City of Tumwater
Drainage Design and Erosion Control Manual

Volume III
Hydrologic Analysis Methods and Conveyance Design

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Prepared for
City of Tumwater
Public Works Department

Note:
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Volume III – Hydrologic Analysis Methods and Conveyance Design

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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 on waters of Washington State. As described in Volume I of this manual, BMPs for long-term management of stormwater at developed sites can be divided into three main categories:

1. BMPs addressing the volume and timing of stormwater flows
2. BMPs addressing prevention of pollution from potential sources
3. 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 for 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.

This volume details Tumwater's policies regarding the quantity control of runoff from developed or artificially altered sites. The scope of this volume includes:

- Design criteria and specifications for the construction of runoff quantity control facilities
- Approved methods for estimating peak flow rates, volume of runoff, required input data, and required storage volumes based on site conditions
- Approved materials for use in private and public drainage facilities.

The intent of this volume is to prescribe approved methods and requirements for runoff control to prevent impacts on downstream properties or natural resources to the maximum extent practical. The City of Tumwater (city) recognizes that it is not always possible to fully prevent any impacts downstream; in such extreme cases, the project applicant may be required to provide off-site mitigation as determined by the city.

These regulations and criteria are based on fundamental principles of drainage, hydraulics, hydrology, environmental considerations and publications, manuals, and texts accepted by the professional engineering community. The engineer is responsible for being knowledgeable and proficient with the necessary design methodologies identified within the manual. Here is a partial listing of publications that may be used as reference documents:

- Washington State Department of Ecology (Ecology) 2012 Stormwater Management Manual for Western Washington, as Amended in December 2014 (2014 Ecology Manual; Ecology 2014)
- Any Ecology-approved stormwater management manual
- Low Impact Development: Technical Guidance Manual for Puget Sound, by Washington State University Extension and Puget Sound Partnership (WSU and PSP 2012)
- *Applied Handbook of Hydrology* (Singh 2016)
- *Handbook of Hydraulics* (Brater and King 1996)
- The following references published by the Washington State Department of Transportation (WSDOT):
    - Hydraulics Manual
    - Standard Specifications for Road, Bridge, and Municipal Construction
    - Standard Plans
- *Soil Survey of Thurston County, Washington* published by the U.S. Department of Agriculture (USDA) Soil Conservation Service (SCS 1990). (Also refer to the Natural Resources Conservation Service [NRCS] Web Soil Survey at <http://websoilsurvey.sc.egov.usda.gov>.)
- City of Tumwater Road Standards, latest amendment
- Other information sources acceptable to the city and based on general use by the professional engineering community.

The most current edition of all publications shall be used.

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** discusses hydrologic design standards and methods of hydrologic analysis, including the use of hydrograph methods for designing BMPs, an

overview of various computerized modeling methods, and analysis of closed depressions.

- **Chapter 3** describes flow control BMPs and provides design specifications for infiltration, detention, and retention facilities. This volume's focus is on flow control. Additional water quality design considerations are addressed in Volume V.

This volume includes two appendices. Appendix III-A describes the Santa Barbara Urban Hydrograph (SBUH) and Soil Conservation Service Unit Hydrograph (SCSUH) computer models, as well as the Rational Method. Appendix III-B includes number of charts and tables useful in designing conveyance systems with non-continuous hydrologic models. They include: design storm rainfall totals, isopluvial maps for western Washington, common Tumwater soil types and hydrologic groupings, NRCS curve numbers, and roughness coefficients. Appendix III-B includes several nomographs that may be useful for culvert sizing.

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 Volume I, Chapter 2). After the minimum requirements have been determined, this volume should be consulted to design flow management facilities. These facilities can then be included in any required stormwater plan submittals (see Volume I, Chapter 3).

Chapter 2 – Hydrologic Analysis Methods and Requirements

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:

- For designing runoff treatment BMPs, a calibrated continuous simulation hydrologic model based on the U.S. Environmental Protection Agency's (U.S. EPA) Hydrological Simulation Program-Fortran (HSPF) program, or an approved equivalent model (e.g., the Western Washington Hydrology Model [WWHM] or MGSFlood), must be used to calculate runoff and determine the water quality design flow rates and volumes. Acceptable methodologies are described in detail below. Design standards and sizing criteria for water quality BMPs are provided in Volume V.

- For designing flow control BMPs, a calibrated continuous simulation runoff model based on the U.S. EPA's HSPF, or an approved equivalent model (e.g., WWHM or MGSFlood), must be used. Flow control BMP criteria are discussed in detail in Volume V. The circumstances under which different methodologies apply are summarized in Table 2.1, below.

- For conveyance system design, the designer may use an approved continuous simulation runoff model or a single event hydrologic model to determine the peak flow rate. The peak flow rate from a continuous runoff model will vary depending on the time step used in the model. Therefore, the length of the time step must be sufficiently short relative to the time of concentration of the watershed to provide for reasonable conveyance system design flows. For most situations in Tumwater, a 15-minute (maximum) time step will be sufficient for conveyance system design. If the project is in a predominantly urbanized watershed with a time of concentration less than about 15 minutes (roughly 10 acres in size), the conveyance design must either use a 5-minute time step (if available), or use an event-based model for conveyance sizing. Conveyance design is discussed in detail in Chapter 3.

- Ecology has developed the HSPF-based WWHM. By default, WWHM uses rainfall/runoff relationships developed for specific basins in the Puget Sound region to all parts of western Washington.

- One other HSPF-based continuous runoff model has been approved by Ecology for use in the city of Tumwater: MGSFlood. Though MGSFlood uses different, extended precipitation files, its features and (more importantly) its runoff estimations are very similar to those predicted by WWHM.

- Use of other continuous simulation runoff models must receive prior concurrence from the city before being used for facility design.
- Free WWHM software and documentation can be found at the Ecology web site: https://ecology.wa.gov/Regulations-Permits/Guidance-technical-assistance/Stormwater-permittee-guidance-resources/Stormwater
- A professional version of WWHM with expanded capabilities can be purchased from Clear Creek Solutions, Inc. at <http://www.clearcreeksolutions.com/>.
- MGSFlood software can be downloaded for evaluation or purchase from MGS Engineers LLC at <http://www.mgsengr.com/mgsfloodhome.html>.

Table 2.1. Summary of the Application Design Methodologies.

Method	BMP/Conveyance Designs in Western Washington		
	Treatment	**Flow Control**	**Conveyance**
Continuous Runoff Models (WWHM or approved alternatives. See below.)	Method applies to all BMPs	Method applies throughout western Washington	Method applies throughout western Washington
Soil Conservation Service Unit Hydrograph (SCSUH)/ Santa Barbara Urban Hydrograph (SBUH)	Not applicable	Not applicable	Acceptable
Rational Method	Not applicable	Not applicable	Acceptable for certain conveyance design only (see Section 2.2.7)

Where large master-planned developments are proposed, the city may require a basin-specific calibration of HSPF rather than use of the default parameters in the above-referenced models. Basin-specific calibrations may be required for projects that will occupy more than 320 acres.

2.2 Hydrologic Analysis Methods Used for Runoff Modeling and BMP Design

This section provides a discussion of the methodologies to be used for calculating stormwater runoff from a project site. It includes a discussion of modeling to demonstrate achievement of flow control and runoff treatment standards, as well as descriptions of general modeling methodologies including continuous simulation models and single event models, such as the Santa Barbara Unit Hydrograph (SBUH).

The project engineer shall verify that a particular modeling approach will be acceptable. The project engineer shall provide clear and complete information (e.g., input and output files, annotation of key outputs or reports to highlight and clarify key results and conclusions, and discussion of results) to enable the city to conduct its review. See Volume I for additional submittal details and requirements.

2.2.1 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. Detailed flow control requirements are provided in Volume I. This section provides general guidance for analyzing design performance relative to flow control standards.

There are three flow-related standards stated in Volume I, Section 2.4 of this manual: Minimum Requirement #5: On-Site Stormwater Management; Minimum Requirement #7: Flow Control; and Minimum Requirement #8: Wetlands Protection.

Minimum Requirement #5 allows the user to demonstrate compliance with the LID Performance Standard of matching developed discharge durations to predeveloped durations for the range of predeveloped discharge rates from 8 percent of the 2-year peak flow to 50 percent of the 2-year peak flow. If the postdevelopment flow duration values exceed any of the predevelopment flow levels between 8 percent and 50 percent of the 2-year predevelopment peak flow values, then the LID Performance Standard has not been met.

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 percent of the 2-year recurrence interval peak flow up to the full 50-year peak flow. (Note that Minimum Requirement #7 also includes discharge requirements for projects in closed depression areas, discussed in more detail in Section 2.2.5 below.)

- The continuous runoff models compute the predevelopment 2- through 100-year recurrence interval flow values and compute the postdevelopment runoff 2- through 100-year recurrence interval flow values from the outlet of the proposed stormwater facility.

- The model uses discharge data from the applicable BMP(s) to compare the predevelopment and postdevelopment 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 percent and 100 percent of the 2-year recurrence interval 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 percent of the 2-year and 100 percent of the 50-year recurrence interval 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.

Minimum Requirement #8 specifies that total discharge to a wetland must not deviate by more than 20 percent on a single event basis, and must not deviate by more than 15 percent on a monthly basis. Flow components feeding the wetland under both pre- and postdevelopment scenarios are assumed to be the sum of the surface, interflow, and groundwater flows from the project site. Ecology has added the capability to model flows to wetlands and analyze the daily and monthly flow deviations (per these requirements) to WWHM2012. Refer to Minimum Requirement #8 in Volume I and the 2014 Ecology Manual, Volume I, Appendix I-D for additional requirements related to wetlands.

In some cases, the Administrator may require a site to provide flow control for off-site drainage when bypassing around the site is not feasible. See Chapter 3 for conveyance requirements for off-site drainage.

2.2.2 Hydrologic Analysis of Runoff Treatment BMPs

This section addresses hydrologic analysis requirements for treatment facilities. Requirements discussed in this section include design volumes and flows, areas needing treatment and sequencing of facilities. Additional requirements for individual facilities are discussed in Volume V.

Water Quality Design Storm Volume

The 91st percentile, 24-hour runoff volume estimated by an approved continuous runoff model shall be used as the water quality design storm volume.

Water Quality Design Flow Rate

Downstream of detention facilities: The full 2-year recurrence interval release rate from a detention facility (using an approved continuous runoff model) designed to meet the flow duration standard shall be used as the design flow rate.

Preceding detention facilities or when detention facilities are not required: The flow rate at or below which 91 percent of the runoff volume, as estimated by an approved continuous runoff model, shall be used as the design flow rate. The 91 percent volume for treatment facilities is designed to achieve the applicable performance goal at the water quality design flow rate (e.g., 80 percent total suspended solids removal).

- Offline facilities: When runoff flow rates exceed the water quality design flow rate and treatment facilities are not preceded by an equalization or storage basin, the treatment facility should continue to receive and treat the water quality design flow rate to the applicable treatment performance goal. Only the portion of flow rates that exceed the water quality design flow may be bypassed around a treatment facility.

Treatment facilities preceded by an equalization or storage basin may identify a lower water quality design flow rate, provided that at least 91 percent of the estimated runoff volume in the time series of an approved continuous runoff model is treated to the applicable performance goals (e.g., 80 percent total suspended solids removal at the water quality design flow rate and 80 percent total suspended solids removal on an annual average basis).

- Online facilities: Runoff flow rates in excess of the water quality design flow rate can be routed through the facility provided a net pollutant reduction is maintained, and the applicable annual average performance goal is likely to be met. When on-line runoff treatment facilities experience flows greater than the water quality design flow rate, it is assumed that no pollutant removal is occurring. For this reason, water quality design flow rates for on-line facilities are higher than design flow rates for off-line facilities with the same drainage characteristics.

Treatment facilities that are located downstream of detention facilities should only be designed as on-line facilities.

Flows Requiring Treatment

Runoff from pollution-generating hard or pervious surfaces must be treated. Pollution-generating hard surfaces (PGHS) are those hard surfaces considered to be a significant source of pollutants in stormwater runoff. PGHS include pollution-generating impervious surfaces (PGIS) and pollution-generating permeable pavements. Permeable pavements subject to pollution-generating activities are also considered pollution-generating pervious surfaces (PGPS) because of their infiltration capability. The glossary in Volume I provides additional definitions and clarification of these terms.

PGHS, PGIS, and PGPS include those surfaces that receive direct rainfall, or run-on or blow-in of rainfall, and are subject to: vehicular use; industrial activities; or storage of erodible or leachable materials, wastes, or chemicals. Erodible or leachable materials, wastes, or chemicals are those substances which, when exposed to rainfall, measurably alter the physical or chemical characteristics of the rainfall runoff. Examples include erodible soils that are stockpiled, uncovered process wastes, manure, fertilizers, oily substances, ashes, kiln dust, and garbage dumpster leakage. Metal roofs are also considered to be PGIS unless they are coated with an inert, non-leachable material (e.g., baked enamel coating). Roofs subject to venting significant amounts of dusts, mists or fumes from manufacturing, commercial, or other indoor activities are also PGIS.

A surface, whether paved or not, shall be considered subject to vehicular use if it is regularly used by motor vehicles. The following are considered regularly-used surfaces: roads, unvegetated road shoulders, bike lanes within the traveled lane of a roadway, driveways, parking lots, unfenced fire lanes, vehicular equipment storage yards, and airport runways.

The following are not considered regularly-used surfaces: paved bicycle pathways separated from and not subject to drainage from roads for motor vehicles, restricted access fire lanes, and infrequently used maintenance access roads.

Pollution generating pervious surfaces (PGPS) are any pervious surfaces that receive direct rainfall, or run-on or blow-in of rainfall, and are subject to vehicular use; industrial activities (as further defined in the glossary); storage of erodible or leachable materials, wastes, or chemicals; the use of pesticides and fertilizers; or loss of soil. Typical PGPS include permeable pavements subject to vehicular use, lawns, and landscaped areas, including: golf courses, parks, cemeteries, and sports fields (including natural and artificial turf).

Summary of Areas Needing Treatment

- All runoff from PGHS is to be treated through the water quality facilities specified in Volume I, Chapter 4.

- Lawns and landscaped areas specified are pervious but also generate runoff into street drainage systems. In those cases, the runoff from the pervious areas must be estimated and added to the runoff from hard surface areas to size treatment facilities.

- Runoff from backyards can drain into native vegetation in areas designated as open space or buffers. In these cases, the area in native vegetation may be used to provide the requisite water quality treatment, provided it meets the requirements outlined in Volume V for dispersion.

- Drainage from hard surfaces that are not pollution-generating need not be treated and may bypass runoff treatment, if it is not mingled with runoff from pollution-generating surfaces.

- Runoff from non-pollution-generating roofs is still subject to flow control per Minimum Requirement #7. The non-pollution-generating roof runoff that is directed to an infiltration trench or drywell must first pass through a catch basin as shown in Downspout Infiltration Systems (see Volume V). Note that metal roofs are considered pollution-generating unless they are coated with an inert, non-leachable material. Roofs that are subject to venting of significant amounts of manufacturing, commercial, or other indoor pollutants are considered pollution-generating.

- Drainage from areas in native vegetation should not be mixed with untreated runoff from streets and driveways, if possible. It is best to infiltrate or disperse this relatively clean runoff to maximize recharge to shallow groundwater, wetlands, and streams (see Volume V for flow dispersion requirements).

- If runoff from non-pollution generating surfaces reaches a runoff treatment BMP, flows from those areas must be included in the sizing calculations for the facility. Once runoff from non-pollution generating areas is mixed with runoff from pollution-generating areas, it cannot be separated before treatment.

2.2.3 Sequence of Facilities

The enhanced treatment and phosphorus removal menus, described in Volume I, Section 4.3, include treatment options in which more than one type of treatment facility is used. In those options, the sequence of facilities is prescribed. This is because the specific pollutant removal role of the second or third facility in a treatment often assumes that significant solids' settling has already occurred.

There is also the question of whether treatment facilities should be placed upstream or downstream of detention facilities that are needed for flow control purposes. In general, all treatment facilities may be installed upstream of detention facilities. However, not all treatment facilities can function effectively if located downstream of detention facilities. Those facilities that treat unconcentrated flows, such as filter strips, are usually not practical downstream of detention facilities. Other types of treatment facilities present special problems that must be considered before placement downstream is advisable.

For instance, prolonged flows discharged by a detention facility that is designed to meet the flow duration standard of Minimum Requirement #7 may interfere with proper functioning of basic biofiltration swales. Grasses typically specified in the basic biofiltration swale design will not survive. A wet biofiltration swale design would be a better choice.

Oil control facilities for runoff treatment must be located upstream of treatment and detention facilities and as close to the source of oil-generating activity as possible.

Table 2.2 summarizes placement considerations of treatment facilities in relation to detention.

Table 2.2. Treatment Facility Placement in Relation to Detention.

Water Quality Facility	Preceding Detention	Following Detention
Basic biofiltration swale (Volume V, Chapter 25)	OK	OK. Prolonged flows may reduce grass survival. Consider wet biofiltration swale.
Wet biofiltration swale (Volume V, Chapter 25)	OK	OK.
Filter strip (Volume V, Chapter 25)	OK	No. Must be installed before flows concentrate.
Basic or large wet pond (Volume V, Chapter 26)	OK	OK. Less water level fluctuation in ponds downstream of detention may improve aesthetic qualities and performance.
Basic or large combined detention and wet pond (Volume V, Chapter 26)	Not applicable	Not applicable
Wet vault (Chapter 26)	OK	OK.
Stormwater treatment wetland/pond (Volume V, Chapter 26)	OK	OK. Less water level fluctuation and better plant diversity are possible if the stormwater wetland is located downstream of the detention facility.

2.2.4 Runoff Computation Methods

This section describes the advantages, limitations, and applicability of runoff computation methods that may be used in the city of Tumwater.

Single-Event and Continuous Simulation Runoff Models

A continuous simulation runoff model has considerable advantages over the single-event-based methods such as the SCSUH, SBUH, or the Rational Method, which are described in Appendix III-A. HSPF is a continuous simulation model that can simulate a wider range of hydrologic responses than the single-event models. Single-event models 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 an HSPF model are the result of a considerable effort to introduce local parameters and actual rainfall data into the model; Therefore, a continuous simulation runoff model produces better estimations of runoff than the SCSUH, SBUH, or Rational Method, which tend to overestimate peak runoff.

A major weakness of a single-event model 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. Also, single-event approaches, such as the SBUH, assume that flow control ponds are empty at the start of the design event. Continuous runoff models are able to simulate a continuous, long-term record of runoff and soil moisture conditions. They simulate situations where ponds are not empty when another rain event begins.

Finally, single event models do not allow for estimation and analyses of flow durations or water level fluctuations. Flow durations are necessary for discharges to streams. Estimates of water level fluctuations are necessary for discharges to wetlands and for tracking influent water elevations and bypass quantities to properly size treatment facilities.

Single-Event Storms – Hydrograph

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

Because the only application for single-event methods in this manual is to size conveyance systems, only limited discussions of design storms, curve numbers, and calculating peak runoffs are presented in Appendix III-A. If single-event methods are used to size temporary and permanent conveyances, the user should reference other texts and software for assistance.

Conveyance systems can be designed using unit hydrograph analysis methods for estimating storm runoff rates. All storage facilities shall be designed to meet the Minimum Requirement #7 for frequency and duration control using a continuous runoff model. If the engineer decides to use a single-event runoff model for conveyance design, the preferred method is the SBUH method, and the SCSUH method is the second choice. The Rational Method may be used for conveyance sizing on sites of 25 acres or less, and having a time of concentration of less than 100 minutes.

Western Washington Hydrology Model

Since the first version of WWHM was developed and released to public in 2001, the WWHM program has gone through several upgrades incorporating new features and capabilities, including low impact development (LID) modeling capability. For example, WWHM2012 includes modeling elements for stormwater LID BMPs. WWHM users should periodically check Ecology's WWHM web site for the latest releases of WWHM, user manuals, and any supplemental instructions. Refer to Volume III, Section 2.2, of the 2014 Ecology Manual for background information on WWHM, and Appendix III-C of that manual for extensive guidance on LID BMP modeling in WWHM.

More information on the WWHM can be found on Ecology's web site at: https://ecology.wa.gov/Regulations-Permits/.

Rational Method

The Rational Method is a simple method used to estimate peak flows, and it may be used for conveyance sizing on sites 25 acres or less in size and having a time of concentration of less than 100 minutes. See Appendix III-A for details on the method.

2.2.5 Closed Depression Analysis

Analysis of closed depressions requires careful assessment of the existing hydrologic performance to evaluate the impacts of a proposed project. A calibrated, continuous simulation runoff model must be used for closed depression analysis and design of mitigation facilities. The applicable flow control requirements (see Minimum Requirement #7) and the city's critical areas ordinance and rules (if applicable) must be thoroughly reviewed prior to proceeding with the analysis. Detailed information on discharge criteria for projects discharging to closed depressions are described in Volume I, Chapter 2 Minimum Requirement #7.

In all cases, discharge to a closed depression shall be allowed only if the Project Engineer can satisfactorily demonstrate that no significant public health, safety, welfare, or property damage issues are present. See requirements in Volume I, Chapter 2 Minimum Requirement #7.

Chapter 3 – Conveyance Design and Analysis

This chapter presents acceptable methods for the analysis and design of conveyance systems. It also includes sections on hydraulic structures that link the conveyance system to the runoff treatment and flow control BMPs.

This chapter includes:

- Design storm selection (Section 3.1)
- Conveyance route design requirements (Section 3.2)
- Easement and access requirements (Section 3.3)
- Conveyance design methods and criteria (Section 3.44)
    - Channels
    - Culverts
    - Storm sewers
    - Pipe structures (manholes, catch basins, flow splitters)
    - Outfalls
    - Flow spreaders
    - Private drainage systems
- Additional requirements for private drainage systems (Section 3.5)

Where practical, given space and topography, open conveyances are the preferred means of collecting and conveying stormwater.

3.1 Design Event Storm Frequency

Ideally, every conveyance system and hydraulic structure would be designed for the largest amount of flow that could ever occur. Unfortunately, this would require unusually large structures and would add an unjustifiable cost to the projects. Therefore, hydraulic structures are analyzed for a specific storm frequency. When selecting a storm frequency for design purposes, consideration is given to the potential degree of damage to adjacent properties, potential hazard and inconvenience to the public, the number of users, and the initial construction cost of the conveyance system or hydraulic structure. The way in which these factors interrelate can become quite complex.

3.1.1 Required Design Events

The design event recurrence interval is related to the probability that such an event will occur in any 1 year. For example, a peak flow having a 25-year recurrence interval has a 4 percent probability of being equaled or exceeded in any future year. A peak flow having a 2-year recurrence interval has a 50 percent probability of being equaled or exceeded in any future year. The greater the recurrence interval is, the lower the probability that the event will occur in any given year.

The design event for each conveyance system category is as follows:

- The project's internal piped conveyance system shall be designed for a 25-year, 24-hour storm event. In areas where the city determines there is a high risk of damage or vital service interruption, a backwater analysis of the peak flows from the 100-year, 24-hour storm events shall be conducted.

- All open channel conveyance systems shall be designed for the 100-year, 24-hour storm event.

- Piped conveyance under public roads and arterials shall convey a 25-year, 24-hour storm event under fully developed basin conditions. Additional criteria:

    - In the urban area, inside of the long-term urban growth management boundary (boundary is depicted on current zoning maps available at local jurisdictions), the outside driving lane of public roads and streets must not have water over more than 50 percent of the lane for a design event of a 25-year, 24-hour storm.

    - In the area outside of the long-term urban growth management boundary, the design event shall be the 100-year, 24-hour storm.

    - In areas where the city determines there is a high risk of damage or vital service interruption (e.g., more than 6 inches of standing water in the streets), the Administrator may specify up to the 100-year, 24-hour event as the design event.

- Natural channel bridges and culverts shall be designed to convey at least the 100-year, 24-hour storm event under fully developed drainage basin conditions based on the tributary area zoning and maximum allowable impervious surface area. Culvert and bridge designs must also meet applicable fish passage and scour criteria.

The city may require an increased level of protection and/or freeboard on a case-by-case basis.

3.1.2 Considerations

Recent scientific studies have indicated that peak flow rates may increase in the future as a result of climate change. Rosenberg et al. (2010) simulated hydrology in three stream basins using two climate change scenarios and found that peak flows increased between 20 percent and 60 percent for two of the three stream systems. Therefore, designers should consider the risk associated with increased peak flows when designing stormwater infrastructure. The designer may choose to conduct a risk analysis or consider existing research to determine if an additional safety factor should be applied to the design flows.

3.2 Conveyance System Route Design and Off-Site Flows

3.2.1 Conveyance System Alignment

All stormwater conveyance pipes shall be placed under the pavement flow line or lie outside of the pavement, unless otherwise specified below. Perpendicular crossings and cul-de-sacs are exempted from this requirement. For curved sections only of minor local residential streets, private roads, and alleys, pipes may be placed underneath pavement areas, but pipes shall be no closer than 6 feet from the roadway centerline. Pipes under permeable pavement sections will need to ensure flows are prevented from short-circuiting through the pipe zone bedding. Location and layout of conveyance piping on roadway retrofit projects will be determined on a case-by-case basis.

New conveyance system alignments that are not within dedicated tracts or rights-of-way shall be placed within drainage easements that are adjacent and parallel to property lines, unless topography or other existing conditions make that infeasible. The width of the permanent easement must be completely within a single parcel or tract. Requirements for conveyance system tracks and easements are discussed in Section 3.3.

Exception: Streams and natural drainage channels shall not be relocated to meet this routing requirement.

3.2.2 Off-Site Flow

Development projects are required to manage off-site drainage in the manner described below and discharge the flow at the natural location (see also Minimum Requirements #4 in Volume I, Section 2.4.5).

Off-site inflow occurs when an upslope area outside the development drains to the flow control facility in the development. Bypassing off-site inflow around required on-site flow control facilities is required unless written approval is granted by the Administrator. If the existing 100-year peak flow rate from any upstream off-site area is greater than 50 percent of the 100-year developed peak flow rate (undetained) for the project site, then the runoff from the off-site area must not flow to the on-site flow control facility.

The bypass of off-site inflow must be designed so as to achieve both of the following:

1. Any existing contribution of flows to an on-site wetland must be maintained.

2. Off-site flows that are naturally attenuated by the project site under predeveloped conditions must remain attenuated, either by natural means or by providing additional on-site detention so that peak flows do not increase. In other words, after development of the subject site, off-site flows shall be infiltrated within or passed through the project site in the same proportion as occurred prior to development.

The area and existing use of the off-site land area should be included in any modeling performed for the design of new facilities that receive the off-site flow. If an adjacent site is undeveloped, the potential for increased future flow volumes should be factored into design process. This may require modeling the off-site land area as if it were developed with a detention facility discharging per the minimum requirements of this manual. If the off-site drainage is to be infiltrated on the project site, the infiltration facilities shall be sized to accommodate the correct proportion of off-site flows. See the 2014 Ecology Manual Volume III, Appendix III-B, or the MGS Flood User's Manual for guidance on modeling off-site flow that enters a flow control facility.

Sizing calculations for stormwater treatment facilities must include the entire drainage area that is tributary to the facility (see Section 2.2.2).

3.3 Easements, Access, and Dedicated Tracts

3.3.1 Natural Channels and Stormwater Facilities

When outside public rights-of-way, all constructed drainage facilities and conveyances, and all natural channels (on the project site) used for conveyance of altered flows due to development (including swales, ditches, stream channels, lake shores, wetlands, potholes, estuaries, gullies, ravines, etc.) shall be within easements or dedicated tracts, as required by the city. Easements shall contain the natural features and facilities and shall allow city access for purposes of inspection, maintenance, repair or replacement, flood control, water quality monitoring, and other activities permitted by law.

Drainage facilities shall not be placed within public road rights-of-way, with the exception of city and highway facilities.

Drainage facilities that are designed to function as multi-use recreational facilities shall be placed in separate tracts or in designated open space, and shall be privately maintained and owned.

3.3.2 Maintenance Access for Underground Drainage Facilities

See individual BMP descriptions in Volume V for additional stormwater facility access requirements.

A maintenance access road must be provided for all manholes, catch basins, vaults, or other underground drainage facilities. For such facilities outside public rights-of-way, maintenance access shall be through an access road. This requirement does not apply to on-site stormwater management BMPs.

A minimum 15-foot-wide access easement shall be provided to the facilities from a public street or right-of-way. An easement should be granted through a tract, dedicated to the city, for access to the stormwater facility and should not cross any individual lot within a subdivision. Easements across individual lots for access to a stormwater conveyance, flow control, and treatment facilities are discouraged and shall be allowed only with specific approval by the City of Tumwater, and only upon demonstration that no other alternative exists and measures are in place to ensure that the easement will not be encroached upon by the property owner.

The maintenance access shall be surfaced with a minimum 12-foot width of lattice block pavement, crushed rock or other approved surface to allow year-round equipment access to the facility, and shall be delineated by a gate, fencing, or other measure to indicate that the access easement exists. The easement shall include easement boundary markers, which shall be fiberglass utility markers with a reflective easement tag, located at each corner of the easement, at angle points and at least every 100 feet along the length of the easement. Contact Tumwater Water Resources for additional information on easement marker requirements.

Drainage structures for conveyance without vehicular access must be channeled.

3.3.3 Access to Conveyance Systems

All publicly and privately maintained conveyance systems shall be within dedicated tracts, drainage easements, or public rights-of-way, in accordance with this manual. Exceptions include: roof downspouts, minor yard drains, and footing drains—unless they serve more than one property.

Conveyance systems to be maintained and operated by the City of Tumwater must be located in a dedicated tract or drainage easement granted to the city. Any new conveyance system on private property that is designed to convey drainage from other private properties must be located in a private drainage easement. The easement will permit access for system maintenance or replacement in the case of failure. The owner of the property containing the conveyance system must grant the easement to the owners of property that contributes stormwater to the conveyance system.

Any easement for access to a conveyance system shall include measures to ensure that the easement will not be encroached upon by adjacent lot owners such as delineation by a gate, fencing, signage, or some other measure to indicate to adjacent property owners that an easement exists.

All drainage tracts and easements, public and private, must have a minimum width of 15 feet. In addition, all pipes and channels must be located within the tract, easement, or rights-of-way so that each pipe face or top edge of channel is at least 5 feet from the nearest boundary of the tract, easement, or right-of-way. Pipes greater than 5 feet in diameter and channels with top widths greater than 5 feet shall be placed in wider easements, adjusted accordingly, to meet the required distances from the boundaries. Table 3.1 shows the minimum widths for easements or tracts containing drainage facilities.

Table 3.1. Minimum Easement/Tract Widths for Conveyance Systems for Access, Inspection, and Maintenance.

Conveyance Width	Easement or Tract Width
Channels ≤30 feet wide	Channel Width + 15 feet from top, one side
Channels >30 feet wide	Channel Width + 15 feet from top, both sides
Pipes/Outfalls ≤36 inches	15 feet centered on pipe
Pipes/Outfalls >36 inches but ≤60 inches	20 feet centered on pipe [a]
Pipes/Outfalls >60 inches	30 feet centered on pipe [a]

[a] The City of Tumwater may allow flexibility, or require larger easements, depending on site-specific conditions.

3.3.4 Discharge to Private Property

If the site of a proposed project discharges surface water or stormwater to an adjacent property that has no public drainage facility or defined drainage course (e.g., a natural channel with a measurable annual discharge, such as a stream rated as Type 5 by the Washington State Department of Natural Resources [WDNR]), the proponent should obtain an easement from the adjacent property owner(s) to establish a drainage pathway that connects to a defined drainage system. In the absence of such an easement, the discharge from stormwater management facilities on the project site shall be distributed along the property line in approximately the same flow pattern as existed before development. A downstream analysis shall be conducted to determine any potential impacts of the distributed flow to downstream property.

The Administrator may, under highly unusual circumstances, excuse the proponent from requirements of this section. For example, the adjacent property is a wetland and is not a closed basin, and discharge to the wetland would not significantly alter the hydrology, degrade wetland functions and values, or reduce the value of the property.

3.4 Conveyance Design Methods and Criteria

This section describes methods and criteria for sizing of storm sewers, channels, revetments, and other drainage structures in the conveyance system. Setbacks and easements for conveyances are described in Section 3.3. Section 3.1.2 provides guidance on considering climate change during design.

3.4.1 Open Channel Flow – Hydraulic Analysis

Two hydraulic analysis methods are used to analyze and design conveyance systems:

- The Uniform Flow Analysis Method (Section 3.4.1 below), commonly referred to as the Manning's equation, is used for the design of open conveyances (Section 3.10) and new pipe systems (Section 3.7), as well as for analysis of existing pipe systems. Manning's equation is only valid for pipe flow when the pipe is flowing less than full. If the pipe is surcharged, the backwater method must be used.

- The Backwater Analysis Method (see below) is used to analyze the capacity of both proposed and existing pipe systems when a pipe is surcharged. If the city determines that, as a result of the project, runoff for any event up to and including the 100-year, 24-hour event would exceed the pipes' unsurcharged capacity, a backwater (pressure sewer) analysis shall be required. Results shall be submitted in tabular and graphic format showing hydraulic and energy gradient.

Uniform Flow Analysis – Manning's Equation

Manning's equation can be used for open channel flow or for a pipe that is flowing less than full. Manning's equation is expressed as:

$$V = \frac{1.486}{n} \times R^{0.67} \times S^{0.5}$$

Where: V = velocity (feet per second),

n = Manning's roughness factor (-)

R = hydraulic radius (area/wetted perimeter; feet), and

S = Channel slope (feet/foot)

Manning's equation can also be expressed in terms of discharge (Q):

$$Q = \frac{1.486}{n} \times A \times R^{0.67} \times S^{0.5}$$

Where: A = cross-sectional area of flow (square feet).

Manning's roughness coefficients (n) for open channels are shown in Table 3.2, and for piped conveyances in Table 3.3. A more extensive table of Manning's roughness factors is available in Table A.6 in Appendix III-A.

Table 3.2. Manning's Roughness Coefficients for Open Channel Conveyances.

Channel Lining	Manning's Roughness Factor (n)
Concrete	0.012
Short grass	0.030
Stony bottom and weedy grass	0.035
Cobble bottom and grass banks	0.040
Dense weeds as high as flow	0.080
Dense woody brush as high as flow	0.120
Biofiltration swale	See Volume V

Table 3.3. Manning's Roughness Factors for Pipe Conveyances.

	Type of Pipe Material	Analysis Method	
		Backwater Flow	Manning's Equation Flow[a]
A.	Concrete pipe	0.013	0.015
B.	Annular Corrugated Metal Pipe or Pipe Arch:		
	1. 2-2/3″ x 1/2″ corrugation (riveted)	0.024	0.028
	2. 3″ x 1″ corrugation	0.027	0.031
	3. 6″ x 2″ corrugation (field bolted)	0.030	0.035
C.	Helical 2-2/3″ x 1/2″ corrugation	0.024	0.028
D.	Spiral rib metal pipe	0.016	0.018
E.	Ductile iron pipe cement lined	0.013	0.015
F.	Plastic	0.010	0.012

[a] The roughness values for this method are 15 percent higher in order to account for entrance, exit, junction, and bend head losses.

Backwater Analysis

A backwater (pressure sewer) analysis is required if the city determines that, as a result of a proposed project, runoff for any event up to and including the 100-year, 24-hour storm event would cause damage or interrupt vital services. When a backwater analysis is required, the design engineer shall analyze the 25- and 100-year, 24-hour design storm events.

- For the 25-year event, there shall be a minimum of 6 inches of freeboard between the water surface and the top of any manhole or catch basin.

- For the 100-year event, overtopping of the pipe conveyance system may occur; however, for conveyance systems in streets, the additional flow shall not extend beyond half the lane width of the outside lane of the traveled way, and it shall not exceed 4 inches in depth at its deepest point.

- Off-channel storage on private property is allowed with recording of the proper easements (see Section 3.3). The additional flow shall be analyzed by open channel flow methods.

A backwater profile analysis computer program, such as the King County Backwater computer program by King County Department of Natural Resources, is recommended over hand calculations. The subroutine, BPIPE, of King County Backwater may be used for quick computation of backwater profiles, given a range of flows through the existing or proposed pipe system.

3.4.2 Channels

Channels can be either roadside ditches, grass-lined swales, or a combination thereof. Consideration must be given to public safety when designing open conveyances adjacent to traveled ways and when accessible to the public. Where space and topography permit, channels are the preferred means of collecting and conveying stormwater.

Channels shall be designed by one of the following methods (refer to Appendix III-A):

- Manning's equation (for uniform flow depth, flow velocity, and constant channel cross-section)
- Direct Step Backwater Method (using the energy equation)
- Standard Step Backwater Method (using a computer program).

Flow velocities must be low enough to prevent channel erosion based on the characteristics of the native soil or the compacted fill material. For velocities above 5 feet per second, channels shall either: 1) have rock-lined bottoms and side slopes to the roadway shoulder top with a minimum thickness of 8 inches, or 2) be stabilized in a fashion acceptable to the city. Water quality shall not be degraded due to passage through an open conveyance. Channels must be stabilized against erosion in compliance with minimum standards for erosion control set forth in Volume II. Table 3.4 provides minimum criteria to prevent damage. Manning's roughness coefficients (n) for open channels are shown in Table 3.2.

Table 3.4. Design Criteria – Open Channels.

Channel Lining	Maximum Design Velocity (fps)	Maximum Design Slope H:V	Minimum Filter Blanket (inches)[c]
Vegetation	5	3	not applicable
Geotextile	[a]	[a]	not applicable
Lattice Block Paving Systems	12	2	[a]
Quarry Spalls, 18-inch diameter	15[b]	2	4
Hand-Placed Riprap, 2-foot thick	12	2	4
Gabions	30	[a]	4
Concrete	30	Design	not applicable

[a] Per manufacturer's instructions

[b] See Riprap Design, *Journal of Hydraulic Engineering* (Maynord et al. 1989).

[c] See Gradation Design of sand and gravel filters, Chapter 26 of Part 633 National Engineering Handbook (NRCS 1994).

fps = feet per second

H:V = horizontal to vertical

Channels having a slope less than 6 percent and having peak velocities less than 5 feet per second shall be lined with vegetation. Check dams for erosion and sedimentation control may be used for stepping down channels being used for biofiltration.

Channel side slopes shall not exceed 2:1 for undisturbed ground (cuts) as well as for disturbed ground (embankments). All constructed channels shall be compacted to a minimum 95 percent compaction as verified by a Modified Proctor test. Channel side slopes adjacent to roads shall not exceed 4:1 and must meet all other AASHTO and City of Tumwater road standards.

Channels shall be designed with a minimum freeboard of 6 inches when the design flow is 10 cubic feet per second or less, and 1 foot when the design discharge is greater than 10 cubic feet per second.

Check dams for erosion and sedimentation control may be used for stepping down channels being used for biofiltration.

3.4.3 Culverts

For the purposes of this manual, culverts are single runs of pipe that are open at each end and have no structures such as manholes or catch basins. Approved pipe materials are detailed below in Section 3.4.4, Storm Sewers. Galvanized or aluminized pipe are not permitted in marine environments or where contact with salt water may occur, even infrequently through backwater events.

Flow capacity shall be determined by analyzing inlet and outlet control for headwater depth. Nomographs used for culvert design shall be included in the submitted Drainage Control Plan. Appendix III-B includes several nomographs that may be useful for culvert sizing.

All culverts shall be designed to convey the flows per Section 3.1, Design Event Storm Frequency. The maximum design headwater depth shall be 1.5 times the diameter of the culvert, with no saturation of roadbeds. For culverts that convey streams, the maximum design headwater depth shall be below the culvert crown. Minimum culvert diameters are:

- For cross culverts under public roadways: minimum 18 inches; 12 inches if grade and cover do not allow for 18 inches
- For roadside culverts, including driveway culverts: minimum 12 inches
- For culverts on private property: minimum 8 inches

Inlets and outlets shall be protected from erosion by rock lining, riprap, or biostabilization, as detailed in Table 3.5.

Table 3.5. Open Conveyance Protection.

Velocity at Design Flow (fps)				
Greater Than	**Less Than or Equal To**	**Protection**	**Thickness**	**Minimum Height Required Above Design Water Surface**
0	5	Grass Lining[a]	not applicable	0.5 foot
5	8	Riprap[a,b]	1 foot	2 feet
8	12	Riprap[c]	2 feet	2 feet
12	20	Slope mattress, gabion, etc.	varies	1 foot

[a] Bioengineered lining allowed for design flow up to 8 fps.

[b] Riprap shall be in accordance with Section 9-13.1 of the WSDOT/APWA standard specifications.
Riprap shall be a reasonably well-graded assortment of rock with the following gradation:
Maximum stone size 12 inches
Median stone size 8 inches
Minimum stone size 2 inches

[c] Riprap shall be reasonably well graded assortment of rock with the following gradation:
Maximum stone size 24 inches
Median stone size 16 inches
Minimum stone size 4 inches

Note: Riprap sizing governed by side slopes on channel, assumed ~3:1.

Debris and access barriers are required on inlet and outlet ends of all culverts greater than 18 inches in diameter. Culverts greater than 36 inches in diameter within stream corridors are exempt.

Minimum culvert velocity shall be 2 feet per second and maximum culvert velocity shall be 15 feet per second. Thirty (30) feet per second may be used with an engineered outlet protection design. The city may waive the minimum requirement in cases where topography and existing drainage systems make it impractical to meet the standard. No maximum velocity for ductile iron or high density polyethylene (HDPE) pipe shall be established, but outlet protection shall be provided.

All corrugated, high density polyethylene pipe (CPEP) and polyvinyl chloride (PVC) culverts and pipe systems shall have concrete or rock headwalls at exposed pipe ends.

Bends are not permitted in culvert pipes.

The following design criteria apply to culverts:

- Minimum cover to be provided:
  - 2 feet under roads
  - 1 foot under roadside applications and on private property, exclusive of roads

  If the minimum cover cannot be provided on a flat site, use ductile iron pipe and analyze for loadings.

- Maximum culvert length: 250 feet
- Minimum separation from other pipes:
    - 6 inches vertical (with bedding) and in accordance with the sewer or water purveyor design criteria
    - 3 feet horizontal

Trench backfill shall be bankrun gravel or suitable native material compacted to 95 percent Modified Proctor test to a depth of 2 feet; below 2 feet, compacted to 90 percent in 8-inch to 12-inch lifts.

All driveway culverts shall be of sufficient length to provide a minimum 3:1 slope from the edge of the driveway to the bottom of the ditch. Culverts shall have beveled end sections to match the side slope. Shallow fords may be substituted for culverts on residential driveway crossings of swales.

Culverts in stream corridors must meet any fish passage requirements of the Washington Department of Fish and Wildlife (WDFW).

3.4.4 Storm Sewers

Analysis Methods

Two methods of hydraulic analysis using Manning's equation are used for the analysis of pipe systems. The first method is the Uniform Flow Analysis Method, commonly referred to as the Manning's equation, and is used for the design of new pipe systems and analysis of existing pipe systems. The second method is the Backwater Analysis Method (see Section 3.4.1) and is used to analyze the capacity of both proposed and existing pipe systems.

When using the Manning's equation for design, each pipe within the system shall be sized and sloped such that its barrel capacity at normal full flow is equal to or greater than the required conveyance capacity as identified in Section 3.1. Manning's roughness coefficients (n) are shown in Table 3.3 for design of pipe systems. Manning's "n" values used for final pipe design must be documented in the Drainage Control Plan.

Nomographs may also be used for sizing the pipes. For pipes flowing partially full, the actual velocity may be estimated from engineering nomographs by calculating Q_{full} and V_{full} and using the ratio of Q_{design}/Q_{full} to find V and d (depth of flow). Appendix III-B includes several nomographs that may be useful for culvert sizing.

Acceptable Pipe Sizes

Storm drainage pipes are subject to the following minimum diameters:

- Private drainage system: 4 inches
- Public right-of-way: 12 inches (exception: laterals connecting catch basins to main lines may be 8 inches)

The Administrator may waive these minimums in cases where topography, design flows, or existing drainage systems make it impractical to meet the standard. For culverts, see Section 3.4.3.

Pipe Materials

All storm drainage pipe, except as otherwise indicated, shall be: rubber-gasketed concrete pipe; double-walled CPEP, with a smooth internal diameter (AASHTO M-294 Type-S); ADS N-12 plastic pipe (up to 24-inch diameter only); or approved equal Pipe shall be coupled with a company-produced PVC coupling or approved equal, except for perforated pipe and major underground detention facilities. ADS N-12 pipe shall have a minimum cover of 2 feet, measured from the top of pipe to the top of paved surface. The rubber-gasket requirement may be waived if it can be shown that joint leakage will not be detrimental to the road prism.

When extreme slope conditions or other unusual topographic conditions exist, other pipe materials and methods, including (but not limited to) PVC, HDPE, or ductile iron pipe, may be used with prior city approval.

If other pipe materials are used, they shall meet the following minimum requirements and shall have prior city approval:

- Ductile iron, Class 50 or 52
- Reinforced concrete pipe
- Corrugated aluminum pipe
- Aluminum spiral rib pipe
- Aluminized Type 2 corrugated steel (meeting AASHTO treatments M274 and M56)
- Corrugated high density polyethylene pipe (CPEP) – smooth interior (maximum 24-inch diameter) meeting AASHTO standard M-294
- Corrugated, high density polyethylene pipe (CPEP) – single wall, fully corrugated, meeting AASHTO standard M-252 (permitted only outside public right-of-way and for use in temporary storm sewer systems and as downspout/footing/yard drain collectors on private property)
- Polyvinyl chloride (PVC) sewer pipe (standard dimension ratio [SDR] 35, meeting requirements of ASTM D3034)
- High density polyethylene (HDPE) pipe. Pipe must comply with requirement of Type III C5P34 per ASTM D1248 and have the Plastics Pipe Institute (PPI) recommended designation of PE3408 and have an ASTM D3350 cell

classification of 345434C or 345534C. Pipe shall have a manufacturer's recommended hydrostatic design stress rating of 800 psi based on a material with a 1,600 psi design basis determined in accordance with ASTM D2837-69. Pipe shall have a suggested design working pressure of 50 psi at 73.4 degrees Fahrenheit and SDR of 32.5. Designs using HDPE pipe shall include consideration of the material's thermal expansion/contraction properties for anchoring.

Pipe material, joints, and protective treatment shall meet WSDOT Standard Specifications, Sections 7-04 and 9-05, and AASHTO and ASTM treatment standards as amended by the city. The proponent is responsible for contacting the city to determine the allowable pipe materials that can be used.

Pipe Slope and Velocity

Minimum velocity is 2 feet per second at design flow. The city may waive that minimum in cases where topography and existing drainage systems make it impractical to meet the standard.

Maximum slopes, velocities, and anchor spacings are shown in Table 3.6. If velocities exceed 15 feet per second for the conveyance system design event, anchors must be provided at bends and junctions.

Table 3.6. Maximum Pipe Slopes and Velocities.

Pipe Material	Pipe Slope Above Which Pipe Anchors Are Required and Minimum Anchor Spacing	Maximum Slope Allowed	Maximum. Velocity at Full Flow
Spiral Rib, PVC, CPEP (single wall)[a]	20% (1 anchor per 100 LF of pipe)	30%[b]	30 fps
Concrete or CPEP (smooth interior)[a]	10% (1 anchor per 50 LF of pipe)	20%[b]	30 fps
Ductile Iron[c]	40% (1 anchor per pipe section)	None	None
HDPE[d]	50% (1 anchor per 100 LF of pipe—cross-slope installations only)	None	None

[a] Not allowed in landslide hazard areas.

[b] Maximum slope of 200 percent allowed for these pipe materials with no joints (one section) with structures at each end and properly grouted.

[c] Restrained joints required on slopes greater than 25 percent. Above-ground installation is required on slopes greater than 40 percent to minimize disturbance to steep slopes:

[d] Butt-fused pipe joints required. Above-ground installation is required on slopes greater than 40 percent to minimize disturbance to steep slopes.

CPEP = corrugated, high density polyethylene pipe

fps = feet per second

HDPE = high density polyethylene

LF = linear feet

PVC = polyvinyl chloride pipe

Downsizing of pipes is only allowed under special conditions. Such conditions include but are not limited to: no hydraulic jump can occur, downstream pipe slope is significantly greater than the upstream slope, and velocities remain in the 3 to 8 feet per second range. Downsizing of downstream culverts within a closed system with culverts 18 inches in diameter or smaller will not be permitted.

Pipes on Steep Slopes

Steep slopes (greater than 30 percent) shall require all drainage to be piped from the top to the bottom in HDPE pipe (butt fused) or ductile iron pipe (welded or mechanically restrained). They shall not be gasketed, slip fit, or banded. On steep slopes, HDPE pipe may be laid on the surface or in a shallow trench, anchored, protected against sluicing, and hand compacted.

For HDPE systems longer than 100 feet and on slopes exceeding 25 percent, the pipe must be secured at the upstream end and the downstream end must be placed in a 4-foot section of the next larger pipe size. This sliding sleeve connection allows for high thermal expansion/contraction.

Pipe System Layout Criteria

Pipes must be laid true to line and grade with no curves, bends, or deflections in any direction (except for HDPE and ductile iron with flanged, restrained, mechanical joint bends not greater than 30 degrees, on steep slopes).

A break in grade or alignment or changes in pipe material shall occur only at catch basins or manholes.

Connections to a pipe system shall be made only at catch basins or manholes. No wyes or tees are allowed, except on private roof/footing/yard drain systems on pipes 8 inches in diameter or less, with clean-outs upstream of each wye or tee.

Access barriers are required on all pipes 18 inches and larger exiting a closed pipe system. Debris barriers (trash racks) are required on all pipes entering a pipe system.

Cover Requirements, Trench Design, Pipe Strength

Provide 6 inches minimum vertical and 3 feet minimum horizontal clearance (outside surfaces) between storm drain pipes and other utility pipes and conduits. The City of Tumwater’s Development Standards for Water and Sewer Systems will apply for crossings of or parallel runs with Tumwater sewer lines and for crossings of water lines. Additional requirements for crossings of septic transport lines or water supply lines may apply. Contact Tumwater Development Services at 360-754-4180 for more information.

Suitable pipe cover over storm pipes in road rights-of-way shall be calculated for HS-20 loading by the Project Engineer. Pipe cover is measured from the finished grade elevation to the top of the outside surface of the pipe. Pipe manufacturer recommendations are acceptable, if verified by the Project Engineer.

Minimum cover for PVC SDR 35 shall be 3 feet in areas subject to vehicular traffic. Maximum cover shall be 30 feet or per the manufacturer's recommendations and as verified with calculations from the Project Engineer.

Pipe cover in areas not subject to vehicular loads, such as landscape planters and yards, may be reduced to a 1 foot minimum.

Where open channels or ponds discharge into storm drains, trash racks are required on all storm sewer system inlet pipes of 18 inches in diameter or larger. Trash racks must be removable with ordinary hand tools.

Pipe Connections

Where a minimal fall is necessary between inlet and outlet pipes in a structure, pipes must be aligned vertically by one of the following, in order of preference:

- Match pipe crowns
- Match 80 percent diameters of pipes
- Match pipe inverts

Where inlet pipes are higher than outlet pipes, drop manhole connections or increased durability in the structure floor may be required.

3.4.5 Manholes and Catch Basins

All catch basins and manholes shall meet WSDOT standards such as Type 1L, Type 1, and Type 2. The following criteria shall be used when designing a conveyance system that uses catch basins or manholes.

Unless otherwise required by the city, Type 1 catch basins shall be used at the following locations or for the following situations:

- When overall structure height does not exceed 8 feet, or when invert does not exceed 5 feet.
- When pipe sizes do not exceed 18 inches and connect at right angles to the long side of the structure; or 12 inches connecting to the short side.
- When all pipes tying into the structure connect at or very near to right angles.

Unless otherwise required by the city, Type 1L catch basins must be used at the following locations or for the following situations:

- When overall structure height does not exceed 8 feet or when invert does not exceed 5 feet.
- When any pipes tying into the structure exceed 18 inches connecting to the long side, or 15 inches connecting to the short side at or very near to right angles.

Unless otherwise required by the city, Type 2 (48-inch minimum diameter) catch basins shall be used at the following locations or for the following situations:

- When overall structure height does not exceed 15 feet.
- When all pipes tying into the structure do not exceed the limits set forth by the manufacturers. Type 2 catch basins over 4 feet in height shall have standard ladders.

Where an approved connection of a private storm drainage system into a city system occurs, a minimum of a Type 1 catch basin shall be used in Tumwater.

Maximum spacing on main storm sewers between access structures, whether catch basins or manholes, shall be 300 feet (Table 3.7).

Table 3.7. Maximum Surface Runs Between Inlet Structures on the Paved Roadway Surface in Tumwater.

Roadway Slope (percent)	Maximum Spacing (feet)
0.5 to 1.0	150
1.0 to 3.0	200
>3.0	300

Catch basin (or manhole) diameter shall be determined by pipe diameter and orientation at the junction structure. A plan view of the junction structure, drawn to scale, is required when more than four pipes enter the structure on the same plane, or if angles of approach and clearance between pipes is of concern. The plan view (and sections if necessary) must ensure a minimum distance (of solid concrete wall) between pipe openings of 8 inches for 48-inch- and 54-inch-diameter catch basins, and 12 inches for 72-inch- and 96-inch-diameter catch basins.

Catch basin evaluation of structural integrity for H-20 loading is required for multiple junction catch basins and other structures that exceed the manufacturer’s recommendations.

The WSDOT Hydraulics Manual can be used to determine inlet grate capacity when capacity is of concern. When verifying capacity, assume grate areas on slopes are 80 percent free of debris, and “vaned” grates are 95 percent free of debris. In sags or low spots, assume grates are 50 percent free of debris, and “vaned” grates are 75 percent free of debris.

The maximum slope of the ground surface shall be 3:1 for a radius of 5 feet around a catch basin grate.

Catch basins shall be provided within 50 feet of the entrance to a pipe system to provide for silt and debris removal.

Maximum spacing of structures for storm drainage conveyance lines running within an easement area shall be 350 feet for pipe grades greater than 0.3 percent and 200 feet for grades less than 0.3 percent. Structures not acting as points of entry for stormwater shall have locking lids and solid covers.

Locking lids will be installed on all structures containing restrictor or flow devices. The locking lids shall be of a quality and design acceptable to the city.

A metal frame and grate for catch basin and inlet, WSDOT Standard Plan B-2a or B-2b or preapproved city standard grate that is deemed bicycle safe, shall be used for all structures collecting drainage from the paved roadway surface.

When the road profile equals or exceeds 6 percent between structures, an asphalt berm shall be installed around the inlet of the structure or the catch basin may be recessed into the curb per this detail.

Table 3.8 presents the allowable structures and pipe sizes allowed by size of structure. All catch basins, inlets, etc., shall be marked.

Table 3.8. Allowable Structure and Pipe Sizes.

Catch Basin Type[a]	Maximum Pipe Diameter	
	Spiral Rib CPEP, HDPE, PVC[b] (inches)	Concrete and Ductile Iron (inches)
Inlet[c]	12	12
Type 1[d]	15	15
Type IL[d]	18	18
Type 2: 48-inch diameter	30	24
Type 2: 54-inch diameter	36	30
Type 2: 72-inch diameter	54	48
Type 2: 96-inch diameter	72	72

[a] Catch basins, including manhole steps, ladder, and handholds shall conform to the WSDOT Standard Plans or an approved equal based upon submittal for approval.

[b] Maintain the minimum side wall thickness per Section 4.6.8.6.

[c] Normally allowed only for use in privately maintained drainage systems and must discharge to a catch basin immediately downstream.

[d] Maximum 5 vertical feet allowed between grate and invert elevation.

Note: The proponent shall check with the City of Tumwater to determine the allowable pipe materials.

Each catch basin or grated manhole in a storm drainage system must have a storm drain marker pertaining to pollution prevention. Contact Public Works, Water Resources, or the Administrator to obtain free markers. Refer to Volume IV, Appendix IV-D for additional information.

3.4.6 Flow Splitter Designs

Many water quality facilities can be designed as flow-through or on-line systems, with flows above the water quality design flow or volume simply passing through the facility at a lower pollutant removal efficiency. However, it is sometimes desirable to restrict flows to water quality treatment facilities and bypass the remaining higher flows around them through off-line facilities. This can be accomplished by splitting flows in excess of the water quality design flow upstream of the facility and diverting higher flows to a bypass pipe or channel. The bypass typically enters a flow control facility or the downstream receiving drainage system, depending on flow control requirements. In most cases, it is a designer's choice whether water quality facilities are designed as on-line or off-line; an exception is oil/water separators, which must be designed off-line.

A crucial factor in designing flow splitters is to ensure that low flows are delivered to the treatment facility up to the water quality design flow rate. Above this rate, additional flows are diverted to the bypass system with minimal increase in head at the flow splitter structure to avoid surcharging the water quality facility under high flow conditions. Flow splitters may be used for purposes other than diverting flows to water quality facilities. However, the following information is generally focused on using flow splitters in association with water quality facilities.

Flow splitters are typically manholes or vaults with concrete baffles. In place of baffles, the splitter mechanism may be a half T-section with a solid top and an orifice in the bottom of the T-section. A full T option may also be used, as described below under "General Design Recommendations." Two possible design options for flow splitters are shown in Figure 3.1 and Figure 3.2. Other equivalent designs that achieve the result of splitting low flows and diverting higher flows around the facility are also acceptable.

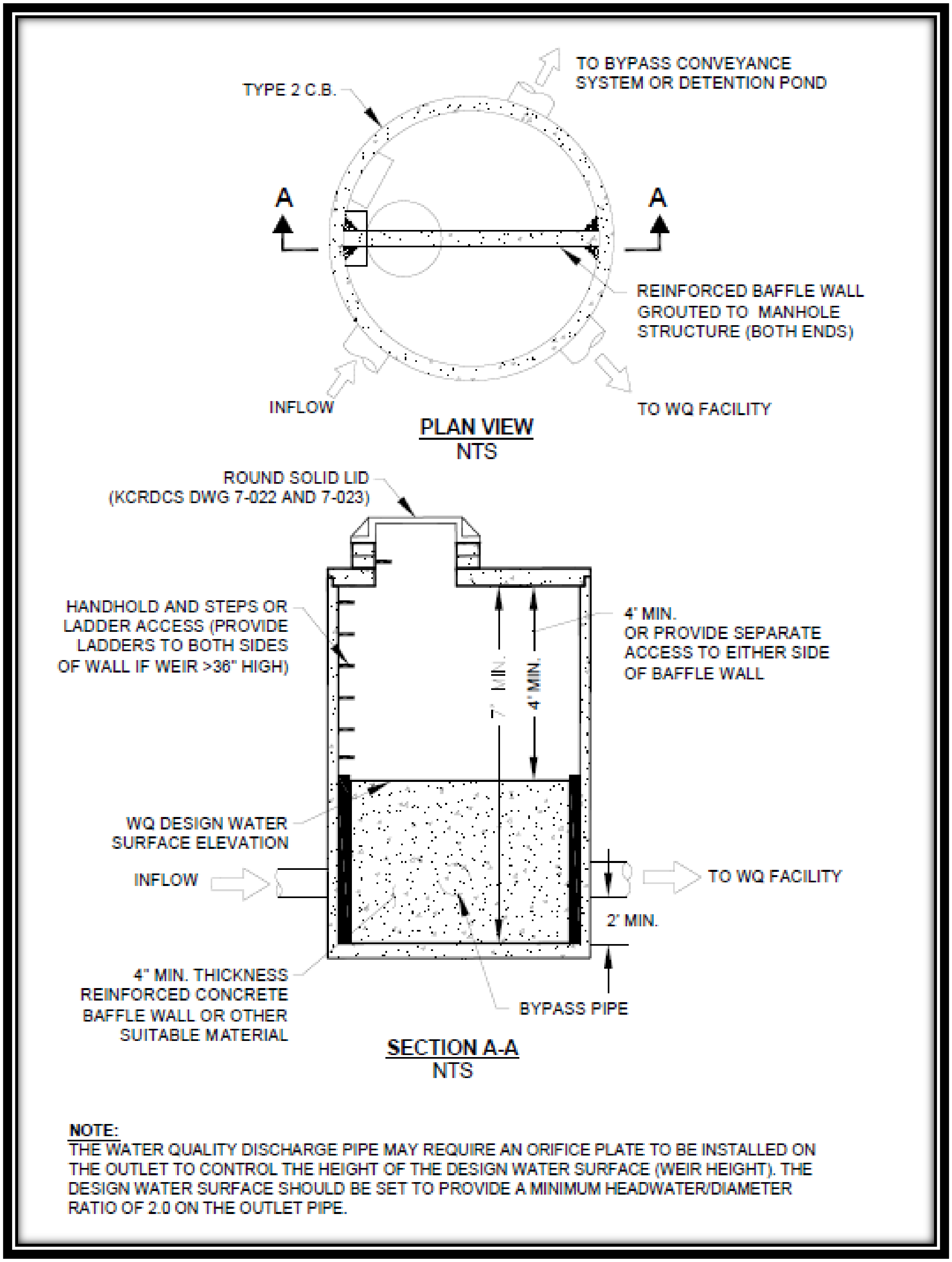
General Design Recommendations

- Unless otherwise specified, a flow splitter shall be designed to deliver the specified water quality design flow rate to the water quality treatment facility (see also Volume V). Flows modeled using a continuous simulation runoff model should use 15-minute time steps.

- The top of the weir shall be located at the water surface for the design flow. Remaining flows enter the bypass line.

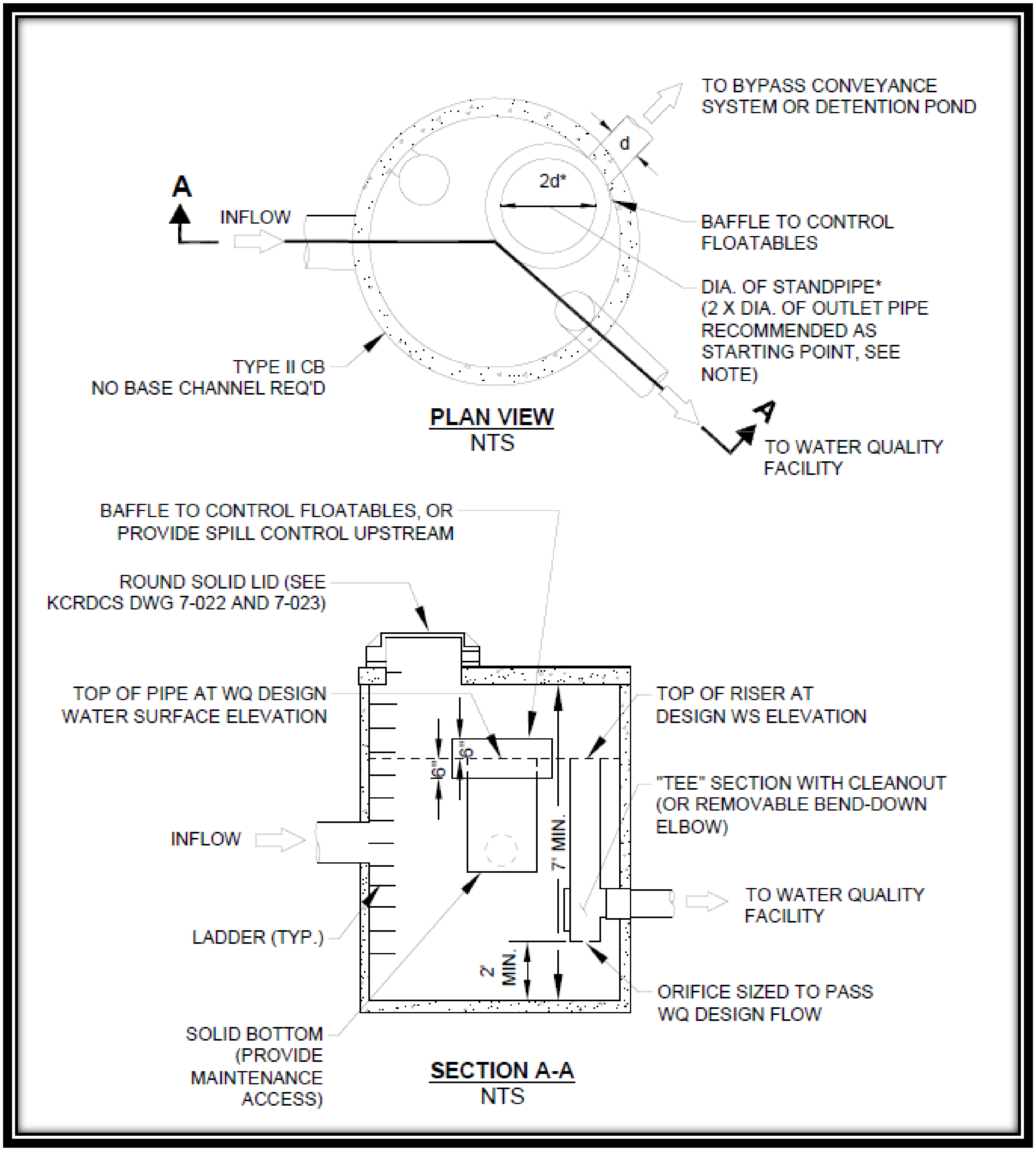
- The maximum head should be minimized for flow in excess of the water quality design flow. Specifically, flow to the water quality facility at the 100-year water surface shall not increase the design water quality flow by more than 10 percent.

- Design Options A and B (shown in Figure 3.1 and Figure 3.2, respectively) or an equivalent design may be used.



Source: King County Surface Water Design Manual, 2016.

Figure 3.1. Flow Splitter, Option A.



Source: King County Surface Water Design Manual, 2016.

Figure 3.2. Flow Splitter, Option B.

- As an alternative to using a solid top plate in Option B (Figure 3.2), a full T-section may be used, with the top of the T-section at the 100-year water surface. This alternative would route emergency overflows (if the overflow pipe were plugged) through the water quality facility rather than back up from the manhole.

- Special applications, such as roads, may require the use of a modified flow splitter. The baffle wall may be fitted with a notch and adjustable weir plate to proportion runoff volumes other than high flows.

- For ponding facilities, backwater effects must be included in designing the height of the standpipe in the manhole.

- Ladder or step-and-handhold access must be provided. If the weir wall is higher than 36 inches, two ladders, one to either side of the wall, shall be used.

Materials

- The splitter baffle may be installed in a Type 2 manhole or vault.

- The baffle wall shall be made of reinforced concrete or another suitable material resistant to corrosion, and have a minimum 4-inch thickness. The minimum clearance between the top of the baffle wall and the bottom of the manhole cover shall be 4 feet; otherwise, dual access points shall be provided.

- All metal parts must be corrosion resistant. Examples of preferred materials include aluminum, stainless steel, and plastic. Zinc and galvanized materials are not allowed because of aquatic toxicity. Painted metal parts shall not be used because of poor longevity.

3.4.7 Outfalls

All piped discharges to streams, rivers, ponds, lakes, or other open bodies of water are designated outfalls, and shall provide for energy dissipation to prevent erosion at or near the point of discharge. Properly designed outfalls are critical to reducing the chance of adverse impacts as the result of concentrated discharges from pipe systems and culverts, both on site and downstream. Outfall systems include rock splash pads, flow dispersal trenches, gabion or other energy dissipaters, and tightline systems. A tightline system is typically a continuous length of pipe used to convey flows down a steep or sensitive slope with appropriate energy dissipation at the discharge end.

Outfalls to streams, wetlands, or other waters of the State may be subject to local review through the State Environmental Policy Act (SEPA) process, Shoreline Management Act, Tumwater Marine Bluff requirements, and other applicable regulations. They may also be subject to state or federal requirements, including hydraulic and permitting requirements of WDFW, WDNR, U.S. Army Corps of Engineers. The requirements of these other reviews and permitting processes shall take precedence where more restrictive than those stated herein.

General Design Criteria for Outfall Features

All energy dissipation at outfalls shall be designed for peak flows from a 100-year, 24-hour storm event. For outfalls with a maximum flow velocity of less than 10 feet per second, a rock splash pad is acceptable. For velocities equal to or greater than 10 feet per second, an engineered energy dissipater must be provided. See Table 3.9 for a summary of the rock protection requirements at outfalls.

Outfalls must be protected against under-cutting. Scour, sedimentation, anchor damage, and other potentially damaging effects must also be considered. Pipe and fittings materials shall be corrosion resistant, such as aluminum, plastic, HDPE, and fiberglass. Galvanized or coated steel will not be acceptable.

The design is limited to slopes of 2:1 or flatter where native vegetation is well established or where slope armoring is engineered to the Administrator's satisfaction. For sites where the Project Engineer determines and the Administrator agrees that the standard is impractical because of lack of space, danger of erosion, etc., alternate outfall designs may be acceptable. Other outfall designs will be allowed upon approval of the Administrator.

Table 3.9. Rock Protection at Outfalls.

Discharge Velocity at Design Flow (fps)	Required Protection Minimum Dimensions				
	Type	Thickness	Width	Length	Height
0 to 5	Rock lining[a]	1 foot	Diameter + 6 feet	8 feet or 4 x diameter, whichever is greater	Crown + 1 foot
5+ to 10	Riprap[b]	2 feet	Diameter + 6 feet or 3 x diameter, whichever is greater	12 feet or 4 x diameter, whichever is greater	Crown + 1 foot
12+	Engineered Design	As required	As required	As required	Crown + 1 foot

[a] **Rock lining** shall be quarry spalls with gradation as follows:

Passing 8-inch-square sieve: 100 percent
Passing 3-inch-square sieve: 40 to 60 percent maximum
Passing 0.75-inch-square sieve: 0 to 10 percent maximum

[b] **Riprap** shall be reasonably well graded with gradation as follows:

Maximum stone size: 24 inches (nominal diameter)
Median stone size: 16 inches
Minimum stone size: 4 inches

fps = feet per second

Note: Riprap sizing governed by side slopes on outlet channel is assumed to be approximately 3:1.

Outfalls with flow velocity under 12 feet per second and discharge under 2 cubic feet per second for the conveyance system design event (Section 3.1) are to be provided (at minimum) with a splash pad (e.g., rock, gabions, concrete).

An engineered energy dissipater is required for outfalls where flow is 2 cubic feet per second or greater, or velocity is 12 feet per second or greater for the conveyance system design event (Section 3.1). Examples are gabion splash blocks, stilling basins, drop pools, hydraulic jump pools, baffled aprons, bubble-up structures, etc.

Outfalls on Steep Slopes

Outfall pipes on steep slopes (refer to Table 3.6) must be anchored and must be fused or butt-welded or mechanically restrained. They may not be gasketed, slip fit, or banded.

On steep slopes, HDPE pipe may be laid on the surface or in a shallow trench, anchored, protected against sluicing, and hand compacted.

HDPE outfall systems must be designed to address the material limitations as specified by the manufacturer—in particular, thermal expansion and contraction. The coefficient of thermal expansion and contraction for HDPE is on the order of 0.001 inch per foot per Fahrenheit degree. Sliding connections to address this thermal expansion and contraction must be located as close to the discharge end of the outfall system as is practical.

HDPE systems longer than 100 feet must be secured at the upstream end, and the downstream end must be placed in a 4-foot section of the next larger pipe size. This sliding sleeve connection allows for high thermal expansion/contraction.

HDPE shall comply with the requirements of Type III C5P34, as tabulated in ASTM D1248, and shall have the PPI recommended designation of PE3408 and the ASTM D3350 cell classification of 345434C or 345534C. The pipe shall have a manufacturer’s recommended hydrostatic design stress rating of 800 psi based on a material with a 1,600 psi design basis determined in accordance with ASTM D2837-69. The pipe shall have a suggested design working pressure of 50 psi at 73.4 degrees Fahrenheit and an SDR of 32.5.

General Design Criteria to Protect Aquatic Species and Habitat

Outfall structures shall be located where they minimize impacts to fish, shellfish, and their habitats. However, new pipe outfalls can also provide an opportunity for low-cost fish habitat improvements. For example, an alcove of low-velocity water can be created by constructing the pipe outfall and associated energy dissipater back from the stream edge and digging a channel, over-widened to the upstream side, from the outfall to the stream (see Figure 3.4). Overwintering juvenile and migrating adult salmonids may use the alcove as shelter during high flows. Potential habitat improvements should be discussed with a WDFW biologist prior to inclusion in design.

Bank stabilization, bioengineering, and habitat features may be required for disturbed areas. Outfalls that discharge to Puget Sound or a major water body may require tide gates. Contact the city for specific requirements.

Outfalls to streams, wetlands, or other waters of the State may be subject to review through the SEPA process, Shoreline Management Act, and other applicable regulations. Outfalls also may be subject to hydraulic project permitting requirements of WDFW, WDNR, and the U.S. Army Corps of Engineers, which shall take precedence where more restrictive than those stated herein.

Rock Splash Pad

At a minimum, all outfalls must be provided with a rock splash pad (see Figure 3.5) except as specified below and in Table 3.9.

The flow dispersal trenches (see also Figures 3.6 and 3.7) shall only be used when both criteria below are met:

- An outfall is necessary to disperse concentrated flows across uplands where no conveyance system exists and the natural (existing) discharge is unconcentrated
- The 100-year peak discharge rate is less than or equal to 0.5 cubic feet per second.

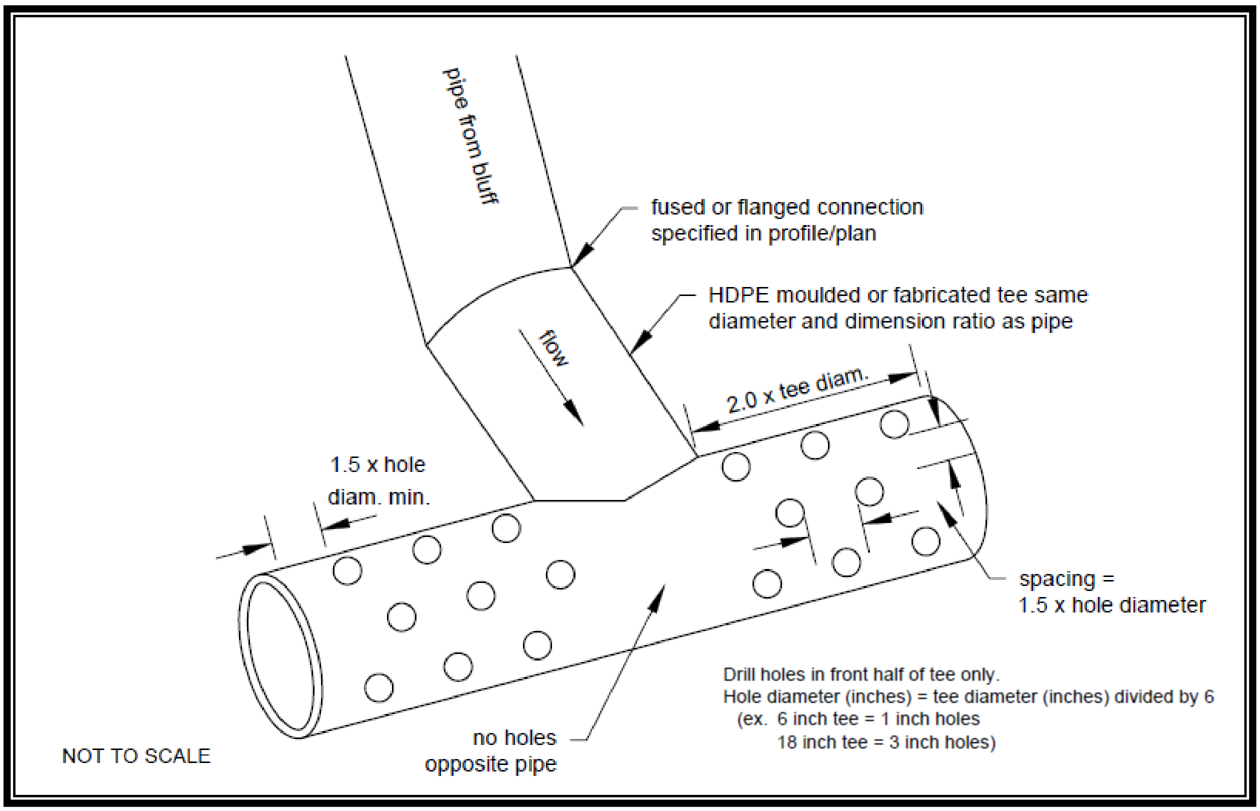


Figure 3.3. Diffuser Tee (example of energy-dissipating end feature).

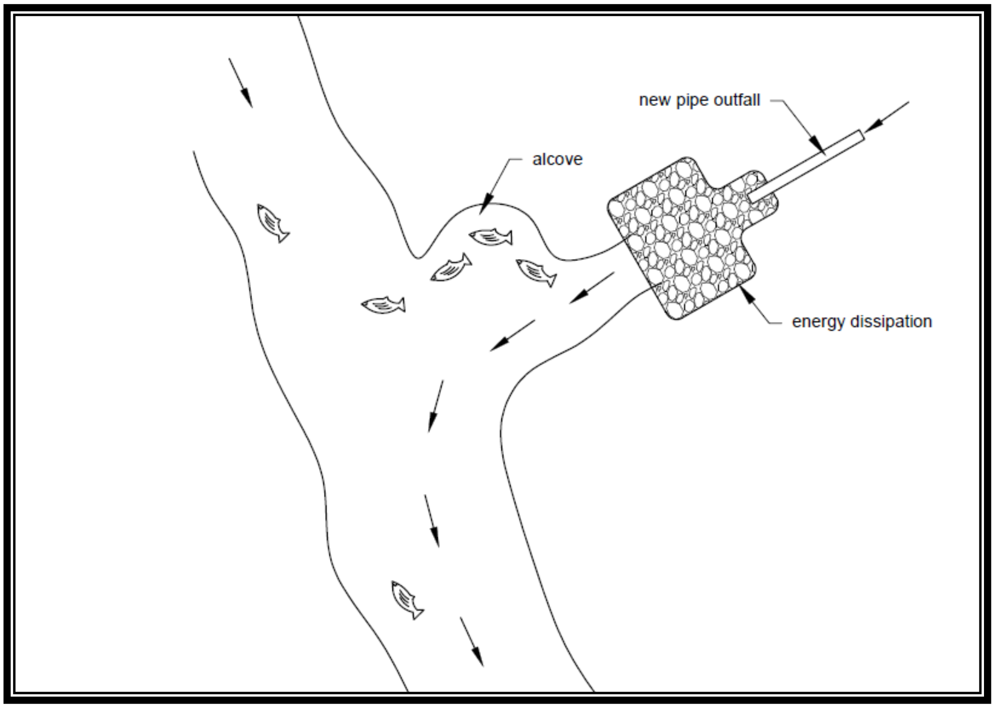


Figure 3.4. Example Fish Habitat Improvement at New Outfall.

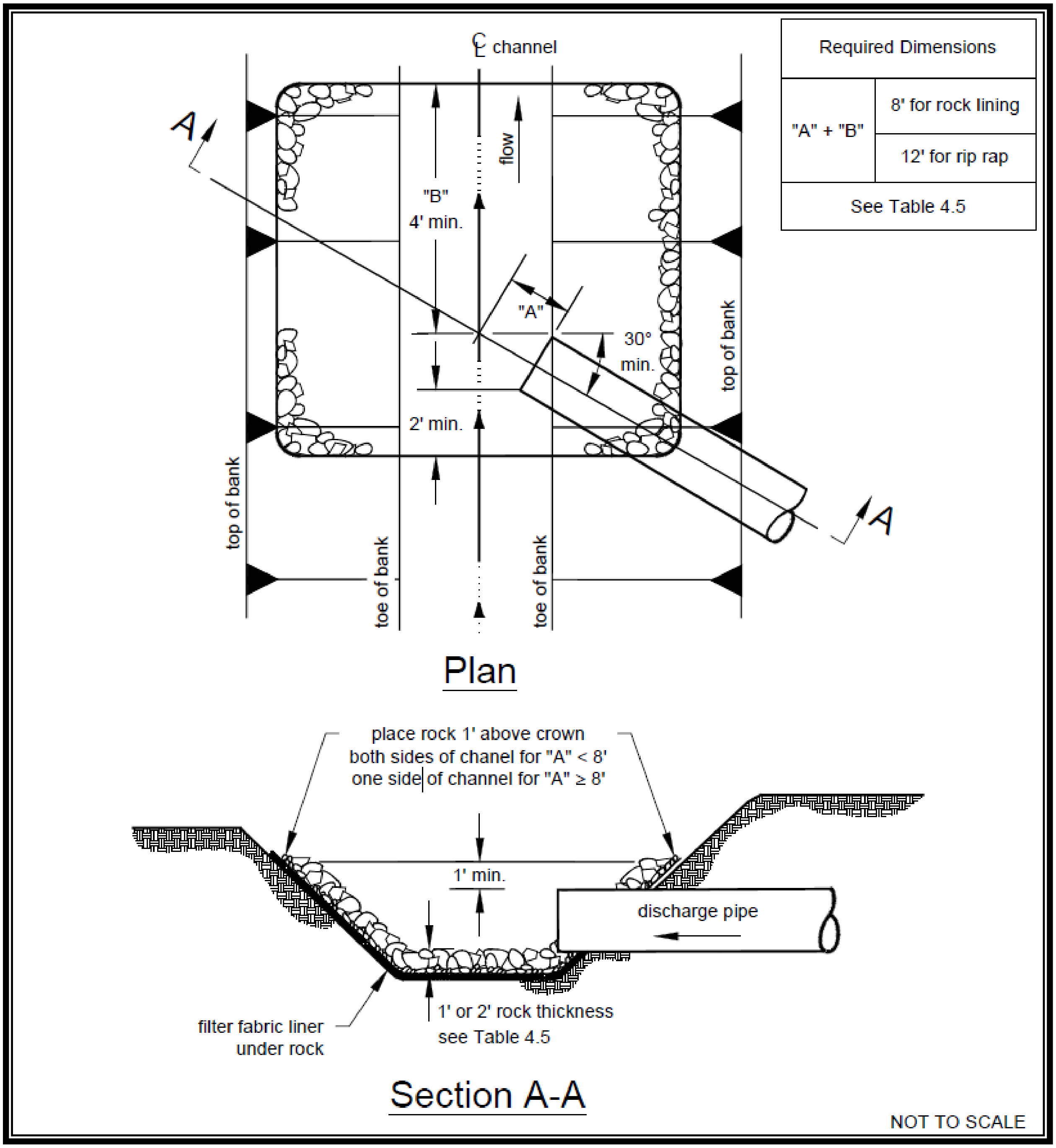


Figure 3.5. Pipe/Culvert Outfall Discharge Protection.

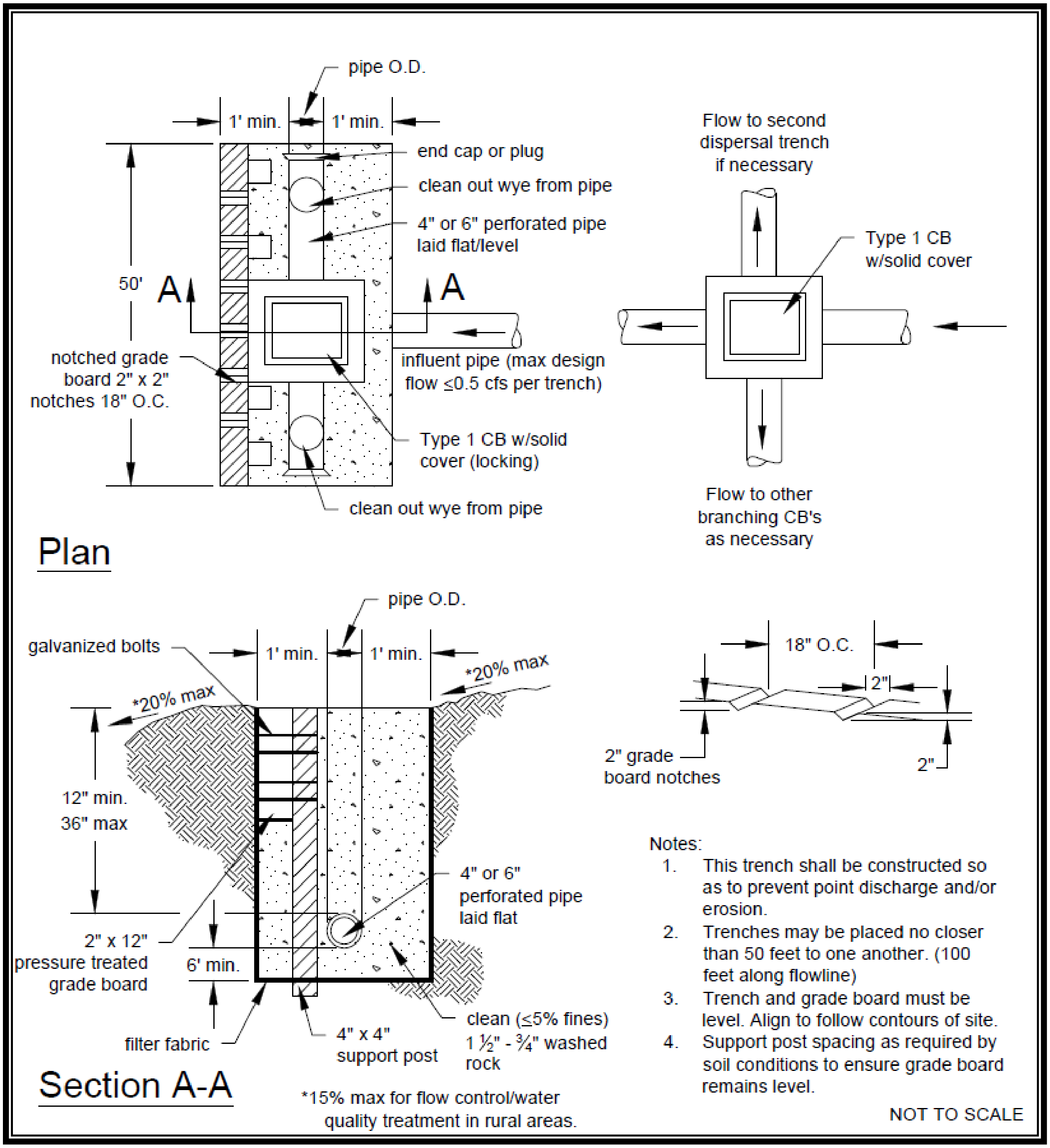
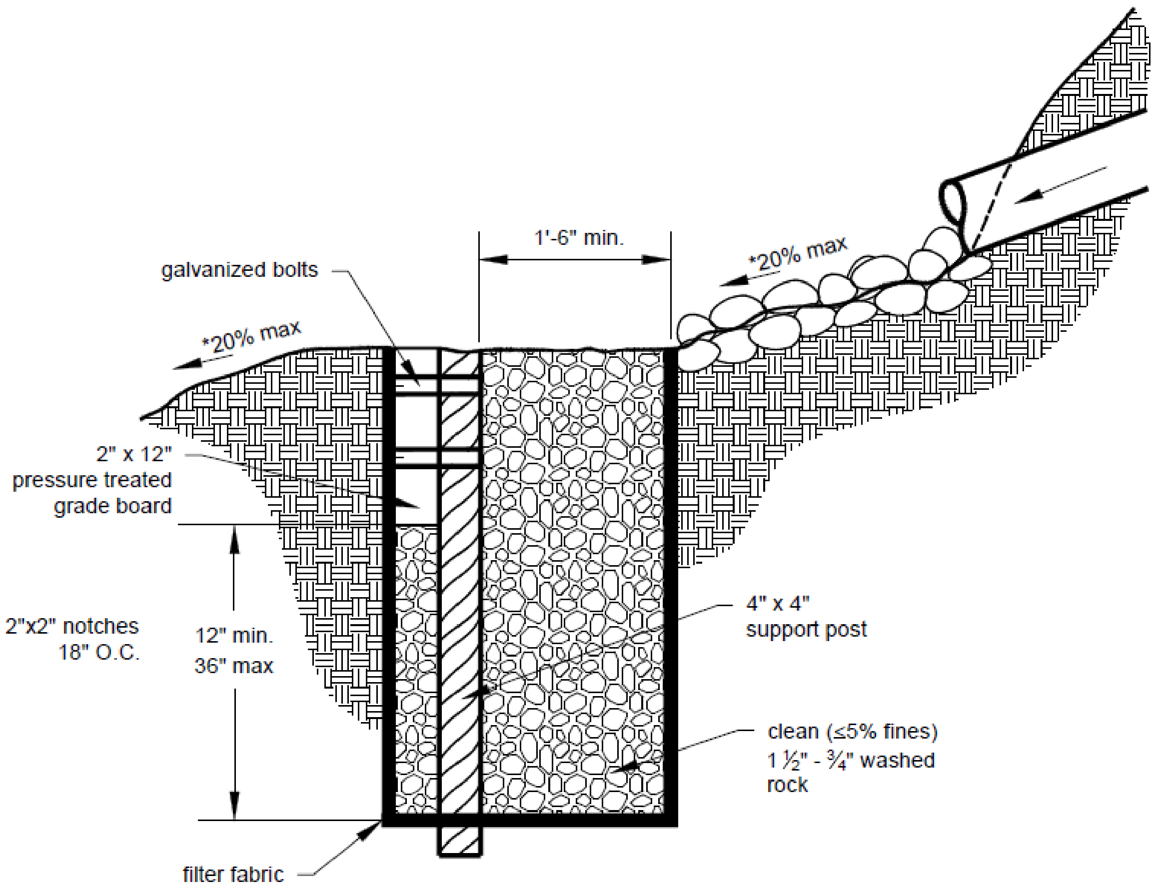


Figure 3.6. Flow Dispersal Trench.



*15% max for flow control/water quality treatment in rural areas.

Section A-A

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.

NOT TO SCALE

Figure 3.7. Alternative Flow Dispersal Trench.

Tightline Systems

Tightline systems may be needed to prevent aggravation or creation of a downstream erosion problem. The following general design criteria apply to tightline systems:

- Outfall tightlines may be installed in trenches with standard bedding on slopes up to 20 percent. In order to minimize disturbance to slopes greater than 20 percent, it is recommended that tightlines be placed at grade with proper pipe anchorage and support.

- Except as indicated above, tightlines or conveyances that traverse the marine intertidal zone and connect to outfalls shall be buried to a depth sufficient to avoid exposure of the line during storm events or future changes in beach elevation. If nonnative material is used to bed the tightline, such material shall be covered with at least 3 feet of native bed material or equivalent.

- HDPE pipe tightlines must be designed to address the material limitations, particularly thermal expansion and contraction and pressure design, as specified by the manufacturer. The coefficient of thermal expansion and contraction for solid wall polyethylene (SWPE) pipe is on the order of 0.001 inch per foot per Fahrenheit degree. Sliding sleeve connections shall be used to address this thermal expansion and contraction. These sleeve connections consist of a section of the appropriate length of the next larger size diameter of pipe into which the outfall pipe is fitted. These sleeve connections shall be located as close to the discharge end of the outfall system as is practical.

- Due to the ability of HDPE pipe tightlines to transmit flows of very high energy, special consideration for energy dissipation must be made. Details of a sample gabion mattress energy dissipater have been provided in Figure 3.8. Flows of very high energy will require a specifically engineered energy dissipater structure.

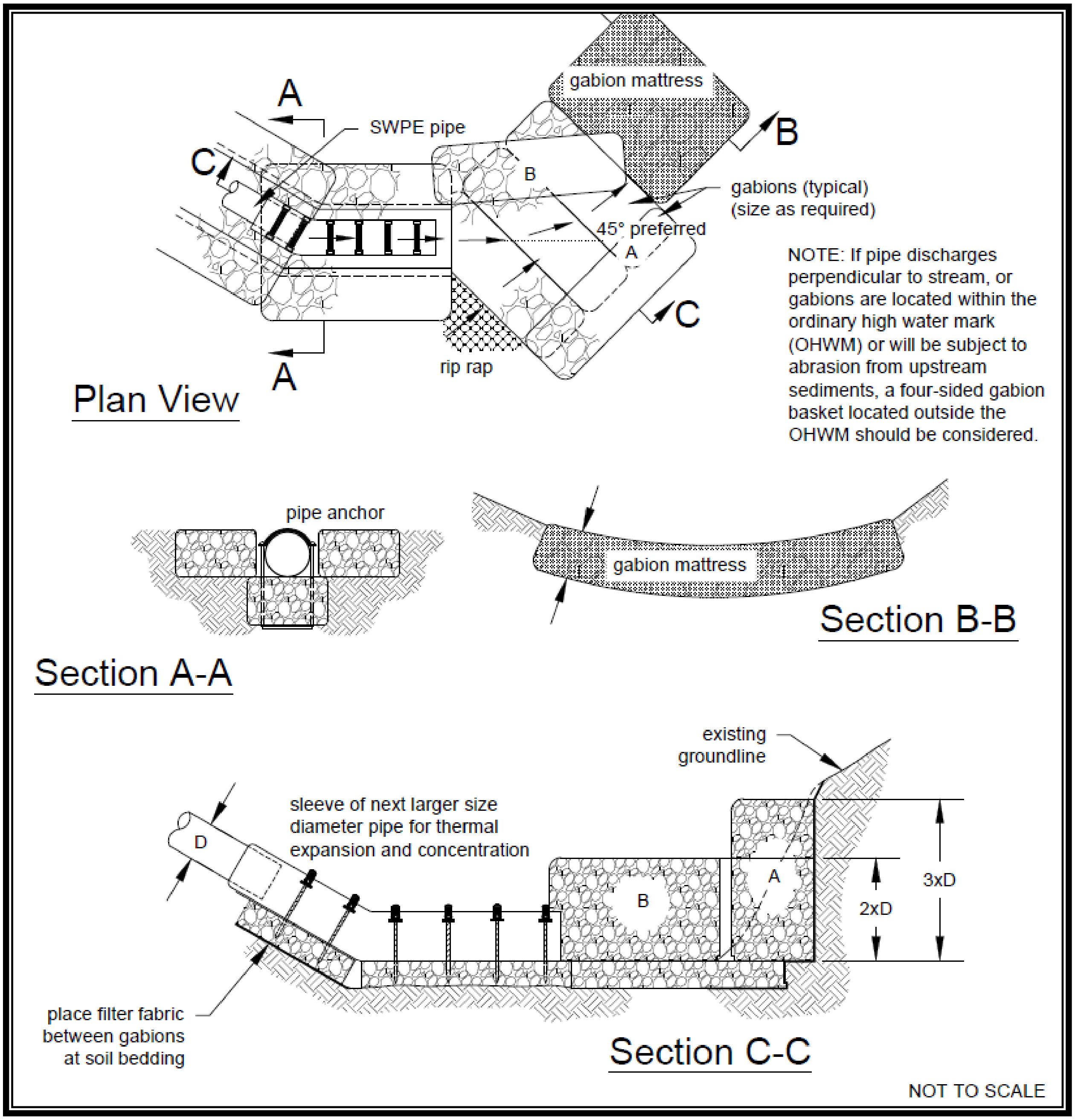


Figure 3.8. Gabion Outfall Detail.

3.4.8 Flow Spreading Options

Flow spreaders function to uniformly spread flows across the inflow portion of several types of stormwater management facilities (e.g., biofiltration swales, filter strips, bioretention areas). There are five flow spreader options presented in this section:

- Option A – Anchored plate
- Option B – Concrete sump box

- Option C – Notched curb spreader
- Option D – Through-curb ports
- Option E – Interrupted curb.

Options A through C can be used for spreading flows that are concentrated. Any one of these options can be used when spreading is required by the facility design criteria. Options A through C can also be used for unconcentrated flows and in some cases must be used, such as to correct for moderate grade changes along a filter strip.

Options D and E are only for flows that are already unconcentrated and enter a filter strip, bioretention area, or continuous inflow biofiltration swale. Other flow spreader options are possible with approval from the City of Tumwater.

General Design Criteria

- Where flow enters the flow spreader through a pipe, it is recommended that the pipe be submerged to the extent practical to dissipate energy as much as possible.
- For higher inflows (velocities greater than 5 feet per second for the 100-year recurrence interval storm), a Type 1 catch basin shall be positioned in the spreader and the inflow pipe shall enter the catch basin with flows exiting through the top grate. The top of the grate shall be lower than the level spreader plate, or if a notched spreader is used, lower than the bottom of the V-notches.

Option A – Anchored Plate (Figure 3.9)

- An anchored plate flow spreader shall be preceded by a sump having a minimum depth of 8 inches and minimum width of 24 inches. If not otherwise stabilized, the sump area shall be lined to reduce erosion and to provide energy dissipation.
- The top surface of the flow spreader plate shall be level, projecting a minimum of 2 inches above the ground surface of the water quality facility, or V-notched with notches 6 to 10 inches on center and 1 to 6 inches deep (use shallower notches with closer spacing). Alternative designs may also be used.
- A flow spreader plate shall extend horizontally beyond the bottom width of the facility to prevent water from eroding the side slope. The horizontal extent shall be such that the bank is protected for all flows up to the 100-year recurrence interval flow or the maximum flow that will enter the water quality facility.
- Flow spreader plates shall be securely fixed in place.
- Flow spreader plates may be made of either wood, metal, fiberglass reinforced plastic, or other durable material. If wood, pressure treated 4- by 10-inch lumber or landscape timbers are acceptable.

- Anchor posts shall be 4-inch-square concrete, tubular stainless steel, or other material resistant to decay.

Option B – Concrete Sump Box (Figure 3.10)

- The wall of the downstream side of a rectangular concrete sump box shall extend a minimum of 2 inches above the treatment bed. This serves as a weir to spread the flows uniformly across the bed.

- The downstream wall of a sump box shall have “wing walls” at both ends. Side walls and returns shall be slightly higher than the weir so that erosion of the side slope is minimized.

- Concrete for a sump box can be either cast-in-place or precast, but the bottom of the sump shall be reinforced with wire mesh for cast-in-place sumps.

- Sump boxes shall be placed over bases that consists of 4 inches of crushed rock, 5/8-inch minus to help ensure the sump remains level.

Option C – Notched Curb Spreader (Figure 3.11)

- Notched curb spreader sections shall be made of extruded concrete laid side-by-side and level. Typically, five “teeth” per 4-foot section provide good spacing. The space between adjacent “teeth” forms a V-notch.

Option D –Through-Curb Ports (Figure 3.12)

- Unconcentrated flows from paved areas entering filter strips, bioretention areas, or continuous inflow biofiltration swales can use curb ports or interrupted curbs (Option E) to allow flows to enter the strip or swale. Curb ports use fabricated openings that allow concrete curbing to be poured or extruded while still providing an opening through the curb to admit water to the water quality facility.

- Openings in the curb shall be at regular intervals but at least every 6 feet (minimum). The width of each curb port opening shall be a minimum of 11 inches. Approximately 15 percent or more of the curb section length shall be in open ports, and no port shall discharge more than about 10 percent of the flow.

Option E – Interrupted Curb (No Figure)

- Interrupted curbs are sections of curb placed to have gaps spaced at regular intervals along the total width (or length, depending on facility) of the treatment area. At a minimum, gaps shall be every 6 feet to allow distribution of flows into the treatment facility before they become too concentrated. The opening shall be a minimum of 2 inches. As a general rule, no opening shall discharge more than 10 percent of the overall flow entering the facility.

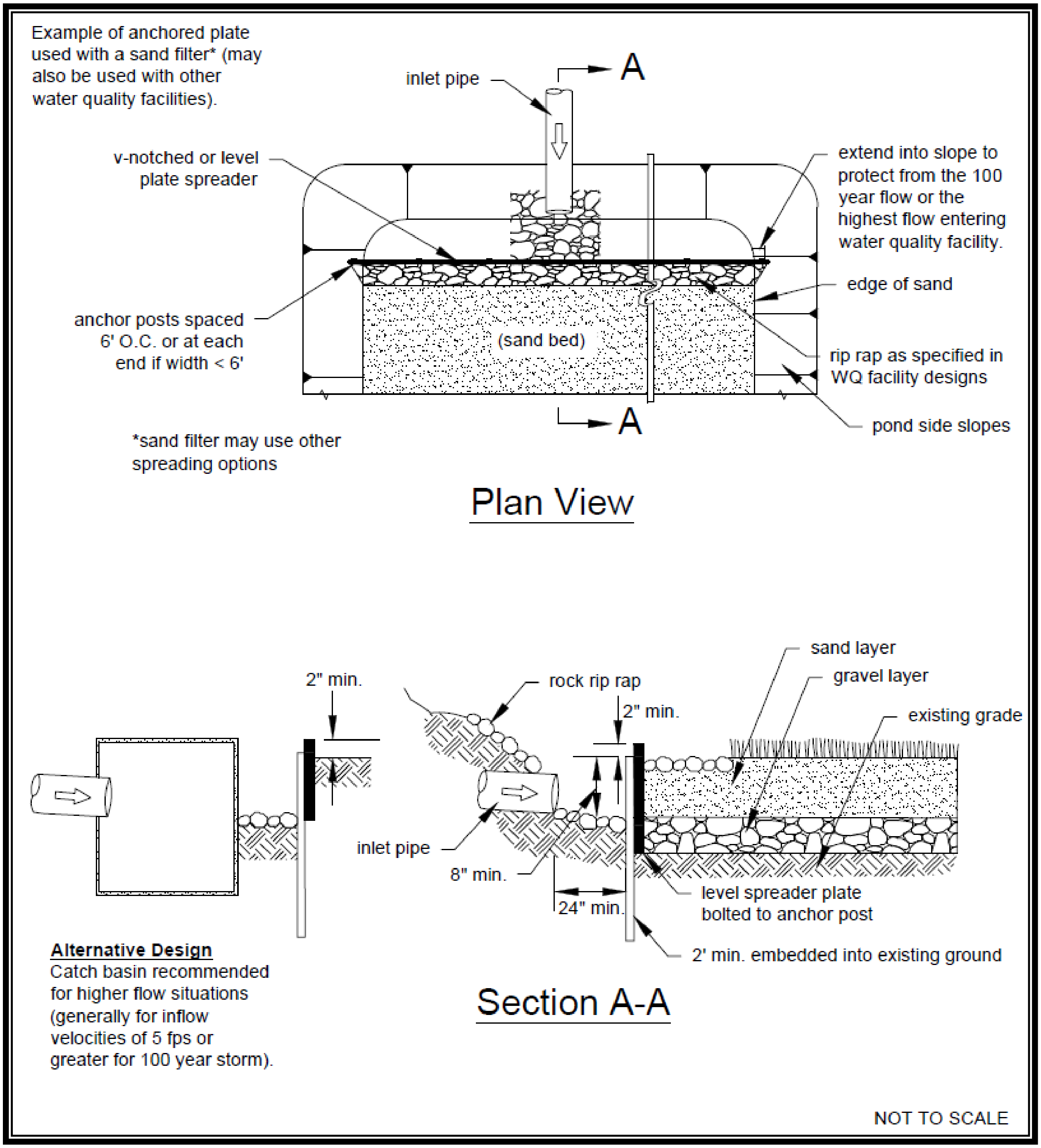


Figure 3.9. Flow Spreader Option A: Anchored Plate.

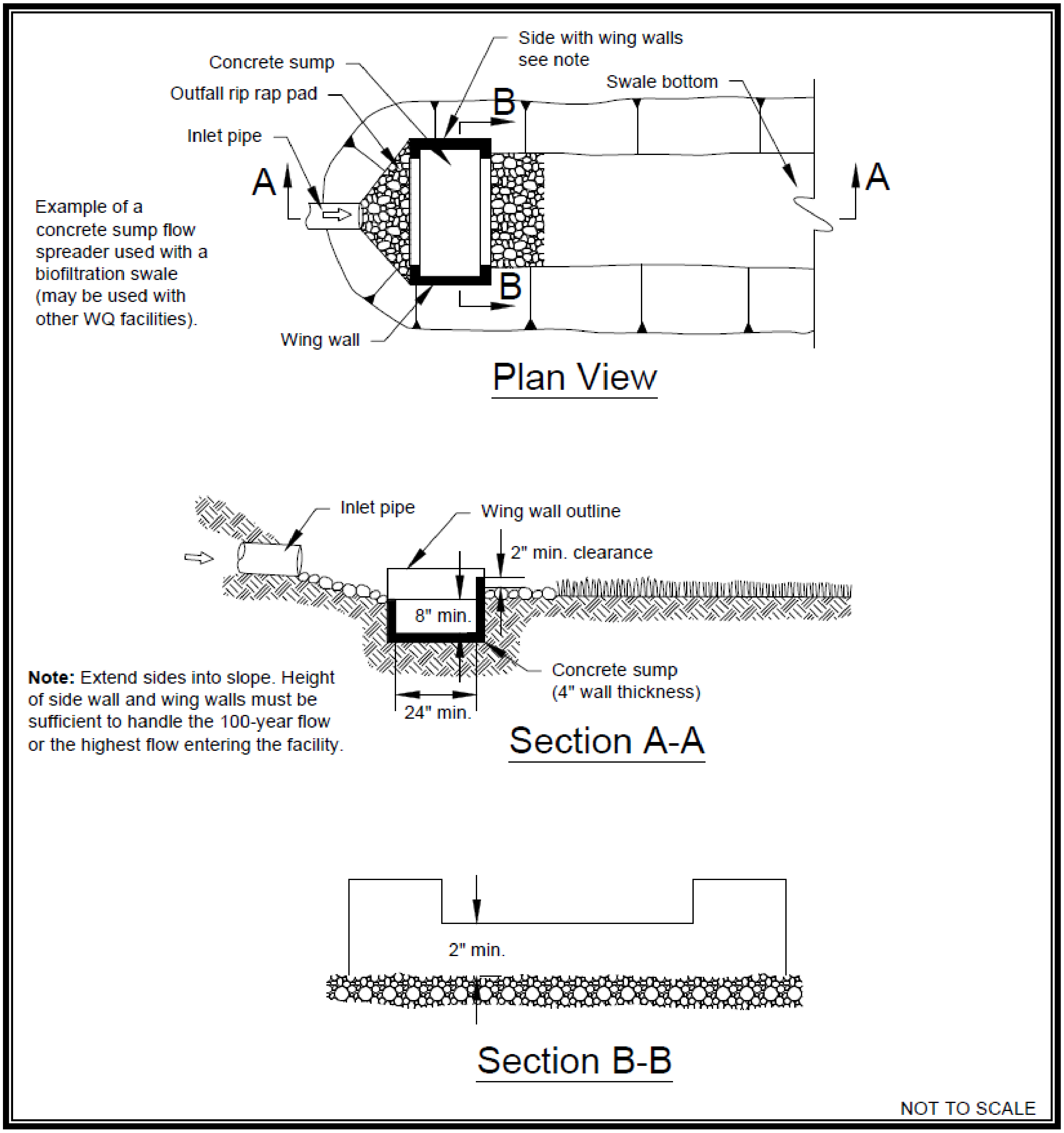


Figure 3.10. Flow Spreader Option B: Concrete Sump Box.

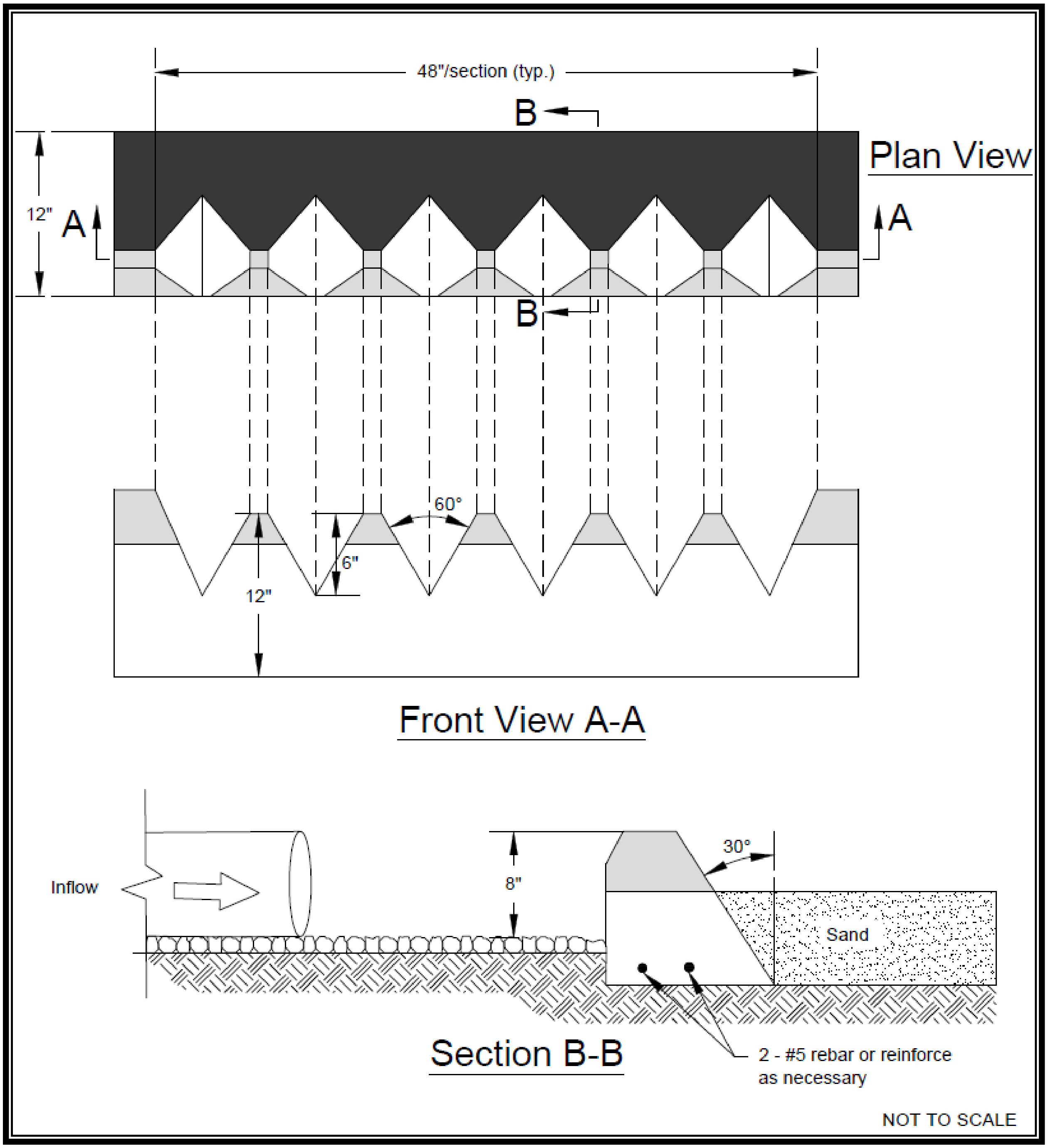


Figure 3.11. Flow Spreader Option C: Notched Curb Spreader.

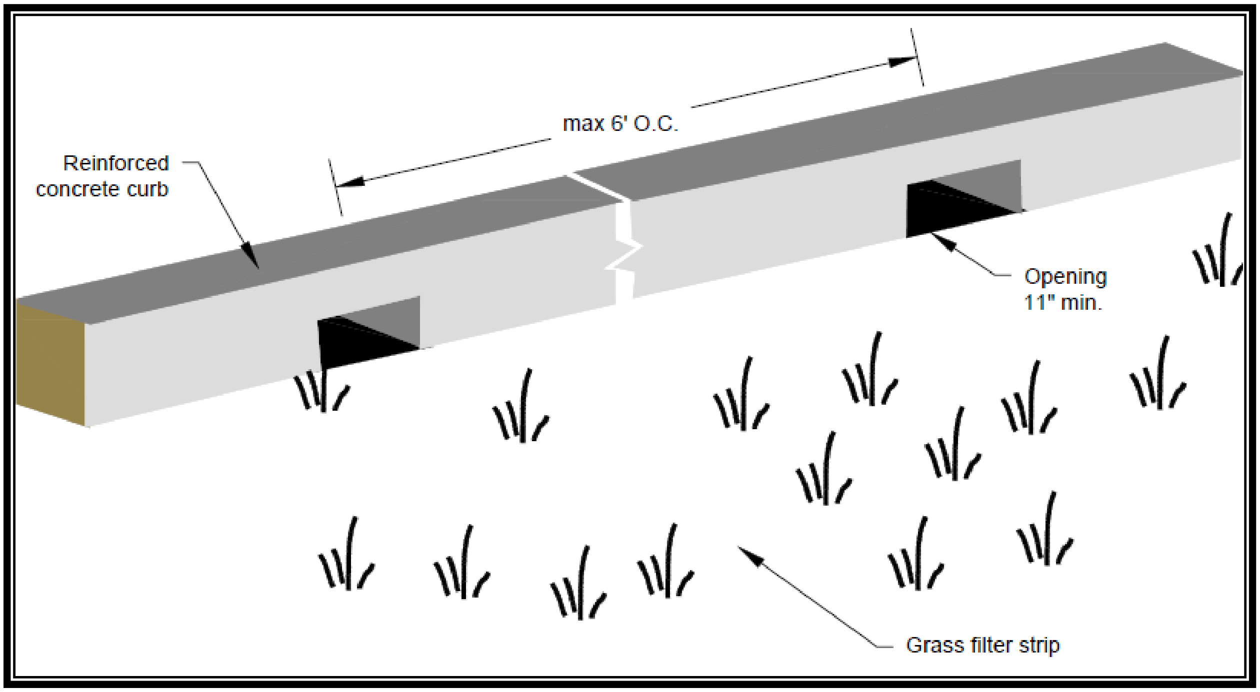


Figure 3.12. Flow Spreader Option D: Through-Curb Port.

3.5 Private Drainage Systems

The engineering analysis for a private drainage system is the same as for a city system. Refer to Section 3.4 for conveyance requirements that also apply to private drainage systems.

Private stormwater conveyance piping shall not be located within the public right-of-way. Where soils or other conditions prohibit infiltration on individual parcels (as determined by the Administrator), stormwater may be conveyed to the stormwater facilities associated with the residential or commercial development. In that case, the stormwater conveyance system located in the public right-of-way shall be sized to accommodate the additional stormwater.

Acceptable Pipe Size

The minimum diameter for storm sewer on private property is 4 inches. When private stormwater (e.g., roof, lot, or footing drains) cannot be infiltrated on individual lots, the minimum standard piping connection to the public system shall be 8-inch PVC.

Discharge Locations

Stormwater will not be permitted to discharge directly onto city roads or into a city system without the prior approval of the city. Discharges to a city system shall be into a structure such as an inlet, catch basin, manhole, through an approved sidewalk underdrain or curb drain, or into an existing or created city ditch. Multiple roof drains shall be terminated at a common junction structure outside of the right-of-way (i.e., catch basin or manhole). The connection from the common junction structure to the city's storm system shall be through an 8-inch main connecting to a city catch basin or manhole. The 8-inch main used for connection shall begin at the right-of-way, the connection to the catch basin or manhole shall be cored. Concentrated drainage will not be allowed to discharge across sidewalks, curbs, or driveways.

All buildings are required to have roof downspouts and subsurface drains directed to either an infiltration system, dispersion system, or to the storm drainage system.

Drainage Stub-Outs

If drainage outlets (stub-outs) are to be provided for each individual lot, the stub-outs shall conform to the requirements outlined below. Note that all applicable minimum requirements in Volume I, Chapter 2, in particular Minimum Requirement #5, must also be addressed for the project site.

- Each outlet shall be suitably located at the lowest elevation on the lot, so as to service all future roof downspouts and footing drains, driveways, yard drains, and any other surface or subsurface drains necessary to render the lots suitable for their intended use. Each outlet shall have free-flowing, positive drainage to an approved stormwater conveyance system or to an approved discharge location.

- Outlets on each lot shall be located with a 5-foot-high, 2- by 4-inch stake marked "storm" or "drain." For stub-outs to a surface drainage, the stub-out shall visibly extend above surface level and be secured to the stake.

- The developer and/or contractor is responsible for coordinating the locations of all stub-out conveyance lines with respect to the utilities (e.g., power, gas, telephone, television).

- All individual stub-outs shall be privately owned and maintained by the lot home owner including from the property line to the riser on the main line.

Volume III References and Information Sources

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Appendix III-A – Single-Event Model Guidance

The only approved use of a single event model is for the sizing of conveyance systems. Approved continuous simulation runoff models must be used for the design of water quality and quantity BMPs.

III-A.1 Rational Method

The only approved use of the Rational Method is for the sizing of conveyance systems in smaller drainage basins (25 acres or less in size). This method provides an estimate of peak discharge (Q_p in cubic feet per second [cfs]) using the following formula:

$$Q_p = CIA$$

Where: C = runoff coefficient (unitless),

A = area of watershed (acres), and

I = rainfall intensity (inches per hour) for a chosen frequency expressed as:

$$I = \frac{m}{(T_c)^n}$$

Where: m, n are regression coefficients (unitless), and

T_c = time of concentration (in hours).

Runoff coefficient (C) values are listed in Table A.1 for a range of land cover types. Regression coefficients (m, n) for determining rainfall intensity are listed in Table A.2. Time of concentration (T_c) is calculated as described under the “Time of Concentration” heading, below.

III-A.2 SBUH or SCSUH Methods

The applicant shall use the western Washington Soil Conservation Service (SCS) curve numbers, not the national SCS curve numbers. The western Washington SCS curve numbers are included in Table A.7. (Tables A.3 through A.8 can be found at the end of this section, prior to Figures A.1 through A.4.) Individual curve numbers for a drainage area may be averaged into a "composite" curve number for use in either the SCS or SBUH methods. The Natural Resources Conservation Service (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 (SCS 1972) contains a detailed description of the development and use of the curve number method.

NRCS has developed "curve number" values based on soil type and land use. They can be found in "Urban Hydrology for Small Watersheds," Technical Release 55 (TR-55; NRCS 1986). The combination of soil type and land use 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 A.8 shows the hydrologic soil group of most soils in the city and provides a brief description of the four groups. For details on other soil types refer to TR-55.

Isopluvial Maps

Included in this appendix are the 2-, 10-, 25-, 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 (NOAA 1973). The applicant has the option of using the National Oceanic and Atmospheric Administration (NOAA) isopluvials for design purposes or using the design storm precipitation values listed in Table A.3. The listed values can be used to an elevation of 650 feet, mean sea level (MSL). Above 650 feet MSL, the applicant shall use the NOAA isopluvials for selection of the design storm precipitation, unless otherwise approved by the city.

The Project Engineer shall use the best engineering judgment in selecting the runoff totals for the project site.

Time of Concentration

Time of concentration is the sum of the travel times for sheet flow, shallow concentrated flow, and channel flow. For lakes and submerged wetlands, the travel time can be determined with storage routing techniques if the stage-storage versus discharge relationship is known or it may be assumed to be zero.

Sheet Flow

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 A.5 gives Manning's n_s values for sheet flow for various surface conditions.

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 (minutes)

n_s = sheet flow Manning's effective roughness coefficient (Table A.5)

L = flow length (feet)

P_2 = 2-year, 24-hour rainfall (inches)

S_o = slope of hydraulic grade line (land slope, feet/foot)

The maximum allowable distance for sheet flow shall be 300 feet, the remaining overland flow distance shall be shallow concentrated flow until the water reaches a channel.

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 A.5 in which average velocity is a function of watercourse slope and type of channel.

The average velocity of flow, once it has measurable depth, shall be computed using the following equation:

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

Where: V = velocity (feet per second)

k = time of concentration velocity factor (feet per second)

S_o = slope of flowpath (feet/foot)

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

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

Where: R = an assumed hydraulic radius

n = Manning's roughness coefficient for open channel flow (see Table A.6)

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 kc values from Table A.5 used in the Velocity Equation, above, or water surface profile information can be used to estimate average flow velocity.

Lakes or Wetlands

This travel time is normally very small and can be assumed as zero. Where significant attenuation may occur due to storage effects, the flows shall be routed using a "level pool routing" technique.

Limitations

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

- Manning's kinematic solution shall not be used for sheet flow longer than 300 feet.
- In watersheds with storm drains, carefully identify the appropriate hydraulic flowpath to estimate Tc.
- 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 a level pool routing technique used to determine the outflow rating curve through the culvert or bridge.

Design Storm Hyetographs

The standard design hyetograph is the SCS Type 1A 24 hour rainfall distribution, resolved into 6 minute time intervals (for conveyance sizing) (See Table A.9) with the design storm values as shown in Table A.3. Various interpretations of the hyetograph are available and may differ slightly from distributions used in other unit hydrograph based computer simulations. Other distributions will be accepted with adequate justification and as long as they do not increase the allowable release rates.

For project sites with tributary drainage areas above elevation 1,000 feet 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 that occurs during major storm events.

The MSL "factor" is computed as follows:

$$M_s \text{ (in inches)} = 0.004\ (MB_{el} - 1000)$$

Where: M_s = rainfall amount to be added to P_r

Mb_{el} = the mean tributary basin elevation above sea level (in feet)

Estimates of Interception

If interception (the volume of precipitation trapped on vegetation) is modeled, the values shown in Table A.4 shall be used as user inputs.

Subbasin Delineation

Within an overall drainage basin, it may be necessary to delineate separate subbasins based on similar land uses and/or runoff characteristics or when hydraulically "self-contained" areas are found to exist. When this is necessary, separate hydrographs shall be generated, routed, and recombined, after travel time is considered, into a single hydrograph to represent runoff flows into the quantity or quality control facility.

Hydrograph Routing

All hydrographs shall be routed by use of a level pool routing technique. Methods are described in the *Handbook of Applied Hydrology* (Singh 2016) and elsewhere.

It is recommended that all such routing be conducted with the use of a computer program.

Hydrograph Phasing Analysis

Where flows from multiple basins or subbasins having different runoff characteristics and/or travel times combine, the design engineer shall sum the hydrographs after shifting each hydrograph according to its travel time to the discharge location of interest. The resultant hydrograph shall be either routed downstream as required in the downstream analysis, or routed through the control facility.

Hydrologic Soil Groups

For purposes of runoff computations using SBUH or SCSUH methods, soils in Tumwater have the hydrologic soil group designations as listed in Table A.8.

Table A.1. Runoff Coefficients for Rational Method Calculations.

Type of Cover	Flat	Rolling (2 percent to 10 percent)	Hilly (over 10 percent)
Pavement and Roofs	0.90	0.90	0.90
Earth Shoulders	0.50	0.50	0.50
Drives and Walks	0.75	0.80	0.85
Gravel Pavement	0.50	0.55	0.60
City Business Areas	0.80	0.85	0.85
Suburban Residential	0.25	0.35	0.40
Single Family Residential	0.30	0.40	0.50
Multi Units, Detached	0.40	0.50	0.60
Multi Units, Attached	0.60	0.65	0.70
Lawns, Very Sandy Soil	0.05	0.07	0.10
Lawns, Sandy Soil	0.10	0.15	0.20
Lawns, Heavy Soil	0.17	0.22	0.35
Grass Shoulders	0.25	0.25	0.25
Side Slopes, Earth	0.60	0.60	0.60
Side Slopes, Turf	0.30	0.30	0.30
Median Areas, Turf	0.25	0.30	0.30
Cultivated Land, Clay and Loam	0.50	0.55	0.60
Cultivated Land, Sand and Gravel	0.25	0.30	0.35
Industrial Areas, Light	0.50	0.70	0.80
Industrial Areas, Heavy	0.60	0.80	0.90
Parks and Cemeteries	0.10	0.15	0.25
Playgrounds	0.20	0.25	0.30
Woodland and Forests	0.10	0.15	0.20
Meadows and Pasture Land	0.25	0.30	0.35
Pasture with Frozen Ground	0.40	0.45	0.50
Unimproved Areas	0.10	0.20	0.30

Source: WSDOT Hydraulics Manual (2007).

Table A.2. Regression Coefficients for Rational Method Calculations.

	2-Year MRI		5-Year MRI		10-Year MRI		25-Year MRI		50-Year MRI		100-Year MRI	
Location	**m**	**n**	**m**	**n**	**m**	**n**	**m**	**n**	**m**	**n**	**m**	**n**
Olympia	3.82	0.466	4.86	0.472	5.62	0.474	6.63	0.477	7.40	0.478	8.17	0.480
Centralia and Chehalis	3.63	0.506	4.85	0.518	5.76	0.524	7.00	0.530	7.92	0.533	8.86	0.537
Tacoma	3.57	0.516	4.78	0.527	5.70	0.533	6.93	0.539	7.86	0.542	8.79	0.545

Source: WSDOT Hydraulics Manual (2007).

MRI = Mean Recurrence Interval (frequency)

Table A.3. Tumwater Design Storm Precipitation Values.

Return Frequency 24-Hour Storm Event (years)	Precipitation (inches)
0.5	1.79
2	2.80
5	3.75
10	4.35
25	5.10
50	5.65
100	6.15

Note: The 7-day, 100-year storm volume is 12 inches.

Table A.4. Interception Values for Various Land Covers.

Land Cover	Interception (inches)
Heavy Forest	0.15
Light Open Forest	0.12
Pasture and Shrubs	0.10
Lawn	0.05
Bare Ground	0.03
Pavement	0.02

Note: Values shown are about 1/2 of those for dry antecedent conditions found in references.

Table A.5. "n" and "k" Values Used in Time Calculations for Hydrographs.

"n" Sheet Flow Equation Manning's Values (for the initial 300 ft. of travel)	**n_s[a]**
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 (s≤ 0.20 ft/ft)	0.06
Cultivated soil with residue cover (s> 0.20 ft/ft)	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
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 (n = 0.024)	21
7. Concrete pipe (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[b]

[a] Manning values for sheet flow only, from Overton and Meadows 1976 (See TR-55; SCS1986).
"k" Values Used in Travel Time/Time of Concentration Calculations.

[b] Determined from Table A.5.

Source: Ecology's *Stormwater Management Manual for the Puget Sound Basin*, February 1992.

Table A.6. Values of the Roughness Coefficient "n".	
Type of Channel and Description	**Manning's "n"**
A. Constructed Channels	
a. Earth, straight and uniform	
1. Clean, recently completed	0.018
2. Gravel, uniform section, clean	0.025
3. With short grass, few weeds	0.027
b. Earth, winding and sluggish	0.025
1. No vegetation	0.025
2. Grass, some weeds	0.030
3. Dense weeds or aquatic plants in deep channels	0.035
4. Earth bottom and rubble sides	0.030
5. Stony bottom and weedy banks	0.035
6. Cobble bottom and clean sides	0.040
c. Rock lined	
1. Smooth and uniform	0.035
2. Jagged and irregular	0.040
d. Channels not maintained, weeds and brush uncut	
1. Dense weeds, high as flow depth	0.080
2. Clean bottom, brush on sides	0.050
3. Same as above, highest stage of flow	0.070
4. Dense brush, high stage	0.100
B. Natural Streams	
B-1. Minor Streams (top width at flood stage <100 feet)	
a. Streams on plain	
1. Clean, straight, full stage no rifts or deep pools	0.030
2. Same as above, but more stones and weeds	0.035
3. Clean, winding, some pools and shoals	0.040
4. Same as above, but some weeds	0.040
5. Same as 4, but more stones	0.050
6. Sluggish reaches, weedy deep pools	0.070
7. Very weedy reaches, deep pools, or floodways with heavy stand of timber and underbrush	0.100
b. Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stages	
1. Bottom: gravel, cobbles, and few boulders	0.040
2. Bottom: cobbles with large boulders	0.050
B-2. Flood Plains	
a. Pasture, no brush	
1. Short grass	0.030
2. High grass	0.035

Table A.6 (continued). Values of the Roughness Coefficient "n".

Type of Channel and Description	Manning's "n"
B-2. Flood Plains (continued)	
b. Cultivated areas	
1. No crop	0.030
2. Mature row crops	0.035
3. Mature field crops	0.040
c. Brush	
1. Scattered brush, heavy weeds	0.050
2. Light brush and trees	0.060
3. Medium to dense brush	0.070
4. Heavy, dense brush	0.100
d. Trees	
1. Dense willows, straight	0.150
2. Cleared land with tree stumps, no sprouts	0.040
3. Same as above, but with heavy growth of sprouts	0.060
4. Heavy stand of timber, a few down trees, little undergrowth, flood stage below branches	0.100
5. Same as above, but with flood stage reaching branches	0.120

Ref: Ecology's *Stormwater Management Manual for the Puget Sound Basin*, February 1992.

Table A.7. Runoff Curve Numbers for Selected Agricultural, Suburban, and Urban Areas.				
(Sources: TR 55, 1986, and Stormwater Management Manual, 1992.)				
	Curve Numbers for Hydrologic Soil Group			
Cover Type and Hydrologic Condition	**A**	**B**	**C**	**D**
Curve Numbers for Predevelopment 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 Postdevelopment Conditions				
Open Space (lawns, parks, golf courses, cemeteries, landscaping, etc.)[a]				
Fair condition (grass cover on 50% to 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[b], driveways, etc. (excluding right-of-way)	98	98	98	98
Permeable Pavement				
Landscaped area	77	85	90	92
50% landscaped area/50% impervious	87	91	94	96
100% impervious area	98	98	98	98
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

Table A.7 (continued). Runoff Curve Numbers for Selected Agricultural, Suburban, and Urban Areas.

Curve Numbers for Postdevelopment Conditions (continued)		
Single-Family Residential[c] (shall only be used for subdivisions >50 acres)		
Dwelling Unit/Gross Acre	**Average Percent Impervious Area[c,d]**	**Curve Number**
1.0 DU/GA	15	Separate curve number shall be selected for pervious and impervious portions of the site or basin
1.5 DU/GA	20	
2.0 DU/GA	25	
2.5 DU/GA	30	
3.0 DU/GA	34	
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	
PUDs, condos, apartments, commercial businesses, industrial areas and subdivisions <50 acres		
	Percent 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 2 of the Soil Conservation Service's Technical Release No. 55 (210-VI-TR-55, Second Ed., June 1986).		

[a] Composite curve numbers may be computed for other combinations of open space cover type.

[b] Where roof runoff and driveway runoff are infiltrated or dispersed according to the requirements in Volume V, 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."

[c] Assumes roof and driveway runoff is directed into street/storm system.

[d] All the remaining pervious area (lawn) are considered to be in good condition for these curve numbers.

Table A.8. Major Soil Groups in Tumwater.

Soil Type	Hydrologic Soil Group	Soil Type	Hydrologic Soil Group
Alderwood	C	Mukilteo	C/D
Baldhill	B	Newberg	B
Baumgard	B	Nisqually	B
Bellingham	C	Norma	D
Boistfort	B	Olympic	B
Bunker	B	Pheeney	C
Cagey	C	Pilchuck	C
Cathcart	B	pits	not applicable
Centralia	B	Prather	C
Chehalis	B	Puget	D
Delphi	B	Puyallup	B
Dupont	D	Rainier	C
Dystric Xerochrepts	C	rock outcrop	not applicable
Eld	B	Raught	B
Everett	A	Riverwash	D
Everson	D	Salkum	B
Galvin	D	Scamman	D
Giles	B	Schneider	B
Godfrey	D	Semiahmoo	C
Grove	A	Shalcar	D
Hoogdal	C	Shalcar Variant	D
Hydraquents	D	Skipopa	D
Indianola	A	Spana	D
Jonas	B	Spanaway	B
Kapowsin	D	Sulton	C
Katulas	C	Tacoma	D
Lates	C	Tenino	C
Mal	C	Tisch	D
Mashel	B	Vailton	B
Maytown	C	Wilkeson	B
Mckenna	D	Xerorthents	C
Melbourne	B	Yelm	C

Hydrologic Soil Group Classifications, as Defined by the Natural Resources 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. Soils have a moderate rate of water transmission (0.15 to 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. Soils have a low rate of water transmission (0.05 to 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. Soils have a very low rate of water transmission (0 to 0.05 in/hr).

Source: NRCS Web Soil Survey, which also contains descriptions for each map unit.

Table A.9. SCS Type IA Storm Rainfall Distribution, 6-Minute Intervals.

Time (hours)	Incremental Rainfall (inches)	Cumulative Rainfall (inches)	Time (hours)	Incremental Rainfall (inches)	Cumulative Rainfall (inches
0	0	0	3.8	0.004	0.109
0.1	0.002	0.002	3.9	0.003	0.112
0.2	0.002	0.004	4	0.004	0.116
0.3	0.002	0.006	4.1	0.004	0.12
0.4	0.002	0.008	4.2	0.003	0.123
0.5	0.002	0.01	4.3	0.004	0.127
0.6	0.002	0.012	4.4	0.004	0.131
0.7	0.002	0.014	4.5	0.004	0.135
0.8	0.002	0.016	4.6	0.004	0.139
0.9	0.002	0.018	4.7	0.004	0.143
1	0.002	0.02	4.8	0.004	0.147
1.1	0.003	0.023	4.9	0.005	0.152
1.2	0.003	0.026	5	0.004	0.156
1.3	0.003	0.029	5.1	0.005	0.161
1.4	0.003	0.032	5.2	0.004	0.165
1.5	0.003	0.035	5.3	0.005	0.17
1.6	0.003	0.038	5.4	0.005	0.175
1.7	0.003	0.041	5.5	0.005	0.18
1.8	0.003	0.044	5.6	0.005	0.185
1.9	0.003	0.047	5.7	0.005	0.19
2	0.003	0.05	5.8	0.005	0.195
2.1	0.003	0.053	5.9	0.005	0.2
2.2	0.003	0.056	6	0.006	0.206
2.3	0.004	0.06	6.1	0.006	0.212
2.4	0.003	0.063	6.2	0.006	0.218
2.5	0.003	0.066	6.3	0.006	0.224
2.6	0.003	0.069	6.4	0.007	0.231
2.7	0.003	0.072	6.5	0.006	0.237
2.8	0.004	0.076	6.6	0.006	0.243
2.9	0.003	0.079	6.7	0.006	0.249
3	0.003	0.082	6.8	0.006	0.255
3.1	0.003	0.085	6.9	0.006	0.261
3.2	0.003	0.088	7	0.007	0.268
3.3	0.003	0.091	7.1	0.007	0.275
3.4	0.004	0.095	7.2	0.008	0.283
3.5	0.003	0.098	7.3	0.008	0.291
3.6	0.003	0.101	7.4	0.009	0.3

Table A.9 (continued). SCS Type IA Storm Rainfall Distribution, 6-Minute Intervals.

Time (hours)	Incremental Rainfall (inches)	Cumulative Rainfall (inches)	Time (hours)	Incremental Rainfall (inches)	Cumulative Rainfall (inches)
3.7	0.004	0.105	7.5	0.01	0.31
7.6	0.021	0.331	11.4	0.004	0.641
7.7	0.024	0.355	11.5	0.004	0.645
7.8	0.024	0.379	11.6	0.004	0.649
7.9	0.024	0.403	11.7	0.004	0.653
8	0.022	0.425	11.8	0.004	0.657
8.1	0.014	0.439	11.9	0.003	0.66
8.2	0.013	0.452	12	0.004	0.664
8.3	0.01	0.462	12.1	0.004	0.668
8.4	0.01	0.472	12.2	0.003	0.671
8.5	0.008	0.48	12.3	0.004	0.675
8.6	0.009	0.489	12.4	0.004	0.679
8.7	0.009	0.498	12.5	0.004	0.683
8.8	0.007	0.505	12.6	0.004	0.687
8.9	0.008	0.513	12.7	0.003	0.69
9	0.007	0.52	12.8	0.004	0.694
9.1	0.007	0.527	12.9	0.003	0.697
9.2	0.006	0.533	13	0.004	0.701
9.3	0.006	0.539	13.1	0.004	0.705
9.4	0.006	0.545	13.2	0.003	0.708
9.5	0.005	0.55	13.3	0.004	0.712
9.6	0.006	0.556	13.4	0.004	0.716
9.7	0.005	0.561	13.5	0.003	0.719
9.8	0.006	0.567	13.6	0.003	0.722
9.9	0.005	0.572	13.7	0.004	0.726
10	0.005	0.577	13.8	0.003	0.729
10.1	0.005	0.582	13.9	0.004	0.733
10.2	0.005	0.587	14	0.003	0.736
10.3	0.005	0.592	14.1	0.003	0.739
10.4	0.004	0.596	14.2	0.004	0.743
10.5	0.005	0.601	14.3	0.003	0.746
10.6	0.005	0.606	14.4	0.003	0.749
10.7	0.004	0.61	14.5	0.004	0.753
10.8	0.005	0.615	14.6	0.003	0.756
10.9	0.005	0.62	14.7	0.003	0.759
11	0.004	0.624	14.8	0.004	0.763
11.1	0.004	0.628	14.9	0.003	0.766
11.2	0.005	0.633	15	0.003	0.769
11.3	0.004	0.637	15.1	0.003	0.772

Table A.9 (continued). SCS Type IA Storm Rainfall Distribution, 6-Minute Intervals.

Time (hours)	Incremental Rainfall (inches)	Cumulative Rainfall (inches)	Time (hours)	Incremental Rainfall (inches)	Cumulative Rainfall (inches)
15.2	0.004	0.776	19	0.003	0.887
15.3	0.003	0.779	19.1	0.003	0.89
15.4	0.003	0.782	19.2	0.002	0.892
15.5	0.003	0.785	19.3	0.003	0.895
15.6	0.003	0.788	19.4	0.002	0.897
15.7	0.004	0.792	19.5	0.003	0.9
15.8	0.003	0.795	19.6	0.003	0.903
15.9	0.003	0.798	19.7	0.002	0.905
16	0.003	0.801	19.8	0.003	0.908
16.1	0.003	0.804	19.9	0.002	0.91
16.2	0.003	0.807	20	0.003	0.913
16.3	0.003	0.81	20.1	0.002	0.915
16.4	0.003	0.813	20.2	0.003	0.918
16.5	0.003	0.816	20.3	0.002	0.92
16.6	0.003	0.819	20.4	0.002	0.922
16.7	0.003	0.822	20.5	0.003	0.925
16.8	0.003	0.825	20.6	0.002	0.927
16.9	0.003	0.828	20.7	0.003	0.93
17	0.003	0.831	20.8	0.002	0.932
17.1	0.003	0.834	20.9	0.002	0.934
17.2	0.003	0.837	21	0.003	0.937
17.3	0.003	0.84	21.1	0.002	0.939
17.4	0.003	0.843	21.2	0.002	0.941
17.5	0.003	0.846	21.3	0.003	0.944
17.6	0.003	0.849	21.4	0.002	0.946
17.7	0.002	0.851	21.5	0.002	0.948
17.8	0.003	0.854	21.6	0.003	0.951
17.9	0.003	0.857	21.7	0.002	0.953
18	0.003	0.86	21.8	0.002	0.955
18.1	0.003	0.863	21.9	0.002	0.957
18.2	0.002	0.865	22	0.002	0.959
18.3	0.003	0.868	22.1	0.003	0.962
18.4	0.003	0.871	22.2	0.002	0.964
18.5	0.003	0.874	22.3	0.002	0.966
18.6	0.002	0.876	22.4	0.002	0.968
18.7	0.003	0.879	22.5	0.002	0.97
18.8	0.003	0.882	22.6	0.002	0.972
18.9	0.002	0.884	22.7	0.002	0.974
22.8	0.002	0.976			
22.9	0.002	0.978			

Table A.9 (continued). SCS Type IA Storm Rainfall Distribution, 6-Minute Intervals.

Time (hours)	Incremental Rainfall (inches)	Cumulative Rainfall (inches)	Time (hours)	Incremental Rainfall (inches)	Cumulative Rainfall (inches)
23	0.002	0.98			
23.1	0.002	0.982			
23.2	0.002	0.984			
23.3	0.002	0.986			
23.4	0.002	0.988			
23.5	0.002	0.99			
23.6	0.002	0.992			
23.7	0.002	0.994			
23.8	0.002	0.996			
23.9	0.002	0.998			
24	0.002	1			

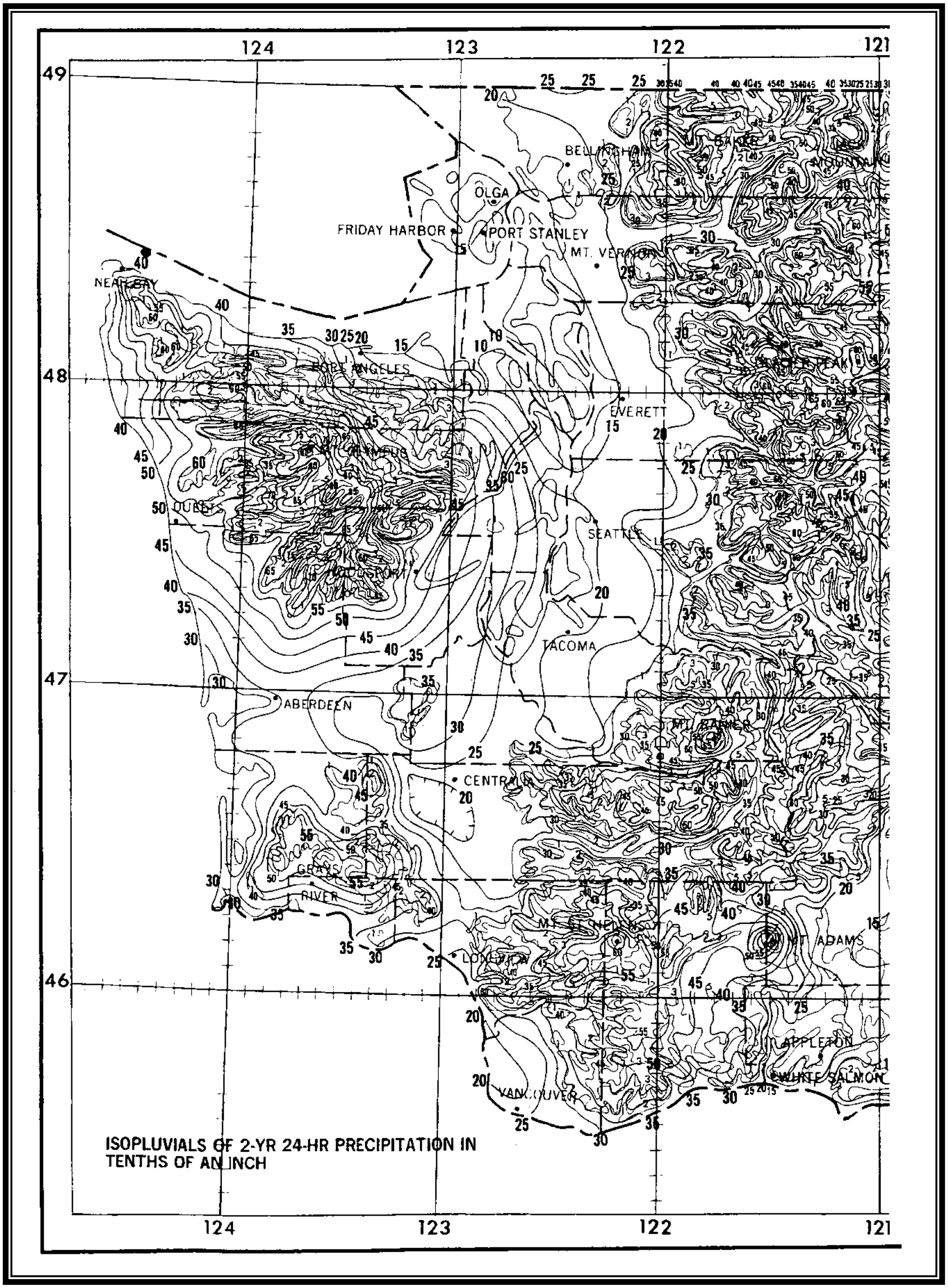


Figure A.1. Western Washington Isopluvial 2-Year, 24-Hour.

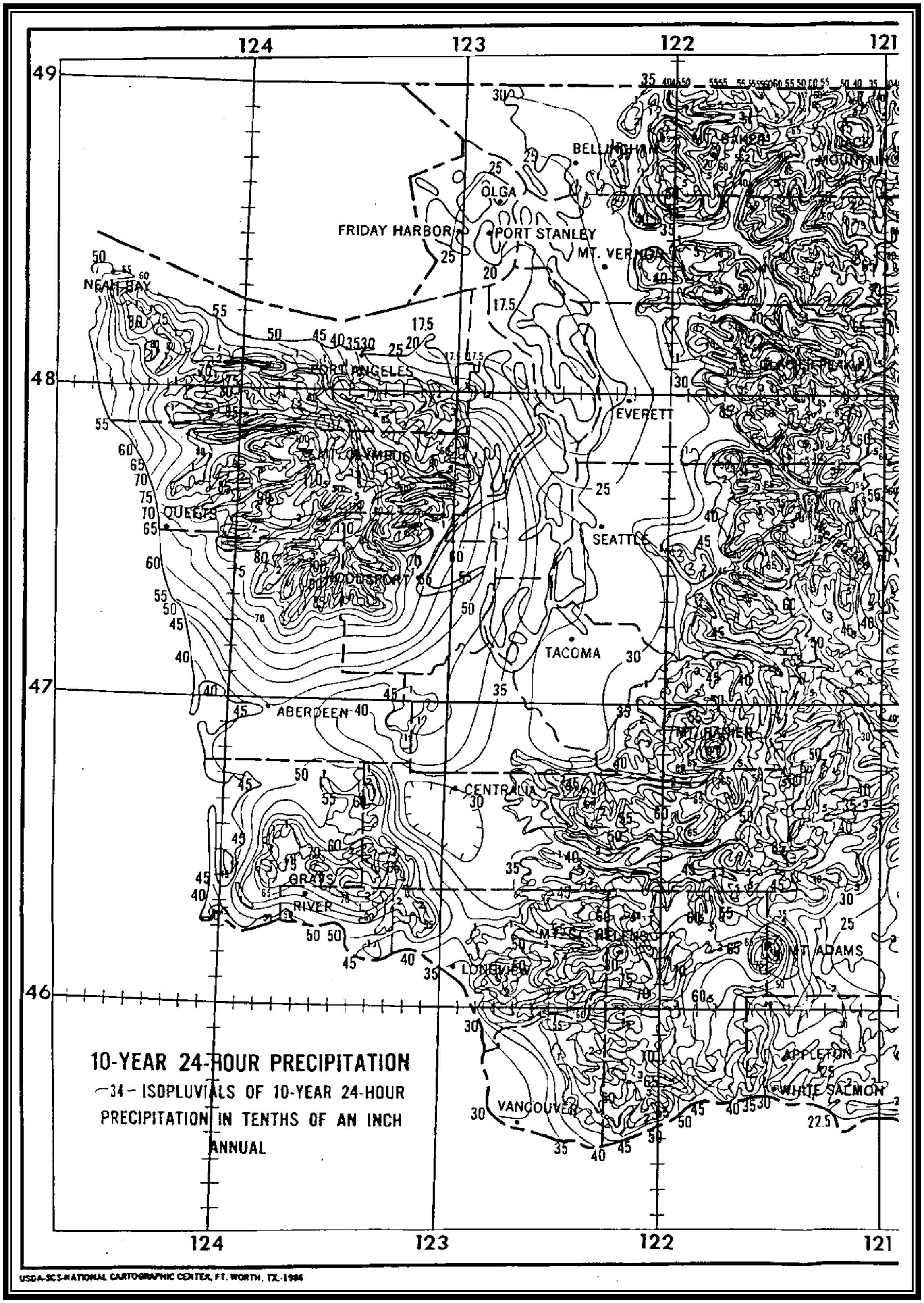
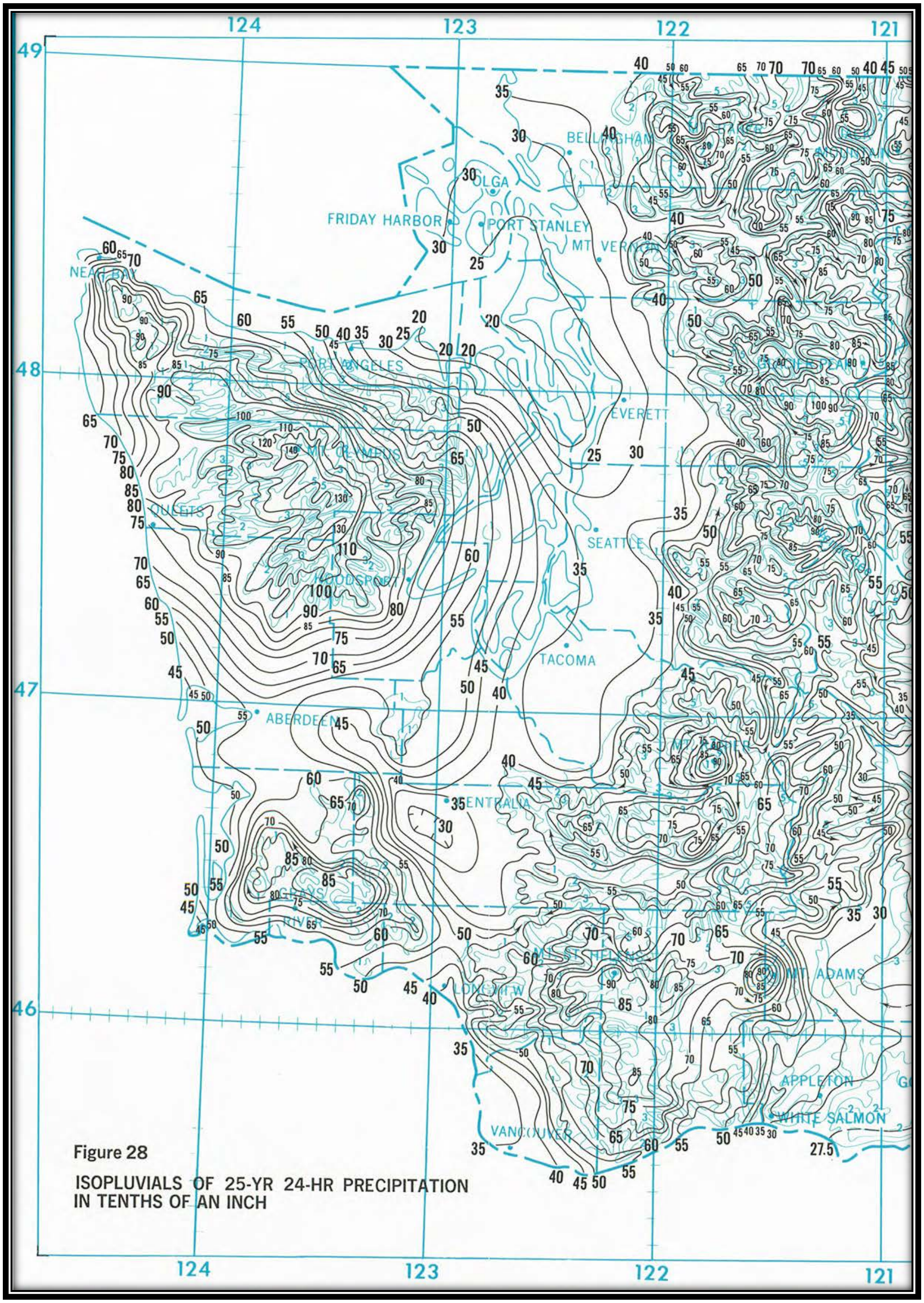


Figure A.2. Western Washington Isopluvial 10-Year, 24-Hour.



Source: NOAA 1973

Figure A.3. Western Washington Isopluvial 25-Year, 24-Hour.

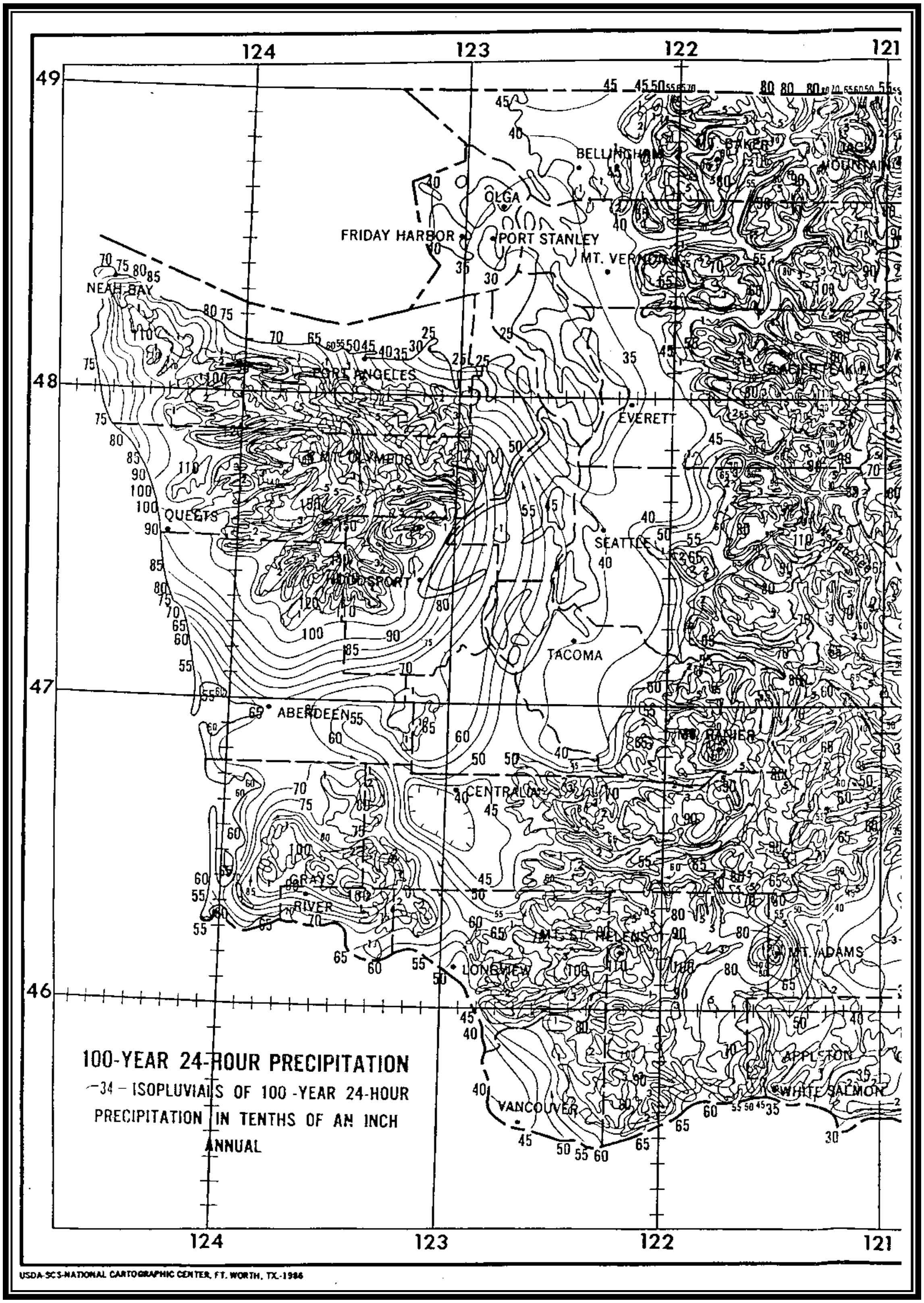


Figure A.4. Western Washington Isopluvial 100-Year, 24-Hour.

Appendix III-B – Nomographs for Various Culvert Sizing Needs

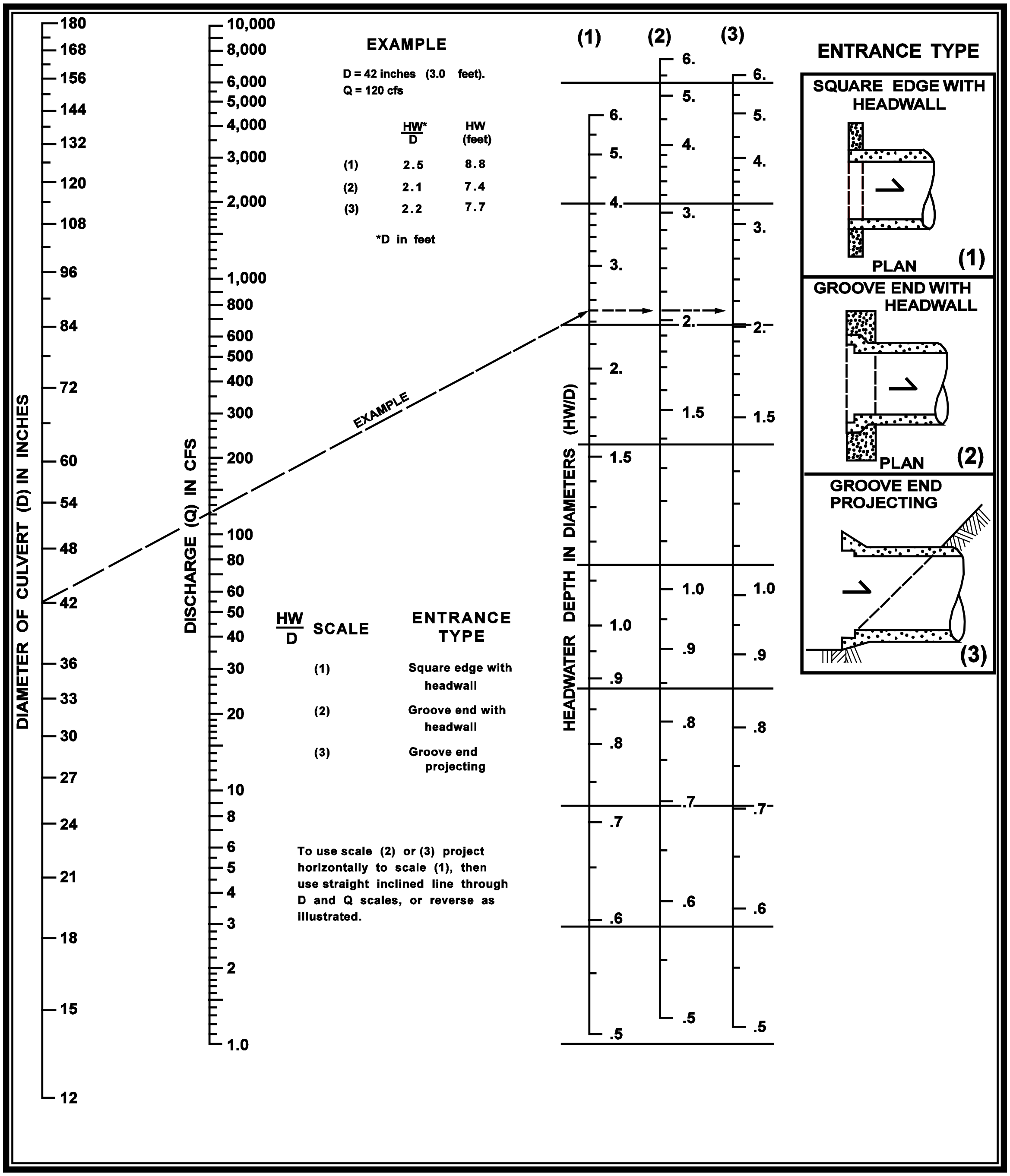


Figure B.1. Headwater Depth for Smooth Interior Pipe Culverts with Inlet Control.

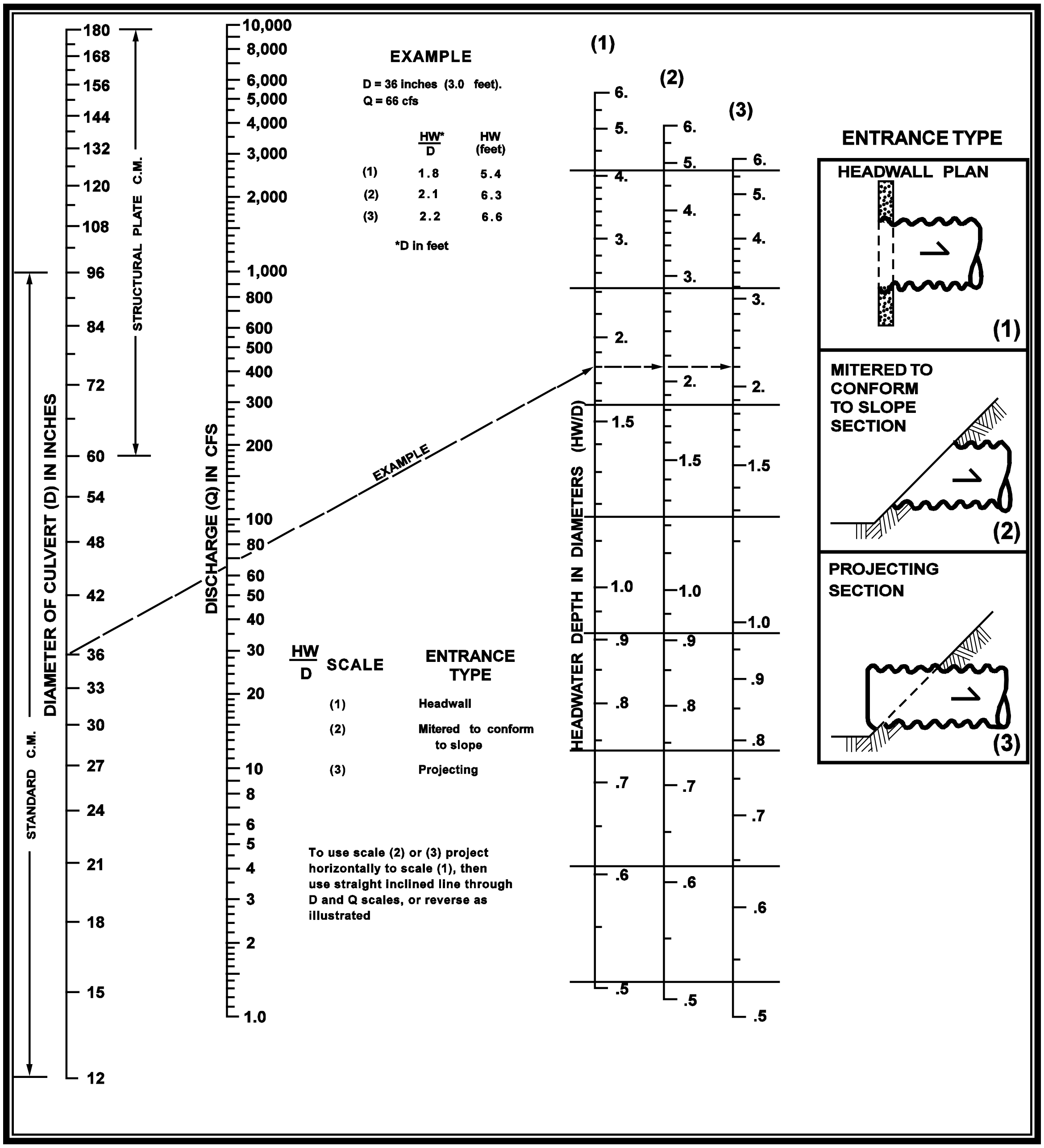


Figure B.2. Headwater Depth for Corrugated Pipe Culverts with Inlet Control.

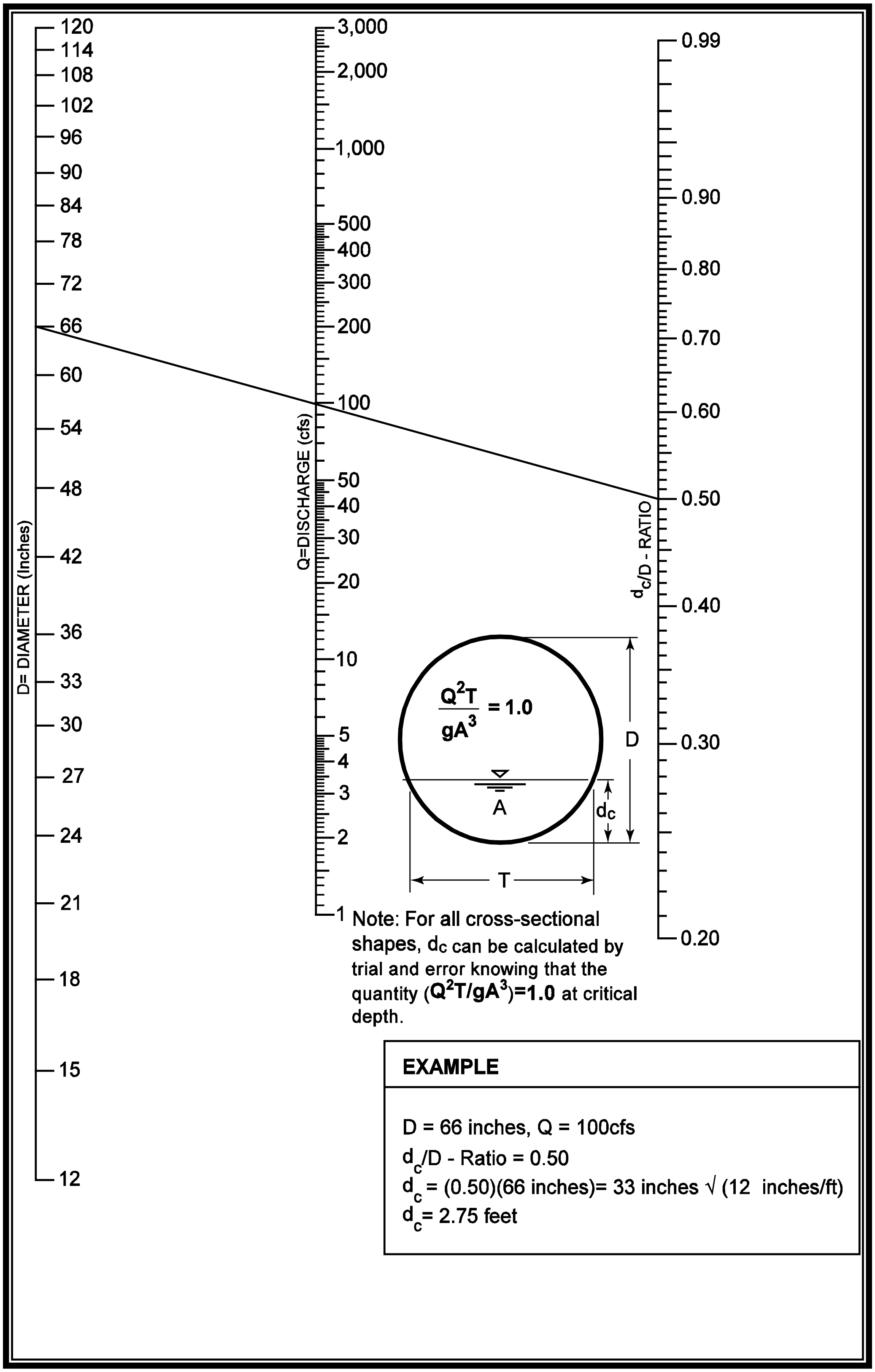


Figure B.3. Critical Depth of Flow for Circular Culverts.

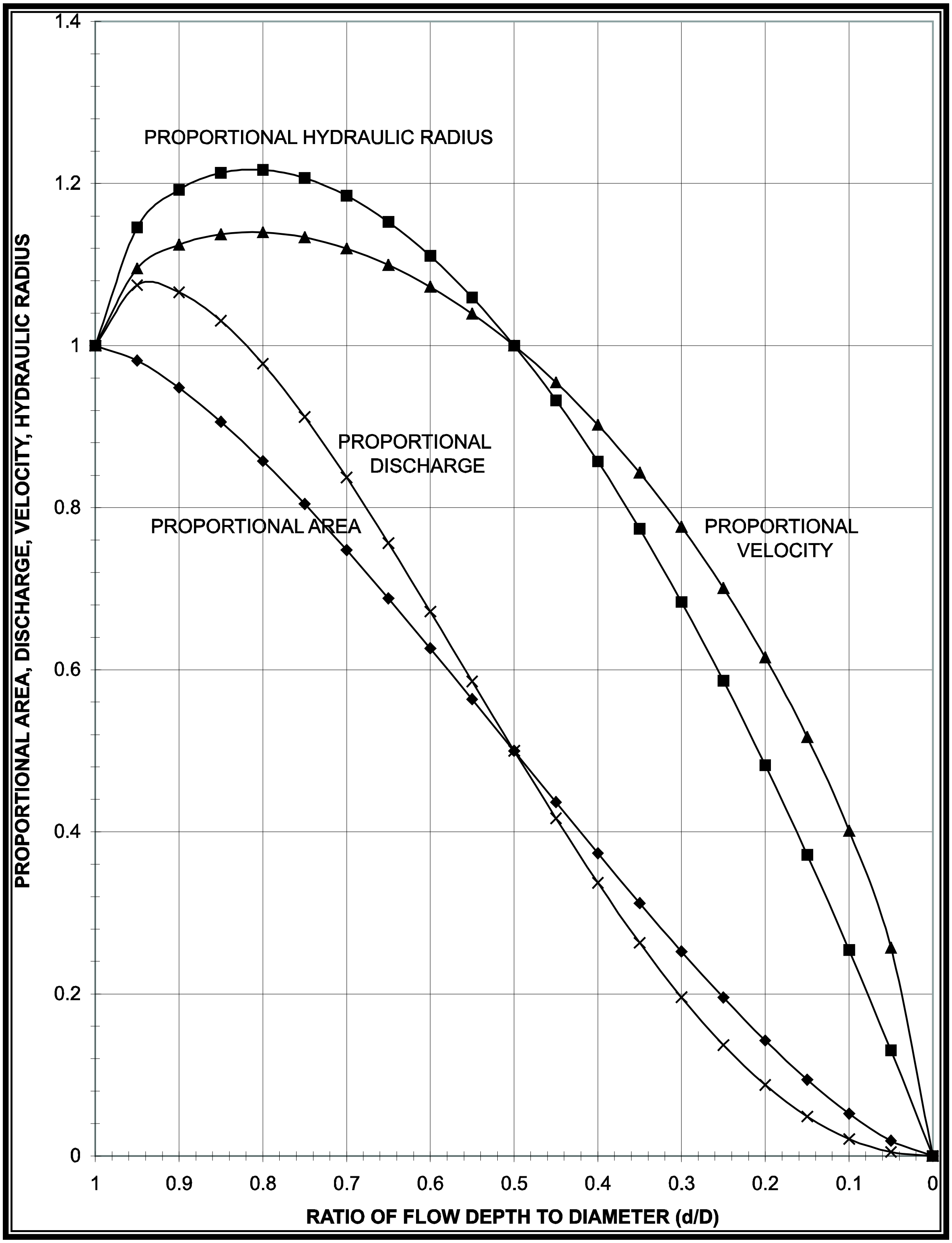


Figure B.4. Circular Channel Ratios.