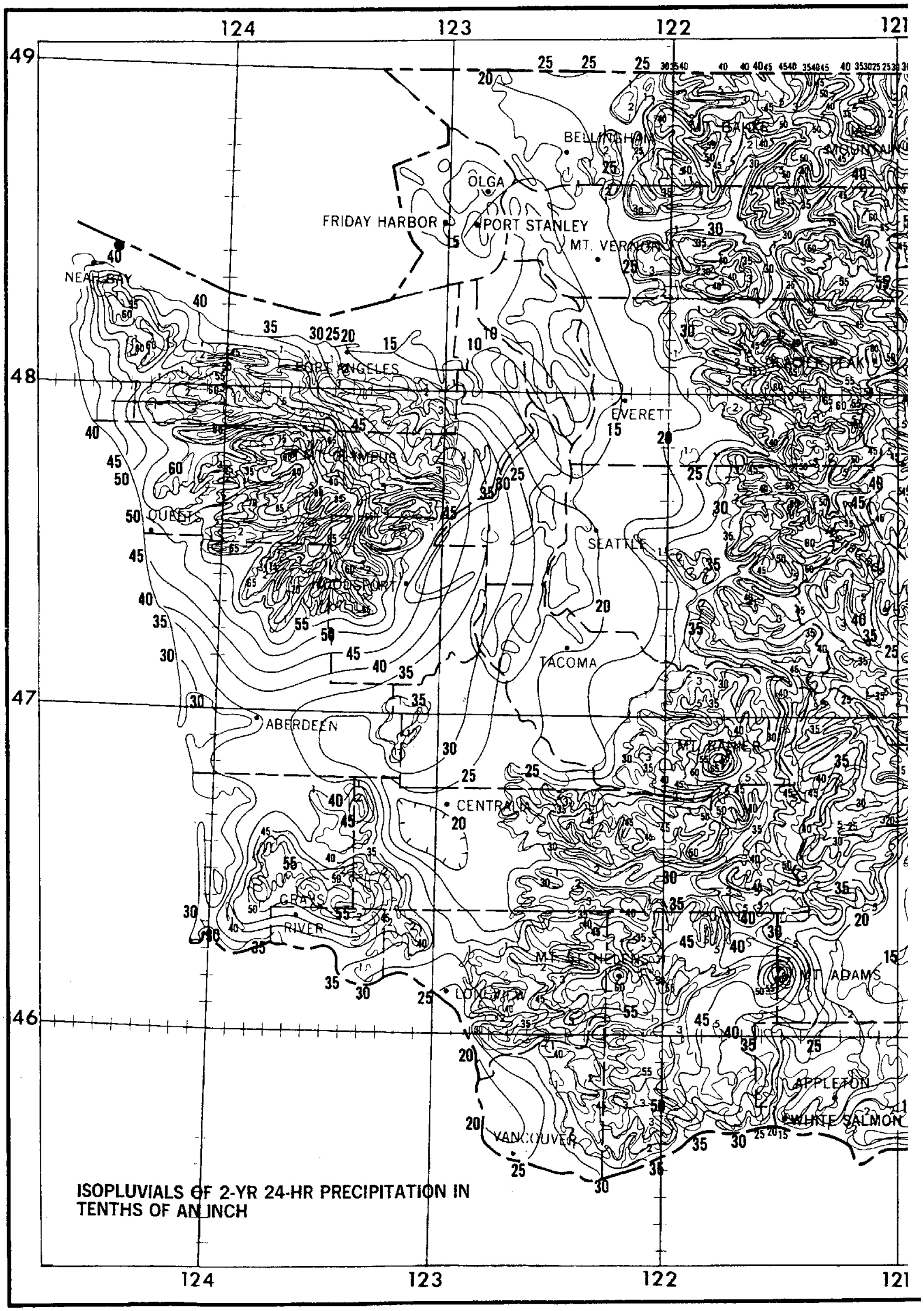
Wiltsie, Edward, Stormwater Facilities Performance Study, Infiltration Pond Testing and Data Evaluation, August 10, 1998

Woodward-Clyde, BMP Design Recommendations, November 1995

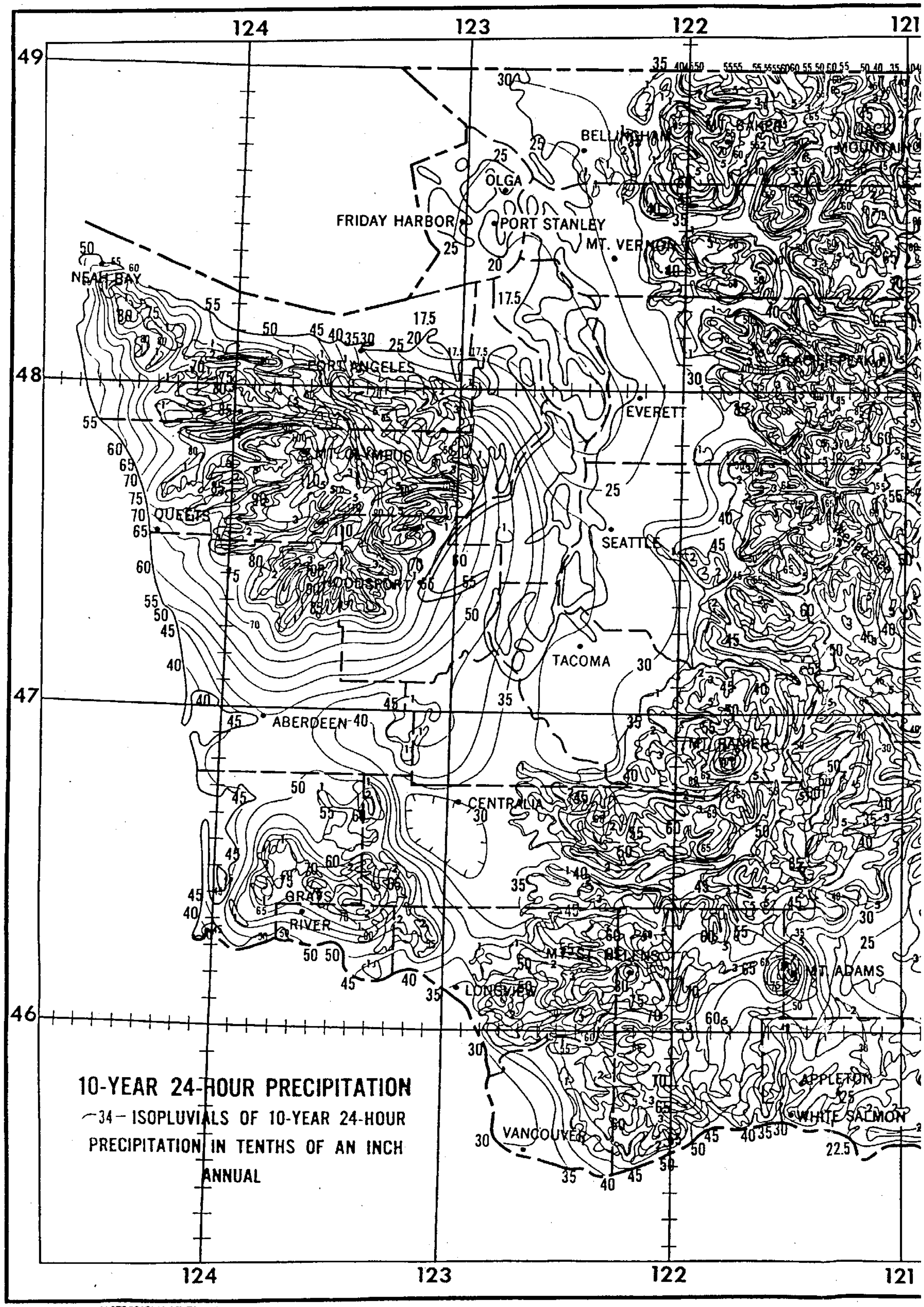
Appendix III-A
Isopluvial Maps for Design Storms

Included in this appendix are the 2, 10 and 100-year, 24-hour design storm and mean annual precipitation isopluvial maps for Western Washington. These have been taken from NOAA Atlas 2 "Precipitation - Frequency Atlas of the Western United States, Volume IX, Washington.

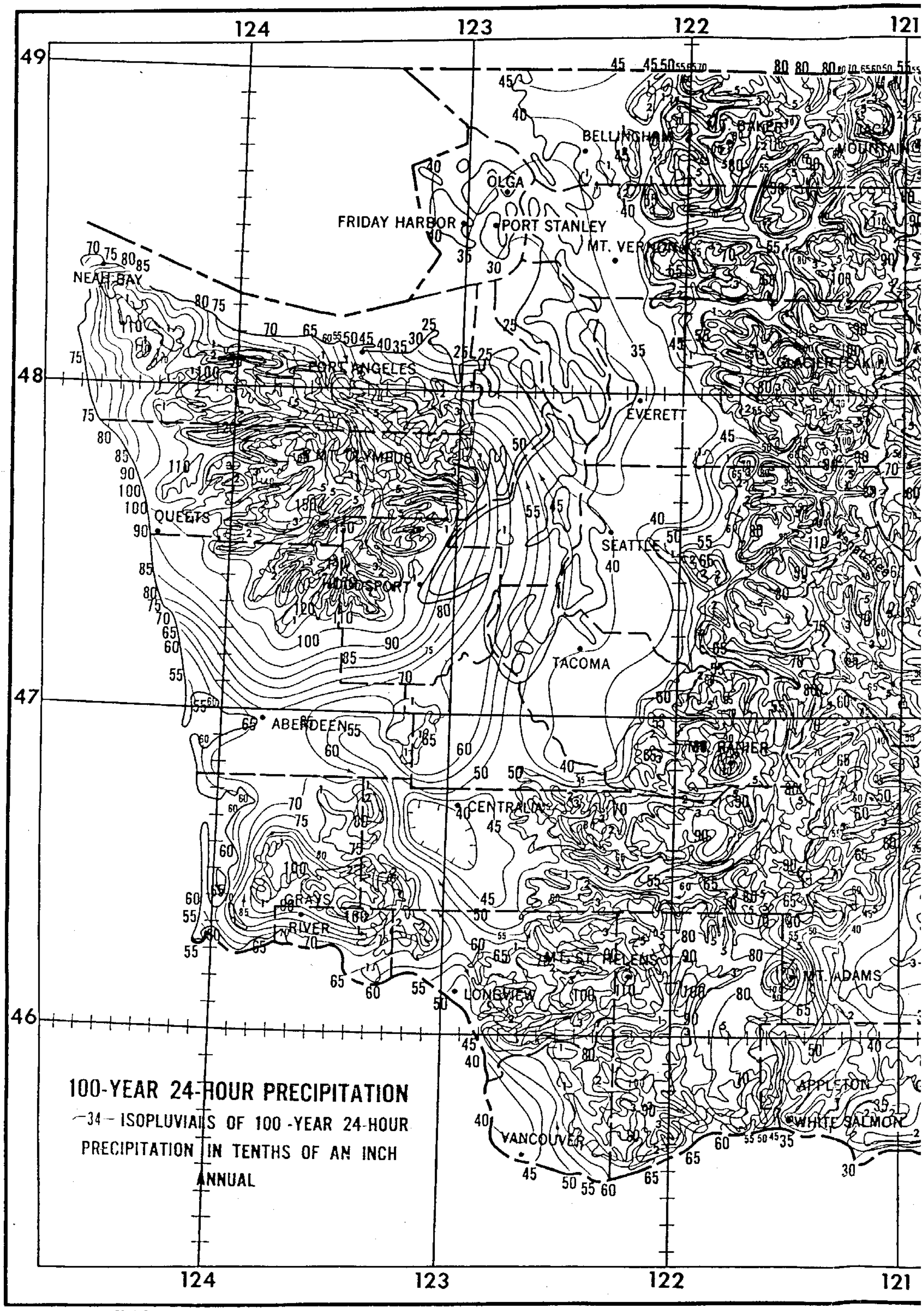
Western Washington Isopluvial 2-year, 24 hour



Western Washington Isopluvial 10-year, 24 hour



Western Washington Isopluvial 100-year, 24 hour



USDA-SCS-NATIONAL CARTOGRAPHIC CENTER, FT. WORTH, TX.-1986

Appendix III-B
Western Washington Hydrology Model – Information, Assumptions, and Computation Steps

The information and assumptions used in the Western Washington Hydrology Model (WWHM) are described in this document.

WWHM Limitations

The WWHM has been created for the specific purpose of sizing stormwater control facilities for new development and redevelopment projects in Western Washington. The WWHM can be used for a range of conditions and developments; however, certain limitations are inherent in this software. These limitations are described below.

The WWHM uses the EPA HSPF software program to do all of the rainfall-runoff and routing computations. Therefore, HSPF limitations are included in the WWHM. For example, backwater or tailwater control situations are not explicitly modeled by HSPF. This is also true in the WWHM.

In addition, the WWHM is limited in its routing capabilities. The user is allowed to input a single stormwater control facility and runoff is routed through this facility. If the proposed development site contains multiple facilities in series or involves routing through a natural lake, pond, or wetland in addition to a stormwater control facility then the user should use HSPF to do the routing computations and additional analysis.

Routing effects become more important as the drainage area increases. For this reason it is recommended that the WWHM not be used for drainage areas greater than one-half square mile (320 acres). The WWHM can be used for small drainage areas down to less than an acre in size.

WWHM Information and Assumptions

1. Precipitation data.

Length of record.
The WWHM uses long-term (43-50 years) precipitation data to simulate the potential impacts of land use development in western Washington. A minimum period of 20 years is required to simulate enough peak flow events to produce accurate flow frequency results. A 40 to 50-year record is preferred. The actual length of record of each precipitation station varies, but all exceed 43 years.

Rainfall distribution.
The precipitation data are representative of the different rainfall regimes found in western Washington. A total of 17 precipitation stations are used. These stations represent rainfall at elevations below 1500 feet. Snowfall and melt are not included in the WWHM.

The primary source for precipitation data is National Weather Service stations. The secondary source is precipitation data collected by local jurisdictions. During development of WWHM, county engineers at 19 western Washington counties were contacted to obtain local precipitation data. Only King County provided local data.

The following precipitation stations have been included in the WWHM:

Precipitation Station	Years of Data	County Coverage
Astoria, OR	1955-1998 = 43	Wahkiakum
Blaine	1948-1998 = 50	Whatcom, San Juan
Burlington	1948-1998 = 50	Skagit, Island
Clearwater	1948-1998 = 50	Jefferson (west)
Darrington	1948-1996 = 48	Snohomish (northeast)
Everett	1948-1996 = 48	Snohomish (excluding northeast)
Frances	1948-1998 = 50	Pacific
Landsburg	1948-1997 = 49	King (east)
Longview	1955-1998 = 43	Cowlitz, Lewis (south)
McMillian	1948-1998 = 50	Pierce
Montesano	1955-1998 = 43	Grays Harbor
Olympia	1955-1998 = 43	Thurston, Mason (south), Lewis (north)
Port Angeles	1948-1998 = 50	Clallam (east)
Portland, OR	1948-1998 = 50	Clark, Skamania
Quilcene	1948-1998 = 50	Jefferson (west), Mason (north), Kitsap
Sappho	1948-1998 = 50	Clallam (west)
SeaTac	1948-1997 = 49	King (west)

The records were reviewed for length, quality, and completeness of record. Annual totals were checked along with hourly maximum totals. Using these checks, data gaps and errors were corrected, where possible. A "Quality of Record" summary was produced for each precipitation record reviewed.

The reviewed and corrected data were placed in multiple WDM (Watershed Data Management) files. One WDM file was created per county and contains all of the precipitation data to be used by the WWHM for that particular county.

Computational time step.
The computational time step used in the WWHM is one hour. The one-hour time step was selected to better represent the temporal variability of actual precipitation than daily data.

2. Precipitation multiplication factors.

Precipitation multiplication factors increase or decrease recorded precipitation data to better represent local rainfall conditions. This is particularly important when the precipitation gage is located some distance from the study area.

Precipitation multiplication factors were developed for western Washington. The factors are based on the ratio of the 24-hour, 25-year rainfall intensities for the representative precipitation gage and the surrounding area represented by that gage's record. The 24-hour, 25-year rainfall intensities were determined from the NOAA Atlas 2 (*Precipitation-Frequency Atlas of the Western United States, Volume IX – Washington, 1973*).

These multiplication factors were created for the Puget Sound lowlands plus all western Washington valleys and hillside slopes below 1500 feet elevation. The factors were placed in the WWHM database and linked to each county's map. They are transparent to the general user. The advanced user will have the ability to change the precipitation multiplication factor for a specific site. However, such changes will be recorded in the WWHM output.

3. Pan evaporation data.

Pan evaporation data are used to determine the potential evapotranspiration (PET) of a study area. Actual evapotranspiration (AET) is computed by the WWHM based on PET and available moisture supply. AET accounts for the precipitation that returns to the atmosphere without becoming runoff. Soil moisture conditions and runoff are directly influenced by PET and AET.

Evaporation is not highly variable like rainfall. Puyallup pan evaporation data are used for all of the 19 western Washington counties.

Pan evaporation data were assembled and checked for the same time period as the precipitation data and placed in the appropriate county WDM files.

Pan evaporation data are collected in the field, but PET is used by the WWHM. PET is equal to pan evaporation times a pan evaporation coefficient. Depending on climate, pan evaporation coefficients for western Washington range from 0.72 to 0.82.

NOAA Technical Report NWS 33, *Evaporation Atlas for the Contiguous 48 United States*, was used as the source for the pan evaporation coefficients. Pan evaporation coefficient values are shown on Map 4 of that publication.

As with the precipitation multiplication factors, the pan evaporation coefficients have been placed in the WWHM database and linked to each county's map. They will be transparent to the general user. The advanced user will have the ability to change the coefficient for a specific site. However, such changes will be recorded in the WWHM output.

4. Soil data.

Soil type, along with vegetation type, greatly influences the rate and timing of the transformation of rainfall to runoff. Sandy soils with high infiltration rates produce little or no surface runoff; almost all runoff is from groundwater. Soils with a compressed till layer slowly infiltrate water and produce larger amounts of surface runoff during storm events.

The WWHM uses three predominate soil type to represent the soils of western Washington: till, outwash, and saturated

Till soils have been compacted by glacial action. Under a layer of newly formed soil lies a compressed soil layer commonly called "hardpan". This hardpan has very poor infiltration capacity. As a result, till soils produce a relatively large amount of surface runoff and interflow. A typical example of a till soil is an Alderwood soil (SCS class C).

Outwash soils have a high infiltration capacity due to their sand and gravel composition. Outwash soils have little or no surface runoff or interflow. Instead, almost of their runoff is in the form of groundwater. An Everett soil (SCS class A) is a typical outwash soil.

Outwash soils over high groundwater or an impervious soil layer have low infiltration rates and act like till soils. Where groundwater or an impervious soil layer is within 5 feet from the surface, outwash soils may be modeled as till soils in the WWHM.

Saturated soils are usually found in wetlands. They have a low infiltration rate and a high groundwater table. When dry, saturated soils have a high storage capacity and produce very little runoff. However, once they become saturated they produce surface runoff, interflow, and groundwater in large quantities. Mukilteo muck (SCS class D) is a typical saturated soil.

The user will be required to investigate actual local soil conditions for the specific development planned. The user will then input the number of acres of outwash (A/B), till (C), and saturated (D) soils for the site conditions.

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 may be modeled as till soils.

Additional soils will be included in the WWHM if appropriate HSPF parameter values are found to represent other major soil groups.

The three predominate soil types are represented in the WWHM by specific HSPF parameter values that represent the hydrologic characteristics of these soils. More information on these parameter values is presented below.

5. Vegetation data.

As with soil type, vegetation types greatly influence the rate and timing of the transformation of rainfall to runoff. Vegetation intercepts precipitation, increases its ability to percolate through the soil, and evaporates and transpires large volumes of water that would otherwise become runoff.

The WWHM will represent the vegetation of western Washington with three predominate vegetation categories: forest, pasture, and lawn (also known as grass).

Forest vegetation represents the typical second growth Douglas fir found in the Puget Sound lowlands. Forest has a large interception storage capacity. This means that a large amount of precipitation is caught in the forest canopy before reaching the ground and becoming available for runoff. Precipitation intercepted in this way is later evaporated back into the atmosphere. Forest also has the ability to transpire moisture from the soil via its root system. This leaves less water available for runoff.

Pasture vegetation is typically found in rural areas where the forest has been cleared and replaced with shrub or grass lots. Some pasture areas may be used to graze livestock. The interception storage and soil evapotranspiration capacity of pasture are less than forest. Soils may have also been compressed by mechanized equipment during clearing activities. Livestock can also compact soil. Pasture areas typically produce more runoff (particularly surface runoff and interflow) than forest areas.

Lawn vegetation is representative of the suburban vegetation found in typical residential developments. Soils have been compacted by earth moving equipment, often with a layer of topsoil removed. Sod and ornamental bushes replace native vegetation. The interception storage and evapotranspiration of lawn vegetation is less than pasture. More runoff results.

Predevelopment default land conditions are forest, although the user has the option of specifying pasture if there is documented evidence that pasture vegetation was native to the predevelopment site. If this option is used, the change will be recorded in the WWHM output.

Forest vegetation is represented by specific HSPF parameter values that represent the forest hydrologic characteristics. As described above, the existing regional HSPF parameter values for forest are based on undisturbed second-growth Douglas fir forest found today in western Washington lowland watersheds.

Postdevelopment vegetation will reflect the new vegetation planned for the site. The user has the choice of forest, pasture, and landscaped vegetation. Forest and pasture are only appropriate for postdevelopment vegetation in parcels separate from standard residential or non-standard residential/commercial. Development areas must only be designated as forest or pasture where legal restrictions can be documented that protect these areas from future disturbances. The WWHM assumes the pervious land portion of the standard residential and non-standard residential/commercial is covered with lawn vegetation, as described above.

6. Development land use data.

Development land use data are used to represent the type of development planned for the site and are used to determine the appropriate size of the required stormwater mitigation facility.

For the purposes of the WWHM in western Washington developed land is divided into two major categories:

1. standard residential, and
2. non-standard residential/commercial.

Standard residential

Standard residential development makes specific assumptions about the amount of impervious area per lot and its division between driveways and rooftops. Streets and sidewalk areas are input separately. Ecology has selected a standard impervious area of 4200 square feet per residential lot, with 1000 square feet of that as driveway, walkways, and patio area, and the remainder as rooftop area.

Impervious, as the name implies, allows no infiltration of water into the pervious soil. All runoff is surface runoff. Impervious land typically consists of paved roads, sidewalks, driveways, and parking lots. Roofs are also impervious.

For the purposes of hydrologic modeling, only effective impervious area is categorized as impervious. Effective impervious area (EIA) is the area where there is no opportunity for surface runoff from an impervious site to infiltrate into the soil before it reaches a conveyance system (pipe, ditch, stream, etc.). An example of an EIA is a shopping center parking lot where the water runs off the pavement and directly goes into a catch basin where it then flows into a pipe and eventually to a stream. In contrast, some homes with impervious roofs collect the roof runoff into roof gutters and send the water down downspouts. When the water reaches the base of the downspout it can be directed either into a pipe or dumped on a splash block. Roof water dumped on a splash block then has the opportunity to spread out into the yard and soak into the soil. Such roofs are not considered to be effective impervious area. For hydrologic modeling purposes, runoff credits are given to developments that contain houses that have roof runoff systems that disperse roof runoff and allow it to drain into the soil. A runoff credit is given by assuming in the modeling that the roof area behaves hydrologically as lawn rather than EIA.

The non-effective impervious area uses the adjacent or underlying soil and vegetation properties. Vegetation often varies by the type of land use. Standard residential and non-standard residential/commercial are both assumed to have lawn as their typical pervious area vegetation.

The assumption is made in the WWHM that the EIA equals the TIA (total impervious area). This is consistent with King County's determination of EIA acres for new developments. Where appropriate, the TIA can be reduced through the use of runoff credits (more on that below).

For standard residential developments the user will input the impervious area in the public right-of-way (streets and sidewalks). In addition, the user will input the number of residential lots and the number of acres associated directly with these residential lots (public right-of-way acreages and non-residential lot acreages excluded). The number of residential lots and the associated number of acres will be used to compute the average number of residential lots per acre. This value together with the number of residential lots and the impervious area in the public right-of-way will be used by the model to calculate the TIA for the proposed development.

Runoff credits will be given reducing runoff from standard residential lots. Runoff credits can be obtained using any or all of the three methods described below.

1. Infiltrate roof runoff
2. Disperse roof runoff
3. Use porous pavement for driveway areas

Credit is given for disconnecting the roof runoff from the development's stormwater conveyance system and infiltrating on the individual residential lots. The WWHM assumes that this infiltrated roof runoff does not contribute to the runoff flowing to the stormwater detention pond site. It disappears from the system and does not have to be mitigated.

Credit is also given for disconnecting the roof runoff from the development's stormwater conveyance system and dispersing it on the surface of individual lots. This runoff is assumed to be the equivalent of runoff from lawn vegetation.

The third option for runoff credit is the use of porous pavement for private driveway areas. Specific HSPF parameters for porous pavement have not been developed for the WWHM. Ecology has made the assumption that porous pavement runoff is the equivalent to the conversion of 147 square feet (1000*0.147) of impervious area to lawn vegetation. This assumption is used in the WWHM calculations.

Forest and pasture vegetation areas are only appropriate for separate undeveloped parcels dedicated as open space, wetland buffer, or park within the total area of the standard residential development. ***Development areas must only be designated as forest or pasture where legal restrictions can be documented that protect these areas from future disturbances.***

<u>Non-standard residential/commercial</u>

Non-standard residential/commercial development includes residential developments for which the standard residential developments assumptions are inappropriate, plus commercial, industrial, schools, roads, multi-family residential (apartments, condos), and other non-single family residential developments. For this type of development the user will input the roof area, landscape area, street/sidewalk/parking areas, and any appropriate non-developed forest and pasture areas. Developed runoff will be calculated based on these categories and their areas. The only explicit runoff credit available to the user is porous pavement for streets, sidewalks, and parking lots. The credit works the same way as for standard residential. It is specified as 14.7% of the total street/sidewalk/ parking impervious area. The user can also implicitly obtain other runoff credits by decreasing or eliminating roof area where roof runoff is infiltrated or by modeling roof area as lawn where roof runoff is dispersed in accordance with the infiltration and dispersion requirements in Chapter 3. This will decrease surface runoff.

Forest and pasture vegetation areas are only appropriate for separate undeveloped parcels dedicated as open space, wetland buffer, or park within the total area of the development. ***Development areas must only be designated as forest or pasture where legal restrictions can be documented that protect these areas from future disturbances.***

Other Development Options and Model Features

The WWHM allows the flexibility of bypassing a portion of the development area around a flow control facility and/or having offsite inflow that is entering the development area pass through the flow control facility. Three options are available to the user:

A. Design Basin: usual development situation with no offsite inflow and no flow bypass.
B. Bypass: a portion of the development does not drain to a stormwater detention facility. Onsite runoff from a proposed development project may bypass the flow control facility provided that all of the following conditions are met.
    1. Runoff from both the bypass area and the flow control facility converges within a quarter-mile downstream of the project site discharge point, and
    2. The flow control facility is designed to compensate for the uncontrolled bypass area such that the net effect at the point of convergence downstream is the same with or without bypass, and
    3. The 100-year peak discharge from the bypass area will not exceed 0.4 cfs, and
    4. Runoff from the bypass area will not create a significant adverse impact to downstream drainage systems or properties, and
    5. Water quality requirements applicable to the bypass area are met.

C. Offsite Inflow: an upslope area outside the development drains to the flow control facility in the development. If the existing 100-year peak flow rate from any upstream offsite area is greater than 50% of the 100-year developed peak flow rate (undetained) for the project site, then the runoff from the offsite area must not flow to the onsite flow control facility. The bypass of offsite runoff must be designed so as to achieve the following:
    1. Any existing contribution of flows to an onsite wetland must be maintained, and
    2. Offsite flows that are naturally attenuated by the project site under predeveloped conditions must remain attenuated, either by natural means or by providing additional onsite detention so that peak flows do not increase.

For each of these options the user inputs the number of acres in the different categories for the predevelopment and postdevelopment land use. The WWHM computes the runoff from each separately and adds them together, as appropriate, to check to see if the stormwater standards have been satisfied. The following

WWHM uses this information to compute the corresponding number of acres of pervious and impervious land and assign these acres to specific PERLNDs and IMPLNDs. These terms are HSPF-speak for pervious land categories and impervious land categories. WWHM uses the PERLNDS and IMPLNDS to compute postdevelopment runoff.

Application of WWHM in Re-developments Projects

WWHM allows only forest or pasture as the predevelopment land condition in the Design Basin screen. This screen does not allow other types of land uses such as impervious and landscaped areas to be entered for existing condition. However, WWHM can be used for redevelopment projects by modeling the existing developed areas that are not subject to the flow control requirements of Volume I as offsite areas. For the purposes of predicting runoff from such an existing developed area, enter the existing area in the Offsite Inflow screen. This screen is designed to predict runoff from impervious and landscaped areas in addition to the forest and pasture areas. If the existing 100-year peak flow rate from the existing developed areas that are not subject to flow control is greater than 50% of the 100-year developed peak flow rate (undetained but subject to the flow control requirements of Volume I), then the runoff from the offsite area must not be allowed to flow to the onsite flow control facility.

7. PERLND and IMPLND parameter values.

In WWHM (and HSPF) pervious land categories are represented by PERLNDs; impervious land categories (EIA) by IMPLNDs. An example of a PERLND is a till soil covered with forest vegetation. This PERLND has a unique set of HSPF parameter values. For each PERLND there are 16 parameters that describe various hydrologic factors that influence runoff. These range from interception storage to infiltration to active groundwater evapotranspiration. Only four parameters are required to represent IMPLND.

The PERLND and IMPLND parameter values to be used in the WWHM are listed below. These values are based on regional parameter values developed by the U.S. Geological Survey for watersheds in western Washington (Dinicola, 1990) plus additional HSPF modeling work conducted by AQUA TERRA Consultants.

PERLND Parameters

	TF	TP	TL	OF	OP	OL	SF	SP	SL
Name									
LZSN	4.5	4.5	4.5	5.0	5.0	5.0	4.0	4.0	4.0
INFILT	0.08	0.06	0.03	2.0	1.6	0.80	2.0	1.8	1.0
LSUR	400	400	400	400	400	400	100	100	100
SLSUR	0.10	0.10	0.10	0.10	0.10	0.10	0.001	0.001	0.001
KVARY	0.5	0.5	0.5	0.3	0.3	0.3	0.5	0.5	0.5
AGWRC	0.996	0.996	0.996	0.996	0.996	0.996	0.996	0.996	0.996
INFEXP	2.0	2.0	2.0	2.0	2.0	2.0	10.0	10.0	10.0
INFILD	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0
BASETP	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
AGWETP	0.0	0.0	0.0	0.0	0.0	0.0	0.7	0.7	0.7
CEPSC	0.20	0.15	0.10	0.20	0.15	0.10	0.18	0.15	0.10
UZSN	0.5	0.4	0.25	0.5	0.5	0.5	3.0	3.0	3.0
NSUR	0.35	0.30	0.25	0.35	0.30	0.25	0.50	0.50	0.50
INTFW	6.0	6.0	6.0	0.0	0.0	0.0	1.0	1.0	1.0
IRC	0.5	0.5	0.5	0.7	0.7	0.7	0.7	0.7	0.7
LZETP	0.7	0.4	0.25	0.7	0.4	0.25	0.8	0.8	0.8

PERLND types:

TF = Till Forest
TP = Till Pasture
TL = Till Lawn
OF = Outwash Forest
OP = Outwash Pasture
OL = Outwash Lawn
SF = Saturated Forest
SP = Saturated Pasture
SL = Saturated Lawn

PERLND parameters:

LZSN = lower zone storage nominal (inches)
INFILT = infiltration capacity (inches/hour)
LSUR = length of surface overland flow plane (feet)
SLSUR = slope of surface overland flow plane (feet/feet)
KVARY = groundwater exponent variable ($inch^{-1}$)
AGWRC = active groundwater recession constant (day^{-1})
INFEXP = infiltration exponent
INFILD = ratio of maximum to mean infiltration
BASETP = base flow evapotranspiration (fraction)
AGWETP = active groundwater evapotranspiration (fraction)
CEPSC = interception storage (inches)
UZSN = upper zone storage nominal (inches)
NSUR = roughness of surface overland flow plane (Manning's n)
INTFW = interflow index
IRC = interflow recession constant (day^{-1})
LZETP = lower zone evapotranspiration (fraction)

A more complete description of these PERLND parameters is found in the HSPF User Manual (Bicknell et al, 1997).

PERLND parameter values for other additional soil/vegetation categories will be investigated and added to the WWHM, as appropriate.

IMPLND Parameters

	EIA
Name	
LSUR	400
SLSUR	0.01
NSUR	0.10
RETSC	0.10

IMPLND parameters:

LSUR = length of surface overland flow plane (feet)
SLSUR = slope of surface overland flow plane (feet/feet)
NSUR = roughness of surface overland flow plane (Manning's n)
RETSC = retention storage (inches)

A more complete description of these IMPLND parameters is found in the HSPF User Manual (Bicknell et al, 1997).

The PERLND and IMPLND parameter values will be transparent to the general user. The advanced user will have the ability to change the value of a particular parameter for that specific site. However, such changes will be recorded in the WWHM output.

Surface runoff and interflow will be computed based on the PERLND and IMPLND parameter values. Groundwater flow will not be computed, as it is assumed that there is no groundwater flow from small catchments that reaches the surface to become runoff. This is consistent with King County procedures (King County, 1998).

8. Guidance for flow control standards.

Flow control standards are used to determine whether or not a proposed stormwater facility will provide a sufficient level of mitigation for the additional runoff from land development. Guidance is provided on the standards that must be met to comply with the Ecology Stormwater Management Manual.

There are two flow control standards stated in the Ecology Manual: Minimum Requirement #7 - Flow Control and Minimum Requirement #8 - Wetlands Protection (See Volume I). Minimum Requirement #7 specifies flow frequency and flow duration ranges for which the postdevelopment runoff cannot exceed predevelopment runoff. Minimum Requirement #8 specifies that discharges to wetlands must maintain the hydrologic conditions, hydrophytic vegetation, and substrate characteristics necessary to support existing and designated beneficial uses.

Minimum Requirement #7 specifies that stormwater discharges to streams shall match developed discharge durations to predeveloped durations for the range of predeveloped discharge rates from 50% of the 2-year peak flow up to the full 50-year peak flow. In addition, the developed peak discharge rates should not exceed the predeveloped peak discharge rates for 2-, 10-, and 50-year return periods. In general, matching discharge durations between 50% of the 2-year and 50-year will result in matching the peak discharge rates in this range.

The WWHM uses the predevelopment peak flow value for each water year to compute the predevelopment 2- through 100-year flow frequency values. The postdevelopment runoff 2- through 100-year flow frequency values are computed from the outlet of the proposed stormwater facility. The user must enter the stage-surface area-storage-discharge table (HSPF FTABLE) for the stormwater facility. The model then routes the postdevelopment runoff through the stormwater facility. As with the predevelopment peak flow values, the maximum developed flow value for each water year will be selected by the model to compute the developed 2- through 100-year flow frequency.

The actual flow frequency calculations are made using the federal standard Log Pearson Type III distribution described in Bulletin 17B (United States Water Resources Council, 1981). This standard flow frequency distribution is provided in U.S. Geological Survey program J407, version 3.9A-P, revised 8/9/89. The Bulletin 17B algorithms in program J407 are included in the WWHM calculations.

The WWHM compares the postdevelopment 2-year flow frequency value with the predevelopment flood value. If the postdevelopment value is greater than the predevelopment value then flow frequency requirement has not been met. The same test is conducted for the 5-, 10-, 25-, 50-, and 100-year flow frequency values. The results are reported in the WWHM output.

The second half of the Minimum Requirement #7 is based on flow duration. The WWHM will use the entire predevelopment and postdevelopment runoff record to compute flow duration. The standard requires that postdevelopment runoff flows must not exceed the flow duration values of the predevelopment runoff between the predevelopment flow values of 50 percent of the 2-year flow and 100 percent of the 50-year flow.

Flow duration is computed by counting the number of flow values that exceed a specified flow level. The specified flow levels used by WWHM in the flow duration analysis are listed below.

1. 50% of the 2-year predevelopment peak flow.
2. 100% of the 2-year predevelopment peak flow.
3. 100% of the 50-year predevelopment peak flow.

In addition, flow durations are computed for 97 other incremental flow values between 50 percent of the 2-year predevelopment peak flow and 100 percent of the 50-year predevelopment peak flow.

There are three criteria by which flow duration values are compared:

1. If the postdevelopment flow duration values exceed any of the predevelopment flow levels between 50% and 100% of the 2-year predevelopment peak flow values (100 Percent Threshold) then the flow duration requirement has not been met.
2. If the postdevelopment flow duration values exceed any of the predevelopment flow levels between 100% of the 2-year and 100% of the 50-year predevelopment peak flow values more than 10 percent of the time (110 Percent Threshold) then the flow duration requirement has not been met.
3. If more than 50 percent of the flow duration levels exceed the 100 percent threshold then the flow duration requirement has not been met.

The results are provided in the WWHM report.

Minimum Requirement #8 specifies that discharges to wetlands must maintain the hydrologic conditions, hydrophytic vegetation, and substrate characteristics necessary to support existing and designated beneficial uses. Criteria for determining maximum allowed exceedences in alterations to wetland hydroperiods are provided in guidelines cited in Guide Sheet 2B of the Puget Sound Wetland Guidelines (Azous and Horner, 1997).

Because wetland hydroperiod computations are relatively complex and are site specific, they will not be included in the WWHM. HSPF is required for wetland hydroperiod analysis. Ecology intends to determine the appropriate steps in using HSPF to compute hydroperiod based on joint recommendations from wetland biologists and HSPF modelers.

WWHM Computation Steps

STEP 1. Location project site.
Drag flag to project site on map.
Zoom in or out on map, as needed.
Click on NEXT.

STEP 2. Input land area data.
Type in Name of Development, Development Address, City/County, and Project Description.
Select Standard Residential or Non-standard/Commercial.
For Standard Residential:

- For Design Basin (project site) input number of acres of
    - Predeveloped Soils
        - Outwash (A&B)
        - Till (C)
        - Saturated (D)
    - Select Predeveloped Vegetation (Forest or Pasture)
    - Residential Acres
        - Lot Acres (excluding public right-of-ways and non-residential parcels)
        - Streets/Sidewalks (public right-of-ways)
        - Forest (undeveloped parcels only)
        - Pasture (undeveloped parcels only)
        - Landscaped Area (parks, etc., excluding residential lots)
        - Each of the above must be separated into A/B soils and C soils.
        - (NOTE: Saturated (D) soil areas should not be included in the developed area acres as these are wetland areas.)
        - Number of Lots (corresponding to the total Lot Acres)
        - (NOTE: the user is allowed a maximum of 10 lots per acre.)
        - Roof Runoff Credits (if appropriate)
            - Infiltrate (percent of lots)
            - Disperse (percent of lots)
        - Pavement Credit (if appropriate)
            - Porous Pavement (private pavement areas only)
- Repeat information (if necessary) for Bypass.
- Repeat information (if necessary) for Offsite Inflow.
- Click NEXT.
- Compute Runoff now? Click Yes.

For Non-standard/Commercial:

- For Design Basin (project site) input number of acres of
    - Predeveloped Soils
        - Outwash (A&B)
        - Till (C)
        - Saturated (D)
    - Select Predeveloped Vegetation (Forest or Pasture)
    - Non-standard Residential/Commercial
        - Impervious Area (Roof acres)

Landscaped Area (pervious acres)
Streets/Sidewalks/Parking
Forest (undeveloped parcels only)
Pasture (undeveloped parcels only)
Each of the above must be separated into A/B soils and C soils.
(NOTE: Saturated (D) soil areas should not be included in the developed area acres as these are wetland areas.)
Pavement Credit (if appropriate)
Porous Pavement (percent of total Streets/Sidewalks/Parking)

Repeat information (if necessary) for Bypass.
Repeat information (if necessary) for Offsite Inflow.
Click NEXT.
Compute Runoff now? Click Yes.

STEP 3. Compute runoff.
HSPF executes for predevelopment and postdevelopment conditions.
Click NEXT.

STEP 4. Perform flow frequency analysis.
Click PERFORM to compute flow frequency values for predevelopment (black), flow duration values for predevelopment (red), and flow frequency values for postdevelopment without stormwater detention facility (blue).

2-Year to 100-Year flow frequency results are reported for the predevelopment and postdevelopment without detention facility conditions. If the difference in the 100-year flow frequency values is less than 0.1 cfs then no facility is required.

If needed to size a stormwater facility, click EXPORT to export either predevelopment, postdevelopment (without routing through stormwater facility), or postdevelopment (with routing through stormwater facility) runoff time series to a separate file.

Click NEXT.

STEP 5. Check stormwater detention facility.
Enter Facility Name/ID and Type of Facility.
Add Table from File or enter manually the following information:

Stage/depth (feet) starting zero
Area (acres) of surface corresponding to stage, starting at zero
Storage (acre-feet) of facility corresponding to stage, starting at zero
Discharge1 (cfs) from facility to surface flow, corresponding to stage, starting at zero

If the stormwater facility is an infiltration facility then click on Infiltration and enter:

Discharge2 (cfs) from facility to subsurface flow, corresponding to stage, starting at zero

NOTE:

(1) the first row of values must be zero;
(2) stage must increase from one row to next;
(3) storage must increase from one row to next;
(4) maximum number of total values cannot exceed 500.

Click NEXT.
Compute Runoff now? Click Yes.
Postdevelopment runoff is routed through the stormwater facility.
Click NEXT.

STEP 6. Perform flow frequency analysis for postdevelopment runoff with stormwater detention facility.
Click NEXT.

STEP 7. Compare flow duration statistics for predevelopment and postdevelopment runoff with stormwater detention facility.
Click lower PERFORM to compute flow frequency values for postdevelopment with stormwater detention facility (black) and flow duration values for postdevelopment with facility (red).

2-Year to 100-Year flow frequency results are reported for the postdevelopment with detention facility. Compare with predevelopment conditions.

Change 110 Percent Threshold (default value) if required by Ecology or local jurisdiction, otherwise use default value.

Click Perform Statistical Analysis to compare predevelopment and postdevelopment flow duration values.

Identify which values passed or failed the test.
Click NEXT.

STEP 8. Ecology Flow Control Standard Summary.
Summary information shows whether the stormwater facility passes or fails the Ecology flow control standard.

To view a complete listing of the input and output information:
(1) save the file (File, Save Project As, enter File name, Save)
(2) click Generate Report
(3) view and/or print Report

To exit the WWHM: File, Exit

References for Western Washington Hydrology Model

Beyerlein, D.C. 1996. Effective Impervious Area: The Real Enemy. Presented at the Impervious Surface Reduction Research Symposium, The Evergreen State College. Olympia, WA.

Bicknell, B.R., J.C. Imhoff, J.L. Kittle Jr, A.S. Donigian Jr, and R.C. Johanson. 1997. Hydrological Simulation Program – Fortran User's Manual for Version 11. EPA/600/R-97/080. National Exposure Research Laboratory. Office of Research and Development. U.S. Environmental Protection Agency. Research Triangle Park, NC.

Dinicola, R.S. 1990. Characterization and Simulation of Rainfall-Runoff Relations for Headwater Basins in Western King and Snohomish Counties, Washington. Water-Resources Investigations Report 89-4052. U.S. Geological Survey. Tacoma, WA.

King County. 1998. Surface Water Design Manual. Department of Natural Resources. Seattle, WA.

United States Water Resources Council. 1981. Guidelines for Determining Flood Flow Frequency. Bulletin #17B of the Hydrology Committee. Washington, DC.