Washington State
Department of Transportation

Hydraulics Manual

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Environmental and Engineering Programs
Hydraulics Office

Americans with Disabilities Act (ADA) Information

English

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1-1 Introduction

This *Hydraulics Manual* provides the guidance for designing hydraulic features related to the Washington State Department of Transportation's (WSDOT's) transportation design including hydrology, culverts, open-channel flow, drainage collection and conveyance systems, fish passage, and pipe materials. These hydraulic features maintain safe driving conditions and protect the roadway from surface and subsurface water. The chapters contained in the *Hydraulics Manual* are based on the Federal Highway Administration's (FHWA's) Hydraulic Engineering Circulars (HECs) that are located at www.fhwa.dot.gov/bridge/hydpub.htm.

The *Hydraulics Manual* makes frequent references to WSDOT's *Highway Runoff Manual*, which provides WSDOT's requirements for managing stormwater discharges to protect water quality, beneficial uses of the state's waters, and the aquatic environment in general. The intent is to use the two manuals in tandem for complete analysis and design of stormwater facilities for roadway and other transportation infrastructure projects. Projects should consult WSDOT's *Design Manual* (2018b) for general hydraulic design guidance. Design-build projects should also consult the *Design Manual*.

In addition to the guidance in the *Hydraulics Manual*, the Project Engineer's Office (PEO) should use good engineering judgment and be mindful of the legal and ethical obligations of WSDOT concerning hydraulic issues. Drainage facilities must be designed to convey the water across, along, or away from the highway in the most economical, efficient, and safe manner possible without damaging the highway or adjacent properties and without causing permit violations. Furthermore, care must be taken so highway construction does not interfere with or damage any of these facilities.

This chapter explains WSDOT policy regarding hydraulic design and hydraulic reports. In Section 1-2, the roles and responsibilities of the PEO, Region Hydraulics Engineer (RHE), and the WSDOT Headquarters (HQ) Hydraulics Section are defined. WSDOT has specific documentation requirements for the hydraulic report, which are specified in Section 1-3. Each hydraulic feature is designed based on specific design frequencies and, in some cases, a specific design tool or software. A summary of the design frequency and recommended design tools or software for most hydraulic features contained in the *Hydraulics Manual* is provided in Section 1-4. Section 1-5 defines the process for reviewing and issuing concurrence of a hydraulic report.

1-2 Responsibility

The PEO is responsible for the preparation of correct and adequate drainage design. Actual design work may be performed by the PEO, by another WSDOT office, or by a private consulting engineer; however, in all cases, it is the PEO's responsibility to complete the design work and verify that a hydraulic report is prepared as described in Section 1-3. In addition, the hydraulic report shall follow the review process outlined in Section 1-5. The PEO is also responsible for initiating the application for hydraulic-related permits required by various local, state, and federal agencies.

While the PEO is responsible for the preparation of hydraulic reports and plans, specifications, and estimates (PS&E) for all drainage facilities except bridges, assistance from the RHE and the HQ Hydraulics Section may be requested for any drainage facility design. The RHE and HQ Hydraulics Section offer technical assistance to PEOs, WSDOT consultants, and local programs for the items listed below:

1. Hydraulic design of drainage facilities (culverts, storm sewers, stormwater best management practices [BMPs], siphons, channel changes, etc.).
2. Hydraulic design of structures (culverts, headwalls, fish ladders, etc.).
3. Hydraulic support for bridge scour, bridge foundations, water surface profiles, and analysis of floodwaters through bridges.
4. Analysis of streambank erosion along roadways and river migration and the design of channel stabilization countermeasures and environmental mitigation.
5. Floodplain studies, flood predictions, and special hydrological analysis (snowmelt estimates, storm frequency predictions, etc.).
6. Analysis of closed drainage basins and unusual or unique drainage conditions.
7. Downstream analysis to identify and evaluate impacts from the project on the hydraulic conveyance system downstream of the project site. The analysis shall be broken into three sections:
   a. Review of resources
   b. Inspection of drainage conveyance systems in the site area
   c. Analysis of downstream effects
8. Wind and wave analysis on open-water structures.
9. Technical support to local programs for hydraulic or bridge-related needs.
10. Providing the Washington State Attorney General's Office with technical assistance on hydraulic issues.

The roles and responsibilities of the RHE and the HQ Hydraulics Section are outlined in Figure 1-1. The HQ Hydraulics Section also takes primary responsibility for the following:

Updating information in the *Hydraulics Manual* periodically.

1. Providing technical information for the *Highway Runoff Manual* updates.
2. Maintaining WSDOT's *Standard Plans for Road, Bridge, and Municipal Constuction* (*Standard Plans*; 2018c); *Standard Specifications for Road, Bridge, and Municipal Construction* (*Standard Specifications*; 2018d); and General Special Provisions involving drainage-related items.
3. Designing water supply and sewage disposal systems for safety rest areas. The PEO is responsible for contacting individual fire districts to collect local standards and forward the information to the HQ Hydraulics Section.
4. Reviewing and concurring with Type A hydraulic reports, unless otherwise delegated to the RHE by the HQ Hydraulics Section.
5. Providing the regions with technical assistance on hydraulic issues that are the primary responsibility of the PEO.
6. Providing basic hydrology and hydraulics training material to the regions. Either region or HQ personnel can perform the actual training. (See the HQ Hydraulics Section web page for information on course availability: www.wsdot.wa.gov/design/hydraulics/training.htm.)

1-3 Hydraulic Reports

The hydraulic report is intended to serve as a complete documented record containing the engineering justification for all drainage and stormwater installations and modifications that occur as a result of the project. The primary use of a hydraulic report is to facilitate design review and to assist in PS&E preparation. The hydraulic report should be well written, show conditions before and after construction, and be defensible in a court of law. This section contains specific guidance for developing, submitting, and archiving a hydraulic report.

A *Highway Runoff Manual* certificate number is required for the stormwater designer that designs a new stormwater BMP on WSDOT right of way (ROW) or modifies an existing stormwater BMP on WSDOT ROW or where a stormwater BMP is designed or modified and will be turned back to WSDOT ownership. The *Highway Runoff Manual* certificate number is given to those who have successfully passed the *Highway Runoff Manual* training course. The link to the training course is www.wsdot.wa.gov/Design/Hydraulics/Training.htm.

1-3.1 *Hydraulic Report Types*

There are three types of hydraulic reports: specialty report, type A, and type B. Figure 1-1 provides guidance for selecting the report type; however, consult the RHE for final selection.

Figure 1-1 Hydraulic Report Selection Table

Report Type	Description	Concurrence[1]		PE Stamp
		RHE	HQ Hydraulics Section	
Specialty Report[2]	Projects with any of the following components: • Culverts greater than 48 inches in diameter or large-span culverts[2] • Bridge • Fish Passage • Bank Protection • Large woody material • River structures (e.g., barbs, engineered logjams, levees) • Channel realignment/modifications or restoration • Any fills in floodplain or floodway • Pump stations • Hydraulic connectivity zones • Siphons		X	X[3]
A[2]	Projects with any of the following components: • Water quality treatment facility • Flow control facility • Storm sewer systems that discharge into a stormwater treatment or flow control facility • Create, modify, or remove any existing or new BMP (Full or partial treatment BMP) • Fish Passage stormwater treatment assessment for full or partial treatment[6] • Region facilities projects[5]	X[4][5]		X
B[2]	Projects without Type A components and with any of the following components: • Culverts up to 48 inches in diameter[2] • Storm sewer systems with 10 or less catch basins/manholes that do not discharge into a treatment or flow control facility • Paving/Safety Restoration and Preservation Projects	X		X

Notes:

HQ = Washington State Department of Transportation Headquarters
PE = Professional Engineer
RHE = Region Hydraulics Engineer

[1]In no case may the Project Engineer's Office provide concurrence on their own design.

[2]For design-build projects, the identified concurring RHE or HQ Hydraulics Section engineer shall be involved in developing the scope and the Request for Proposal. The identified concurring hydraulics engineer shall have rejection authority as per the Request for Proposal.

[3]The PE stamp shall be either by the HQ Hydraulics Section or by a licensed engineer approved by the HQ Hydraulics Section.

[4]The HQ Hydraulics Section is delegating final review authority and concurrence for all Type A hydraulic reports to a person designated by the assistant regional administrator for development in each region.

[5]Facilities designed by the RHE will have concurrence from the HQ Hydraulics Section.

[6]All fish passage projects shall complete a stormwater assessment for the feasibility of full or partial stormwater treatment BMP's. See HRM for more information.

1-3.2 *Preparing a Hydraulic Report*

This section provides guidance for developing a hydraulic report.

1-3.2.1 Hydraulic Report Content and Outline

The hydraulic report checklist identifies the required subject matter that the hydraulic report should contain (see Appendix 1A). PEOs shall provide a well-organized report such that an engineer with no prior knowledge of the project could read and fully understand the hydraulic/hydrologic designs made in the project. The report shall contain enough information to allow reproduction of the design in its entirety, but at the same time PEOs should be concise and avoid duplicate information that could create confusion. Because the software used for analysis will change over time, all assumptions and input parameters shall be clearly documented to allow the analysis to be reproduced in other software in the future, if needed.

In addition, a hydraulic report outline has been developed as a starting point (see Appendix 1B). Although use of the outline is not mandatory, organizing reports in the outline format may expedite the review process. Since some regions have modified the outline to meet specific regional needs or requirements, PEOs should contact their RHE to determine the correct outline before starting a report. Once the relevant outline is selected, it is recommended that PEOs read through the outline, determine which sections are applicable to the project, and delete those that are not. Either the RHE or the HQ Hydraulics Section can be contacted for assistance in preparing a hydraulic report.

The author should not copy sections of the *Hydraulics Manual* into the hydraulic report since it would add redundant information to the report. Instead, authors should reference the relevant section in the hydraulic report narrative.

1-3.2.2 Deviations to the Hydraulics Manual

If the author deviates from the requirements in the *Hydraulics Manual*, they must clearly state why a deviation is necessary and document all the steps used in the analysis in the written portion of the hydraulic report. Deviations from either manual require approval prior to submitting a hydraulic report for review. Requests for a deviation shall go through the RHE to the HQ Hydraulics Section engineering staff.

1-3.2.3 Design Tools and Software

The design tools and programs described in the *Hydraulics Manual* and in the *Highway Runoff Manual* shall be utilized whenever possible. To determine if software and/or a design tool is recommended, PEOs shall review Section 1-4 or check the expanded list on the HQ Hydraulics Section web page: www.wsdot.wa.gov/design/hydraulics/programdownloads.htm. If a PEO wishes to use a design tool or software other than those recommended, they must request concurrence by the 10 percent milestone for the hydraulic report through the RHE (see Appendix 1A).

1-3.2.4 Contract or Scope of Work

PEOs should use caution when referencing the hydraulic report outline in contracts or scopes of work for consultants. Never contract or scope a consultant to only finish or complete the hydraulic report outline. The consultant should use the hydraulic report outline to develop the report in accordance with the *Hydraulics Manual*; the hydraulic report shall address all of the applicable minimum requirements in the *Highway Runoff Manual*. Contact the RHE and/or HQ Hydraulics Section to review the contract or scope prior to hiring a consultant.

1-3.3 *Hydraulic Report Submittal and Archiving*

Hydraulic reports shall be submitted to the following offices.

1-3.3.1 Review Copies

PEOs shall submit a complete electronic and/or hard copy, depending on the reviewer's preference, of the hydraulic report to the appropriate concurring authority (RHE and/or HQ Hydraulics Section; see Figure 1-1) for review. To allow the most efficient hydraulic report review, PEOs shall follow the hydraulic review process outlined in Section 1-5 and shown in Figure 1-2. Final concurrence of the hydraulic report will be issued once the report complies with the *Hydraulics Manual* and the *Highway Runoff Manual* and all reviewer comments are satisfactorily addressed.

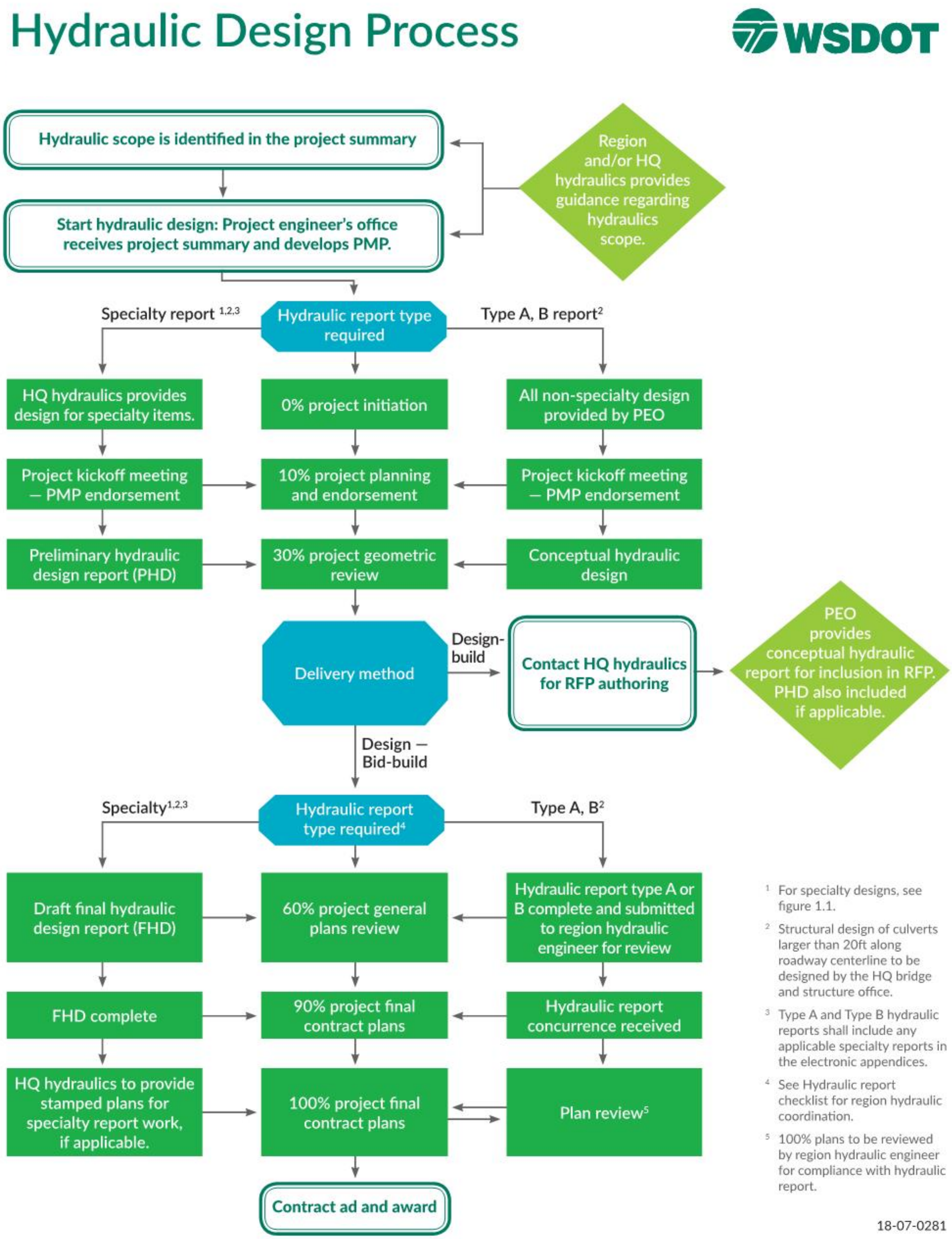
1-3.3.2 Final Copies

Upon concurrence, two hard copies and a searchable electronic copy of the hydraulic report and the original approval letter shall be sent to the offices noted below. Electronic copies shall include the entire contents of the hydraulic report (including the appendices files) in a PDF format.

1. Send one PDF or a hard copy to the Construction Office (whichever they prefer) for reference during construction.
2. Send one PDF and one hard copy to the RHE to be kept in a secure location as the record of copy for 10 years and then follow the state retention schedule.
3. Send one PDF to the HQ Hydraulics Section. The HQ Hydraulics Section will retain this copy for at least 10 years and then follow the state retention schedule.
4. Archive the original concurrence letter and original hydraulics report with the design documentation package.

The 10-year report retention period begins after construction is complete. However, WSDOT employees are directed to preserve electronic, paper, and other evidence as soon as they are aware of an incident that may reasonably result in an injury, claim, or legal action involving the department per WSDOT Secretary's Executive Order E 1041 (wsdot.wa.gov/docs/operatingrulesprocedures/1041.pdf). In some instances, this may extend beyond the 10-year retention period.

Figure 1-2 Hydraulic Design Process



1-3.4 *Hydraulic Report Revisions and Supplements*

At times, a hydraulic report may need to be revised due to design changes within a proposed project. There are two ways to submit a change:

1. **Revision.** A revision is a correction to the existing report either due to an error or omitted design documentation. The PEO shall submit the revision along with a new title page that is stamped and signed by the PE with the same date as the revision or later.
2. **Supplement.** A supplement is a change that was not part of the original scope of work. The same approval process is required as with the original report; however, the supplement shall be a stand-alone document that references the original report. The supplement shall indicate what the existing design was and how the existing design has changed as well as describe why the change was necessary.

Either type of change shall be included in a submittal package with the changes clearly documented as well as supporting analysis and data including any revised plans, calculations, and other updates, as warranted, to support the change. The package shall be submitted to the concurring authority following the guidance in Section 1-3.3 and as shown on Figure 1-1.

1-3.5 *Hydraulic Reports and Design-Build Projects*

Design-build projects present design and schedule challenges so PEOs should coordinate the hydraulic design with both the RHE and the HQ Hydraulics Section throughout the project. In addition to the guidance in the *Hydraulics Manual* and the *Highway Runoff Manual*, PEOs shall also consult the *Design-Build Manual* (WSDOT 2018f).

Prior to the Request for Proposal phase of the project, a conceptual design hydraulic report is prepared that serves as the basis of a bid and further development by the selected design-build contractor. Refer to the design-build Request for Proposal template for more information on required reporting.

1-3.6 *Developers and Utility Agreements*

Developers, state and local agencies, utilities, and others designing stormwater facilities within the WSDOT ROW shall assume the same responsibility as the PEO and prepare hydraulic reports in compliance with the policy outlined in Chapter 1. Developers, state and local agencies, utilities, and others discharging stormwater to the WSDOT ROW may need a permit. For more information on requirements and permits for discharging to the WSDOT ROW and/or building on the WSDOT ROW, consult the *Utilities Manual* (WSDOT 2018e), the *Development Services Manual* (WSDOT 2016), and the *Local Agency Guidelines* manual (WSDOT 2018a).

1-3.7 *Downstream Analysis*

A downstream analysis identifies and evaluates the impacts and risks, if any, a project will have on the drainage conveyance system, properties, and sensitive areas that are downstream of the project site. All projects that propose to discharge stormwater from WSDOT ROW and meet the requirements below are required to provide a downstream analysis as part of the hydraulic report, see the hydraulic report outline in Appendix 1B.

- Projects that add 5,000 square feet or more of new, impervious surface area.
- Projects where known drainage or erosion problems indicate there may be impacts on the downstream conveyance system, properties, or sensitive areas.

- Projects that add less than 5,000 square feet of new, impervious surface and where the project is within 300 feet of a stream or if the project's stormwater discharges into a stream within 0.25-mile downstream of WSDOT's ROW.
- Projects that alter existing drainage.

1-3.7.1 Downstream Analysis Reports

At a minimum, the analysis must include the area of the project site to a point 0.25-mile downstream of the site and upstream to a point where any backwater conditions cease. The results of the analysis must be documented in the project hydraulic report. Potential impacts to be assessed in the report also include but are not limited to changes in flows for extreme events, changes in flood duration, bank erosion, channel erosion, and nutrient loading changes from the project site. The analysis is divided into three parts that follow sequentially:

1. Review of resources.
2. Inspection of drainage conveyance systems in the site area.
3. Analysis of downstream effects.

1-3.7.2 Review of Resources

The PEO reviews available resources to assess the existing conditions of the drainage conveyance systems in the project vicinity. Resource data commonly includes aerial photographs, area maps, floodplain maps, wetland inventories, stream surveys, habitat surveys, engineering reports concerning the entire drainage basin, the Climate Impacts Vulnerability Assessment Statewide map (WSDOT 2011), geographic information system (GIS) and LiDAR information, and any previously completed downstream analyses. All of this information shall encompass an area 0.25-mile downstream of the project site's discharge point from WSDOT's ROW.

The background information is used to review and establish the existing conditions of the drainage conveyance system. This baseline information is used to determine whether the project will improve upon existing conditions, have no impact, or degrade existing conditions if no mitigating measures are implemented. The RHE and HQ Environmental Services Office staff will be able to provide most of this information. Other resource information sources include the Washington State Department of Ecology (Ecology), the Washington Department of Fish and Wildlife (WDFW), and local agencies.

1-3.7.3 Inspection of Drainage Conveyance System

The PEO must inspect the downstream conveyance system and identify any existing problems that might relate to stormwater runoff. The PEO will physically inspect (if possible) the drainage conveyance system at the project site and downstream from WSDOT ROW for a distance of at least 0.25-mile. The inspection shall include any problems or areas of concern that were noted during the resource review process or in conversations with local residents and the WSDOT Maintenance Office. The PEO shall also identify existing or potential conveyance capacity problems in the drainage system, existing or potential areas where flooding may occur, existing or potential areas of extensive channel destruction erosion, and existing or potential areas of significant destruction of aquatic habitat (runoff treatment or flow control) that can be related to stormwater runoff. If areas of potential and existing impacts related to project site runoff are established, actions must be taken to minimize impacts to downstream resources.

1-3.7.4 Analysis of Downstream Effects

This final step analyzes information gathered in the first two steps of the downstream analysis. It is necessary to determine if the project will create any drainage conveyance problems downstream or make any existing problems worse. The PEO must analyze downstream effects to determine corrective or preventive actions that may be necessary. If the project is within a medium or high vulnerability location according to the Climate Impacts Vulnerability Assessment Map (WSDOT 2011), the PEO must run extreme events (for example, the 100-year storm event) and evaluate the impacts and stability of the conveyance system. The PEO shall perform a risk assessment based on the extreme events showing impacts to the conveyance system and to downstream properties and sensitive areas.

PEOs shall consult the *Highway Runoff Manual* for further guidance on the design flow for runoff treatment and flow control BMP design. In some cases, analysis of downstream effects may indicate that no corrective or preventive actions are necessary. If corrective or preventive actions are necessary, the following options must be considered:

- Design the on-site treatment and/or flow control facilities to provide a greater level of runoff control than stipulated in the minimum requirements in Chapter 3 of the *Highway Runoff Manual*.
- Take a protective action separate from meeting Minimum Requirements Nos. 5 and 6 in the *Highway Runoff Manual* for runoff treatment and flow control. In some situations, a project will have negative impacts even when the minimum requirements are met; for example, a site where the project discharges runoff into a small, closed-basin wetland even though a detention pond was installed to comply with Minimum Requirement 6. The total volume of runoff draining into the wetland will change, possibly affecting habitat and plant species in the area. If a situation is encountered where there will be downstream impacts resulting from the project, the corrective action must be applied to the project based on a practicability analysis.
- If a project is flow control exempt, the conveyance system downstream of the project site shall be inspected to ensure adequate capacity. The PEO shall also analyze and document any changes to the downstream conveyance system, properties, and sensitive areas. If there are any negative impacts, the PEO shall perform a risk analysis showing what would happen if no actions were taken to minimize the negative impacts.

1-4 Storm Frequency Policy and Recommended Software/Design Tools

It is not practical to design hydraulic structures for the largest possible flow since this would result in unreasonably large and costly structures. Therefore, specific storm frequencies have been selected for various types of hydraulic structures. Selected storm frequencies for design purposes have considered the potential degree of damage to the roadway and adjacent property, potential hazard and inconvenience to the public, the number of users on the roadway, and the initial construction cost of the hydraulic structure.

The way in which these factors interrelate can be quite complex. WSDOT policy regarding design storm frequency for typical hydraulic structures has been established so the PEO does not have to perform a risk analysis for each structure on each project. The design storm frequency is referred to in terms of mean recurrence interval (MRI) of precipitation. Figure 1-3 lists the MRIs to be used for the design of new hydraulic structures. Based on past experience, these will give acceptable results in most cases. A more detailed discussion of MRI can be found in Chapter 2. New hydraulic structures shall also consider climate resiliency for final design size.

Occasionally, the cost of damages may be so great or the need to preserve the level of services using the roadway during higher storm events may be so important that a higher MRI is appropriate. As this is a departure from conventional design, it must go to the RHE and the HQ Hydraulics Section early for discussion and concurrence. Good engineering judgment must be used to recognize these instances, and the design should be modified accordingly. In high-risk areas, a statistical risk analysis (benefit/cost) may be needed to arrive at the most suitable frequency. This must go to the RHE and the HQ Hydraulics Section early for discussion and concurrence.

Figure 1-3 lists hydrology and hydraulic methods and approved software and design tools. A more detailed discussion of these hydrologic methods can be found in Chapter 2. Copies of the software or design tools can be found on the HQ Hydraulics Section web page: www.wsdot.wa.gov/design/hydraulics/programdownloads.htm.

PEOs proposing to use software that has not been approved need to perform a side-by-side comparison with an approved one. This should be done early in the schedule. Contact the RHE for additional guidance.

Figure 1-3 Design Frequencies, Hydrologic Methods, and Modeling Tools

Type of Structure	MRI[1] (Years)	Hydrologic Method	Hydraulic Design Tools and Software[2]
Gutters	10	Rational	Inlet spreadsheet
Storm Sewer Inlets • On longitudinal slope • Vertical curve sag/closed contour location	10 50	Rational	Inlet spreadsheet Sag spreadsheet
Storm Sewers[3][4] • Laterals • Trunk Lines	25 25	SBUH/SCS Curve Number Method Rational	StormShed3G or Storm sewer spreadsheet[5]
Ditches[4][6]	10	SBUH/SCS or Rational	StormShed3G or Manning's
Standard Culverts • Design for HW/D ratio[7] • Check for high flow damage	25 100	Published flow records Flood reports (flood insurance study) USGS regression Rational SBUH/SCS Curve Number Method	HY-8 or HEC-RAS
Bottomless Culverts[8] • Design for HW depth	100	Same as standard culverts (except Rational Method)	HY-8, HEC-RAS, or SRH-2D[9]
Temporary Bypass Pipes • Design for HW depth	2[7][8][9]	Published Flow records SBUH/SCS Continuous Simulation	StormShed3G, HY-8, HEC-RAS, or Manning's
Bridges/Fish Passage Culverts • Conveyance design and foundation scour • Check for high flow damage	100 500	Same as standard culverts (except Rational Method)	HEC-RAS (1D) or SRH-2D[9]
Stormwater BMP	See the *Highway Runoff Manual*		

Notes:

BMP = best management practice
HEC-RAS = Hydrologic Engineering Center's River Analysis System
HW/D = headwater/diameter
MRI = mean recurrence interval
SBUH/SCS = Santa Barbara Urban hydrograph/Soil Conservation Service
SRH-2D = Sedimentation and River Hydraulics – 2D Model
USGS = U.S. Geological Survey
WSDOT = Washington State Department of Transportation

[1]See the *Highway Runoff Manual* for further guidance on selecting design storms.
[2]If a different method or software is selected, the reason for not using the standard WSDOT method shall be explained and approved as part of the 10 percent submittal. The following web link contains a detailed description of all current programs and design tools recommended by WSDOT: www.wsdot.wa.gov/design/hydraulics/programdownloads.htm.
[3]When tying into existing system, the hydrologic methods used shall be the Rational Method.
[4]Storm sewers and ditches shall be designed to the same design frequency as the farthest downstream BMP.
[5]Must obtain prior approval from Region Hydraulics Engineer to use this method for designing storm sewers.
[6]More design guidance for roadside ditches can be found Chapter 4.
[7]For temporary culvert design, see Chapter 3.
[8]For non-fish bearing watercourses.
[9]In Federal Emergency Management Agency (FEMA) floodplains, use the same modeling methodology as FEMA for that floodplain.

1-5 Hydraulic Report Review Schedule

Hydraulic reports developed for WSDOT must be reviewed and receive concurrence by the HQ Hydraulics Section or RHE (per Figure 1-1) prior to the project advertisement date. The HQ Hydraulics Section has delegated concurrence authority to all HQ Hydraulics Section engineers and to some RHEs. PEOs shall contact the RHE to verify the hydraulic report review process.

To help facilitate an efficient design and review process, a hydraulic report review process has been developed. The review will consist of several checkpoints or milestones of the design as it is being developed, followed by a complete review of the report. The purpose of the milestones is to establish communication between the PEO, the RHE and/or the HQ Hydraulics Section, and other internal and external stakeholders during the hydraulic design. Each prescribed milestone is considered complete when the corresponding checklist (see Appendix 1A) is completed, along with deliverables, and submitted to the RHE reviewer(s).

1-5.1 *Milestones and Scheduling*

WSDOT has developed the Project Management and Reporting System to track and manage projects. This system utilizes a master deliverable list (MDL) to identify major elements that occur during most projects. The MDL is intended to be a starting point for creating a work breakdown structure and identifies specific offices the PEO should communicate with during project schedule development. The current MDL identifies three options for hydraulics (see Section 1-3):

1. Type A report
2. Type B report
3. Specialty report

Regardless of the type of report, the milestones identified in Figure 1-4 apply. At the 10 percent milestone, all projects with hydraulic features shall develop an approved hydraulic schedule. At a minimum, the schedule shall include the milestones with agreed-upon dates by the PEO, the RHE, and the HQ Hydraulics Section. Figure 1-4 should be used as a starting place. For Primavera users, a template that includes the milestones is available on the HQ Hydraulics Section web page: www.wsdot.wa.gov/design/hydraulics/default.htm. Additional guidance will be provided in future revisions to the *Hydraulics Manual*.

Figure 1-4 Hydraulic Report Review Schedule

Percent	Milestone	Project Alignment	Estimated Task Durations[1]	Date of Completion
0	Define project	Project definition complete MDL No. 320	TBD	TBD
10	Develop approved schedule	TBD	TBD	TBD
30	Design planning checklist complete	Design approved MDL #1685	TBD	TBD
60	Conceptual design complete	Complete prior to starting design	TBD	TBD
90	Draft hydraulic report submitted for review and concurrence	TBD	Estimate six weeks for PEO to write and compile report contents. Once report is completed, allow eight weeks for region review, comments, and resolution of comments by PEO.	TBD
TBD	Revisions and supplements	Complete prior to hydraulic report archive	TBD	TBD
100	Hydraulic report archived	Complete prior to project design approval	TBD	TBD

Notes:
MDL = master deliverable list
PEO = Project Engineer's Office
TBD = to be determined
[1]Allow additional time for projects submitted around major holidays.

1-6 Appendices

Appendix 1A Hydraulic Report Checklist

Appendix 1B Hydraulic Report Outline

Appendix 1A Hydraulic Report Checklist

Please see the following link for the Hydraulic Report Checklist:

www.wsdot.wa.gov/design/hydraulics/default.htm

Note that an updated checklist is planned. Contact the RHE for the current checklist.

Appendix 1B Hydraulic Report Outline

Please see the following link for the Hydraulic Report Outline:

www.wsdot.wa.gov/design/hydraulics/default.htm

Chapter 2 Hydrology

2-1 Introduction

This chapter presents WSDOT's procedures and acceptable methodologies for hydraulics and hydrologic analyses for roadway hydraulic features design. The procedures and methodologies presented in this chapter assume that the PEO has a basic understanding of the science of hydrology and its principles. Additionally, the PEO should be familiar with the regulations and requirements of various state and federal agencies that regulate water-related construction, as they may be applicable to proposed improvements

WSDOT uses several methods for determining runoff rates and/or volumes. Experience has shown these methods to be accurate, convenient, and economical. The following methods will be discussed in detail in subsequent sections of this chapter:

1. Rational Method
2. Santa Barbara Urban Hydrograph (SBUH) Method
3. Continuous Simulation Hydrologic Model (MGSFlood)
4. Published Flow Record
5. USGS Regional Regression Equations
6. Existing Hydrologic Studies
7. Basin Transfer of Gage Data

   Two other methods—documented reporting and high-water mark observations—shall be used wherever possible to calibrate the results of the above statistical and empirical methods. Where calculated results vary from on-site observations, further investigation may be required. The additional two methods are summarized below:

8. Documented Reporting

   Documented testimony of long-time residents should be given serious consideration by the PEO. The PEO must be aware of any bias that residents may have. Independent calculations should be made to verify this type of reporting and observations. The information furnished by residents of the area should include, but not be limited to, the following:

   a. Dates of past floods
   b. High-water marks
   c. Amount of drift
   d. Any changes in the river channel that may be occurring (i.e., streambed stability—is the channel widening, migrating, or meandering)
   e. Estimated velocity
   f. Description of flooding characteristics between normal flow to flood stage

9. High-Water Mark Observations

High-water marks can be used to reconstruct discharge from past flood events on existing structures or on the bank of a stream or ditch. These marks, along with other data, can be used to determine discharge by methods discussed in Chapter 3 or Chapter 4.

Additional hydrologic procedures are available including complex computer models, which can give the PEO accurate flood flow predictions. However, these methods, which require costly field data and large amounts of data preparation and calculation time, can rarely be justified for a single hydraulic structure. The HQ Hydraulics section shall be contacted before a procedure other than those listed above is used in a hydrologic analysis.

For simplicity and uniformity, the HQ Hydraulics section and the RHE will normally require one of the first six methods listed above. Exceptions will be permitted if adequate justification is provided and approved by the RHE.

Section 2-2 discusses how to select the appropriate method of assessing hydrology for a given site. Sections 2-3 and 2-4 discuss other important considerations, including the size of the basin and things to consider in cold climate areas. The remainder of the chapter describes each of the methods in more detail, followed by some examples in Section 2-11.

2-2 Selecting a Method

The first step in performing a hydrologic analysis is to determine the most appropriate method. The methods for determining runoff rates and volumes are summarized below, and Figure 2-1 provides a comparison table. Subsequent sections provide a more detailed description of each method. Additional guidance will be provided in future revisions to the *Hydraulics Manual*.

- **Rational Method** (Kuichling 1889)**:** This method is used when peak discharges for basins up to 200 acres must be determined. This method does not provide a time series of flow nor flow volume. It is a simple and accurate method, especially when the basin is primarily impervious. The Rational Method is appropriate for culvert design, pavement drainage design, and storm sewer design. It is also appropriate for some stormwater facility designs in eastern Washington.
- **SBUH Method** (Stubchaer 1975): This method is used when estimation of a runoff hydrograph is necessary. The SBUH Method also can be used when retention and detention must be evaluated. The SBUH Method can be used for drainage areas up to 1,000 acres. The SBUH Method can be used for stormwater facility designs in eastern Washington and for culvert and storm sewer designs through the entire state.
- **Continuous Simulation Hydrologic Model:** For western Washington, calibrated continuous simulation hydrologic models, based on the Hydrological Simulation Program-Fortran (HSPF) routine, have been created for computing peak discharges and runoff volumes. These models are used for stormwater facility designs in western Washington and estimating seasonal runoff for temporary stream diversions. WSDOT uses the continuous simulation hydrologic model MGSFlood when calculating runoff treatment rates and volumes for stormwater facility design. Programs other than MGSFlood may be used if approved by HQ Hydraulics Section.

- **Published Flow Record:** This method shall be used whenever there is appropriate stream gauge data available. This is a collection of data rather than a predictive analysis like the other methods listed. USGS, cities, counties, and other agencies gather streamflow data on a regular basis. This collected data can be analyzed statistically to predict flood flows for the river and is typically more accurate than simulated flows. Published flow records are most appropriate for culvert and bridge design.
- **USGS Regional Regression Equations** (Mastin et. al. 2016)**:** This method can be used when there is no appropriate stream gauge data available. It is a set of regression equations that were developed using data from streamflow gauging stations. The regression equations are simple to use but are less accurate than published flow records. USGS regression equations are appropriate for culvert and bridge design and are intended for use in rural and predominately undeveloped basin areas. PEOs should consult the USGS regression equation documentation for limitations when computing flows in urban basins (basins with greater than 5 percent impervious area).
- **Existing Hydrologic Studies:** This method uses existing studies or models of the watershed of interest, including FEMA flood insurance studies, smaller urban drainages, city- or countywide drainage master plans, and calibrated HSPF models. Often these values are accurate since they were developed from an in-depth analysis. Flood report data can be derived from FEMA and other approved sources, including the HQ Hydraulics Section. Obtained data may be appropriate for culvert and bridge design.
- **Basin Transfer of Gauge Data with Regional USGS Equations:** When a project is located on an ungauged stream, but there is a stream nearby with a substantial flow record, it is possible to extrapolate flows from one basin to the other, provided certain criteria are met. The watersheds of the gauged and ungauged streams must have similar geology and soils, elevation range, vegetation, and canopy cover, and must be roughly the same size. The concept is simple:

  $$Q_{\text{ungauged}} = Q_{\text{gauged}}(A_{\text{ungauged}}/A_{\text{gauged}})$$

  Where

  Q = discharge

  A = drainage area

  The USGS offers a spreadsheet called Flood Q Tools that includes the Flood Q Ratio Tool, which incorporates weighting of the ratio-based discharge. The weighting function uses the appropriate regional regression equation. Flood Q Tools can be found at: pubs.er.usgs.gov/publication/sir20165118pubs.er.usgs.gov/publication/sir20165118.

  The Flood Q Ratio Tool puts bounds on the ungauged site – it must be within 50 percent of the area of the gauged basin and on the same stream. However, if no other tools are available, it may be used to estimate flows on a different stream, provided all other parameters (basin size, soils, elevation, etc.) are similar. This tool also has the functionality of using the regression-based weighting of the Q derived from the area ratio. Additional inputs for this technique are mean annual precipitation and percent canopy cover (for Regions 1 and 2) in the ungauged basin.

Figure 2-1 Methods for Estimating Runoff Rates and Volumes

Method	Assumptions	Data Needs
Rational	• Basins <200 acres • Time of concentration <1 hour • Storm duration less than or equal to concentration time • Rainfall uniformly distributed in time and space • Runoff is primarily overland flow • Negligible channel storage (such as detention ponds, channels with significant volume, and floodplain storage)	• Time of concentration (minutes) • Drainage area (acreage) • Runoff coefficient (C values) • Rainfall intensity (use m,n values to calculate inches/hour)
Santa Barbara Urban Hydrograph	• Rainfall uniformly distributed in time and space • Runoff is based on surface flow • Small to medium basins <1,000 acres • Urban type area (pavement usually suffices) • Regional Storms (eastern Washington)[1] • Short-duration storm for stormwater conveyance • Long-duration storm for stormwater volume • Type 1A Storm (western Washington)[1] (stormwater conveyance)	• Curve Number (CN values) • Drainage area (acreage) • Digital precipitation values in the Washington State Department of Transportation GIS, National Oceanic and Atmospheric Administration Atlas, or (Isopluvials) precipitation values
Continuous Simulation Hydrologic Model (Western Washington)	• HSPF routine for stormwater best management practices for flow control facilities, such as detention and infiltration ponds, and water quality facilities, such as vegetated filter strips and bioswales. • Elevations below 1,500 feet	• Drainage basin area (acreage) • Land cover (impervious, vegetation), Soils (outwash, till, saturated) • Climatic Region (mean annual precipitation)
Published Flow Record	• Basins with stream gauge data • Appropriate station and/or generalized skew coefficient relationship applied	• Ten or more years of gauged flood records (contact the HQ Hydraulics Section for additional guidance)
U.S. Geological Survey (USGS) Regional Regression Equations	• Appropriate for culvert and bridge design • Midsized and large basins • Simple but lack accuracy of flow records for basins with more than 5% total impervious area	• 2016 Regional Equations • Annual precipitation (inches) • Drainage area (square miles) • StreamStats web application
Existing Hydrologic Studies	• Appropriate for culvert and bridge design • Midsized and large watersheds • Report accuracy varies so confirm level of accuracy with entity that the report derives from	• Available from Federal Emergency Management Agency or local flood administrative agency typically the City or County
Basin Transfer of Gauge Data with Regional USGS equations	• Similar hydrologic characteristics and size ratio	• Discharge and area for gauged watershed • Area for ungauged watershed

Notes:

HSPF = Hydrological Simulation Program-Fortran

[1]The *Highway Runoff Manual* provides detailed guidance for design storms.

2-3 Drainage Basin

The size of the drainage basin is one of the most important parameters regardless of which method of hydrologic analysis is used.

2-3.1 *Off-Site Basins*

To determine the basin area, use the StreamStats web application, Quad maps, or ArcMap/GIS Workbench. These tools cannot be used in urban areas and all subbasins should be delineated by variation in soil and drainage characteristics.

All basins shall be field verified to the maximum extent feasible. Select the best available topographic map (GIS or other approved mapping software) or best available data that cover the entire area contributing surface runoff to the point of interest. In areas under urban influence, flow paths do not always follow topography due to the presence of streets, buildings, and enclosed drainage (catch basins/pipes). In most cases, drainage patterns and catchment areas cannot be deduced from an in-office terrain analysis. Field verification of how the impervious areas and pervious areas are connected or disconnected to the flow paths may be required.

2-3.2 *On-Site Basins*

On-site basins areas shall be determined by using the most recent survey data and being field verified by the PEO.

2-4 Cold Climate Considerations

Snowmelt and rain on snow is a complicated process and can result in greater runoff rates. There are two parts to this section: the first part focuses on calculating the impacts of snow melt and the second section provides additional considerations for PEOs when evaluating the impacts of snow melt in a project location.

2-4.1 *Calculating Snow Melt*

When the project is listed as a mountainous route, per the WSDOT Highway Log, or is over an elevation of 1,500 feet, the project shall consider snow melt impacts. The PEO shall apply the method described in this section, consult the RHE, the WSDOT Maintenance Office, the PEO, and historical data. Then in the hydraulic report, the PEO shall describe in detail what value (if any) was determined to most accurately represent snowmelt at a project location.

The first question PEOs should consider is whether or not snow melt effects will impact a project. In particular, PEOs should check the snow record to determine the maximum monthly average snow depths for the project location. Snow depths can be found at the following websites or by contacting the RHE or HQ Hydraulics Section:

- Washington Climate Summaries
- Washington Snow Map

The following equation uses a factor of 5, developed from the energy budget equation by the U.S. Army Corps of Engineers (USACE), and available snow for eastern Washington cities to convert depth to snow water equivalent. This amount shall be added to the 100-year, 24-hour precipitation value when designing for flood conditions for rain on snow or snowmelt. The equation below should only be applied when the average daily snow depth within the month at a project location meets or exceeds 2 inches:

$$\text{Snow/water equivalent} = \frac{\text{Average snow depth (maximum per month (inches/day))}}{5}$$

The snow/water equivalent shall not be greater than 1.5 inches.

2-4.2 *Additional Considerations*

Regardless of snowmelt impacting a project site, PEOs should consider the following issues to provide adequate road drainage and prevent flood damage to downstream properties.

1. **Roadside drainage:** During the design phase, consideration should be given to how roadside snow will accumulate and possibly block and erode inlets and other flow paths for water present during the thawing cycle. If it is determined that inlets could be blocked by the accumulation of plowed snow, consideration should be given to an alternate course of travel for runoff. This will help prevent the water ponding that sometimes occurs in certain areas due to snowmelt and rain not having an open area in which to drain off the roadway. This may require coordination with the WSDOT Maintenance Office.
2. **Retention ponds:** When detention or retention ponds are located near the roadway, the emergency spillway should be located outside of any snow storage areas that could block overflow passage, or an alternative flow route should be designated. This may require coordination with the WSDOT Maintenance Office.
3. **Frozen ground:** Frozen ground coupled with snowmelt or rain on snow can cause unusually adverse conditions. These combined runoff sources are generally reflected in the USGS regression equations and in the historic gauge records. No corrections or adjustments typically need to be made to these hydrology methods for frozen ground or snowmelt. For smaller basins, the SBUH Method and the Rational Method are typically used to determine peak volume and peak runoff rates. The curve number (CN) value for the SBUH Method and the runoff coefficient for the Rational Method typically do not need to be increased to account for frozen ground in snowy or frozen areas as consideration has been given to this in the normal precipitation amounts and in deriving the snowmelt equation.

2-5 Rational Method

2-5.1 General

The Rational Method is used to predict peak flows for small drainage areas, which can be either natural or developed. The Rational Method can be used for culvert design, pavement drainage design, storm sewer design, and some eastern Washington stormwater facility design. The greatest accuracy is obtained for areas smaller than 100 acres and for developed conditions with large portions of impervious surface (pavement, roof tops, etc.).

Basins up to 200 acres may be evaluated using the rational formula (Equations 2-1a and 2-1b); however, results for large basins often do not properly account for effects of infiltration and thus are less accurate. PEOs should never perform a Rational Method analysis on a mostly undeveloped basin that is larger than the lower limit specified for the USGS regression equations, since the USGS regression equations will yield a more accurate flow prediction for that size of basin. The formula for the Rational Method is as follows:

$$Q = \frac{CIA}{K_C} \tag{2-1a}$$

Where:

Q = Runoff in cubic feet per second (cfs)

C = Runoff coefficient in dimensionless units

I = Rainfall intensity in inches per hour

A = Drainage area in acres

K_c = Conversion factor of 1 for English

When several subareas within a drainage basin have different runoff coefficients, the rational formula can be modified as follows:

$$Q = \frac{I\Sigma CA}{K_C} \tag{2-1b}$$

Where:

$$\sum CA = C_1 \times A_1 + C_2 \times A_2 + \ldots C_n \times A_n$$

Hydrologic information calculated by the Rational Method shall be submitted as a calculation package within the hydraulic report using this spreadsheet (link below), or other similar forms approved by the HQ Hydraulics Section that best describes the project's hydraulic information (www.wsdot.wa.gov/publications/fulltext/hydraulics/programs/hydrology.xls).

This spreadsheet contains all the required input information and the resulting discharge. The description of each area should be identified by name or station so the area may be easily located. A plan sheet or map showing the delineation of these areas shall be included with the hydraulic report along with the appropriate calculations.

2-5.2 Runoff Coefficients

The runoff coefficient "C" represents the percentage of rainfall that becomes runoff. The Rational Method implies that this ratio is fixed for a given drainage basin. In reality, the coefficient may vary with respect to prior wetting and seasonal conditions. The use of an average coefficient for various surface types is quite common, and it is assumed to stay constant through the duration of the rainstorm.

When considering frozen ground, PEOs should review Section 2-4.2, No. 3. In a high growth rate area, runoff factors should be projected that will be characteristic of developed conditions 20 years after project construction. Even though local stormwater practices (where they exist) may reduce potential increases in runoff, prudent engineering should still make allowances for predictable growth patterns.

The coefficients in Figure 2-2 are applicable for peak storms of 10-year frequency. Less frequent, higher intensity storms will require the use of higher coefficients because infiltration and other losses have a proportionally smaller effect on runoff. Generally, when designing for a 25-year frequency, the coefficient shall be increased by 10 percent; when designing for a 50-year frequency, the coefficient shall be increased by 20 percent; and when designing for a 100-year frequency, the coefficient shall be increased by 25 percent. The runoff coefficient shall not be increased above 0.95, unless approved by the RHE. Higher values may be appropriate for steeply sloped areas and/or longer return periods, because in these cases infiltration and other losses have a proportionally smaller effect on runoff.

Figure 2-2 Runoff Coefficients for the Rational Method – 10-Year Return Frequency

Cover Type	Flat	Rolling (2% to 10%)	Hilly (Over 10%)
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

2-5.3 *Time of Concentration*

Time of concentration (T_c) is defined as the time for runoff to travel from the hydraulically most distant point of the watershed to a point of interest in the watershed. Travel time (T_t) is the time it takes water to travel from one location to another in a watershed. T_t is a component of T_c, which is computed by summing all the travel times for consecutive components of the drainage flow path. This concept assumes that rainfall is applied at a constant rate over a drainage basin, which would eventually produce a constant peak rate of runoff.

Actual precipitation does not fall at a constant rate. A precipitation event usually begins with less rainfall intensity, builds to peak intensity, and eventually tapers down to no rainfall. Because rainfall intensity is variable, the time of concentration is included in the Rational Method so that the PEO can determine the proper rainfall intensity to apply across the basin. The intensity that should be used for designing is the highest intensity that will occur with the entire basin contributing flow to the flow rate location being studied. This may be a much lower intensity than the maximum intensity due to it taking several minutes before the entire basin is contributing flow; the maximum intensity lasts for a much shorter time, so the rainfall intensity that creates the greatest runoff is less than the maximum by the time the entire basin is contributing flow.

Most drainage basins consist of different types of ground covers and conveyance systems that flow must navigate. These are referred to as flow segments. It is common for a basin to have overland and open-channel flow segments. Urban drainage basins often have flow segments that flow through a storm sewer pipe in addition to overland and open-channel flow segments. A travel time (the amount of time required for flow to move through a flow segment) must be computed for each flow segment. The time of concentration is equal to the sum of all the flow segment travel times.

For a few drainage areas, a unique situation occurs where the time of concentration that produces the largest amount of runoff is less than the time of concentration for the entire basin. This can occur when two or more subbasins have dramatically different types of cover (i.e., different runoff coefficients). The most common case would be a large paved area together with a long, narrow strip of natural area. In this case, the PEO shall check the runoff produced by the paved area alone to determine if this scenario would cause a greater peak runoff rate than the peak runoff rate produced when both land segments are contributing flow based on a shorter time of concentration for the pavement-only area. The scenario that produces the greatest runoff shall be used, even if the entire basin is not contributing flow to this peak runoff rate.

The procedure for determining the time of concentration for overland flow was developed by the Natural Resources Conservation Service (NRCS; formerly known as the Soil Conservation Service [SCS]) and is described below. It is sensitive to slope, type of ground cover, and channel size. If the total time of concentration is less than five minutes, a minimum of five minutes shall be used as the duration, (see Section 2-5.4 for details). Figure 2-3 lists ground cover coefficients.

The time of concentration can be calculated as in Equations 2-2 and 2-3:

$$T_t = \frac{L}{K\sqrt{S}} = \frac{L^{1.5}}{K\sqrt{\Delta H}} \quad (2\text{-}2)$$

$$T_C = T_{t1} + T_{t2} + \ldots T_{tnz} \quad (2\text{-}3)$$

Where:

T_t = Travel time of flow segment in minutes
T_c = Time of concentration in minutes
L = Length of segment in feet
ΔH = Elevation change across segment in feet
K = Ground cover coefficient in feet
S = Slope of segment $\frac{\Delta H}{L}$ in feet per feet

Figure 2-3 Ground Cover Coefficients

Type of Cover	Flow depth (inches)	K (feet)
Forest with heavy ground cover	--	150
Minimum tillage cultivation	--	280
Short pasture grass or lawn	--	420
Nearly bare ground	--	600
Small roadside ditch with grass	--	900
Paved area	--	1,200
Gutter flow	4	1,500
	6	2,400
	8	3,100
Storm sewers	12-inch diameter	3,000
	18-inch diameter	3,900
	24-inch diameter	4,700
Open-Channel Flow (n = 0.040) Narrow Channel (w/d =1)	12	1,100
	24	1,800
	48	2,800
Open-Channel Flow (n = 0.040) Wide Channel (w/d =9)	12	2,000
	24	3,100
	48	5,000

Notes:
-- = not applicable
w/d = width/depth ratio

2-5.4 Rainfall Intensity

After the appropriate storm frequency for the design has been determined (see Chapter 1) and the time of concentration has been calculated, the rainfall intensity can be calculated. PEOs shall never use a time of concentration that is less than 5 minutes for intensity calculations, even when the calculated time of concentration is less than 5 minutes. The 5-minute limit is based on two ideas:

1. Shorter times give unrealistic intensities. Many Intensity-Duration-Frequency curves are constructed from curve-smoothing equations and not based on actual data collected at intervals shorter than 15 to 30 minutes. Making the curves shorter involves extrapolation, which is not reliable.
2. Rainfall takes time to generate runoff within a defined basin, thus it would not be realistic to have less than 5 minutes for a time of concentration.

Rainfall intensity is the average of the most intense period enveloped by the time of concentration and is not instantaneous rainfall. Equation 2-4 calculates rainfall intensity.

$$I = \frac{m}{(T_C)^n} \tag{2-4}$$

Where:

I = Rainfall intensity in inches per hour
T_c = Time of concentration in minutes
m and n = Coefficients in dimensionless units (Figure 2-4)

The coefficients (m and n) have been determined for all major cities for the 2-, 5-, 10-, 25-, 50-, and 100-year MRI. The coefficients listed in Figure 2-4 are accurate from 5-minute durations to 1,440-minute durations (24 hours). These equations were developed from the 1973 National Oceanic and Atmospheric Administration Atlas 2, *Precipitation-Frequency Atlas of the Western United States*, Volume IX-Washington (Miller et al.).

The PEO, with RHE assistance, shall interpolate between the two or three nearest cities listed in Figure 2-4 when working on a project in an unlisted location. Consult with the HQ Hydraulics Section if help is needed with interpolating which values to use.

Figure 2-4 Inches to Rainfall Coefficients

Location	2-Year MRI		5-Year MRI		10-Year MRI		25-Year MRI		50-Year MRI		100-Year MRI	
	m	n	m	n	m	n	m	n	m	n	m	n
Aberdeen and Hoquiam	5.10	0.488	6.22	0.488	7.06	0.487	8.17	0.487	9.02	0.487	9.86	0.487
Bellingham	4.29	0.549	5.59	0.555	6.59	0.559	7.90	0.562	8.89	0.563	9.88	0.565
Bremerton	3.79	0.480	4.84	0.487	5.63	0.490	6.68	0.494	7.47	0.496	8.26	0.498
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
Clarkston and Colfax	5.02	0.628	6.84	0.633	8.24	0.635	10.07	0.638	11.45	0.639	12.81	0.639
Colville	3.48	0.558	5.44	0.593	6.98	0.610	9.07	0.626	10.65	0.635	12.26	0.642
Ellensburg	2.89	0.590	5.18	0.631	7.00	0.649	9.43	0.664	11.30	0.672	13.18	0.678
Everett	3.69	0.556	5.20	0.570	6.31	0.575	7.83	0.582	8.96	0.585	10.07	0.586
Forks	4.19	0.410	5.12	0.412	5.84	0.413	6.76	0.414	7.47	0.415	8.18	0.416
Hoffstadt Cr. (SR 504)	3.96	0.448	5.21	0.462	6.16	0.469	7.44	0.476	8.41	0.480	9.38	0.484
Hoodsport	4.47	0.428	5.44	0.428	6.17	0.427	7.15	0.428	7.88	0.428	8.62	0.428
Kelso and Longview	4.25	0.507	5.50	0.515	6.45	0.509	7.74	0.524	8.70	0.526	9.67	0.529
Leavenworth	3.04	0.530	4.12	0.542	5.62	0.575	7.94	0.594	9.75	0.606	11.08	0.611
Metaline Falls	3.36	0.527	4.90	0.553	6.09	0.566	7.45	0.570	9.29	0.592	10.45	0.591
Moses Lake	2.61	0.583	5.05	0.634	6.99	0.655	9.58	0.671	11.61	0.681	13.63	0.688
Mt. Vernon	3.92	0.542	5.25	0.552	6.26	0.557	7.59	0.561	8.60	0.564	9.63	0.567
Naselle	4.57	0.432	5.67	0.441	6.14	0.432	7.47	0.443	8.05	0.440	8.91	0.436
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
Omak	3.04	0.583	5.06	0.618	6.63	0.633	8.74	0.647	10.35	0.654	11.97	0.660
Pasco and Kennewick	2.89	0.590	5.18	0.631	7.00	0.649	9.43	0.664	11.30	0.672	13.18	0.678
Port Angeles	4.31	0.530	5.42	0.531	6.25	0.531	7.37	0.532	8.19	0.532	9.03	0.532
Poulsbo	3.83	0.506	4.98	0.513	5.85	0.516	7.00	0.519	7.86	0.521	8.74	0.523
Queets	4.26	0.422	5.18	0.423	5.87	0.423	6.79	0.423	7.48	0.423	8.18	0.424
Seattle	3.56	0.515	4.83	0.531	5.62	0.530	6.89	0.539	7.88	0.545	8.75	0.5454
Sequim	3.50	0.551	5.01	0.569	6.16	0.577	7.69	0.585	8.88	0.590	10.04	0.593
Snoqualmie Pass	3.61	0.417	4.81	0.435	6.56	0.459	7.72	0.459	8.78	0.461	10.21	0.476
Spokane	3.47	0.556	5.43	0.591	6.98	0.609	9.09	0.626	10.68	0.635	12.33	0.643
Stevens Pass	4.73	0.462	6.09	0.470	8.19	0.500	8.53	0.484	10.61	0.499	12.45	0.513
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
Vancouver	2.92	0.477	4.05	0.496	4.92	0.506	6.06	0.515	6.95	0.520	7.82	0.525
Walla Walla	3.33	0.569	5.54	0.609	7.30	0.627	9.67	0.645	11.45	0.653	13.28	0.660
Wenatchee	3.15	0.535	4.88	0.566	6.19	0.579	7.94	0.592	9.32	0.600	10.68	0.605
Yakima	3.86	0.608	5.86	0.633	7.37	0.644	9.40	0.654	10.93	0.659	12.47	0.663

2-6 Single-Event Hydrograph Method: Santa Barbara Urban Hydrograph

The SBUH Method is best suited for WSDOT projects where conveyance systems are being designed and for some stormwater treatment facilities in eastern Washington. The SBUH Method was developed to calculate flow occurring from surface runoff and is most accurate for drainage basins smaller than 100 acres, although it can be used for drainage basins up to 1,000 acres. The SBUH Method should not be used where groundwater flow can be a major contributor to the total flow. While not all WSDOT projects are in urban basins, typically the paved surfaces (similar to urban areas) that generate the majority of the total flow may make use of SBUH applicable for highway projects.

An SBUH analysis requires the PEO to understand certain characteristics of the project site, such as drainage patterns, predicted rainfall, soil type, area to be covered with impervious surfaces, type of drainage conveyance, and—for eastern Washington—the flow-control BMPs that are to be provided. The physical characteristics of the site and the design storm determine the magnitude, volume, and duration of the runoff hydrograph. Other factors, such as the conveyance characteristics of channel or pipe, merging tributary flows, and type of BMPs, will alter the shape and magnitude of the hydrograph. The key elements of a single-event hydrograph analysis are listed below and described in more detail in this section:

- Design storm hyetograph
- Runoff parameters
- Hydrograph synthesis
- Hydrograph routing
- Hydrograph summation

There are several commercially available computer programs that include the SBUH Method. See Chapter 1.

2-6.1 *Design Storm Hyetograph*

The SBUH Method requires the input of a rainfall distribution or a design storm hyetograph. The design storm hyetograph is rainfall depth versus time for a given design storm frequency and duration. For this application, it is presented as a dimensionless table of unit rainfall depth (incremental rainfall depth for each time interval divided by the total rainfall depth) versus time. The type of design storm used depends on the project locations as noted below:

- **Eastern Washington:** For projects in eastern Washington, the design storms are usually the short-duration storm for conveyance design and the regional storm for volume-based stormwater facilities. (Design storms are discussed further in the *Highway Runoff Manual*.) However, occasionally with large basins and long concentration periods, the long duration regional (or Type 1A) storm will produce larger flow (Qs).
- **Western Washington:** For projects in western Washington, the design storm for conveyance is the Type 1A storm. For designs other than conveyance, see Section 2-7 for a description of the Continuous Simulation Method.

Along with the design storm, precipitation depths are needed and shall be selected for the city nearest to the project site using PRISM data available from ArcGIS Workbench as the primary data source for the most accurate results from its interpolation methodology, followed by utilizing an isopluvial map that clearly identifies the location within the map contours (see Appendix 2A).

2-6.2 *Runoff Parameters*

The SBUH Method requires input of parameters that describe physical drainage basin characteristics. These parameters provide the basis from which the runoff hydrograph is developed. This section describes the three key parameters (contributing drainage basin areas, runoff CN, and runoff time of concentration) that, when combined with the rainfall hyetograph in the SBUH Method, develop the runoff hydrograph.

The proper selection and delineation of the contributing drainage basin areas to the BMP or structure of interest is required in the hydrograph analysis. The contributing basin area(s) used should be relatively homogeneous in land use and soil type. If the entire contributing basin is similar in these aspects, the basin can be analyzed as a single area. If significant differences exist within a given contributing drainage basin, it must be divided into subbasin areas of similar land use and soil characteristics. Hydrographs should then be computed for each subbasin area and summed to form the total runoff hydrograph for the basin. Contributing drainage basins larger than 100 acres shall be divided into subbasins. By dividing large basins into smaller subbasins and then combining calculated flows, the timing aspect of the generated hydrograph is typically more accurate.

2-6.2.1 Curve Numbers

The NRCS has conducted studies into the runoff characteristics of various land types. The NRCS 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. CNs are chosen to depict average conditions—neither dry, nor saturated. The PEO shall use the CNs listed in the *Highway Runoff Manual*, or the NRCS website, or the GIS workbench.

The factors that contribute to the CN value are known as the soil-cover complex. The soil-cover complexes have been assigned to one of four hydrologic soil groups, according to their runoff characteristics. These soil groups are labeled Types A, B, C, and D, with Type A generating the least amount of runoff and Type D generating the most. The *Highway Runoff Manual* shows the hydrologic soil groups of most soils in Washington State. The different soil groups can be described as follows:

- **Type A** – Soils having high infiltration rates, even when thoroughly wetted, and consisting chiefly of deep, well drained to excessively drained sands or gravels. These soils have a high rate of water transmission.
- **Type B** – Soils having moderate infiltration rates when thoroughly wetted and consisting chiefly of moderately fine to moderately coarse textures. These soils have a moderate rate of water transmission.
- **Type C** – Soils having slow infiltration rates when thoroughly wetted and consisting chiefly of soils with a layer that impedes downward movement of water or soils with moderately fine to fine textures. These soils have a slow rate of water transmission.
- **Type D** – Soils having very slow infiltration rates when thoroughly wetted and consisting chiefly of clay soils with a high swelling potential, soils with a permanent high water table, soils with a hardpan or clay layer at or near the surface, and shallow soils over bedrock or other nearly impervious material. These soils have a very slow rate of water transmission and typically comprise areas such as wetlands.

The HQ Materials Laboratory can also perform a soil analysis to determine the soil group for the project site. This should be done only if an NRCS soils map cannot be located for the county in which the site is located, the available SCS map does not characterize the soils at the site (many NRCS maps show "urban land" in highway ROWs and other heavily urbanized areas where the soil properties are uncertain), or there is reason to doubt the accuracy of the information on the NRCS map for the particular site.

When performing an SBUH analysis for a basin, it is common to encounter more than one soil type. If the soil types are similar (within 20 CN points), a weighted average can be used. If the soil types are significantly different, the basin should be separated into smaller subbasins (previously described for different land uses). Pervious ground cover and impervious ground cover should always be analyzed separately. If the computer program StormShed3D is used for the analysis, pervious and impervious land segments will automatically be separated, but the PEO will have to combine and manually weigh similar pervious soil types for a basin.

2-6.2.2 Antecedent Moisture Condition

The moisture condition in a soil at the onset of a storm event, referred to as the antecedent moisture condition (AMC), has a significant effect on both the volume and rate of runoff. Recognizing this, the SCS developed three AMCs: I, II, and III.

- **AMC I:** Soils are dry but not to the wilting point.
- **AMC II:** Average conditions.
- **AMC III:** Heavy rainfall, or light rainfall and low temperatures, has occurred within the last five days, and soil is near saturated or saturated.

Figure 2-5 gives seasonal rainfall limits for the three AMCs. These derive from the amount of rainfall in any five days.

Figure 2-5 Total Five-Day Antecedent Rainfall

Antecedent Moisture Condition	Dormant Season (inches)	Growing Season (inches)
I	Less than 0.5	Less than 1.4
II	0.5 to 1.1	1.4 to 2.1
III	Over 1.1	Over 2.1

The CN values generally listed are for AMC II, if the AMC falls into either group I or III, the CN value will need to be modified to represent project site conditions. The *Highway Runoff Manual* provides further information regarding when the AMC should be considered and conversions for the CN for different AMCs for the case of Ia = 0.2S. For other conversions, see the National Engineering Handbook (NRCS 2010).

2-6.2.3 Time of Concentration

Time of concentration (T_c) is defined as the time for runoff to travel from the hydraulically most distant point of the watershed to a point of interest in the watershed. Travel time (T_t) is the time it takes water to travel from one location to another in a watershed. T_t is a component of T_c, which is computed by summing all the travel times for consecutive components of the drainage flow path. While this section starts the same as Section 2-5.3, the analysis described in this section is more detailed because water traveling through a basin is classified by flow type.

The different flow types include: sheet flow; shallow, concentrated flow; open-channel flow; or some combination of these. Classifying flow type is best determined by field inspection and using the parameters described below:

- **Sheet flow** is flow over plane surfaces. It usually occurs in the headwater areas of streams and for short distances on evenly graded slopes. With sheet flow, the friction value (ns, which is a modified Manning's roughness coefficient) is used. These ns values are for shallow flow depths up to about 0.1 foot and are used only for travel lengths up to 150 feet on impervious surfaces without curb and 100 feet on pervious surfaces. The *Highway Runoff Manual* provides the Manning's n values for sheet flow at various surface conditions.

  For sheet flow of up to 100 feet, use Manning's kinematic solution (Equation 2-5) to directly compute T_t:

  $$T_t = (0.42\ (nsL)0.8)/((P2)0.527(so)0.4) \qquad (2\text{-}5)$$

  Where:

  T_t = travel time (minutes)

  ns = sheet flow Manning's coefficient (dimensionless)

  L = flow length (feet)

  P2 = 2-year, 24-hour rainfall (inches)

  so = slope of hydraulic grade line (land slope, feet/foot [ft/ft])

- **Shallow flow** – After the maximum sheet flow length, sheet flow is assumed to become shallow concentrated flow. The average velocity for this flow can be calculated using the k_s values from the *Highway Runoff Manual*. Average velocity is a function of watercourse slope and type of channel. After computing the average velocity using the Velocity Equation (Equation 2-6), the travel time (T_t) for the shallow concentrated flow segment can be computed by dividing the length of the segment by the average velocity.

- **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 on USGS quadrangle maps. For developed drainage systems, the travel time of flow in a pipe is also represented as an open channel. The k_c values from the *Highway Runoff Manual* used in the Velocity Equation can be used to estimate average flow velocity. Average flow velocity is usually determined for bank full conditions. After average velocity is computed, the travel time (T_t) for the channel segment can be computed by dividing the length of the channel segment by the average velocity.

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

  $$V = (k)(so0.5) \qquad (2\text{-}6)$$

  Where:

  V = velocity (feet per second [ft/s])

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

  so = slope of flow path (ft/ft)

  Regardless of how water moves through a watershed, when estimating travel time (T_t), the following limitations apply:

The *Highway Runoff Manual* shows suggested n and k values for various land covers to be used in travel time calculations. Stormshed3G will calculate time of concentration with inputs of slope and the appropriate coefficient. For small basins, a minimum time of concentration of 5 minutes shall be entered. Additional guidance will be provided in future revisions to the *Hydraulics Manual*.

2-7 Continuous Simulation Hydrologic Model (Western Washington Only)

When designing stormwater facilities in western Washington, the PEO must use an Ecology-approved continuous simulation hydrologic model to meet the requirements of the most current version of the *Highway Runoff Manual*. A continuous simulation hydrologic model captures the back-to-back effects of storm events that are more common in western Washington. These events are associated with high volumes of flow from sequential winter storms rather than high peak flow from short duration events, as is characteristic in eastern Washington.

WSDOT uses MGSFlood (see *Highway Runoff Manual*), which uses the HSPF routines for computing runoff from rainfall on pervious and impervious land areas. In addition, MGSFlood has the BMP design criteria built into the software and will help the sizing of the stormwater facility to meet the *Highway Runoff Manual*-required runoff treatment and flow control flow rates and volumes. WSDOT also uses MGSFlood to estimate seasonal flows for temporary stream diversion designs. Refer to the HQ Hydraulics Section web page for a detailed example of this modeling approach.

MGSFlood does have limitations that the PEO should understand before using the program, regarding the project location, conveyance design, and the basin size. MGSFlood is for projects in western Washington with elevations below 1,500 feet. The program does not include routines for simulating the accumulation and melting of snow, and its use should be limited to areas where snowmelt is typically not a major contributor to floods or to the annual runoff volume. MGSFlood is not used for permanent conveyance design but is capable for conveyance design when a small-time step, such as 5 or 15 minutes is used. For projects located in western Washington that fall outside the modeling guidelines described in this paragraph, contact the RHE or HQ Hydraulics Section staff for assistance.

2-7.1 *Modeling Requirements*

MGSFlood should be used once the PEO has selected the BMP(s) for the project site and has determined the input values for precipitation, delineated drainage basin areas, and soil characteristics. Each of these input values are further described in the sections below.

2-7.1.1 Precipitation Input

There are two methods for transposing precipitation time series that are available in the continuous simulation model: extended precipitation time series selection and precipitation station selection. The PEO will generally select the extended precipitation time series unless it is not available for a project site, then the precipitation station is selected. Both methods are further described below.

1. **Extended Precipitation Time Series Selection** – Uses a family of prescaled precipitation and evaporation time series (Figure 2-6). These time series were developed by combining and scaling precipitation records from widely separated stations, resulting in record lengths in excess of 100 years. Extended hourly precipitation and evaporation time

series have been developed using this method for most of the lowland areas of western Washington where WSDOT projects are constructed. These time series should be used for stormwater facility design for project sites.

2. **Precipitation Station Selection** – For project sites located outside the extended time series region, a second precipitation scaling method is used (Figure 2-7). A source gauge is selected, and a single scaling factor is applied to transpose the hourly record from the source gauge to the site of interest (target site). The current approach for single-factor scaling, as recommended in Ecology's *Stormwater Management Manual for Western Washington* (Ecology 2014), is to compute the scaling factor as the ratio of the 25-year, 24-hour precipitation for the target and source sites. Contact the RHE or HQ Hydraulics Section staff if assistance is needed in selecting the appropriate gauge.

Figure 2-6 Extended Precipitation Time Series Regions

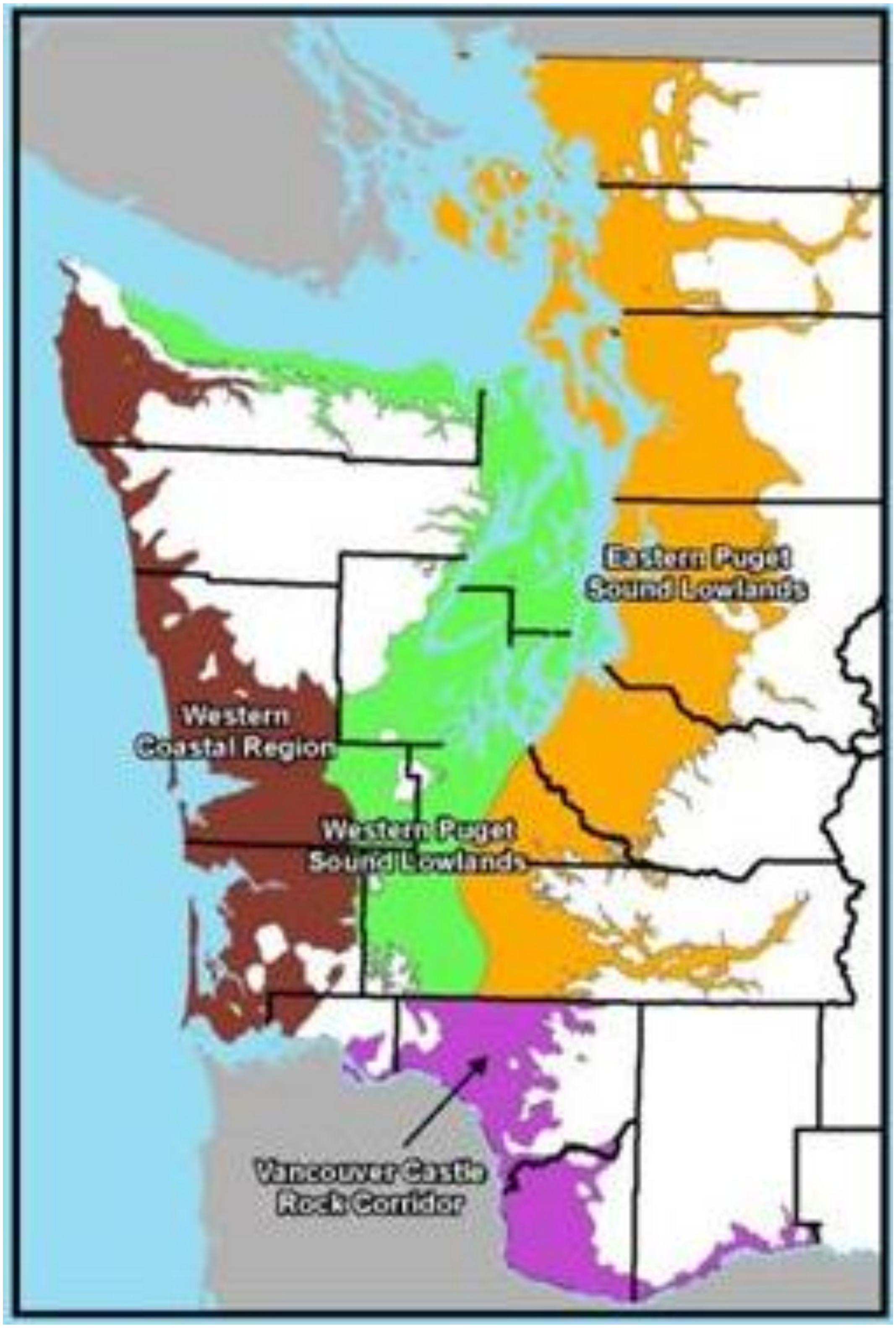
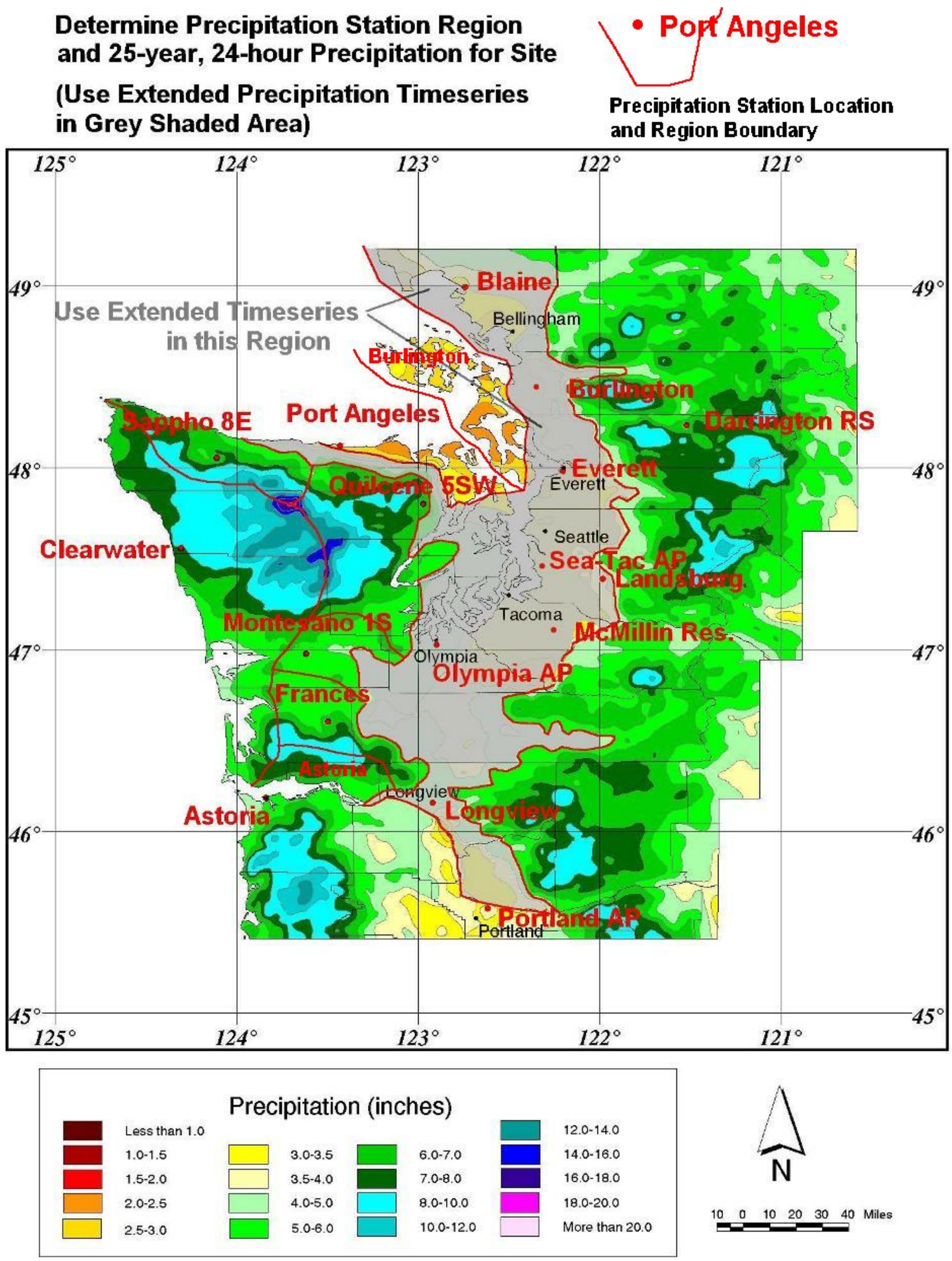


Figure 2-7 Precipitation Station Selection Outside Extended Precipitation Time Series Regions



2-7.1.2 Hydrologic Soil Groups

For each basin, land cover is defined in units of acres for predeveloped and developed conditions. Soils must be classified into one of three categories for use in MGSFlood: till, outwash, or saturated soil (as defined by the USGS). Mapping of soil types by the NRCS is the most common source of soil/geologic information used in hydrologic analyses for stormwater facility design. Each soil type defined by the NRCS has been classified into one of four hydrologic soil groups: A, B, C, or D. In western Washington, the soil groups used in MGSFlood generally correspond to the NRCS hydrologic soil groups shown in Figure 2-8.

Figure 2-8 Relationship Between NRCS Hydrologic Soil Group and MGSFlood Soil Group

NRCS Group	MGSFlood Group
A	Outwash
B	Till or Outwash
C	Till
D	Saturated

Note:
NRCS = Natural Resources Conservation Service

NRCS Type B soils can be classified as either glacial till or outwash, depending on the type of soil under consideration. Type B soils underlain by glacial till or bedrock, or that have a seasonally high water table, are classified as till. Conversely, well-drained B-type soils should be classified as outwash. It is important to work with the HQ Materials Laboratory or a licensed geotechnical engineer to confirm the soil properties and near-surface hydrogeology of the site are well understood, as they are significant factors in the final modeling results. The *Highway Runoff Manual* contains some soils classification information for preliminary work.

Wetland soils remain saturated throughout much of the year. The hydrologic response from wetlands is variable, depending on the underlying geology, the proximity of the wetland to the regional groundwater table, and the geometry of the wetland. Generally, wetlands provide some base flow to streams in the summer months and attenuate storm flows via temporary storage and slow release in the winter. Special design consideration must be considered when including wetlands in continuous simulation runoff modeling.

2-8 Published Flow Records

When available, published flow records provide the most accurate data for designing culverts and bridge openings. This is because the values are based on actual measured flows and not calculated flows. The streamflows are measured at a gauging site for several years. A statistical analysis, typically using the USGS Regression Spreadsheet, is then performed on the measured flows to predict the recurrence intervals.

USGS, Ecology, local and state municipalities, and several utility companies work together to maintain gauging sites throughout Washington State. Flood discharges for these gauging sites, at selected exceedance probabilities (based on historical data up to 2014), can be found in the following websites:

- StreamStats
- https://pubs.er.usgs.gov/publication/sir20165118
- USGS

2-9 USGS Regression Equations

While measured flows provide the best data for design purposes, it is not practical to gauge all rivers and streams in the state. A set of equations has been developed by USGS to calculate flows for drainage basins in the absence of a streamflow gauge. The equations were developed by performing a regression analysis on streamflow gauge records to determine which drainage basin parameters are most influential in determining peak runoff rates. In addition, StreamStats or digital precipitation values in WSDOT GIS Workbench can be used.

Estimates of the magnitude and frequency of flood-peak discharges and flood hydrographs are used for a variety of purposes, such as the design of bridges, culverts, and flood-control structures, and for the management and regulation of floodplains.

The equations divide the state into four different hydrologic regions, as shown on the map in Appendix 2B. The various hydrologic regions require different input variables, depending on the hydrologic region. Input parameters that may be required include: total area of the drainage basin; percent of the drainage basin that is in forest cover; and percent of the drainage basin that is in lakes, swamps, or ponds. These variables can be determined by the PEO through use of site maps, aerial photographs, and site inspections.

The PEO must be aware of the limitations of these equations. They were developed for natural rural basins; however, the equations have been updated with current flood events. The equations can be used in urban ungauged areas with additional back-up data (i.e., comparing results to nearest gauge data for calibration and sensitivity analysis, field inspection of high-water lines, and information from local maintenance). PEOs should contact the RHE for further guidance. Also, any river that has a dam and reservoir in it should not be analyzed with these equations. Finally, the PEO must keep in mind that, due to the simple nature of these equations and the broad range of each hydrologic region, the results of the equations contain a wide confidence interval, represented as the standard error.

The standard error is a statistical representation of the accuracy of the equations. Each equation is based on many rivers and the result represents the mean of all the flow values for the given set of basin characteristics. The standard error shows how far out one standard deviation is for the flow that was just calculated. For a bell-shaped curve in statistical analysis, 68 percent of all the samples are contained within the limits set by one standard deviation above the mean value and one standard deviation below the mean value. It can also be viewed as indicating that 50 percent of all the samples are equal to or less than the flow calculated with the equation and 84 percent of all samples are equal to or less than one standard deviation above the flow just calculated.

The PEOs shall use the mean value determined from the regression equations with no standard error or confidence interval. If the flows are too low or too high for that basin based on information that the PEO has collected, then the PEO may apply the standard error specific to the regression equation accordingly. The PEO should consult the RHE for assistance.

In addition to the worksheets at the end of this chapter, the USGS has a computation program, PeakFQ, to improve the process of estimating peak flows. The program is available for PEOs use and should be loaded by the Region IT: PeakFQ.

StreamStats is another USGS tool that not only estimates peak flows but can also delineate the basin area and determine the mean annual precipitation as well as other basin characteristics. It should be noted that StreamStats uses GIS PRISM maps and may produce a slightly different result than the map links on Appendix 2A.

2-10 Flood Reports

Flood reports have been developed for many rivers in Washington State. Most of these reports have been developed by FEMA. Other reports have been developed by the USACE and by local agencies.

Many small- and medium-sized streams within urbanizing areas have had some modeling by local government. These can be useful and appropriate to adopt for WSDOT use, following examination of model assumptions and drainage basin delineation.

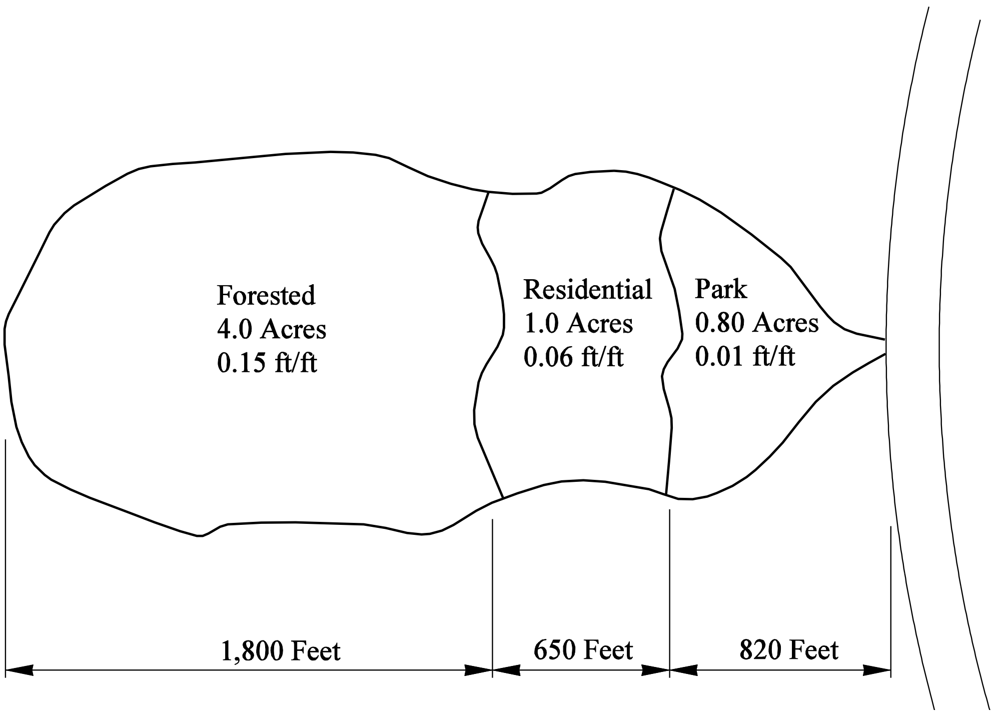
These reports are a good source of flow information since they were developed to analyze the flows during flooding conditions of a particular river or stream. The types of calculations used by the agency conducting the analysis are more complex than the Rational Method or USGS regression equations and are therefore more accurate. The increased time required to perform these complex calculations is not justified for the typical structure that WSDOT is designing; however, if the analysis has already been performed by another agency, then it is in WSDOT's best interest to use this information.

FEMA reports and USACE flood reports are available on the FEMA map service center website. HQ Hydraulics Section should be contacted for local agency reports. HQ Hydraulics Section may also have basin planning documents or action plans that could contain flow rate information.

2-11 Examples

Compute the 25-year runoff for the Spokane watershed shown in Figure 2-9. Three types of flow conditions exist from the highest point in the watershed to the outlet. The upper portion is 4.0 acres of forest cover with an average slope of 0.15 ft/ft. The middle portion is 1.0 acre of single family residential with a slope of 0.06 ft/ft and primarily lawns. The lower portion is a 0.8-acre park with 18-inch-diameter storm sewers with a general slope of 0.01 ft/ft.

Figure 2-9 Rational Formula Example



$$T_c = \sum \frac{L}{K\sqrt{S}} = \frac{1800}{150\sqrt{0.15}} + \frac{650}{420\sqrt{0.06}} + \frac{820}{3{,}900\sqrt{0.01}}$$

$$T_c = 31\text{ min} + 6\text{ min} + 2\text{ min} = 39\text{ min}$$

$$I = \frac{m}{(T_c)^n} = \frac{9.09}{(39)^{0.626}} = 0.93\frac{\text{in}}{\text{hr}}$$

$$\sum CA = 0.22(4.0\text{ acres}) + 0.44(1.0\text{ acres}) + 0.11(0.8\text{ acres}) = 1.4\text{ acres}$$

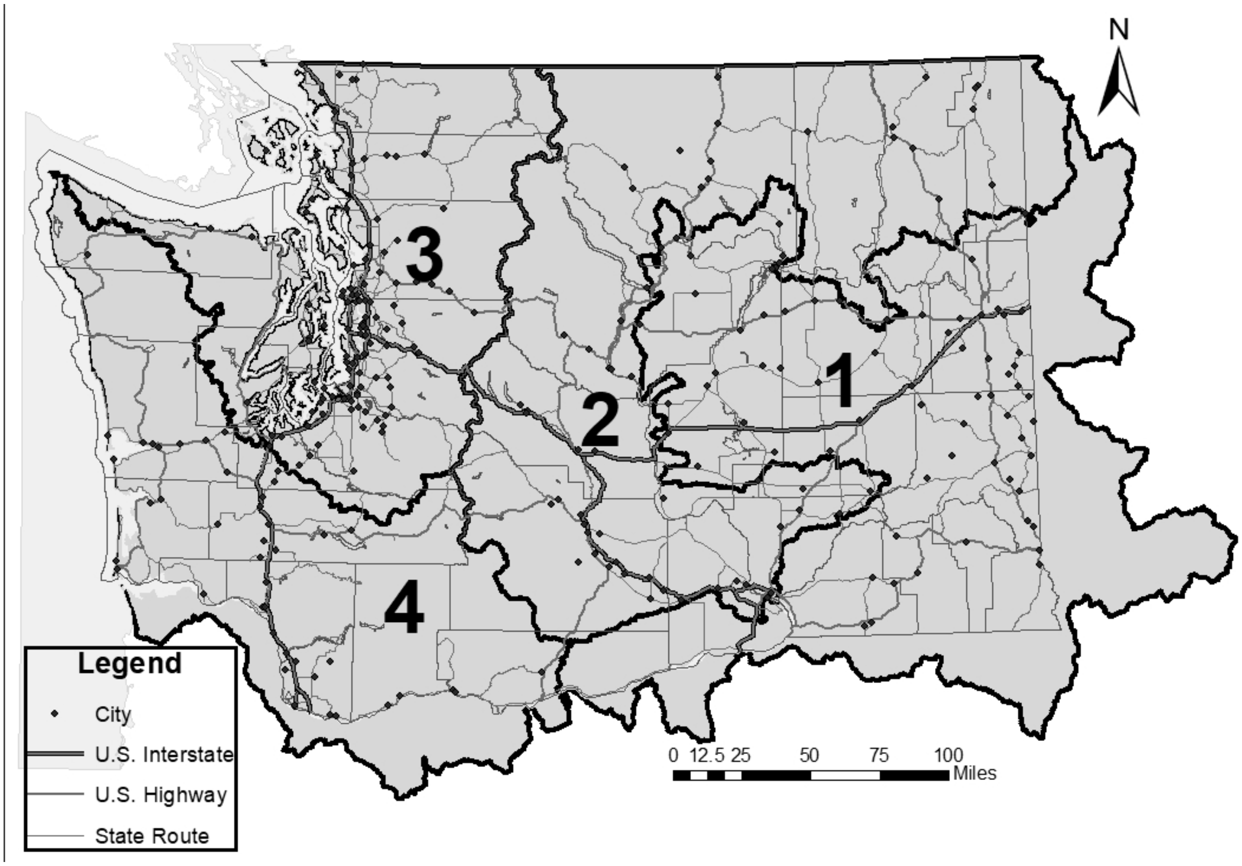
$$Q = \frac{I\left(\sum CA\right)}{K_c} = \frac{(0.93)(1.4)}{1} = 1.31\text{cfs}$$

2-12 Appendices

Appendix 2A	Isopluvial and MAP Web Links and Mean Annual Precipitation Data
Appendix 2B	USGS Regression Equation Zone Map

Appendix 2A Isopluvial and MAP Web Links and Mean Annual Precipitation Data

The 24-hour and 2-hour Isopluvial maps and the mean annual precipitation maps for Washington are available in PDF format through the link below or by using GIS Workbench. Contact your local GIS group for how to extract digital precipitation data using ArcMap.
www.wsdot.wa.gov/Design/Hydraulics



N
3
2
1
4
Legend
City
U.S. Interstate
U.S. Highway
State Route
0 12.5 25 50 75 100
Miles

Chapter 3 Culvert Design

3-1 Introduction

A culvert is a closed conduit under a roadway or embankment used to maintain flow from a natural channel or drainage ditch. A culvert shall convey flow without causing damaging backwater, excessive flow constriction, or excessive outlet velocities.

In addition to determining the design flows and corresponding hydraulic performance of a particular culvert, other factors can affect the ultimate design of a culvert and should be taken into consideration. These factors can include the economy of alternative pipe materials and sizes, horizontal and vertical alignment, environmental concerns, and necessary culvert end treatments.

In some situations, the hydraulic capacity may not be the only consideration for determining the size of a culvert opening. Fish passage requirements often dictate a different type of crossing than would normally be used for hydraulic capacity. Wetland preservation may require upsizing a culvert or replacing a culvert with a bridge. Excessive debris potential may also require an increase in culvert size. Bridges and fish passage culverts are covered in more detail in Chapter 7 but require a PEO approved by the HQ Hydraulics Section to complete the design.

The guidance in this chapter applies only to non-fish-bearing channels. For culverts associated with fish-bearing channels, refer to Chapter 7.

Section 3-2 discusses the data acquisition and documentation required when designing culverts. Culvert design considerations are discussed in detail in Section 3-3, and various end treatments are discussed in Section 3-4. Section 3-5 covers other miscellaneous design considerations that have not been previously discussed.

3-2 Culvert Design Documentation

3-2.1 *Hydraulic Reports*

The PEO shall collect field data and perform an engineering analysis as described in Section 3-2.2 and 3-2.3. Culverts in this size range shall be referred to on the contract plan sheets as "Schedule ____ Culv. Pipe ____ in. Diam." The PEO is responsible for listing all acceptable pipe alternates based on site conditions. The decision regarding which type of pipe material to be installed at a location will be left to the contractor unless a specific material type is called out in the plans and justification is provided in the hydraulic report. See Chapter 8 for a discussion on schedule pipe and acceptable alternates.

Culverts larger than 48 inches in diameter or span will be included as part of a specialty report and are required to be designed by either the HQ Hydraulics Section or by a licensed engineer approved by the HQ Hydraulics Section, as outlined in Chapter 1.

In addition to standard culvert design, the HQ Hydraulics Section can assist in the design of any unique culvert installation. The requirements for these structures will vary, and it is recommended that the HQ Hydraulics Section be contacted early in the design phase to determine what information will be necessary to complete the engineering analysis.

3-2.2 Required Field Data

Information and field data required to complete an engineering analysis for all new culvert installations or draining an area requiring a culvert shall be part of the hydraulic report and include the items that follow:

1. Topographic map showing the contours and the outline of the drainage area.
2. Description drainage area ground cover.
3. Fish passage requirement, if applicable see Chapter 7.
4. Soils investigation per WSDOT's *Design Manual*.
5. Proposed roadway profile and alignment in the vicinity of the culvert.
6. Proposed roadway cross section at the culvert.
7. Corrosion zone location, pH, and resistivity of the site.
8. Investigate a sufficient distance upstream and downstream and any other unique features that can affect design, such as low-lying structures that could be affected by excessive headwater debris, anticipated sediment transport, and other consideration discussed in Section 3-5.

If an existing culvert(s) does not have a history of problems and only needs to be extended or replaced, it is not necessary to gather all the information listed above to determine if it is adequately sized for the flows it receives. Attaining the history of problems at an existing culvert site may be sufficient to complete the analysis. Figure 3-1 is a general outline showing the information and field data requirements for a hydraulic report and specialty report.

For non-fish-bearing channels with spans between 4 and 20 feet, use the culvert design in this chapter. If the channel is fish-bearing and/or the span is greater than 20 feet, refer to Chapter 7 for further guidance.

Figure 3-1 Field Data Requirements for Hydraulic Reports and Specialty Reports

Information and Field Data	New Culvert Site	Extending or Replacing	Specialty Report
1. Topographic survey	R	O	R
2. Ground cover description	R	O	R
3. Ground soil investigation	R	O	R
4. Proposed roadway profile and alignment	R	O	R
5. Proposed roadway cross section	R	O	R
6. Corrosion Zone, pH, resistivity(1)	R(1)	O(1)	R(1)
7. Unique features	R	O	R

Notes:
O = optional
R = required
(1)Only required if replacing with dissimilar material.

3-2.3 *Engineering Analysis*

Collected field data will be used to perform an engineering analysis. The intent of the engineering analysis is to ensure that the PEO considers several issues, including flow capacity requirements, foundation conditions, embankment construction, runoff conditions, soil characteristics, stream characteristics, potential construction problems, estimated cost, environmental concerns, and any other factors that may be involved and pertinent to the design. Additional analysis may be required, if a culvert is installed for flood equalization, to verify that the difference between the floodwater levels is less than one inch on either side of the culvert. The PEO should contact the HQ Hydraulics Section for further guidance on flood equalization. Other miscellaneous design considerations for culverts are discussed in Section 3-5.

Once the engineering analysis is completed, it will be part of the hydraulic report and shall include:

1. Culvert hydrology and hydraulic calculations, as described in Section 3-3 and Figure 3-2.
2. Proposed roadway stationing of the culvert location.
3. Culvert length.
4. Culver diameter.
5. Culvert material.
6. Headwater depths, water surface elevations, and flow rates (Q) for the design flow event (generally the 25-year event and the 100-year flow event).
7. Proposed roadway cross section and roadway profile, demonstrating the maximum and minimum height of fill over the culvert.
8. Appropriate end treatment as described in Section 3-4.
9. Hydraulic features of downstream controls, tailwater, or backwater (storage) conditions.

The information needed for replacement or extension of existing culverts is not the same as that required for new culverts (see Figure 3-2). For a more in detailed diagnostic about what is required for a specialty report for water crossings, see Chapter 7.

Figure 3-2 Information for the Hydraulics and Specialty Reports for New Culverts and for Extending/Replacing Existing Culverts

Engineering Analysis Items	New Culvert Site	Extending or Replacing	Specialty Report
1. Culvert hydraulic and hydrology calculations	R	O	R
2. Roadway stationing at culvert	R	R	R
3. Culvert and stream profile	R	O	R
4. Culvert length and size	R	R	R
5. Culvert material	R	R	R
6. Hydraulic details	R	O	R
7. Proposed roadway details	R	O	R
8. End treatment	R	R	R
9. Hydraulic features	R	O	R

Notes:
O = optional
R = required

3-3 Hydraulic Design of Culverts

A complete theoretical analysis of the hydraulics of a particular culvert installation is time-consuming and complex. Flow conditions vary from culvert to culvert and can also vary over time for any given culvert. The barrel of the culvert may flow full or partially full depending upon upstream and downstream conditions, barrel characteristics, and inlet geometry. However, under most conditions, a simplified procedure is sufficient to determine the type of flow control and corresponding headwater elevation that exist at a culvert during the chosen design flow.

This section includes excerpts from the FHWA's Hydraulic Design Series (HDS) No. 5, *Hydraulic Design of Highway Culverts*. The PEO should refer to the *Hydraulics Manual* for detailed information on the theory of culvert flow or reference an appropriate hydraulics textbook for unusual situations. The HQ Hydraulics Section is also available to provide design guidance.

The general procedure to follow when designing a culvert for a span width of less than 20 feet is summarized in the steps below. Culvert spans over 20 feet are considered bridges and any hydraulic design for bridges is the responsibility of the HQ Hydraulics Section, see Section 3-3.1.2 for further guidance.

1. Calculate the culvert design flows (Section 3-3.1).
2. Determine the allowable headwater elevation (Section 3-3.2).
3. Determine the tailwater elevation at the design flow (Section 3-3.3).
4. Determine the type of control that exists at the design flow(s), either inlet control or outlet control (Section 3-3.4).
5. Calculate outlet velocities (Section 3-3.5).

3-3.1 *Culvert Design Considerations*

3-3.1.1 Flow

The first step in designing a culvert is to determine the design flows to be used. The flow from the basin contributing to the culvert can be calculated using the methods described in Chapter 2. Generally, culverts will be designed to meet criteria for two flows: the 25-year event and the 100-year event. If fish passage is a requirement at a culvert location, contact the HQ Hydraulics Section (see Chapter 7). Guidelines for temporary culverts are described further in Section 3-3.1.7. The PEO will be required to analyze each culvert at each of the design flows, ensuring that the appropriate criteria are met.

3-3.1.2 Additional Requirement for Culverts over 20 Feet

Once a culvert exceeds 20 feet along the centerline of the roadway, it is defined as a bridge and all hydraulic analysis on bridges are the responsibility of the HQ Hydraulics Section (see Chapter 1). The federal definition of a bridge is a structure, including supports, erected over a depression or obstruction, such as water, highway, or railway, and having a track or passageway for carrying traffic or other moving loads with a clear span, as measured along the centerline of the roadway, equal to or greater than 20 feet.

The interior cell walls of a multiple box are ignored as well as the distance between the multiple pipes if the distance between pipes is less than D/2 (i.e., a 16-foot culvert on a 45-degree skew is a bridge; a 10-foot culvert on a 60-degree skew is a bridge; and three 6-foot pipes 2 feet apart is a bridge).

The two primary types of hydraulic analysis performed on bridges are backwater and scour. As noted above, all hydraulic analysis of bridges is performed by the HQ Hydraulics Section; however, it is the responsibility of the PEO to gather field information for the analysis. Chapter 7 contains more information about backwater and scour analysis, along with the PEO list of responsibilities.

3-3.1.3 Alignment and Grade

It is recommended that culverts be placed on the same alignment and grade as the natural channel, especially on year-round streams. This tends to maintain the natural drainage system and minimize downstream impacts.

In many instances, it may not be possible or feasible to match the existing grade and alignment. This is especially true in situations where culverts are conveying only hillside runoff or streams with intermittent flow. If following the natural drainage course results in skewed culverts, culverts with horizontal or vertical bends, or excessive and/or solid rock excavation, it may be more feasible to alter the culvert profile or change the channel alignment up or downstream of the culvert. This is best evaluated on a case-by-case basis, with potential environmental and stream stability impacts being balanced with construction and function ability issues.

3-3.1.4 Allowable Grade

Concrete pipe may be used on any grade up to 10 percent. Corrugated metal pipe and thermoplastic pipe may be used on up to 20 percent grades. For grades over 20 percent, consult with the RHE or the HQ Hydraulics Section for design assistance.

3-3.1.5 Minimum Spacing

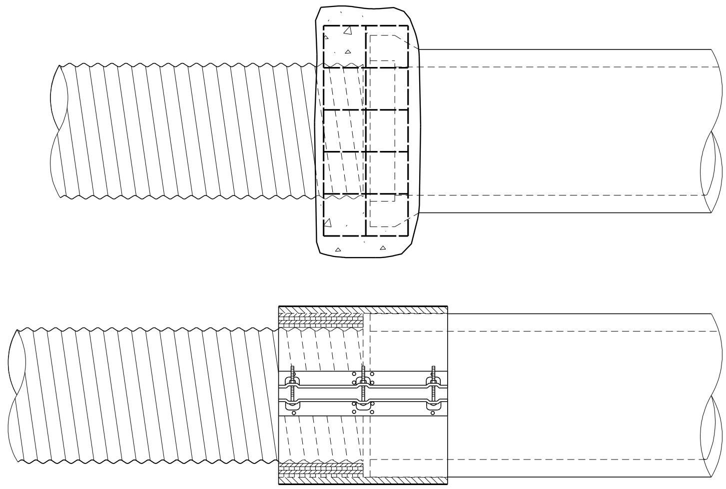
The use of multiple culvert openings is discouraged due to decreased efficiency and less room available to transport large woody material (LWM). Using multiple culverts requires a deviation from the HQ Hydraulics Section, thus needing approval from the RHE.

3-3.1.6 Culvert Extension

Whenever possible, culvert extensions should be done in-kind—use the same pipe material and size and follow the existing slope. All culvert extensions shall follow the guidelines for the culvert sizes noted in Section 3-2.2 and Chapter 1. For in-kind extensions, the PEO shall follow the manufacturer's recommendations for joining pipe. For extensions of dissimilar material or box culverts, the PEO shall follow the guidelines below. For situations not listed, contact the RHE.

- Culvert pipe connections for dissimilar materials must follow Standard Plan B-60.20-01 of WSDOT's Standard Plans, as shown in Figure 3-3.
- For cast-in-place box culvert connections, contact the Bridge Design Office for rebar size and embedment.
- Precast box culvert connections must follow American Society for Testing and Materials (ASTM) C 1433, American Association of State Highway and Transportation Officials (AASHTO) M 259, M 273, and *Standard Specification* 6-02.3(28)

Figure 3-3 Connection for Dissimilar Culvert Pipe



3-3.1.7 Temporary Culverts

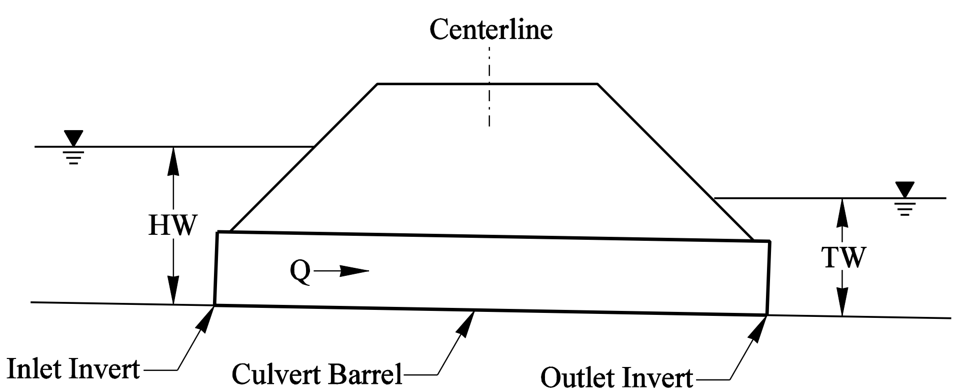
Temporary culverts used for fish passage stream diversion shall refer to Chapter 7. All other temporary culverts for a single construction season shall be sized for the 2-year storm event, unless the PEO can provide hydrologic justification for a different storm event and receive HQ Hydraulics Section or RHE approval.

3-3.2 *Allowable Headwater*

3-3.2.1 General

The depth of water that exists at the culvert entrance at a given design flow is referred to as the headwater. Headwater depth is measured from the invert of the culvert to the water surface, as shown in Figure 3-4.

Figure 3-4 Headwater and Tailwater Diagram



Limiting the amount of headwater during a design flow can be beneficial for several reasons. The potential for debris clogging becomes less as the culvert size is increased. Maintenance is virtually impossible to perform on a culvert during a flood event if the inlet is submerged more than a few feet. Also, increasing the allowable headwater can adversely impact upstream property owners by increasing flood elevations. These factors must be taken into consideration and balanced with the cost effectiveness of providing larger or smaller culvert openings.

If a culvert is to be placed in a stream that has been identified in a FEMA flood insurance study, the floodway and floodplain requirements for that municipality may govern the allowable amount of headwater. In this situation, the PEO shall contact the HQ Hydraulics Section for additional guidance.

3-3.2.2 Allowable Headwater for Circular and Box Culverts and Pipe Arches

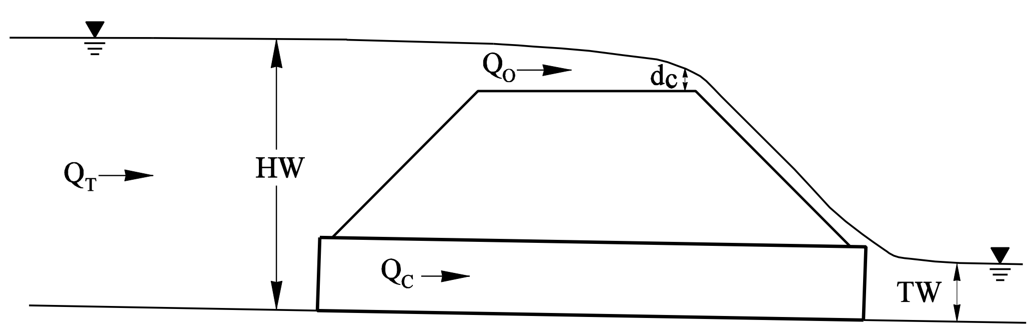
Circular culverts, box culverts, and pipe arches shall be designed such that the ratio of the headwater (HW) to diameter (D) during the 25-year flow event is less than or equal to 1.25 (HWi/D <1.25). HWi/D ratios larger than 1.25 are permitted, provided that existing site conditions dictate or warrant a larger ratio. An example of this might be an area with high roadway fills, little stream debris, and no impacted upstream property owners. The justification for exceeding the HWi/D ratio of 1.25 must be discussed in the HQ Hydraulics Section and, if approved by the RHE, included as a narrative in the hydraulic report.

The headwater that occurs during the 100-year flow event must also be investigated. Two sets of criteria exist for the allowable headwater during the 100-year flow event, depending on the type of roadway over the culvert:

1. If the culvert is under an interstate or major state route that must be kept open during major flood events, the culvert must be designed such that the 100-year flow event can be passed without overtopping the roadway.
2. If the culvert is under a minor state route or other roadway, it is recommended that the culvert be designed such that there is no roadway overtopping during the 100-year flow event. However, there may be situations where it is more cost effective to design the roadway embankment to withstand overtopping rather than provide a structure or group of structures capable of passing the design flow. An example of this might be a low average daily traffic roadway with minimal vertical clearance that, if closed due to overtopping, would not significantly inconvenience the primary users.

Overtopping of the road will begin to occur when the headwater rises to the elevation of the road. The flow over the roadway will be similar to flow over a broad-crested weir, as shown in Figure 3-5. A methodology is available in HDS-5 to calculate the simultaneous flows through the culvert and over the roadway. The PEO must be mindful that the downstream embankment slope must be protected from the erosive forces that will occur. This can generally be accomplished with riprap reinforcement, but the HQ Hydraulics Section should be contacted for further design guidance. Additionally, the PEO should verify the adjacent ditch does not overtop and transport runoff, causing damage to either public or private infrastructure.

Figure 3-5 Roadway Overtopping

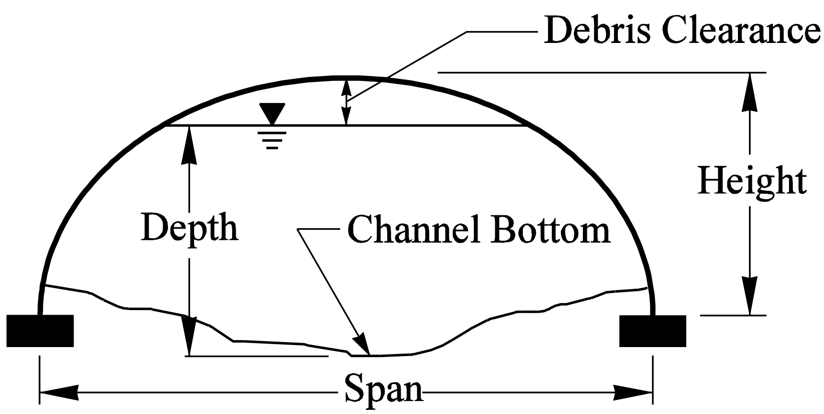


3-3.2.3 Allowable Headwater for Bottomless Culverts

Bottomless culverts with footings shall be designed such that one foot of debris clearance from the water surface to the culvert crown is provided during the 25-year flow event (see Figure 3-6). In many instances, bottomless culverts function similarly to bridges. They typically span the main channel and are designed to pass relatively large flows. If a large arch becomes plugged with debris, the potential for significant damage occurring to either the roadway embankment or the culvert increases.

Excessive headwater at the inlet can also increase velocities through the culvert and correspondingly increase the scour potential at the footings. Sizing a bottomless culvert to meet the one-foot criteria will alleviate many of these potential problems. Bottomless culverts shall also be designed such that the 100-year event can be passed without the headwater depth exceeding the height of the culvert. Flow depths greater than the height can cause potential scour problems near the footings.

Figure 3-6 Typical Bottomless Culvert



3-3.3 *Tailwater Conditions*

The depth of water that exists in the channel downstream of a culvert is referred to as the tailwater and is shown in Figure 3-4. Tailwater is important because it can affect the depth of headwater necessary to pass a given design flow. This is especially true for culverts that are flowing in outlet control, as explained in Section 3-3.4. Generally, one of three conditions will exist downstream of the culvert and the tailwater can be determined as described below.

1. If the downstream channel is relatively undefined and depth of flow during the design event is considerably less than the culvert diameter, the tailwater can be ignored. An example of this might be a culvert discharging into a wide, flat area. In this case, the downstream channel will have little or no impact on the culvert discharge capacity or headwater.

2. If the downstream channel is reasonably uniform in cross section, slope, and roughness, the tailwater may affect the culvert discharge capacity or headwater. In this case, the tailwater can be approximated by solving for the normal depth in the channel using Manning's equation as described in Chapter 4.

3. If the tailwater in the downstream channel is established by downstream controls, other means must be used to determine the tailwater elevation. Downstream controls can include such things as natural stream constrictions, downstream obstructions, or backwater from another stream or water body. If it is determined that a downstream control exists, a method such as a backwater analysis, a study of the stage–discharge relationship of another stream into which the stream in question flows, or the securing of data on reservoir storage elevations or tidal information may be involved in determining the tailwater elevation during the design flow. If a field inspection reveals the likelihood of a downstream control, contact the HQ Hydraulics Section for additional guidance.

3-3.4 *Flow Type*

Refer to HDS-5 for in depth discussions of culvert flow types

3-3.5 *Velocities in Culverts – General*

A culvert, because of its hydraulic characteristics, generally increases the velocity of flow over that in a natural channel. High velocities are most critical just downstream from the culvert outlet and the erosion potential from the energy in the water must be considered in culvert design.

Culverts that produce velocities in the range of 3 to 10 ft/s tend to have fewer operational problems than culverts that produce velocities outside of that range. Varying the grade of the culvert generally has the most significant effect on changing the velocity, but since many culverts are placed at the natural grade of the existing channel, it is often difficult to alter this parameter. Other measures, such as changing the roughness characteristics of the barrel, increasing or decreasing the culvert size, or changing the culvert shape should be investigated when it becomes necessary to modify the outlet velocity.

If velocities are less than about 3 ft/s, siltation in the culvert may become a problem. In those situations, it may be necessary to increase the velocity through the culvert or to provide oversized culverts. An oversized culvert will increase siltation in the culvert, but the larger size may prevent complete blocking and will facilitate cleaning. It is recommended that the PEO consult with the RHE to determine the appropriate culvert size for this application. Additional guidance will be provided in future revisions to the *Hydraulics Manual*.

If velocities exceed about 10 ft/s (3 meters/second), abrasion due to bed load movement through the culvert and erosion downstream of the outlet can increase significantly. Abrasion is discussed in more detail in Chapter 8. Corrugated metal culverts may be designed with extra thickness to account for possible abrasion. Concrete box culverts and concrete arches may be designed with sacrificial steel inverts or extra slab thicknesses to resist abrasion. Thermoplastic pipe exhibits better abrasion characteristics than metal or concrete; see Chapter 8 for further guidance.

Adequate outlet channel or embankment protection must be designed to ensure that scour holes or culvert undermining will not occur. Energy dissipators can also be used to protect the culvert outlet and downstream property, as discussed in Section 3-4.7. Energy dissipators can significantly increase the cost of a culvert and should only be considered when required to prevent a large scour hole or as remedial construction.

Refer to HDS-5 for procedures used to calculate culvert velocities.

3-3.6 *Culvert Hydraulic Calculations Form*

Refer to HDS-5 for culvert calculation forms, charts, and nomographs if using hand calculations for culvert design. However, the FHWA culvert design computer program HY-8 is the preferred WSDOT design method.

3-3.7 *Computer Programs*

Once familiar with culvert design theory as presented in this chapter, the PEO is encouraged to utilize one of several commercially available culvert design software programs. The FHWA has developed a culvert design program named HY-8 that utilizes the same general theory presented in this chapter. HY-8 is a user-friendly, Windows-based software, and the output from the program can be printed and incorporated directly into the hydraulic report. HY-8 is copyright protected but the copyright allows for free software distribution. It is available by contacting either the RHE or the HQ Hydraulics Section at: www.fhwa.dot.gov/engineering/hydraulics/software.cfm

In addition to being user-friendly, HY-8 is advantageous in that the headwater elevations and outlet velocities calculated by the program tend to be more accurate than the values calculated using the methods presented in this chapter. HY-8 computes an actual water surface profile through a culvert using standard step-backwater calculations. The methods in this chapter approximate this approach but make several assumptions to simplify the design. HY-8 also analyzes an entire range of flows input by the user. For example, the program will simultaneously evaluate the headwater created by the Q25 and Q100 flow events, displaying all the results on one screen. This results in a significantly simplified design procedure for multiple flow applications. The HY-8 program contains a help guide accessed internally to aid in the system's operations. Additional guidance will be provided in future revisions to the *Hydraulics Manual*.

3-3.8 *Example*

Refer to HDS-5 for example culvert calculations.

3-4 Culvert End Treatments

The type of end treatment used on a culvert depends on many interrelated and sometimes conflicting considerations. The PEO must evaluate safety, aesthetics, debris capacity, hydraulic efficiency, scouring, and economics. Each end condition may serve to meet some of these purposes, but none can satisfy all these concerns. The PEO must use good judgment to arrive at a compromise as to which end treatment is most appropriate for a specific site. Treatment for safety is discussed in WSDOT's *Design Manual*.

Several different types of end treatments will be discussed in this section. The type of end treatment chosen for a culvert shall be specified in the hydraulic report and the contract plans for each installation.

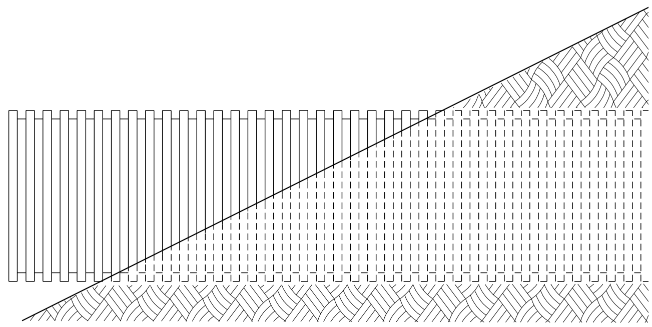
3-4.1 *Projecting Ends*

A projecting end is a treatment where the culvert is allowed to protrude out of the embankment (see Figure 3-7). The primary advantage of this type of end treatment is that it is the simplest and most economical of all treatments. Projecting ends also provide excellent strength characteristics since the pipe consists of a complete ring structure out to the culvert end.

There are several disadvantages to projecting ends. For metal, the thin wall thickness does not provide flow transition into or out of the culvert, significantly increasing head losses (the opposite is true for concrete, the thicker wall provides a more efficient transition). From an aesthetic standpoint, projecting ends may not be desirable in areas exposed to public view. They should only be used when the culvert is located in the bottom of a ravine or in rural areas.

Modern safety considerations require that no projecting ends be allowed in the designated clear zone. (See WSDOT's *Design Manual*) for details on the clear zone and for methods that allow a projecting end to be used close to the traveled roadway.)

Figure 3-7 Projecting End



Projecting ends are also susceptible to flotation when the inlet is submerged during high flows. Flotation occurs when an air pocket forms near the projecting end, creating a buoyant force that lifts the end of the culvert out of alignment. The air pocket can form when debris plugs the culvert inlet or when significant turbulence occurs at the inlet as flow enters culvert. Flotation tends to become a problem when the diameter exceeds 6 feet for metal pipe and 2 feet for thermoplastic pipe. It is recommended that pipes exceeding those diameters be installed with a beveled end and a concrete headwall or slope collar as described in Sections 3-4.2 and 3-4.4. Concrete pipe will not experience buoyancy problems and can be projected in any diameter. However, because concrete pipe is fabricated in relatively short 6- to 12-foot sections, the sections are susceptible to erosion and corresponding separation at the first joint from the end.

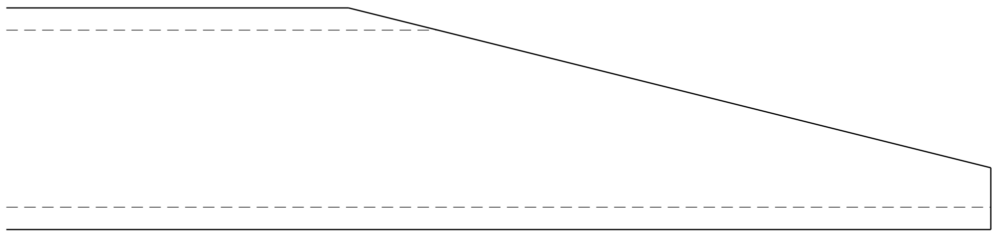
3-4.2 *End Sections*

A beveled end treatment consisting of cutting the end of the culvert at an angle to match the embankment slope surrounding the culvert is referred to as a Flush Bevel. This type of bevel is preferred over others due to increased efficiency and reduced impact on the surrounding environment. For more information about bevels see HDS-5. A typical bevel schematic is shown on Standard Plan B-70.20-00 and in Figure 3-8. A beveled end provides a hydraulically more efficient opening than a projecting end, is relatively cost effective, and is generally considered to be aesthetically acceptable.

Beveled ends should be considered for culverts 6 feet in diameter and less. If culverts larger than 6 feet in diameter are beveled but not reinforced with a headwall or slope collar, the structural integrity of the culvert can be compromised, and failure can occur. The standard beveled end section shall not be used on culverts placed on a skew of more than 30 degrees from the perpendicular to the centerline of the highway; however, a standard beveled end section can be considered if the culvert is rotated until it is parallel with the highway.

Cutting the ends of a corrugated metal or plastic culvert structure to an extreme skew or bevel to conform to the embankment slope destroys the ability of the end portion of the structure to act as a ring in compression. Headwalls, riprap slopes, slope paving, or stiffening of the pipe may be required to stabilize these ends. In these cases, special end treatment shall be provided if needed. The HQ Hydraulics Section can assist in the design of special end treatments.

Figure 3-8 Beveled End Section

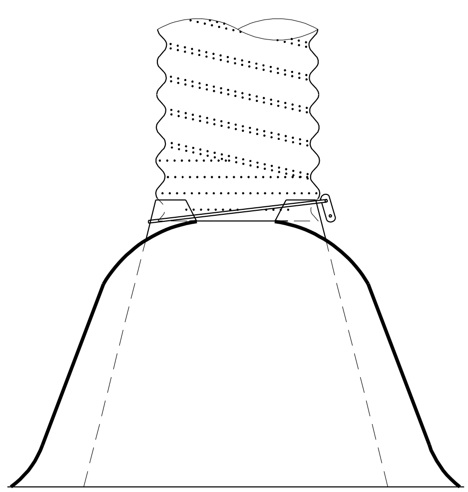


3-4.3 *Flared End Sections*

A metal flared end section is a manufactured culvert end that provides a simple transition from culvert to channel. Flared end sections allow flow to smoothly constrict into a culvert entrance and then spread out at the culvert exit as flow is discharged into the natural channel or watercourse. Flared ends are generally considered aesthetically acceptable since they serve to blend the culvert end into the finished embankment slope.

Flared end sections are typically used only on circular pipe or pipe arches. The acceptable size ranges for flared ends and other details are shown on Standard Plans for Flared End Sections and a detail is shown in Figure 3-9. Flared ends are generally constructed out of steel and aluminum and should match the existing culvert material, if possible. However, either type of end section can be attached to concrete or thermoplastic pipe and the contractor should be given the option of furnishing either steel or aluminum flared end sections for those materials.

Figure 3-9 Flared End Section



A flared end section is usually the most feasible option in smaller pipe sizes and should be considered for use on culverts up to 48 inches in diameter. For diameters larger than 48 inches, end treatments such as concrete headwalls tend to become more economically viable than the flared end sections.

The undesirable safety properties of flared end sections generally prohibit their use in the clear zone for all but the smallest diameters (see WSDOT's *Design Manual* for culvert design). A flared end section is made of light gauge metal and, because of the overall width of the structure, it is not possible to modify it with safety bars. When the culvert end is within the clear zone and safety is a consideration, the PEO must use a tapered end section with safety bars as shown on Standard Plans B-80.20-00 and B-80.40-00. The tapered end section is designed to match the embankment slope and allow an errant vehicle to negotiate the culvert opening in a safe manner.

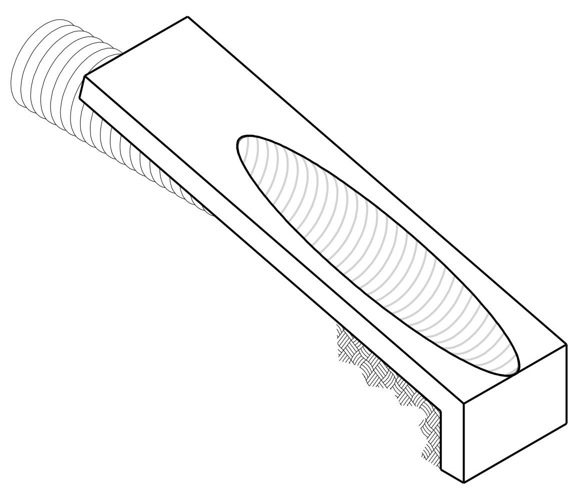
3-4.4 *Headwalls and Slope Collars*

A headwall is a concrete frame poured around a beveled culvert end. It provides structural support to the culvert and eliminates the tendency for buoyancy. A headwall is generally considered to be an economically feasible end treatment for metal culverts that range in size from 6 to 10 feet. Metal culverts smaller than 6 feet generally do not need the structural support provided by a headwall. Headwalls shall be used on all thermoplastic culverts. A typical headwall is shown on Standard Plans B-75.20-02 or in Figure 3-10. When the culvert is within the clear zone, the headwall design can be modified by adding safety bars. Standard Plans B-75.50-01 and B-75.60-00 provide the details for attaching safety bars.

The PEO is cautioned not to use safety bars on a culvert where debris may cause plugging of the culvert entrance even though the safety bars may have been designed to be removed for cleaning purposes. When the channel is known to carry debris, the PEO shall provide an alternate solution to safety bars, such as increasing the culvert size or providing guardrail protection around the culvert end.

Headwalls for culverts larger than 10 feet tend to lose cost-effectiveness due to the large volume of material and forming cost required for this type of end treatment. Instead, a slope collar is recommended for culverts larger than 10 feet in diameter. A slope collar is a reinforced concrete ring surrounding the exposed culvert end. The HQ Hydraulics Section generally performs the design of the slope collar during the structural analysis of the culvert.

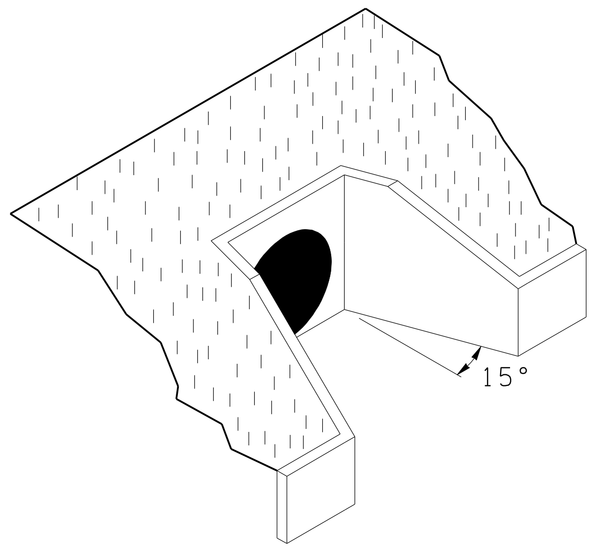
Figure 3-10 Headwall



3-4.5 *Wing Walls and Aprons*

Wing walls and aprons are intended for use on reinforced concrete box culverts. Their purpose is to retain and protect the embankment and provide a smooth transition between the culvert and the channel. Normally, they consist of flared vertical wing walls, a full or partial apron, and bottom and side cutoff walls (to prevent piping and undercutting). Wing walls may also be modified for use on circular culverts in areas of severe scour problems (Figure 3-11). The apron will provide a smooth transition for the flow as it spreads to the natural channel. When a modified wing wall is used for circular pipe, the PEO must address the structural details involved in the joining of the circular pipe to the square portion of the wing wall. The HQ Hydraulics Section can assist in this design.

Figure 3-11 Modified Wing Wall for Circular Pipe



3-4.6 *Improved Inlets*

When the head losses in a culvert are critical, the PEO may consider the use of a hydraulically improved inlet. These inlets provide side transitions as well as top and bottom transitions that have been carefully designed to maximize the culvert capacity with the minimum amount of headwater; however, the design and form construction costs can become quite high for hydraulically improved inlets. For this reason, their use is not encouraged in routine culvert design. It is usually less expensive to simply increase the culvert diameter by one or two sizes to achieve the same or greater benefit.

Certain circumstances may justify the use of an improved inlet. When complete replacement of the culvert is too costly, an existing inlet-controlled culvert may have its capacity increased by an improved inlet. Improved inlets may also be justified in new construction when the length of the new culvert is long (over 500 feet) and the headwater is controlled by inlet conditions. Improved inlets may have some slight advantage for barrel or outlet-controlled culverts, but usually not enough to justify the additional construction costs. If the PEO believes that a site might be suitable for an improved inlet, the HQ Hydraulics Section should be contacted. Also, HDS-5 contains a significant amount of information related to the design of improved inlets.

3-4.7 *Energy Dissipators*

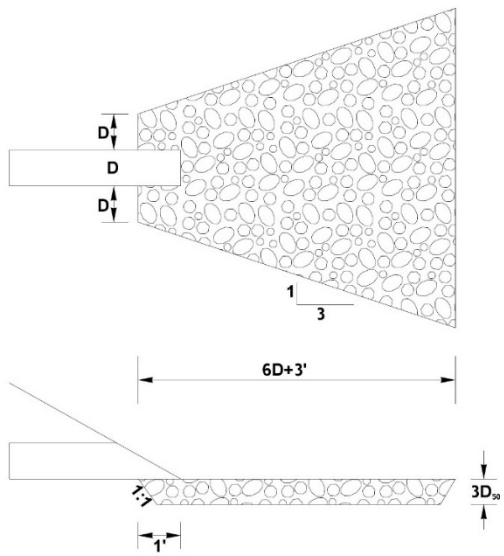
When the outlet velocities of a culvert are excessive (exceeds 6 ft/s), the PEO shall use an energy dissipator. Energy dissipators can be quite simple or very complex, depending on the site conditions. Debris and maintenance problems should be considered when designing energy dissipators.

Typical energy dissipators include:

1. **Riprap Protected Outlets**

   Riprap is frequently hand placed around the outlet end of culverts to protect against the erosive action of the water (Figure 3-12). The material size at the outlet is dependent on the outlet velocity as noted in Figure 3-13. The limits of this protection would typically cover an area that would be vulnerable to scour holes. (See Section 3-4.5 for details on wing walls and aprons.)

Figure 3-12 Riprap Protected Outlet



Note: Evaluate need to extend splash pad made to suit site conditions.

Figure 3-13 Outlet Protection Material Size

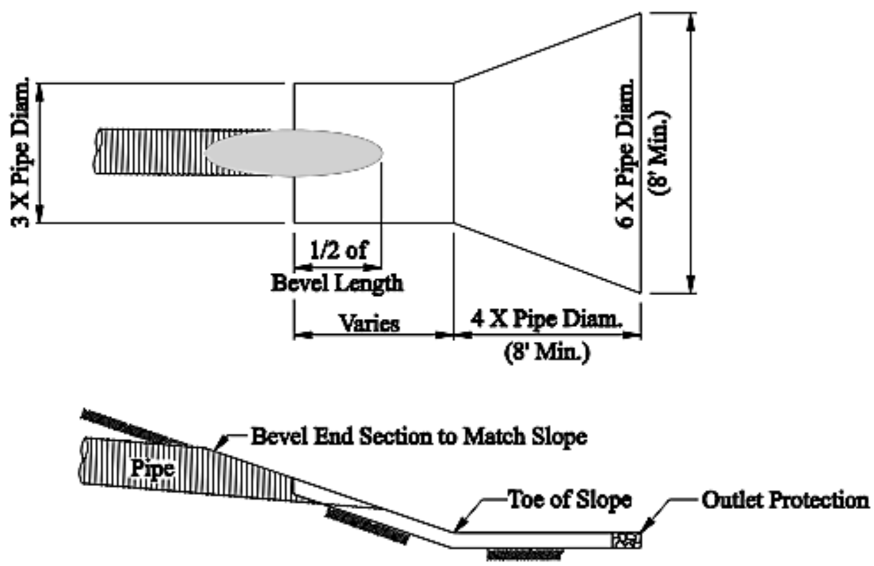
Outlet Velocity (feet/second)	Material
5–7	Quarry Spalls
7–10	Rock for Erosion and Scour Protection Class A
10–15	Rock for Erosion and Scour Protection Class B
>15	Rock for Erosion and Scour Protection Class C

Note:
The Project Engineer's Office should provide geotextile or filter material between outlet material and the existing ground for soil stabilization (see Chapter 4 for information).

2. **Splash Pads**

   If Figure 3-12 indicates that outlet protection is required, another option is a splash pad. Splash pads are constructed in the field at the culvert outlet and used to prevent erosion. Splash pads shall be a minimum of six times the diameter width and four times the diameter length, as shown in Figure 3-14.

Figure 3-14 Splash Pad Detail



Note: Evaluate need to extend splash pad made to suit site conditions.

3. **Other Energy Dissipating Structures**

   Other structures include impact basins and stilling basins/wells designed according to the FHWA's HEC-14, "Hydraulic Design of Energy Dissipators for Culverts and Channels." These structures may consist of baffles, posts, or other means of creating roughness to dissipate excessive velocity. It is recommended that the HQ Hydraulics Section be consulted to assist in the design of these types of structures.

Energy dissipators have a reputation for collecting debris on the baffles, so the PEO should consider this possibility when choosing a dissipator design. In areas of high debris, the dissipator should be kept open and easily accessible to maintenance crews. Provisions should be made to allow water to overtop without causing excessive damage.

3-4.8 *Culvert Debris*

Debris problems can cause even an adequately designed culvert to experience hydraulic capacity problems. Debris may consist of anything from limbs and sticks to logs and trees. Silt, sand, gravel, and boulders can also be classified as debris. The culvert site is a natural place for these materials to settle and accumulate. No method is available for accurately predicting debris problems. Examining the maintenance history of each site is the most reliable way of determining potential problems. Sometimes, upsizing a culvert is necessary to enable it to more effectively pass debris. Upsizing may also allow a culvert to be more easily cleaned.

3-5 Miscellaneous Culvert Design Considerations

3-5.1 *Multiple Culvert Openings*

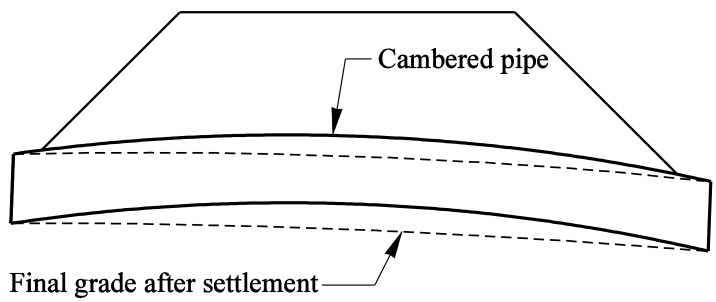
The use of multiple culvert openings is discouraged due to decreased efficiency and less room available to transport LWM. Using multiple culverts requires a deviation from the HQ Hydraulics Section, thus needing approval from the RHE.

3-5.2 *Camber*

When a culvert is installed under moderate to high fills 30 to 60 feet or higher, there may be greater settlement of the fill under the center of the roadway than at the sides. This occurs because at the culvert ends there is little fill while at the centerline of the roadway contains the maximum fill. The difference in surcharge pressure at the elevation of the culvert may cause differential settlement of the fill and can create a low point in the culvert profile. To correct for the differential settlement, a culvert can be constructed with a slight upward curve in the profile, or camber, as shown in Figure 3-15. This is determined by the HQ Geotech.

The camber is built into the culvert during installation by laying the upstream half of the culvert on a flat grade and the downstream half on a steeper grade to obtain the design grade after settlement. The amount of expected camber can be determined by the HQ Materials Laboratory and must be shown on the appropriate profile sheet in the contract plans.

Figure 3-15 Camber Under High Fills



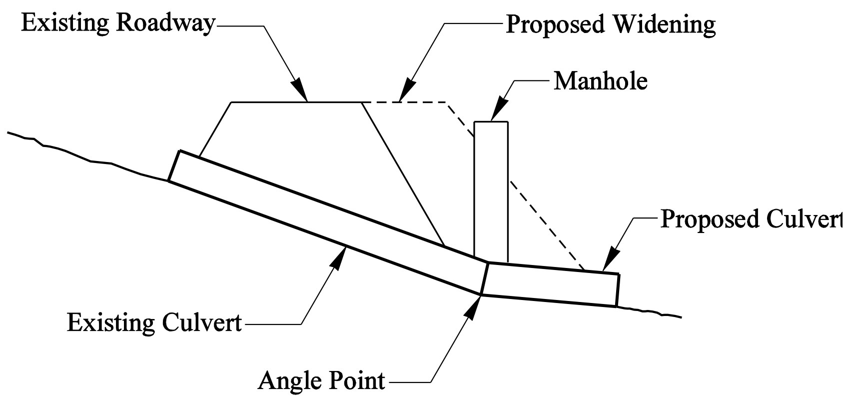
3-5.3 *Horizontal and Vertical Angle Points*

It is recommended that the slope of a culvert remain constant throughout the entire length of the culvert. This is generally easy to accomplish in new embankments. However, in situations where existing roadways are to be widened, it may be necessary to extend an existing culvert at a different slope. The location where the slope changes is referred to as the angle point.

If the new culvert is to be placed at a flatter grade than the existing culvert, it is recommended that a manhole be incorporated into the design at the angle point, as shown in Figure 3-16. The change in slope tends to create a location in the culvert that will catch debris and sediment. Providing access with a manhole will facilitate culvert maintenance.

If the new culvert is to be placed at a steeper slope than the existing culvert, the manhole can be eliminated at the angle point if debris and sedimentation have not historically been a concern at the existing culvert.

Figure 3-16 Culvert Angle Point



3-5.4 *Upstream Ponding*

The culvert design methodology presented in Section 3-3 assumes that the headwater required to pass a given flow through a culvert will be allowed to fully develop upstream of the culvert inlet. Any peak flow attenuation provided by ponding upstream of the culvert inlet is ignored. If a large enough area upstream of the inlet is available for ponding, the design headwater will not occur, and the culvert will not pass the full design flow. However, by ignoring any ponding effects, the culvert design is simplified, and the final results are conservative. Most culverts should be designed using these assumptions.

If it is determined that the ponding characteristics of the area upstream of the inlet need to be taken into consideration, the calculation of flow becomes a flood routing problem which entails a more detailed study. Essentially, the area upstream of the inlet acts as a detention pond and the culvert acts as an outlet structure. The culvert can be designed utilizing flood-routing concepts similar to designing a stormwater detention pond, but that methodology is beyond the scope of the *Hydraulics Manual*. Since the need for this type of culvert design is rare, the HQ Hydraulics Section should be contacted for further assistance.

3-5.5 *Miscellaneous Design Considerations – Siphons*

A siphon is a water conveyance conduit, which operates at subatmospheric pressure over part of its length. These types of culverts shall not be used unless the site has no other practical option of water conveyance. Siphons pose a large safety risk for animals and humans alike so due to their increased danger over other culverts, siphons require a HQ Hydraulics Section specialty report.

Chapter 4 Open-Channel Flow

4-1 Introduction

An open channel is a watercourse that allows part of the flow to be exposed to the atmosphere. This type of channel includes rivers, culverts, stormwater systems that flow by gravity, roadside ditches, and roadway gutters. Open-channel flow design criteria are used in the following areas of transportation design:

- River stabilization (Section 4-6)
- Partially full flow pipes
- Roadside ditches (Section 4-3)
- Bridge design
- Downstream analysis

Proper design requires that open channels have sufficient hydraulic capacity to convey the flow of the design storm. In the case of earth-lined channels or river channels, bank protection is also required if the shear stress is high enough to cause erosion or scouring.

This chapter provides guidance for designing systems with open-channel flow, including determining design velocity (Section 4-2) and critical depth (Section 4-4), designing roadside ditches (Section 4-3) and backwater analysis for river flow (Section 4-5).

River stabilization (Section 4-6) may be necessary for highly erosive, high-energy rivers, to help the river dissipate some of its energy and stabilize the river banks and channel bottom. The success of the rock structures or rock bank protection is dependent on the ability of the rock to withstand the forces of the river; therefore, it is important to properly size the rocks used. The methodology for sizing rocks used in river stabilization is described in HEC-22.

The flow capacity of a culvert is often dependent on the channel up- and downstream from that culvert. For example, the tailwater level is often controlled by the hydraulic capacity of the channel downstream of the culvert. Knowing the flow capacity of the downstream channel, open-channel flow equations can be applied to a typical channel cross section to adequately determine the depth of flow in the downstream channel. This depth can then be used in the analysis of the culvert hydraulic capacity.

Biofiltration swales are shallow, grass-lined, open channels that clean stormwater runoff before it reaches a receiving body. The PEO should route stormwater through biofiltration swales or other approved stormwater BMPs as required in the *Highway Runoff Manual*.

A downstream analysis identifies and evaluates the impacts a project will have on the hydraulic conveyance system downstream of the project site. See Section 1-3.7.

Measurement of flow in channels can be difficult because of the nonuniform channel dimensions and variations in velocities across the channel. Weirs allow water to be routed through the structure of known dimension, permitting flow rates to be measured as a function of depth of flow through the structure.

4-2 Determining Channel Velocities

In open-channel flow, the volume of flow and the rate at which flow travels are useful in designing the channel. For the purposes of the *Hydraulics Manual*, the determination of the flow rate in the channel, also known as discharge, is based on the continuity of flow equation or Equation 4-1. This equation states that the discharge (Q) is equivalent to the product of the channel velocity (V) and the area of flow (A).

$$Q = V\,A \tag{4-1}$$

Where:

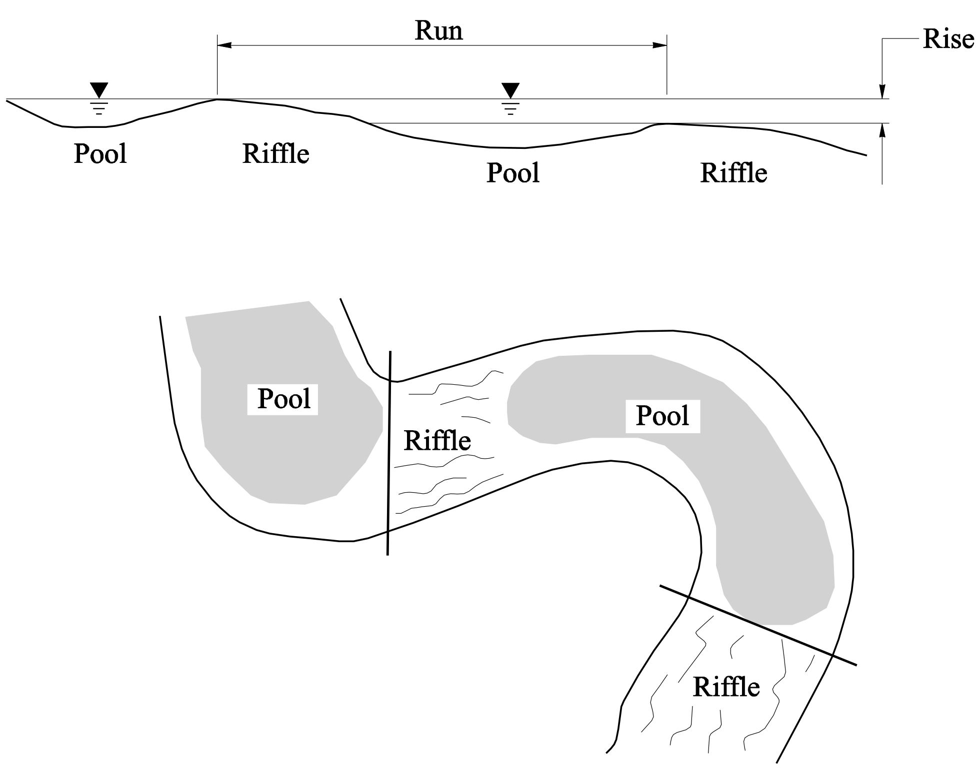
- Q = discharge, cfs
- V = velocity, ft/s
- A = flow area, ft^2

In some situations, the flow area of a channel is known. If it is not, the flow area must be calculated. Computer programs and charts from HDS-4 are available for determining channel geometry or velocities.

4-2.1 *Field Slope Measurements*

By definition, slope is rise over run (or fall) per unit length along the channel centerline or thalweg. Slope is the vertical drop in the river channel divided by the horizontal distance measured along the thalweg of a specific reach. The vertical drop shall be measured from the water surface at the top-of-riffle (end of pool) to the next top-of-riffle to get an accurate representation of the slope in that reach (Figure 4-1).

Figure 4-1 Field Slope Measurement

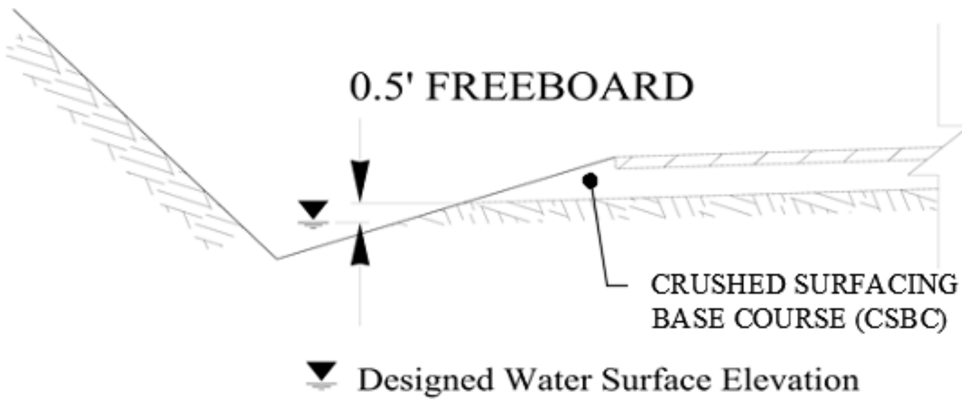


4-3 Roadside Ditch Design Criteria

Roadside ditches are generally located alongside uncurbed roadways with the primary purpose of conveying runoff away from the roadway. Ditches shall be designed to convey the 10-year recurrence interval with a 0.5-foot freeboard (from the ditch design water surface elevation to the bottom of the pavement subgrade or ditch spill) and a maximum side slope of 2H:1V (Figure 4-2). The preferred cross section of a ditch is trapezoidal; however, a "V" ditch can also be used where ROW is limited or the design requirements can still be met. In those cases where the grade is flat, preventing adequate freeboard, the depth of channel should still be sufficient to remove the water without saturating the subgrade shoulder.

To maintain the integrity of the channel, ditches are usually lined. See HDS-4, HEC-15, or the *Standard Specifications* for more information. WSDOT's *Design Manual* also contains design guidance for both paved and grass-lined ditches.

Figure 4-2 Drainage Ditch Detail



Ditches should not be confused with biofiltration swales. In addition to collecting and conveying drainage, biofiltration swales also provide runoff treatment by filtering out sediment. (See the *Highway Runoff Manual* for design guidance for biofiltration swales.) Roadside ditches are to be designed so the integrity or geometry of the roadway is not compromised.

4-4 Critical Depth

Before finalizing a channel design, the PEO must verify that the normal depth of a channel is either greater than or less than the critical depth. Critical depth is the depth of water at critical flow, a very unstable condition where the flow is turbulent and a slight change in the specific energy—the sum of the flow depth and velocity head—could cause a significant rise or fall in the depth of flow. Critical flow is also the dividing point between the subcritical flow regime (tranquil flow), where normal depth is greater than critical depth, and the supercritical flow regime (rapid flow), where normal depth is less than critical depth.

Critical flow tends to occur when passing through an excessive contraction, either vertical or horizontal, before the water is discharged into an area where the flow is not restricted. A characteristic of critical depth flow is often a series of surface undulations over a very short stretch of channel. The PEO should be aware of the following areas where critical flow could occur: culverts, bridges, and near the brink of an overfall.

A discussion of specific energy is beyond the scope of the *Hydraulics Manual*. The PEO should refer to HDS-5 or HEC-14, for further information.

4-5 River Backwater Analysis

Natural river channels tend to be highly irregular in shape so an analysis using Manning's equation, while helpful for making an approximation, is not sufficiently accurate to determine a river water surface profile. The HQ Hydraulics Section is responsible for computing water surface profiles and has several computer programs to calculate the water surface profile of natural river channels. The computation of the water surface profile is called a backwater analysis. The purpose of this section is to state when a backwater analysis is necessary as well as to summarize the minimum design requirements for the analysis and provide the project office with a list of field information required for the HQ Hydraulics Section to perform an analysis. This section will be revised in a future update.

A backwater analysis is performed when designing a bridge that crosses a river designated as a FEMA regulatory floodplain. WSDOT is required by federal mandate to design these bridges to accommodate the 100-year storm event. It is desirable to maintain a 3-foot vertical clearance between the bottom of the bridge and the 100-year water surface elevation. The water surface elevations for the 100-year and 500-year water surface profiles shall be shown on the plans.

Backwater analysis can be useful in culvert design. Computing the water surface profile can help the PEO determine if the culvert is flowing under inlet or outlet control. The region must provide the following information to the HQ Hydraulics Section to complete a river backwater analysis.

1. A topographic surface of the project site with 1- or 2-foot contour intervals is required. The extent of the topographic mapping required is site-specific but shall include all areas within the 100-year floodplain. All bridge and unique attributes of the project area shall be identified.
2. The Manning's roughness coefficients must be established for all parts of the river within the project area. See Appendix 4A for guidance. The HQ Hydraulics Section will need photographs of the channel bed and streambank along the reach of interest to determine the appropriate channel roughness. Photographs are especially important in areas where ground cover changes.

To prevent subsequent difficulties in the backwater analysis, the HQ Hydraulics Section should be contacted to determine the necessary parameters. For additional information about backwater analyses, see FHWA's HDS-7.

4-6 River Stabilization

Because of the abundance of watercourses in Washington State, and the legacy of highway placement along and across their corridors, stabilization of part of the river cross section or alignment is often necessary to protection transportation investments. New roadways and other infrastructure must be placed to minimize interaction with or effects on waterbodies, avoiding them altogether if possible. This section discusses the options available for those cases where action must be taken and provides a subset of suggested techniques and associated technical references for those techniques. This is not a comprehensive guide, and as new techniques arise, all should be considered (in coordination with HQ Hydraulics Section) for their cost benefit in addressing interactions with water bodies.

4-6.1 *Streambank Protection*

Extensive guidance exists for numerous techniques for bank protection, from riprap to revegetation. Many techniques recommended in Pacific Northwest Rivers incorporate LWM. Some of the most pertinent guidance documents are listed below:

- HEC-23, Volumes 1 and 2
- Integrated Streambank Protection Guidelines (ISPG; WDFW 2002)
- Stream Restoration Design (National Engineering Handbook 654, NRCS 2007)
- Bank Stabilization Design Guidelines (Baird et al. 2015)
- WDFW's Stream Habitat Restoration Guidelines (April 2012 Draft)

The techniques are too numerous to discuss in detail in this document. Figure 4-3 lists the most common treatment types and the conditions for which they are most appropriate.

Figure 4-3 Common Treatment Types and Conditions

No Action	Flow-redirection Techniques	Structural Techniques	Biotechnical Techniques	Internal Bank-Drainage Techniques	Avulsion-prevention Techniques	Other Techniques
Allow Bank Erosion to Continue	Groins	Anchor Points	Woody Plantings	Subsurface Drainage Systems	Floodplain Roughness	Channel Modifications
Move structures at risk	Buried Groins	Roughness Trees	Herbaceous Cover	--	Floodplain Grade Control	Riparian Buffer Management
--	Barbs	Riprap	Soil Reinforcement	--	Floodplain Flow Spreader	Spanning Habitat Restoration
--	Engineered Logjams	Log Toes	Coir Logs	--	--	Off-Channel Spawning and Rearing Habitat
--	Drop Structures	Rock Toes	Bank Reshaping	--	--	--
--	Porous Weirs	Log Crib walls	--	--	--	--
--	--	Manufactured Retention Systems	--	--	--	--

Note:
-- = not applicable

Additionally, matrices 1, 2, and 3 in the ISPG provide qualitative ratings of each technique relative to the underlying cause of a problem is site-based (local conditions) or reach-based (watershed conditions).

4-6.2 *Riprap for Bank Stabilization*

Riprap bank protection is a layer of rock placed to stabilize the bank and inhibit lateral erosion. Riprap is deformable, compared to rigid channel linings such as concrete. Rigid channel linings are generally not recommended for the same reasons that flexible linings are recommended. If rigid linings are undermined, the entire rigid lining will be displaced increasing the chances of failure and leaving the bank unprotected. Riprap rock encased in grout is also an example of a rigid channel lining.

There are disadvantages to using riprap bank protection. Replacing streambank vegetation with riprap will create a relatively smooth surface, resulting in higher water velocities. This change will impact the channel downstream, and to some extent upstream, where the riprap ends, creating a higher potential for erosion. Because of impacts to the adjacent channel, the PEO should consider if using riprap for bank protection would solve the problem or create a new problem. In addition, Section 24-046 of Title 222 of the Washington Administrative Code (WAC 222-24-046) states that bioengineering techniques are preferred, that work area is to be minimized to the area needing protection, and that mitigation will be required whenever riprap is used. These aspects should be considered when determining if riprap is appropriate.

Riprap bank protection is primarily used on the outside of curved channels or along straight channels when the streambank serves as the roadway embankment. Riprap on the inside of the curve is only recommended when overbank flow reentering the channel may cause scour. On a straight channel, bank protection shall begin and end at a stable feature in the bank, if possible. Such features may be bedrock outcroppings or erosion-resistant materials, trees, vegetation, or other evidence of stability.

This section does not apply to an existing bridge or when historical evidence indicates that riprap will be needed around a new bridge. In those cases, the region should indicate this information on the Bridge Site Data Sheet (Form 235-001) and refer the riprap design to the HQ Hydraulics Section.

4-6.2.1 Riprap Sizing for Bank Protection

A design procedure for rock riprap channel linings was developed by the University of Minnesota as a part of a National Cooperative Highway Research Program (NCHRP) study under the sponsorship of the AASHTO. The design procedure presented in this section is based on this study and has been modified to incorporate riprap as defined in the *Standard Specifications*.

Once the PEO has completed the analysis in this section, the PEO should consider the certainty of the velocity value used to size riprap along with the importance of the facility. For additional guidance, PEOs can consult *NCHRP Report 568 - Riprap Design Criteria* and *Hydraulic Engineering Circular 23 - Bridge Scour and Stream Instability Countermeasures* (Lagasse et al. 2006).

Manning's formula or computer programs compute the hydraulic capacity of a riprap-lined channel. The appropriate n-values are shown in Figure 4-4.

Figure 4-4 Manning's Roughness Coefficients for Riprap (n)

Type of Rock Lining[1]		n (Small Channels)[2]	n (Large Channels)
Spalls	D_{50} = 0.5 ft	0.035	0.030
Light Loose Riprap	D_{50} = 1.1 ft	0.040	0.035
Heavy Loose Riprap	D_{50} = 2.2 ft	0.045	0.040

Notes:
[1]See the *Standard Specifications*.
[2]Small channels can be loosely defined as less than 1,500 cubic feet per second.

Using Manning's equation, the PEO can determine the slope, the depth of flow, and the side slopes of the channel required to carry the design flow. The PEO, using this information, can then determine the required minimum D_{50} stone size with Equation 4-2.

$$D_{50}=C_R\, d\, S_o \qquad (4\text{-}2)$$

Where:

D_{50} = Particle size of gradation, ft, of which 50 percent by weight of the mixture is finer
C_R = Riprap coefficient (see Figure 4-5)
D = Depth of flow in channel, ft
S_o = Longitudinal slope of channel, ft/ft
B = Bottom width of trapezoidal channel, ft (see Figure 4-5)

Figure 4-5 **Riprap Coefficients**

Channel	Angular Rock 42° of Repose (0.25' < D_{50} < 3')			Rounded Rock 38° of Repose (0.25' < D_{50} < 0.75')		
Side Slopes	**B/d=1**	**B/d=2**	**B/d=4**	**B/d=1**	**B/d=2**	**B/d=4**
1.5H:1V	21	19	18	28	26	24
1.75H:1V	17	16	15	20	18	17
2H:1V	16	14	13	17	15	14
2.5H:1V	15	13	12	15	14	13
3H:1V	15	13	12	15	13	12
4H:1V	15	13	12.5	15	13	12.5
Flat Bottom	12.5	12.5	12.5	12.5	12.5	12.5

Note:
Angular rock should be used for new bank protection as it is better at interlocking and providing a stable slope. Rounded rock is unstable and is not recommended for new bank protection. The coefficients have only been provided to verify if native material is a sufficient size to resist erosion. Rounded rock use in new designs should be limited to the channel bed region and to provide streambed characteristics in a bottomless arch culvert.

a. Example 1 – Riprap Sizing for Bank Protection

A channel has a trapezoidal shape with side slopes of 2H:1V and a bottom width of 10 feet. It must carry a Q_{25} = 1,200 cfs and has a longitudinal slope of 0.004 ft/ft. Determine the normal depth and the type of riprap, if any, that is needed.

After estimating the velocity (Section 4-2) and guessing a roughness coefficient for riprap from Figure 4-4 (for this example, n = 0.035 was chosen for spalls), the normal depth was found to be d = 7.14 ft with a velocity of V = 6.92 ft/s.

Next, use Figure 4-5 to determine what type, if any, riprap is needed.

$$B/d = \frac{10\text{ft}}{7.14\text{ft}} = 1.4$$

Given a side slope of 2H:1V, and a calculated value of B/d = 1.4, C_R is noted to be between 16 and 14 in Figure 4-5 for angular rock. It is allowable to interpolate between B/d columns.

$$D_{50}=C_R\ d\ S_o$$

$$D_{50}=15(7.14\text{ft})(0.004)=0.43\text{ft}$$

From Figure 4-4, "Spalls" would provide adequate protection for a D_{50} of 0.5 foot or less in this channel. If the present streambed has rock that exceeds the calculated D_{50}, then manmade protection is needed.

b. Example 2 – Riprap Sizing for Bank Protection

Repeat the process using a 1 percent slope, and the PEO finds:

$$D=5.75\text{ ft}$$

$$V=9.72\text{ft/s}$$

$$B/d=10/5.75=1.74\text{ft}$$

$$C_R=14.5$$

$$D_{50}=14.5(5.75\text{ft})(0.01)=0.83\text{ft}$$

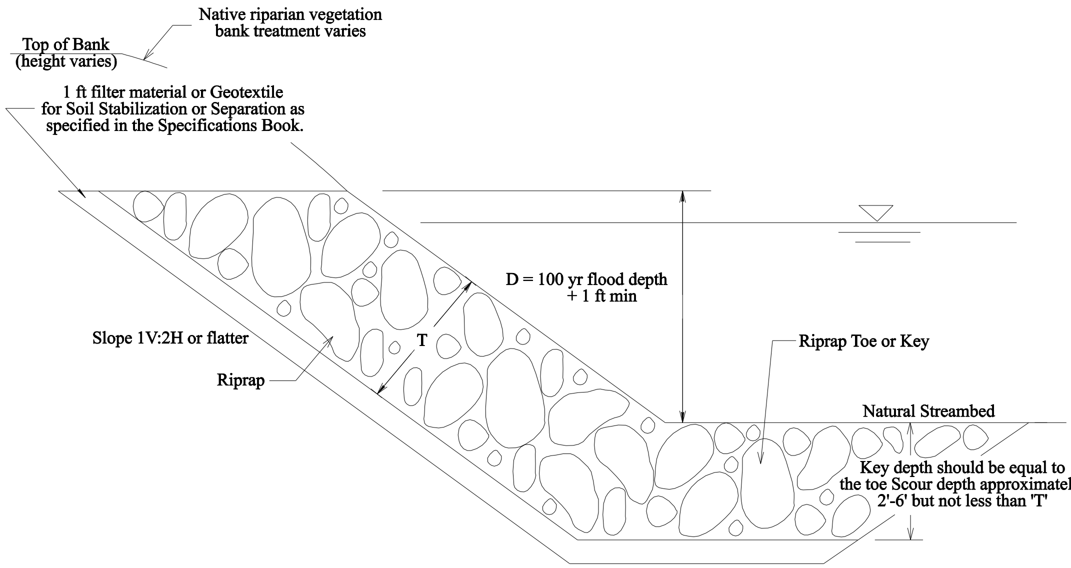
In this case, from Figure 4-4, light loose riprap would be appropriate. Since the roughness coefficient noted in Figure 4-4 for light loose riprap is n = 0.040, the PEO may recalculate the depth and velocity to get a more exact answer but this would only change the normal depth slightly and would not affect the choice of bank protection. In some cases, on very high velocity rivers or rivers that can transport large rocks downstream, even heavy loose riprap may not be adequate to control erosion and specially sized riprap may need to be specified in the contract. The HQ Hydraulics Section and the HQ Materials Laboratory are available for assistance in writing a complete specification for special riprap.

Once the size of riprap is determined, there are several methods in which riprap bank protection can be constructed. Two types of riprap placement, including dumped rock riprap and hand-placed riprap, are discussed in the following sections.

4-6.2.2 Placement of Riprap Bank Protection

Once the type of riprap has been selected from Figure 4-4, the next step is to determine the appropriate installation. Several factors affect the placement of riprap including: the type of filter material best suited for the project site, the thickness of riprap placement, and the depth to key riprap to prevent undermining. Figure 4-6 illustrates a typical cross section of a riprap bank protection installation.

Figure 4-6 Typical Cross Section of Riprap Bank Protection Installation



The filter material acts as a transition between the native soil and the riprap, preventing the piping of fines through the voids of the riprap structure and at the same time allowing relief of the hydrostatic pressure in the soil. There are two types of filters that are used: gravel (filter blanket) or fabric (geotextile). A filter blanket may consist of a 1-foot thick layer of material graded from sand to 6 inches of gravel, (placed in layers from fine to coarse out to the riprap). Filter materials are further described in the *Standard Specifications* and *Design Manual*. If the existing banks are similar to the filter material of sands and gravel, no filter layer may be needed.

The proper selection of a filter material is critical to the stability of the original bank material in that it aids in preventing scour or sloughing. Prior to selecting a filter material, the PEO should first consult with the Region Materials Engineer and the RHE to determine if there is a preference. In areas of highly erodible soil (fine, clay-like soils), the HQ Hydraulics Section should be consulted, and an additional layer of sand may be required. For additional guidance selecting the appropriate filter material, see *Hydraulic Engineering Circular No. 11*.

The thickness of riprap placed (shown as T in Figure 4-6) depends on which type of riprap was selected; quarry spalls, light loose riprap, or heavy loose riprap. Riprap thickness is 2 feet for light loose riprap, 3 feet for heavy loose riprap, and 1 foot for quarry spalls. Care should be taken during construction to ensure that the range of riprap sizes, within each group, is evenly distributed to keep the riprap stable. Riprap is usually extended to 1 foot above the 100-year flood depth of the water as shown in Figure 4-6. However, if severe wave action is anticipated, it should extend farther up the bank.

The PEO and construction inspectors must recognize the importance of a proper toe or key at the bottom of any riprap bank protection. The toe of the riprap is placed below the channel bed to a depth equaling the toe scour depth. If the estimated scour is minimal, the toe is placed at a depth equivalent to the thickness of the riprap and helps to prevent undermining. Without this key, the riprap has no foundation and the installation is certain to fail. Where a toe trench cannot be dug, the riprap shall terminate in a stone toe at the level of the streambed. A stone toe (a ridge of stone) placed along steep, eroding channel banks is one of the most reliable, cost-effective, bank stabilization structures available. The toe provides material, which will fall into a scour hole and prevent the riprap from being undermined. Added care should be taken on the outside of curves or sharp bends where scour is particularly severe. The toe of the bank protection may need to be placed deeper than in straight reaches.

4-6.3 *Channel Stabilization*

Channel stabilization, as opposed to bank stabilization, involves controlling and maintaining the channel cross section, alignment, and gradient, for some given length of the stream. There can be several reasons to stabilize a channel. At WSDOT, it is typically to protect transportation infrastructure such as a culvert or roadway embankment. Some channel stabilization may also be used for fish habitat or passage. The major types of channel stabilization are concrete or rock linings, weirs, dams, and grade-control structures. There are also fish passage features known as roughened channels; see Chapter 7 for more details.

Notably, channel stabilization is a significant modification to natural processes, and is not only technically challenging to design a maintenance-free, sustainable project, but also it is increasingly difficult to obtain the necessary environmental permits from the regulatory agencies. Therefore, such projects should be undertaken only when there are no other feasible options, only in consultation with HQ Hydraulics Section.

Figure 4-7 lists the major categories of channel stabilization techniques, the major materials involved, the risks, as well as references and manuals. Because this topic is so broad and because there is existing guidance, we refer designers to these references for details.

Figure 4-7 Channel Stabilization Techniques

Technique/ Structure	Objectives	Risks	Materials	References
Drop structures	Grade control	Outflanking, becoming fish passage barrier	Logs; concrete; sheet metal	WCDG; ISPG; HEC-23
Engineered Log Jams	Alignment control; avulsion control	Change in flow direction renders ineffective	Steel beams; wood piles; logs with/out rootwads	ISPG
Barbs, groins	Alignment control	Loss of riparian habitat; erosion of adjacent areas; incorrect spacing	Rock; rock with logs	ISPG; HEC-23
Log deflectors	Alignment control	Scour	Logs with rootwads; anchors or piles	ISPG, WCDG
Channel relocation	Alignment control	Design channel doesn't match equilibrium slope and/or shape, resulting in erosion or aggradation	Excavation of natural materials with possible additions of wood or rock	ISPG; National Engineering Handbook 654
Floodplain roughness/ grade control	Avulsion control	Insufficient roughness at bank-overtopping flows	Logs; plantings; seeding; rock	ISPG
Rock weirs	Grade Control	Mass failure; fish barrier	Rock; rock with logs	Rock Weir Design Guidance (BurRec); National Engineering Handbook 654

Notes:

HEC-23 = Hydraulic Engineering Circular No. 23 - Bridge Scour and Stream Instability Countermeasures Experience, Selection, and Design Guidance (Federal Highway Administration 2009)

ISPG = Integrated Streambank Protection Guidelines (Washington Department of Fish and Wildlife 2004)

WCDG = Water Crossing Design Guidelines (Washington Department of Fish and Wildlife 2013)

4-7 Appendices

Appendix 4A Manning's Roughness Coefficients (n)

Appendix 4A Manning's Roughness Coefficients (n)

Figure 4A-1 References for Manning's Roughness Coefficients

Category of Surface	Surfaces Included	Source
Open Channel and Pipe	Closed Conduits Pipes Pavement Gutter Manmade Channels	HEC 22
River, Stream, and Culvert Design for Aquatic Organism Passage	Rigid Channel Minor Streams Floodplains Major Streams Alluvial Beds Sand Beds Gravel Beds Cohesive Soils Composite Roughness Value	HDS 6 HEC 26 (when required for Aquatic Organism Passage) HEC 22 Chow V.T. 1959(1)
Channel Lining	Rigid Channel Unlined Channel Grass Gravel Riprap Gabion	HEC 15
Storm Sewer Conduit(2)	Concrete Pipe Metal Pipe Polyethylene Pipe PVC Pipe	HEC 22
Street and Gutter	Concrete Gutter Asphalt Concrete Pavement	HEC 22
Maintained Vegetation	Grass	HEC 15 Chow V.T. 1959(3)

Notes:
(1)See Figure 4A-2 on following page.
(2)For storm sewer pipes 24 inches or less in diameter, use n = 0.013.
(3)See Figure 4A-3 on following page.

Figure 4A-2 Manning's Roughness Coefficients for Stream Channels

Stream Channels	Manning's n
Minor streams (surface width at flood stage less than 100 feet):	
1. Fairly regular section:	
a. Some grass and weeds, little or no brush	0.030-0.035
b. Dense growth of weeds, depth of flow materially greater than weed height	0.035-0.05
c. Some weeds, light brush on banks	0.035-0.05
d. Some weeds, heavy brush on banks	0.05-0.07
e. Some weeds, dense willows on banks	0.06-0.08
f. For trees within channel, with branches submerged at high stage, increase all above values by 0.01-0.02	
2. Irregular sections, with pools, slight channel meander; increase values given in 1a-e above 0.01-0.02	
3. Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stage:	
a. Bottom of gravel, cobbles, and few boulders	0.04-0.05
b. Bottom of cobbles, with large boulders	0.05-0.07
Floodplains (adjacent to natural streams):	
1. Pasture, no brush:	
a. Short grass	0.030-0.035
b. High grass	0.035-0.05
2. Cultivated areas:	
a. No crop	0.03-0.04
b. Mature row crops	0.035-0.045
c. Mature field crops	0.04-0.05
3. Heavy weeds, scattered brush	0.05-0.07
4. Light brush and trees:	
a. Winter	0.05-0.06
b. Summer	0.06-0.08
5. Medium to dense brush:	
a. Winter	0.07-0.11
b. Summer	0.10-0.16
6. Dense willows, summer, not bent over by current	0.15-0.20
7. Cleared land with tree stumps, 100 to 150 per acre:	
a. No sprouts	0.04-0.05
b. With heavy growth of sprouts	0.06-0.08
8. Heavy stand of timber, a few down trees, little under-growth:	
a. Flood depth below branches	0.10-0.12
b. Flood depth reaches branches	0.12-0.16

Major streams (surface width at flood stage more than 100 feet): Roughness coefficient is usually less than for minor streams of similar description on account of less effective resistance offered by irregular banks or vegetation on banks. Values of n may be somewhat reduced. Follow recommendation in publication cited if possible. The value of n for larger streams of most regular section, with no boulders or brush, may be in the range of 0.028-0.033.

Figure 4A-3 Manning's Roughness Coefficients for Highway Channels and Swales with Maintained Vegetation

Surface	Manning's n	
	Manning's n at Depth of flow <0.7 feet	**Manning's n Depth of flow 0.7-1.5 feet**
Bermudagrass, Kentucky bluegrass, buffalo grass:		
Mowed to 2 inches	0.07-0.045	0.05-0.035
Length 4 to 6 inches	0.09-0.05	0.06-0.04
Good stand, any grass:		
Length about 12 inches	0.18-0.09	0.12-0.07
Length about 24 inches	0.30-0.15	0.20-0.10
Fair stand, any grass:		
Length about 12 inches	0.14-0.08	0.10-0.06
Length about 24 inches	0.25-0.13	0.17-0.09

Note:
Values shown are for velocities of 2 and 6 feet per second.

Chapter 5 Drainage of Highway Pavements

5-1 Introduction

Roadway and structure pavement drainage should be considered early in a project design, while the roadway geometry is still being developed since the hydraulic capacity of gutters and inlets is determined by the longitudinal slope and superelevation of the pavement. The imperviousness of the roadway pavement will result in significant runoff from any rainfall event. To ensure safety to the traveling public, careful consideration must be given to removing the runoff from the roadway through structure pavement drainage facilities.

This chapter provides specific guidance on designing the drainage of highway pavements, including assessing site hydrology (Section 5-2), methods for draining highways (Section 5-3), gutter flow and determining inlet spacing (Section 5-4), and drainage structures and grate types and considerations (Section 5-5). It concludes with a brief discussion of hydroplaning and hydrodynamic drag (Section 5-6).

The flatter the longitudinal profile, the wider the shoulders need to be to accommodate increased spread width. However, for narrow shoulders, superelevation and/or widening transitions can create a gutter profile far different from the centerline profile. The PEO must carefully examine the geometric profile of the gutter to eliminate standing water created by these transitions. These areas should be identified and eliminated. This generally requires geometric changes stressing the need for early consideration of drainage.

Improperly placed superelevation transitions can cause serious problems, especially on bridges. Inlets or other means must pick up gutter flow before the flow crosses to the other side of the pavement. The collection of crossover flow on bridges is complex as effective drain inlets are difficult to place within structure reinforcement. Bridges over waterways and wetlands pose water quality issues and downspouts are not allowed. Also, bridge drain downspouts have a history of plugging problems and are an objectionable aesthetic impact on the structure.

Eliminating inlets on bridges can usually be accomplished by considering drainage early in the design phase. Superelevation transitions, zero gradients, and sag vertical curves should be avoided on bridges. Modern bridges generally use watertight expansion joints so that all surface water can drain off the structure and collect in inlets placed at the bridge ends. Drainage design at bridge ends requires a great deal of coordination between the RHE, PEO, and the HQ Hydraulics Section.

Multilane highways create unique drainage situations. The number of lanes draining in one direction should be considered during the design phase. Contact the RHE for additional design guidance.

5-2 Hydrology

The Rational Method is required for determining peak flow rates for pavement drainage. This method is easy to use for pavement drainage design because the time of concentration is generally taken as five minutes. For more discussion on the Rational Method, see Chapter 2. The design frequency and spread width are also significant variables in the design of pavement drainage.

5-3 Highway Drainage

When highways are built on fill, roadway drainage is usually allowed to flow uncollected to the sides of the roadway and over the side of the fill slope. Where erosion potential is low, this sheet flow of highway drainage does not present any problem to adjacent property owners nor is it a threat to the highway fill.

Curbs are often used before vegetation is established to prevent erosion. Once sufficient vegetation is present to resist erosion and treat runoff, consideration should be given to eliminating the curb in future overlay contracts. However, since most approach slabs include curbs, consideration must be given to dispersing the concentrated flow at the bridge ends before removing the curb. Possible solutions include discharging runoff to an inlet, maintaining curbing until runoff can be properly dispersed, or utilizing a fabric or filter blanket.

A ditch running parallel to the roadway generally drains highways in a cut section. These ditches are designed and sized in accordance with the criteria shown in Chapter 4.

5-3.1 *Downstream End of Bridge Drainage*

The downstream end of bridges need special attention. If a storm sewer inlet system is not provided, a channel should be provided at the end of any significant barrier, which collects and concentrates stormwater away from the bridge.

Bridges with approach slabs generally have an extruded curb beginning at the bridge end and terminating just past the approach slab. The concentrated flow shall be directed into a low-risk erosion area. Inlets shall be located a minimum of 10 feet downstream from an approach slab to avoid approach slab settlement, see Standard Plan B-95.41-00 for typical inlet.

Bridges without approach slabs and curbing pose yet another set of problems. The concentrated flow runs off the bridge slab and flows off the fill slope or drains behind the wing walls and can compromise the integrity of the structure's geotechnical design. To mitigate this effect, all runoff shall be directed away from wing walls, fill slopes, and embankments, so that no material is susceptible to erosion. Bridge drains are designed to reduce the amount of concentrated flows off a structure; however, bridge drains tend to get blocked or clogged from roadside debris during normal use. This clogging creates an excess of concentrated flow off the structure, which must be mitigated to prevent subgrade erosion.

5-3.2 *Slotted Drains and Trench Systems*

Slotted drains and trench systems shall not be used for highway drainage.

5-3.3 *Drop Inlets*

Drop inlets shall not be used for pavement drainage.

5-4 Gutter Flow and Inlet Spacing

When stormwater is collected and carried along the roadside in a gutter, or next to a curb or barrier, the allowable top width of the flow prism (Zd) is dependent on the road classification, as noted in Figure 5-1.

For Design-Bid-Build projects, the PEO shall perform a gutter flow analysis for each construction staging plan of the project using the same allowable spread design criteria in Figure 5-1. Not meeting the criteria in Figure 5-1 is not considered a HM deviation. The purpose of the required analysis is to identify areas of ponding water for the Contractor to be aware of during the construction portion of the project. The gutter spread analysis shall be placed in the TESC Plan, Abbreviated TESC Plan, or Region equivalent document and shall have concurrence from the region hydraulics engineer.

For Design-Build projects, the Design-Builder shall perform a gutter flow analysis for each construction staging plan of the project using the same allowable spread design criteria in Figure 5-1. Not meeting the criteria in Figure 5-1 is not considered a HM deviation. The purpose of the required analysis is to identify areas of ponding water for the Design-Builder to be aware of during the construction of the project and for the Design-Builder to manage the risk accordingly. The gutter spread analysis shall be placed in the TESC Plan, Abbreviated TESC Plan, or Region equivalent document and shall have concurrence from the region hydraulics engineer.

WSDOT uses gutter flow capacity and inlet spacing (on continuous grades and at sumps) equations from the FHWA's HEC-22. WSDOT gutter flow calculations generally assume a uniform gutter section per HEC-22. The following specific sections of HEC-22 are used for gutter flow capacity and inlet spacing:

- 4.3.4 Flow in Sag Vertical Curves
- 4-4 Drainage Inlet Design
- 4-4.4 Interception Capacity of Inlets on Grade
- 4-4.5 Interception Capacity of Inlets in Sag Locations
- 4-4.6.2 Inlet Spacing on Continuous Grades
- 4-4.6.3 Flanking Inlets

Figure 5-1 Design Frequency and Allowable Spread

Road Classification		Design Frequency (years)	Allowable Spread (Z_d)
Interstate, Principal, Minor Arterial, or Divided	<45 mph	10	Shoulder + 2 feet(1)
	≥45 mph	10	Shoulder
	Sag Point	50	Shoulder + 2 feet(1)
Collector and Local Streets	<45 mph	10	Shoulder + one-half/driving lane(2)
	≥45 mph	10	Shoulder
	Sag Point	50	One-half driving lane(2)
Roundabouts	All design speeds	10	Maintain at least 10 feet of driving lane that is free of water
Restricted Turning Lanes	With STOP sign or signalized intersections	10	Shoulder + one-half driving lane
	All other	10	Shoulder
Ferry Terminals	All	10	Driving lane

Notes:

mph = miles per hour

(1)When the lane adjacent to the shoulder is less than 12 feet, there shall be a minimum of 10 feet that is free of water.

(2)In addition to the allowable spread requirement, the depth of flow shall not exceed 0.12 feet at the fog line.

5-4.1 *Capacity of Inlets on a Continuous Grade*

The flow that is not intercepted by an inlet on a continuous grade and continuous run of curb and gutter is considered bypass flow and should be added to the flow traveling toward the next inlet located downstream. The last inlet on a continuous run of curb (that is not a sag or flanking inlet) is permitted to bypass a maximum of 0.10 cfs for the 10-year MRI storm. The bypass flow rate of 0.1 cfs will not usually cause erosion or hydroplaning problems. The PEO shall analyze the spread width of flow after the last inlet on a continuous run of curb until the curb ends or the curb enters into a sump. The spread width analysis shall end at the 50-year water surface elevation determined in the sag analysis. The spread width shall be compliant with Figure 5-1.

In urban situations, with much lower speeds than noted in Figure 5-1, it may not be feasible to use the allowable spread recommended in the *Hydraulics Manual*. In this situation, the PEO should first consider innovative solutions such as increasing the slope of the gutter (from 2 to 5 percent, for example), depressing the inlet, or using a combination curb opening and grate inlet. If it is still not possible to meet the allowable spread in Figure 5-1, the PEO should consider the safety of the intersection, how icing and hydroplaning could affect a driver at this location, and how quickly ponding from the rainfall event will shed off the roadway. The PEO should work with the RHE and Traffic Engineer to develop a solution that best suits the project location and keeps the roadway safe. If, after considering all possible scenarios, it is determined that the spread of runoff is not safe at this location then more drastic measures such as revising the project scope or seeking more funding may be necessary.

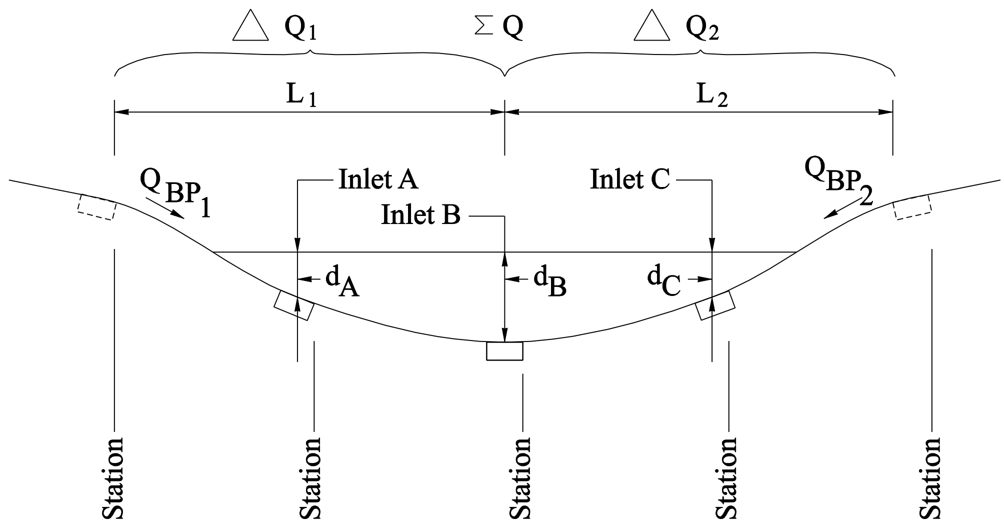
In addition to the requirements above, in areas where a superelevation transition causes a crossover of gutter flow, the amount of flow calculated at the point of zero superelevation shall be limited to 0.10 cfs. The PEO will find, by the time the roadway approaches the zero point, the calculated spread (Zd) will become very wide; because of this, the new inlet shall be placed upstream of the zero point. The flow width criteria will be exceeded at the crossover point, even when the flow is less than 0.10 cfs.

Roundabouts are typically designed to accommodate speed limits of 35 miles per hour or less, generally, the posted advisory speed limits are between 15 to 25 miles per hour. Potentially, runoff from a roundabout is diverted to multiple different directions and, if it is possible, runoff from the upstream roadway should be captured so that flow bypass should be 0.1 cfs or less flowing through the roundabout area. If runoff within a roundabout area is less than 0.1 cfs, no inlets would be necessary. If inlets are placed within a roundabout area, when maintenance is necessary, the roundabout may need to be closed. Curb openings could be used to alleviate ponding water at roundabouts. The inlet spacing spreadsheet may not be fully accurate to calculate the flow spread at roundabouts since runoff at a roundabout could flow off in multiple directions. The PEO should consult the RHE for help with analyzing spread widths inside of roundabouts.

5-4.2 *Capacity of Inlets at Sag Locations*

By definition, a sag is any portion of the roadway where the profile changes from a negative grade to a positive grade. Inlets at sag locations perform differently than inlets on a continuous grade and therefore require a different design criterion. Theoretically, inlets at sag locations may operate in one of two ways: (1) at low ponding depths, the inlet will operate as a weir or (2) high ponding depths (5-inch depth above the grated inlet and 1.4 times the grate opening height for combination inlets), the inlet will operate as an orifice. It is very rare that ponding on a roadway will become deep enough to force the inlet to operate as an orifice. As a result, this section will focus on inlets operating as a weir with flow spilling in from the three sides of the inlet that are exposed to the ponding.

Figure 5-2 Sag Analysis



Where:

Inlet B = sag inlet
Inlet A and Inlet C = flanking inlets
$d_A = d_C = 0.5d_B$

Inlets at sag locations can easily become plugged with debris; therefore, it is good engineering practice to provide some type of relief. This relief can be accomplished by locating flanking inlets, on either side of the sag inlet, so they will operate before water exceeds the allowable spread into the travel lane at the sag. Flanking inlets shall be located so the depth of water at the flanking inlets pond to half the allowable depth at the sag (or $0.5d_{B\ allowable}$); see Figure 5-2. Flanking inlets are only required when the sag is located in a depressed area and water has no outlet except through the system. A tall curb, traffic barrier, retaining wall, or other obstruction that prevents the runoff from flowing off of the traveled roadway, generally represents this condition since it contains this ponded area. However, if runoff is capable of overtopping the curb and flowing away from the roadway before exceeding the allowable sag limits noted in Figure 5-1, flanking inlets are not required. With this situation, there is a low potential for danger to the drivers of the roadway if the inlets do not function as designed. Before flanking inlets are removed in this situation, the PEO should consider the potential damage of water going over the curb. The PEO shall use the guidelines provided in this section for locating flanking inlets. If the PEO suspects flanking inlet are unnecessary, consult the RHE earlier in the design.

Any section of roadway located in a sag should be designed according to the criteria described below and further detailed in the WSDOT Sag Worksheet located on the HQ Hydraulics Section web page (www.wsdot.wa.gov/publications/fulltext/hydraulics/programs/sagworksheetud.xls).

Once an inlet has been placed in a sag location, the total actual flow to the inlet can be determined as shown below. Q_{Total} must be less than $Q_{allowable,}$ as described in Equation 5-1.

$$Q_{TOTAL} = Q_{BP1} + Q_{BP2} + \Delta Q_1 + \Delta Q_2 \quad (5\text{-}1)$$

Where:

$Q_{BP1\&2}$ = Bypass flow from the last inlet on either side of a continuous grade

$\Delta Q_{1\&2}$ = Runoff that is generated from last inlet on either side of the continuous grades; see Figure 5-2

The effective perimeter of the flanking and sag inlets can be determined using the length and widths for various grates provided in Figure 5-11. This would be the sum of the three sides of the inlet where flow spills in and where ponding would occur. Only the sides that receive gutter flow (see Figure 5-8) would be assumed to be 50 percent plugged (except for the Combination Inlet, Standard Plan B-25.20-02, which should be considered 0 percent plugged). This typically will be the grate widths (and not grate length) that are reduced by 50 percent. The total available perimeter that would receive flow is represented by Equation 5-2. This adjustment is in addition to reducing the perimeter to account for the obstruction caused by the bars in the grate. Figure 5-11 lists perimeters for various grates with reductions already made for bars.

$$P_n = L + 2*W/2 \quad (5\text{-}2)$$

Where:

P_n = Effective perimeter of the inlet "n" (sag or flanking inlet)

L = Length of the inlet "n" from Figure 5-2

W = Width of the inlet "n" from Figure 5-2

The allowable capacity of an inlet operating as a weir, that is the maximum $Q_{allowable}$, can be found depending on the inlet layout as described below:

When there is only a single inlet at the sag (no flanking inlets), Equation 5-3 should beused:

$$Q_{allowable} = C_W \times P \times d_{B\,allowable}^{1.5} \quad (5\text{-}3)$$

Where:

C_w = Weir coefficient, 3.0 for English Units

P = effective perimeter of the grate in feet

$d_{B\ allowable}$ = maximum depth of water at the sag inlet in feet

Flanking inlets shall be located laterally from the sag inlet at a distance equal to that required to produce a depth of $0.5d_{B\ allowable}$. $Q_{allowable}$ can be simplified to Equation 5-4 below. Equation 5-4 assumes all grates are the same size and are oriented the same (all rotated or not rotated):

$$\Sigma Q = C_W \times P \times \left[2(0.5d_B)^{1.5} + (d_B)^{1.5}\right] \quad (5\text{-}4)$$

Where:

d_B = depth of water at the sag inlet (ft)

In some applications, locating inlets so water ponds to $0.5d_{B\ allowable}$ is too long of a distance (generally in cases with long flat slopes). The PEO should instead calculate $Q_{allowable}$ using Equation 5-5 and check that the spread width of surface water does not exceed those noted in Figure 5-1.

$$Q_{allowable} = C_w P\left[d_A^{\ 1.5} + d_B^{\ 1.5} + d_C^{\ 1.5}\right] \tag{5-5}$$

Where:

d_N = depth of water at the flanking inlets and the sag (ft)

The actual depth of water over the sag inlet can be found with Equation 5-6 and must be less than $d_{B\ allowable}$. If, however, the inlets are not located at $0.5d_{B\ allowable}$, Equation 5-6 will need to be modified to reflect this.

$$d_B = \left[\frac{Q_{TOTAL}}{(C_{WA}P_A 0.3536 + C_{WB}P_B + C_{WC}P_C 0.3536)}\right]^{\frac{2}{3}} \tag{5-6}$$

Where:

Q_{Total} = Actual flow into the inlet in cfs

C_w = Weir coefficient, 3.0

P_N = Effective grate perimeter, in feet; see Figure 5-11

d_B = Actual depth of ponded water at the inlet in feet

After the analysis is completed, the PEO shall verify the allowable depth and allowable flow have not been exceeded ($Q_{allowable} > Q_{TOTAL}$ and $d_{B\ allowable} > d_B$). If both the allowable depth and allowable flow are greater than the actual, then the maximum allowable spread will not be exceeded and the design is acceptable. If the actual depth or flow is greater than the allowable, then the runoff will spread beyond the maximum limits and the design is not acceptable. In this case, the PEO shall add flanking inlets or use different inlets that have larger openings. Additional flanking inlets should be placed close to the sag inlet to increase the flow interception and reduce the flow into the sag.

5-5 Drainage Structures

There are many variables involved in determining the hydraulic capacity of an inlet structure including depth of flow, grade, superelevation, and placement. The depth of flow next to the curb is a major factor in the interception capacity of an inlet structure. Slight variations in grade or superelevation of the roadway can also have a large effect on flow patterns, and placement of an inlet can result in dramatic changes in its hydraulic capacity. These variables can be found by collecting the following information prior to starting an inlet design: plan sheets, road profiles, curb/barrier profiles, cross sections, superelevations, and contour maps.

Drainage structures should never be placed directly in the wheel path. While many are traffic rated and have lockdown grates, the constant pounding of traffic causes unnecessary stress and wear on the structure, frame, and grate. Inlets shall be installed at the curb/barrier face and at the proper elevation relative to the pavement. The structure offset shown in the plans shall be to the center of grate, not to the center of the structure, to ensure the grate is located along the curb face. There shall be no gap between the structure and the curb/barrier face as this would lead to other issues.

Scuppers in median barriers shall not be used in the following situations:

- Passing runoff from one side of a median barrier to a drainage structure or curb and gutter section on the other side (downstream) of the median barrier.
- Passing runoff through the median barrier so that the runoff continues to flow across highway lanes on the other side (downstream) of the median barrier.

For the above scenarios, flows shall be captured by placing inlets on each side of the median barrier as shown in Standard Plan B-95.20-01, allowing runoff to pass between the structures in a pipe.

Debris floating in the gutter tends to collect at the inlets, plugging part or all of the grate opening. Inlet locations on a continuous grade are calculated using the full width of the grate with no allowance needed for debris. Inlets located in a sag are analyzed with an allowance for debris blocking half of the grate. Areas with deciduous trees and large pedestrian populations are more prone to debris plugging. Bark from logging operations and agricultural areas are also known to cause debris problems. These areas may require additional maintenance.

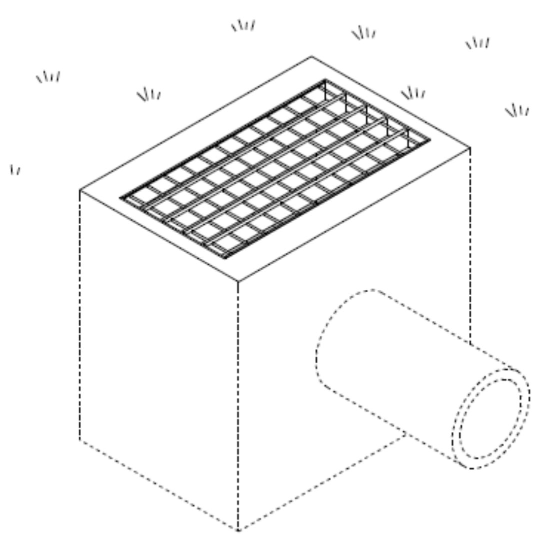
5-5.1 *Inlet Structure Types*

WSDOT uses grate inlets, catch basins, and manholes to capture runoff for WSDOT projects. Each inlet structure type has different variations and advantages for use in certain situations. On top of each inlet structure type is a grate that allows water to flow into the structure. This section will briefly describe each structure type.

5-5.1.1 Grate Inlet Type 1 Structure - Standard Plan B-35.20-00

Grate inlet Type 1 structures are cast-in-place and utilizes a sump by placing the outlet pipe's invert elevation higher than the bottom of the structure (Figure 5-3). This allows suspended sediment within the water to settle and reduce turbidity prior to entering the downstream stormwater system. Type 1 inlet structures require more construction because they are cast-in-place; however, this allows the PEO to tie into existing stormwater infrastructure without modifying the hydraulic gradient.

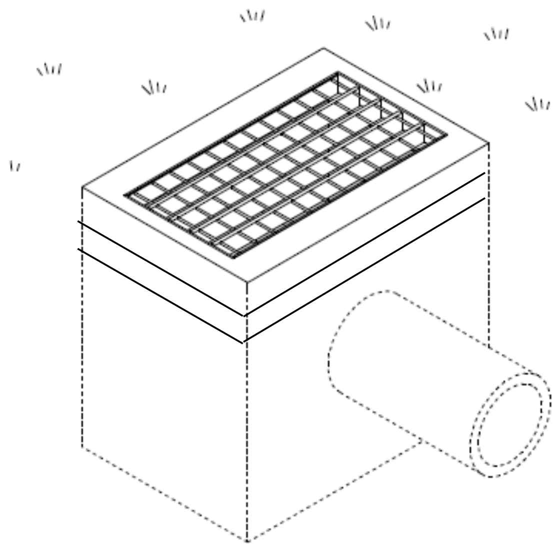
Figure 5-3 Grate Inlet Type 1 Structure



5-5.1.2 Grate Inlet Type 2 Inlet Structure - Standard Plan B-35.40-00

Grate inlet Type 2 structures are constructed using sections of pre-cast reinforced concrete (Figure 5-4). These pre-cast sections can be stacked to meet the required height, thus reducing the construction time and cost. This inlet structure is similar to the grate inlet Type 1 in that they both have an invert elevation higher than the structure bottom. This creates a sump that allows suspended sediment to settle prior to entering the downstream stormwater system. The grate inlet Type 2 should be used in areas where existing infrastructure is easy to tie into.

Figure 5-4 Grate Inlet Type 2 Structure



5-5.1.3 Catch Basins

Catch basins are designed to retain sediment and debris transported by stormwater into a storm sewer system. Catch basins include a sump for the collection of sediment and debris. Catch basin sumps require periodic cleaning to be effective and may become an odor and mosquito nuisance if not properly maintained. Catch basins are used to link long runs of storm sewer pipes and to help change directions of the storm sewer system. See the following:

- Standard Plan B-5.20-02 Catch Basin Type 1
- Standard Plan B-5.40-02 Catch Basin Type 1L
- Standard Plan B-5.60-02 Catch Basin Type 1P (for Parking Lot)
- Standard Plan B-10.20-02 Catch Basin Type 2
- Standard Plan B-10.40-01 Catch Basin Type 2 with Flow Restrictor
- Standard Plan B-10.70-00 Catch Basin T - PVC

5-5.1.4 Manholes

Similar to catch basins, manholes are to convey stormwater as a part of a storm sewer system. They are used to also change the direction of a storm sewer system. Manholes do not have a sump. They can have solid locking lids that block water from entering the manhole. They can also be configured to have a grate to allow water to flow into the manhole. See the following:

- Standard Plan B-15.20-01 Manhole Type 1
- Standard Plan B-15.40-01 Manhole Type 2
- Standard Plan B-15.60-02 Manhole Type 3

5-5.1.5 Concrete Inlet - Standard Plan B-25.60-02

A concrete inlet is used when a sump to catch sediments is not desired and the maximum inside pipe diameter is less than or equal to 15 inches.

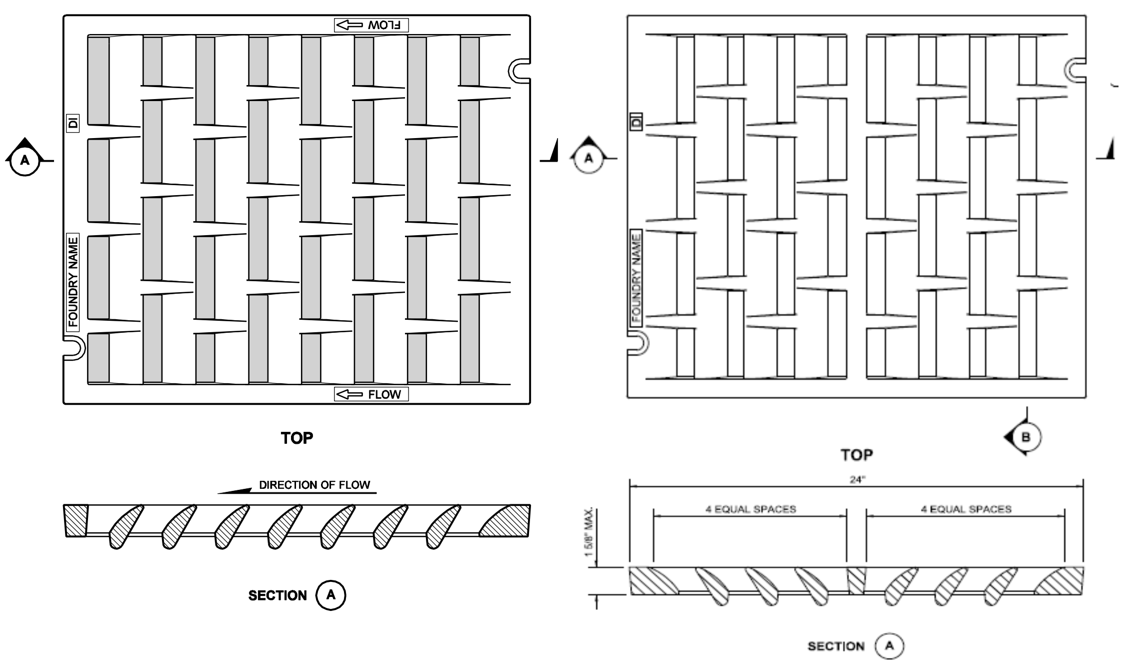
5-5.2 *Grate Types*

Grates are an essential component in ensuring the efficiency of a drainage system. The following grates (except the Rectangular Herringbone Grate) shall be used for new construction, where applicable.

5-5.2.1 Rectangular Bi-Directional Vaned Grate – Standard Plan B-30.40-03

At low velocities, the vaned grate (Figure 5-5) and the herringbone Rectangular Vaned Grate - Standard Plan B-30.30-03 and grate are equally efficient. At higher velocities—greater than 5 ft/s—a portion of the flow tends to skip over the herringbone grate whereas the vaned grate will capture a greater portion of this flow. The vaned grate also has a higher capacity for passing debris and shall be used in place of the herringbone grate in all new installations. Installation of the vaned grate is critical as the grate is directional. If installed backwards the interception capacity is severely limited.

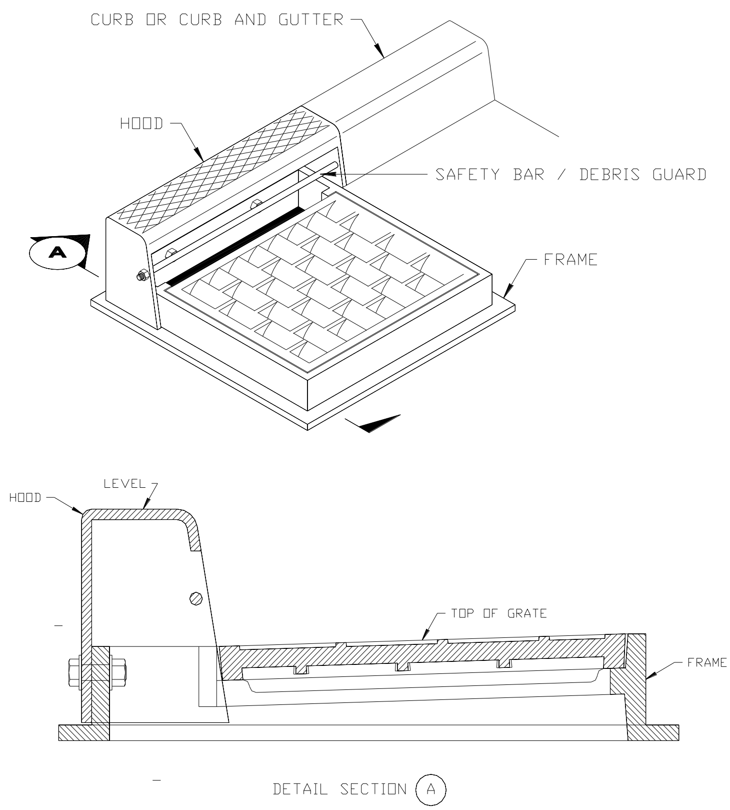
Figure 5-5 Rectangular Vaned Grate and Rectangular Bi-Directional Vaned Grate



5-5.2.2 Combinations Inlet - Standard Plan B-25.20-02

The combination inlet is a vaned grate on a catch basin with a hooded curb cut area (Figure 5-6). The vaned grate is debris efficient, and, if the grate does become clogged, the overflow goes into the hooded opening. These inlets are useful for sag condition installations, although they can also be effective on continuous grades. The interception capacity of a combination inlet is only slightly greater than with a grate alone. Therefore, the capacity is computed neglecting the curb opening and the PEO should follow the same analysis as for a vaned grate alone (see Standard Plan B-30.30-03).

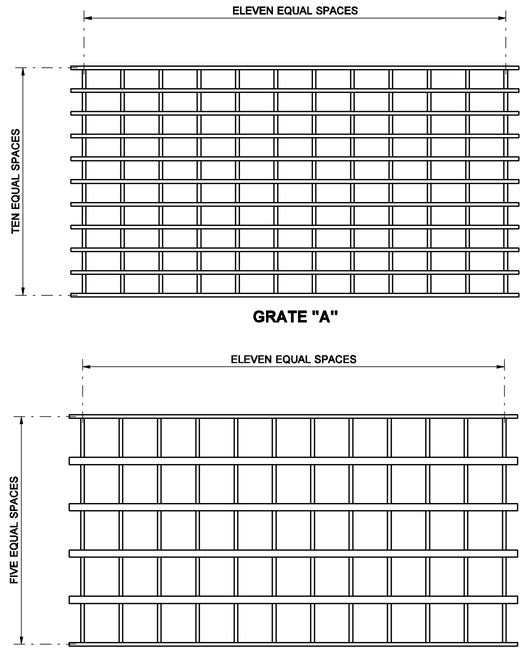
Figure 5-6 Section and Isometric View Combination Inlet Frame, Hood, and Vaned Grate



5-5.2.3 Welded Grates for Grate Inlet, Grate A and Grate B - Standard Plan B-40.20.00

Both welded grates (types A and B) have large openings that can compensate for debris problems (Figure 5-7); however, there are limitations in their usage. Due to structural failure of Grates A and B, neither of these grates can be installed in heavy traffic areas where wheel loads will pass directly over. Grate B has large openings and is useful in ditches or non-paved median locations, in areas where there is no pedestrian or bicycle traffic. Grate A can be used anywhere Grate B is used as well as at the curb line of a wide interstate shoulder. Grate A may occasionally be subject to low-speed traffic or parked on, but it cannot withstand repeated interstate loading or turning vehicles.

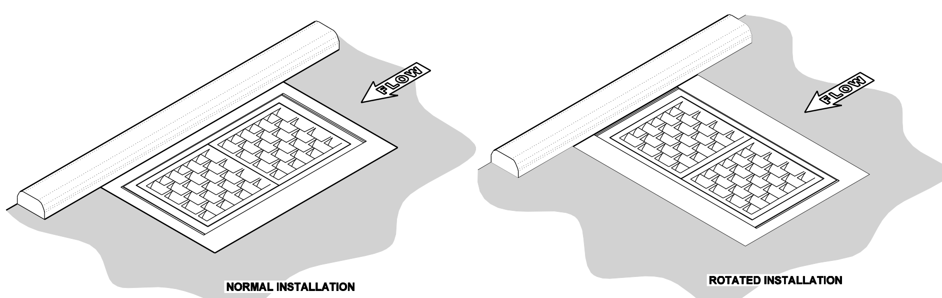
Figure 5-7 Grates A and B



5-5.2.4 Frame and Dual Vaned Grates for Grate Inlet - Standard Plan B-40.40-02

Standard Plan B-40.40-02 has been tested in H-25 loading and was determined compatible with heavy traffic installations. This frame and double-vaned grate should be installed in a Unit H on top of a Grate Inlet Type 2 (Figure 5-8). The frame and vaned grates may be used in either new construction or retrofit situations. When used in areas of highway speeds, lockdown grates shall be specified. This grate can also be rotated 90 degrees to increase the flow interception capacity.

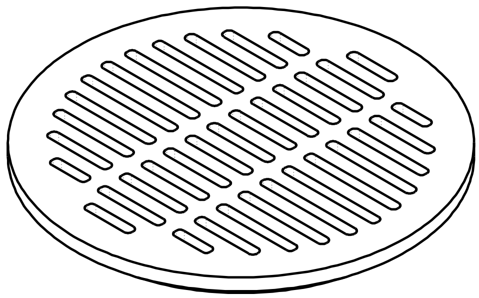
Figure 5-8 Frame and Vaned Grates for Installation on Grate Inlet



5-5.2.5 Circular Grate or Standard Plan B-30.80-01

Circular grates are intended for use with dry wells, see Standard Plan B-20.20-02 and B-20.60-03 for details (Figure 5-9). Install with circular frames (rings) as detailed in Standard Plan B-30.70-04.

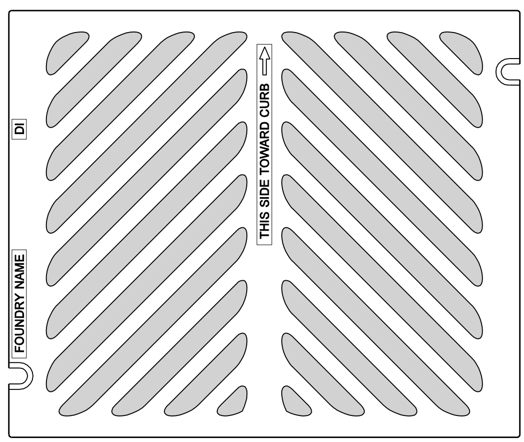
Figure 5-9 Circular Grate



5-5.2.6 Rectangular Herringbone Grate - Standard Plan B-30.50-03

The HQ Hydraulics Section no longer allows herringbone grates (Figure 5-10) to be used on WSDOT projects. Historically, use of the vaned grate was limited due to cost considerations. The cost difference now is minimal, the vaned grate is bicycle safe and is hydraulically superior under most conditions. Herringbone grates shall not be used for new construction.

Figure 5-10 Herringbone Pattern



Grate inlet properties are summarized in Figure 5-11.

Figure 5-11 Properties of Grate Inlets

Standard Plan	Description	Continuous Grade[1]		Sag Location[2] Perimeter Flows as Weir	
		Grate Width (feet)	Grate Length (feet)	Width (feet)	Length (feet)
B-30.50-03[3]	Rectangular Herringbone Grate	1.67	2.0	0.69	0.78
B-30.30-03 or B-30.40-03[4]	Vaned Grate for Catch Basin and Inlet	1.67	2.0	1.31	1.25
B-25.20-02[2]	Combination Inlet	1.67	2.0	1.31	1.25
B-40.20-00	Grate Inlet Type 1 (Grate A or B[5])	2.01 3.89[6]	3.89 2.01[6]	1.67 3.52	3.52 1.67
B-30.80-01	Circular Grate	1.52		2.55[7]	
B-40.40-02	Frame and Vaned (Single or Dual) Grates for Grate Inlet Type 2	1.75[8] 3.52[6]	3.52[8] 1.75[6]	1.29 2.58[6]	2.58 1.29[6]

Notes:

(1)Inlet widths on a continuous grade are not reduced for bar area or for debris accumulation.

(2)The perimeters and areas in this portion of the table have already been reduced for bar area. These values shall be cut in half when used in a sag location as described in Section 5-5.2, except for the Combination Inlet, Standard Plan.

(3)Shown for informational purposes only (see Section 5-5).

(4)For sag conditions, combination inlets shall use a bidirectional vaned grate (as shown in Standard Plan).

(5)Type B grate shall not to be used in areas of pedestrian or vehicular traffic (see Section 5-5 for further discussion).

(6)Rotated installation (see *Standard Plans*).

(7)Only the perimeter value has been provided for use with weir equations.

(8)Normal Installation (see *Standard Plans*).

5-6 Hydroplaning and Hydrodynamic Drag

FHWA's HEC-22 provides an in-depth discussion on the factors that contribute to hydroplaning on roadways and offers rules of thumb to help reduce hydroplaning.

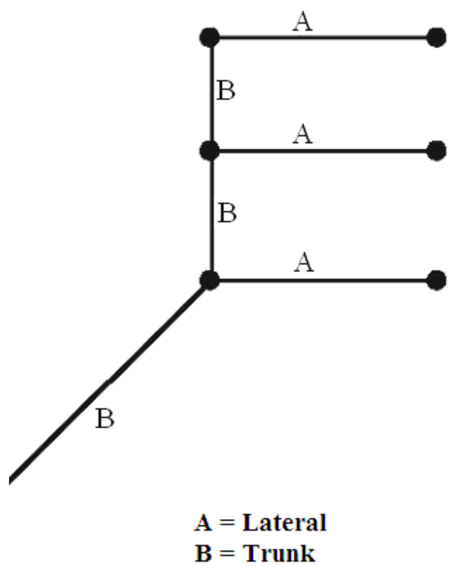
Chapter 6 Storm Sewer

6-1 Introduction

A storm sewer is a pipe network that conveys surface drainage from a surface inlet or through a manhole, to an outlet location. This chapter discusses the criteria for designing storm sewers (Section 6-2); the data and process required to document the design (Section 6-3); methods, tools, and concepts to help develop designs (Section 6-4 through Section 6-6); and pipe materials used for storm sewers (Section 6-8). It also includes a discussion of drywells (Section 6-7) and subsurface drainage (Section 6-9)

Storm sewers are defined as closed-pipe networks connecting two or more inlets, see Figure 6-1. Storm sewer networks typically consist of laterals that discharge into a trunk line. The trunk line then receives the discharge and conveys it to an outlet location.

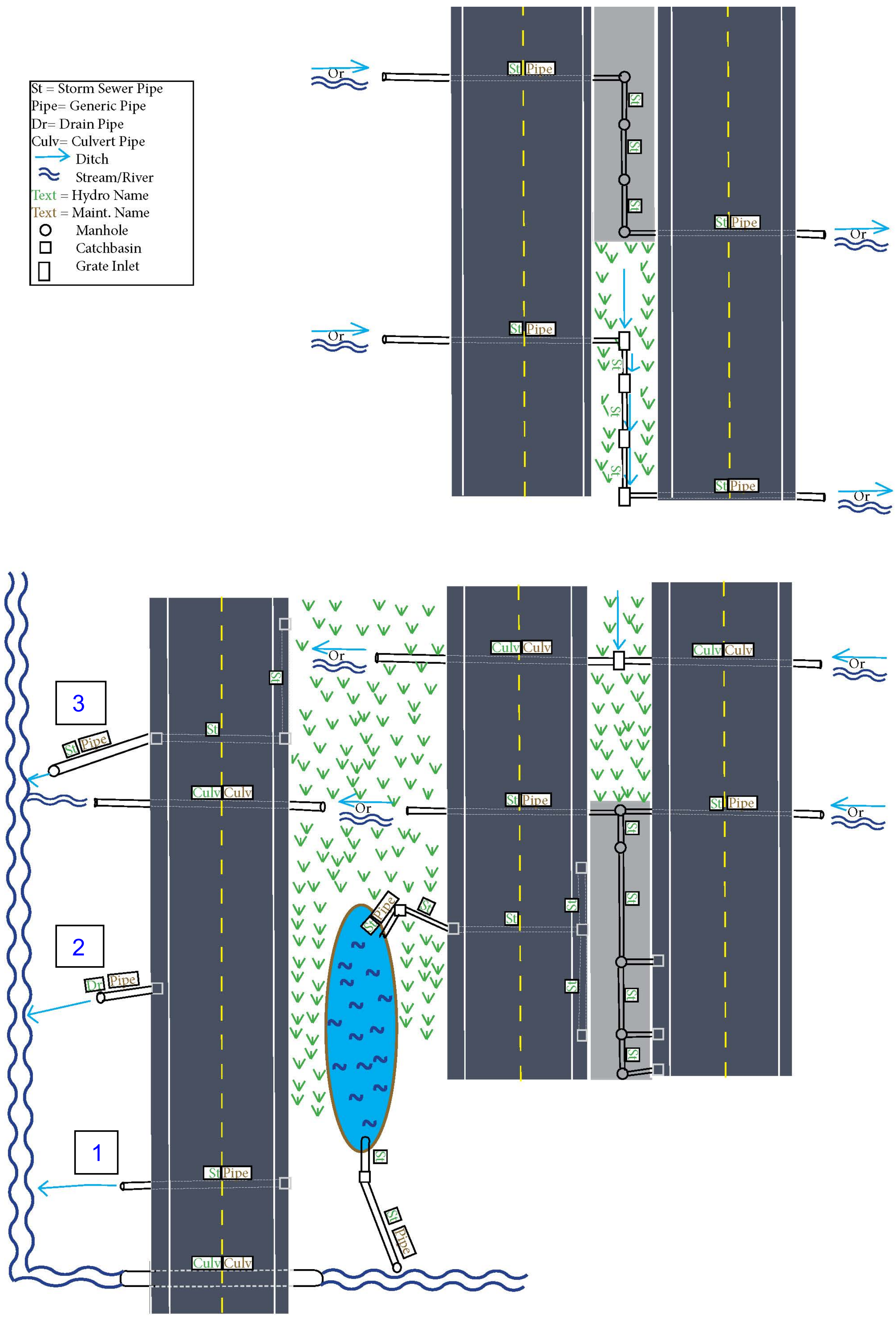
Figure 6-1 Storm Sewer Structure



While this is a typical configuration, there are other configurations that do not meet the storm sewer definition, as shown in Figure 6-2. Configurations with only one inlet and one or two pipes shall be classified as a culvert on the plan sheets. The configurations in Figure 6-2 shall be designed as follows:

1. Storm sewer that does not require pressure testing.
2. Lateral that does not require pressure testing.
3. Storm sewer that does require pressure testing.

Figure 6-2 Storm Sewer Configurations



All storm sewer design shall be based on the design criteria outlined in Section 6-2, which includes limits for runoff rates, pipe flow capacity, hydraulic grade line (HGL), soil characteristics, pipe strength, potential construction problems, and potential runoff treatment issues. Runoff is typically calculated using the Rational Method or the SBUH Method; see Chapters 1 and 2 for further discussion. Based on the runoff rate, the pipe velocity is calculated using Manning's equation, which relates the pipe capacity to the pipe diameter, slope, and roughness. The preference is to have the HGL below the pipe crown. After sizing the pipe, verify that the HGL is below all rim elevations. A storm sewer design may be performed by hand calculations, as described in Section 6-4, or by computer program, as described in Section 6-5.

Additional guidance on pipe sizing with respect to climate resiliency will be provided in future revisions to the *Hydraulics Manual*.

6-2 Design Criteria

Along with determining the required pipe sizes for flow conveyance and the HGL, storm sewer system design should consider the following guidelines:

1. **Soil Conditions**: Soil with adequate bearing capacity must be present to interact with the pipes and support the load imparted by them. Surface and subsurface drainage must be provided to assure stable soil conditions. Soil resistivity and pH must also be known so the proper pipe material will be used. Section 8-5 contains further guidance.
2. **Structure Spacing and Capacity**: Design guidelines for inlet spacing and capacity are detailed in Chapter 5. Structures (catch basins, grate inlets, and manholes) should be placed at all breaks in grade and horizontal alignment. The desired pipe run length between structures is 150 feet and shall not exceed 300 feet for pipes less than 48 inches in diameter and 500 feet for pipes greater than 48 inches in diameter. When grades are flat, pipes are small, or there could be debris issues, the PEO should reduce the spacing. The RHE and local WSDOT Maintenance Office shall be consulted for final determination on maximum spacing requirements. For minimum clearance between culverts and utilities, PEOs should consult the RHE for guidance.
3. **Existing Systems**: Criteria for repair and/or replacement of existing systems be provided in future revisions to the *Hydraulics Manual*. Until then, contact the RHE for guidance when working with existing systems, and refer to Chapter 8 for guidance on trenchless pipe repair methods.
4. **Future Expansion**: If a storm sewer system may be expanded in the future, provision for the expansion shall be incorporated into the current design. Additionally, prior to expanding an existing system, the existing system shall be inspected for structural integrity and hydraulic capacity using the Rational Method.
5. **Velocity**: The design velocity for storm sewers shall be between 3 to 10 feet per second. This velocity is calculated using Manning's equation, under full flow conditions even if the pipe is only flowing partially full with the design storm. The minimum slope required to achieve these velocities is summarized in Figure 6-3.

   When flows drop below 3 feet per second, pipes can clog due to siltation. Flows can be designed to as low as 2.5 feet per second with justification in the hydraulic report. As the flow approaches (and exceeds) 10 feet per second, PEOs should consult the RHE for abrasion design guidance.

Figure 6-3 Minimum Storm Sewer Slopes

Pipe Diameter (inches)	Minimum Slope (feet/foot)	
n=0.013	2.5 feet per second	3 feet per second
12	0.003	0.0044
15	0.0023	0.0032
18	0.0018	0.0025
24	0.0012	0.0017

6. **Pipe Elevations at Structures**: Pipe crowns differing in diameter, branch, or trunk lines shall be at the same elevation when entering structures. For pipes of the same diameter where a lateral is placed so the flow is directed against the main flow through the manhole or catch basin, the lateral invert must be raised to match the crown of the inlet pipe. Matching the crown elevation of the pipes will prevent backflow in the smaller pipe. (A crown is defined as the highest point of the internal surface of the transverse cross section of a pipe.) It is also generally acceptable to have the crown elevation of the upstream pipe in the structure be higher than the crown elevation of the downstream pipe in the same structure.

7. **Minimum Pipe Diameter:** The minimum pipe diameter shall be 12 inches.

8. **Structure Constraints:** During the storm sewer layout design, PEOs should also consider the physical constraints of the structure. Specifically:
   - **Diameter** – Verify the maximum allowable pipe diameter into a drainage structure prior to design. *Standard Plans* for drainage structures have pipe allowances clearly stated in tables for various pipe materials.
   - **Angle** – Verify the layout is constructible with respect to the angle between pipes entering or exiting a structure before finalizing the storm sewer layout. That is, to maintain structural integrity there are minimum clearance requirements that must be met depending on the pipe diameter. PEOs can verify the minimum pipe angle with the Pipe Angle Calculation Worksheet.

9. **Pipe Material**: Storm sewers shall be designed to include all Schedule A pipe options, unless specific site constraints limit options. (See (Section 6-8 for further discussion.)

10. **Increase in Profile Grade:** In cases where the roadway or ground profile grades increase downstream along a storm sewer, a smaller diameter pipe may be sufficient to carry the flow at the steeper grade. However, due to maintenance concerns, WSDOT design practices do not allow pipe diameters to decrease in downstream runs. Consideration could be given in running the entire length of pipe at a grade steep enough to allow use of the smaller diameter pipe. Although this will necessitate deeper trenches, the trenches will be narrower for the smaller pipe and therefore the excavation may not substantially increase. A cost analysis is required to determine whether the savings in pipe costs will offset the cost of any extra structure excavation.

11. **Discharge Location:** A discharge location is where stormwater from WSDOT highways is conveyed off of the ROW by pipe, ditch, or other man-made conveyance. Additional considerations for discharge locations include energy dissipators and tidal gates. Energy dissipators prevent erosion at the discharge location; for design guidance see Chapter 3. Installation of tide gates may be necessary when the discharge location is in a tidal area; consult the RHE for further guidance.

12. **Location:** Wide medians usually offer the most desirable storm sewer location. In the absence of a wide median, a location beyond the pavement edge on state ROW or easement is preferable. It is recommended when a storm sewer is placed beyond the pavement edge that a one-trunk system with connecting laterals be used instead of running two separate trunk lines down each side of the road.
13. **Confined Space and Structure Depths:** PEOs shall consult the local WSDOT Maintenance Office and RHE to ensure that structures can be adequately maintained.

Additional guidance will be provided in future revisions to the *Hydraulics Manual*.

6-3 Data for Hydraulic Reports

Storm sewer system design requires that data be collected and documented in an organized fashion. Hydraulic reports shall include all related calculations, whether performed by hand or computer. See Appendix 1B for guidelines on what information should be submitted and recommendations on how it should be organized.

6-4 Storm Sewer Design - Manual Calculations

Storm sewer design is accomplished in two parts: determine the pipe capacity and then evaluate the HGL. See the Storm Sewer Design spreadsheet to determine the pipe capacity of the storm sewer system, available at:
www.wsdot.wa.gov/Design/Hydraulics/ProgramDownloads.htm.

The Storm Sewer Design spreadsheet does not currently calculate the HGL at each structure so the PEO must calculate them using hand calculations, per Section 6-6 and HEC-22, or use computer software per Section 6-5. Consult with the RHE for more guidance on how to do these HGL calculations.

6-5 Storm Sewer Design – Computer Analysis

There are several commercially available computer programs for storm sewer design. Refer to Chapter 1 for WSDOT-approved software.

6-6 Hydraulic Grade Line

The HGL shall be designed so there is air space between the top of water and the inside of the pipe. In this condition, the flow is operating as gravity flow, and the HGL is the water surface elevation traveling through the storm sewer system. If the HGL becomes higher than the crown elevation of the pipe, the system will start to operate under pressure flow. If the system is operating under pressure flow, the water surface elevation in the catch basin/manhole needs to be calculated to verify the water surface elevation is below the rim (top) elevation. When the water surface elevation exceeds the rim elevation, water will discharge through the inlet and cause severe traffic safety problems. Fortunately, if the storm sewer pipes were designed as discussed in the previous sections, then the HGL will only become higher than the catch basin/manhole rim elevation when energy losses become significant or if the cover over a storm sewer is low (less than 5 feet).

Regardless of the design conditions, the HGL should be evaluated when energy loss becomes significant. Possible significant energy loss situations include high flow velocities through the system (greater than 6.6 ft/s), pipes installed under low cover at flat gradients, inlet and outlet pipes forming a sharp angle at structures, and multiple flows entering a structure.

The HGL can only be calculated after the storm sewer system has been designed. When computer models are used to determine the storm sewer capacity, the model will generally evaluate the HGL. The remainder of this section provides the details for how the analysis is performed.

The HGL is calculated beginning at the most downstream point of the storm sewer outlet and ending at the most upstream point. To start the analysis, the water surface elevation at the storm sewer outlet must be known. Refer to Chapter 3 for an explanation on calculating water surface elevations at the downstream end of a pipe (the tailwater is calculated the same for storm sewer outlet and culverts). Once the tailwater/pond elevation is known, the energy loss (usually called head loss) from friction is calculated for the most downstream run of pipe and the applicable minor losses are calculated for the first structure upstream of the storm sewer outlet. Head losses are added to the water surface elevation at the storm sewer outlet to obtain the water surface elevation at the first upstream structure (also the HGL at that structure, assuming velocities are zero in the structure). The head losses are then calculated for the next upstream run of pipe and structure and are added to the water surface elevation of the first structure to obtain the water surface elevation of the second upstream structure.

This process is repeated until the HGL has been computed for each structure. The flow in most storm sewers is subcritical; however, if any pipe is flowing supercritical, the HGL calculations are restarted at the structure on the upstream end of the pipe flowing supercritical. (Chapter 4 contains an explanation of subcritical and supercritical flow.)

The HGL calculation process is represented in Equation 6-1:

$$\mathrm{WSEL}_{J1} = \mathrm{WSEL}_{OUTFALL} + H_{f1} + H_{e1} + H_{ex1} + H_{b1} + H_{m1} \quad (6\text{-}1)$$

$$\mathrm{WSEL}_{J2} = \mathrm{WSEL}_{J1} + H_{f2} + H_{e2} + H_{ex2} + H_{b2} + H_{m2}$$

$$\mathrm{WSEL}_{Jn+1} = \mathrm{WSEL}_{Jn} + H_{fn+1} + H_{en+1} + H_{exn+1} + H_{bn+1} + H_{mn+1}$$

Where:

WSEL = Water surface elevation at structure noted

H_f = Friction loss in pipe noted

H_e = Entrance head loss at structure noted

H_{ex} = Exit head loss at structure noted

H_b = Bend head loss at structure noted

H_m = Multiple flow head loss at structure noted

If the HGL is lower than the rim elevation of the manhole or catch basin, the design is acceptable. If the HGL is higher than the rim elevation, flow will exit the storm sewer and the design is unacceptable. The most common way to lower the HGL below the rim elevation is to lower the pipe inverts for one or more storm sewer runs or increase the pipe diameter. The HGL shall be designed so that regular maintenance inspections may be achieved without pumping.

Head loss due to friction is a result of the kinetic energy lost as the flow passes through the pipe. The rougher the pipe surface is, the greater the head loss is going to be. Refer to HEC-22 to calculate head loss from friction. Note that for all storm sewer pipes 24 inches or less in diameter, Manning's n shall be 0.013. For all other pipes, refer to Appendix 4A for appropriate Manning's n values.

6-7 Drywells

Prior to specifying a drywell in a design, PEOs shall consult the *Highway Runoff Manual* for additional guidance and design criteria. Drywells are considered Underground Injection Control Wells and are required to be registered with Ecology per WAC 173-218. Refer to the *Highway Runoff Manual*. Additionally, stormwater must be treated prior to discharging into a drywell using a BMP described in the *Highway Runoff Manual*. Finally, all drywells shall be sized following the design criteria outlined in the *Highway Runoff Manual*.

6-8 Pipe Materials for Storm Sewers

When designing a storm sewer network, the PEO shall review Chapter 8 (for Pipe Materials) and the list of acceptable pipe material (schedule pipe) in the *Standard Specifications*. Storm sewer pipe is subject to some use restrictions, which are detailed in Chapter 8 (Storm Sewer Pipe).

Pipe flow capacity depends on the roughness coefficient, which is a function of pipe material and manufacturing method. Fortunately, most storm sewer pipes are 24-inch-diameter or less and studies have shown that most common schedule pipe materials of this size range have a similar roughness coefficient. For calculations, the PEO shall use a roughness coefficient of 0.013 when all 24-inch-diameter schedule pipes and smaller are acceptable. For larger diameter pipes, the PEO shall calculate the required pipe size using the largest Manning's roughness coefficient for all the acceptable schedule pipe values in Appendix 4A. In the event a single pipe alternative has been selected, the PEO shall design the required pipe size using the applicable Manning's roughness coefficient for that material listed in Appendix 4A.

In estimating the quantity of structural excavation for design purposes at any location where alternate pipes are involved, estimate the quantity of structural excavation based on concrete pipe since it has the largest outside diameter.

6-9 Subsurface Drainage

Subsurface drainage is provided for control of groundwater encountered at highway locations. Groundwater, as distinguished from capillary water, is free water occurring in a zone of saturation below the ground surface. The subsurface discharge depends on the effective hydraulic head and on the permeability, depth, slope, thickness, and extent of the aquifer.

The solution of subsurface drainage problems often calls for specialized knowledge of geology and the application of soil mechanics. The PEO should work directly with the RHE as subsurface conditions are determined and recommendations are made for design in the soils report.

Subsurface drainage can be intercepted with underdrain pipe, which is sized by similar methods used to design storm sewer pipe. When an underdrain is installed for seepage control in cuts or side hills or lowering the groundwater table for proper subgrade drainage, the design method used to size storm sewers should be followed. The only difference is that the flow used for the calculations is the predicted infiltration from groundwater into the system instead of flow entering the system from roadway drainage. When subsurface drainage is connected to a storm sewer system, the invert of the underdrain pipe shall be placed above the operating water level in the storm sewer. This is to prevent flooding of the underdrain system, which would defeat its purpose. Additional guidance will be provided in future revisions to the *Hydraulics Manual*.

Chapter 7 Fish Passage

7-1 Introduction

This chapter covers the design requirements for water crossings on state highways over fish bearing waters. See Chapter 3 for the design of non-fish bearing culverts. Most rivers and creeks in Washington State contain one or more species of fish during all or part of the year. This chapter has been updated to reflect the requirements for fish passage projects on WSDOT highways from current WAC Hydraulic Code Rules; the 2017 USACE, Seattle District, Nationwide Permit Regional Conditions; and the 2013 Federal Court Injunction for Fish Passage. This chapter is specific to WSDOT projects. For non-WSDOT projects, it is up to the project owner to determine whether the guidance in this chapter is followed or other guidance is followed to obtain project permits and follow state law. WSDOT is actively monitoring completed fish passage projects and will update this chapter as new information becomes available. See Section 7-7 for more information.

All fish-bearing water crossings within Washington State must meet the requirements of WAC's Hydraulic Code Rules (apps.leg.wa.gov/wac/default.aspx?cite=220-660), and the requirements of the *Hydraulics Manual*, unless a deviation is approved by the HQ Hydraulics section. In Water Resource Inventory Areas (WRIAs) 1 through 23, the design must also meet the requirements of the 2013 Federal Court Injunction for Fish Passage. This chapter uses the WDFW's 2013 Water Crossing Design Guidelines (WCDG) as reference. Other published manuals and guidelines may be used with the approval of the HQ Hydraulics Section and permitting agencies. A list of approved manuals and guidelines can be found on the WDFW Fish Passage Program website (https://wdfw.wa.gov/conservation/habitat/fish_passage/guidance_standards.html).

New bridges and fish-bearing culverts must be designed to meet current fish passage standards and WAC to ensure they do not hinder fish use or migration. WAC requires a person to design water-crossing structures in fish-bearing streams to allow fish to move freely through them at all flows that fish are expected to move.

WSDOT and WDFW have cooperated in a Fish Passage Barrier Removal Program since 1991. PEOs can check the fish barrier database (www.wsdot.wa.gov/Projects/FishPassage/default.htm) or contact the HQ Environmental Services Office biology branch to determine whether the project has any fish barriers within its limits and whether or not the crossing will need to be included as part of the project. All water crossings over fish-bearing waters shall be designed by the HQ Hydraulics Section or by an individual approved by the HQ Hydraulics Section (see Chapter 1).

Section 7-2 discusses requirements for assessing and documenting existing conditions to design a successful and fish-passable water crossing. Section 7-3 provides a discussion of hydraulic analyses required for the design, and Sections 7-4 and 7-5 discuss the design process, considerations, and criteria. Section 7-6 provides guidance on temporary diversions, Section 7-7 describes the WSDOT monitoring process, and Section 7-8 concludes the chapter with a discussion of additional resources.

This chapter uses the term Stream Designer(s) to denote work that either the HQ Hydraulics Section or the individual approved by the HQ Hydraulics Section performs and to separate that work from the work that the PEO would do in the rest of the manual. This chapter assumes the Stream Designer has knowledge of WAC, WDFW's 2013 WCDG, and hydrology and river hydraulics, and, as a result, does not cover every topic in thorough detail. This chapter will outline the process that the HQ Hydraulics Section follows in designing a stream crossing, and what is expected on WSDOT projects. These designs require a specialty report. Additional requirements about specialty reports can be found in Chapter 1. The template used by WSDOT will be provided in a future update.

7-2 Existing Conditions

The first step to designing a water crossing is understanding the behavior of the existing system. A thorough investigation of the site and adjacent stream reach, its history, and any known problems should be performed prior to the field visit and confirmed during the field visit. Prior to the first field visit, the Stream Designer(s) should complete the following:

- Determine whether the project is within a FEMA-mapped floodplain
- Evaluate the watershed conditions/landcover (past, current, and future)
- Investigate the type of soils that are in the basin
- Look at historic aerial photographs, if available, for evidence of lateral migration, avulsion, debris flows, sediment pulses, LWM interactions, significant erosion, etc.
- Discuss site history with WSDOT area maintenance
- Review any available survey data and available historical as-builts
- Confirm pre-field visit investigations and conclusions or document differences
- Review any available watershed studies, watershed analyses, hydrology/drainage studies, reach assessments, sediment budget, and transport investigations, etc.

Through site visits, the Stream Designer will perform the following:

- Determine the reference reach
- Measure bankfull width (BFW)
- Determine sediment size using either a Wolman Pebble Count or a grab sample (as appropriate)
- Investigate channel geometry
- Note any channel-forming features
- Note the presence and function of LWM
- Note the presence and function of large cobbles or boulders

Multiple site visits may be required, both before and after survey has taken place, to ensure all the necessary features were surveyed. The Stream Designer will benefit by reviewing the survey request in the field with the survey crew. The information listed above shall be photographed or otherwise recorded for report documentation and design discussions. The Stream Designer shall coordinate with the PEO for the attendance of the resource agencies and interested Tribes during the reference reach selection and BFW determination.

7-2.1 *Reference Reach*

The following process outlines several steps for locating the best reference reach possible while recognizing that many streams near roadway crossings are modified by human processes and thus are not perfect natural analogs. If a system is highly modified, contact the HQ Hydraulics Section for additional guidance. Figure 7-1 depicts a flow chart that describes the steps below.

7-2.1.1 Examine Adjacent Reaches

Examine the reaches with project stakeholders immediately upstream and downstream from the project reach and evaluate the following:

1. Does the average stream gradient change significantly between upstream and downstream?
2. Are there signs of significant erosion or deposition?
3. Are there any man-made features within the active channel? Within the floodplain?
4. Are there any sudden changes in sediment size distribution?

In evaluating the project reach for the above points, the Stream Designer is trying to determine whether the morphological attributes (gradient, confinement, planform, shape, bed materials, etc.) of the reach reflect what would be expected in the vicinity of the site, and how/to what extent these attributes are modified by artificial features, constraints, or conditions.

Significant changes in gradient is an indication that sediment supply may be a concern, or the crossing is in a transition zone, etc. Large amounts of deposition or erosion have an impact on the overall channel slope and shape that may not be sustainable in the long term. Man-made features within the channel and/or floodplain such as riprap, piers, foundations, levees, or mechanically altered channels could cause the reach to not reflect what the channel would look like under natural conditions. However, if the channel is mechanically altered, mimicking the channel shape is recommended; in these instances, contact the HQ Hydraulics Section for additional guidance.

If the answer to any of the above questions is yes, proceed to Section 7-2.1.2. If the answers to all of the above questions are no, proceed to Section 7-2.1.3

7-2.1.2 Similar Reference Reach

If the adjacent reach is not representative, a similar reference reach will need to be located. A reach within the same watershed is preferable, but if one cannot be located, another watershed may need to be investigated. Locate a similar reference reach using the following steps:

1. Examine a topographic map at the 1:24,000 scale (or finer) for reaches farther upstream and downstream of the culvert reach with similar slope, watershed characteristics, and channel confinement.
2. When a new reach with similar slope, watershed characteristics, and channel confinement is identified, determine the size of the contributing watershed area. Is it similar (+/-20 percent) to the contributing area above the project reach?

If the reach meets criteria a and b, go to Section 7-2.1.3. If it does not, look to adjacent watersheds with similar aspect, elevation, and geology and go back to step (a).

7-2.1.3 Reference Reach Data Collection

After locating an appropriate reference reach, collect data for the specialty report. At a minimum, collect the following information:

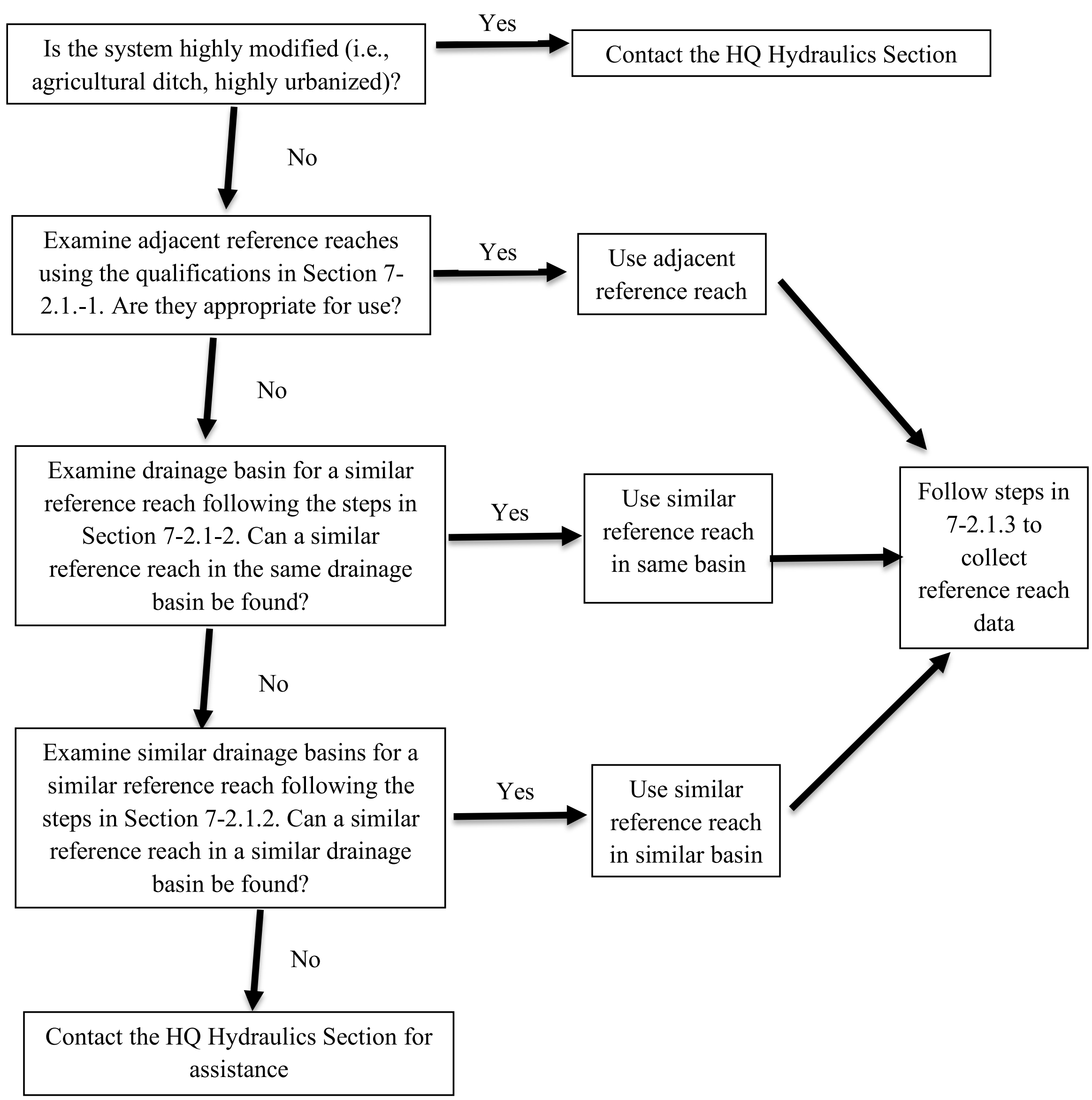
- Stage of channel evolution at the project reach
- Water surface slope during non-flood event

- Channel sinuosity and radius of curvature
- Presence and residual depth of pools
- BFW in at least three representative locations; compare to those measured at project reach
- Pebble counts or grab samples in at least three locations on riffles or pool tailouts (Wolman 1954)
- Note riparian zone vegetation, canopy density
- Note presence and function (or absence) of LWM, especially key pieces (see Chapter 10)
- Record geographic coordinates of reference reach
- Note anthropogenic impacts to the reach

7-2.1.4 Project Constraints

If it is determined that a constraint is present requiring a design reference reach, contact the HQ Hydraulics Section for concurrence requirements for the use of a design reference reach.

Figure 7-1 Reference Reach Determination



7-2.2 *Bankfull Width*

An accurate BFW is critical as it is a driving factor for the minimum required structure size per Section 7-4.3. Appendix C of WDFW's 2013 WCDG is a useful reference in determining an appropriate BFW. A minimum of three measurements shall be used when computing the average BFW. Measure widths that describe prevailing conditions at straight channel sections and outside the influence of any culvert, bridge or other artificial or unique channel constriction.

If there are significant differences between measured and modeled BFW, further evaluation or justification will be required. Hydraulic modeling shall be utilized to verify the appropriate measured BFW by using the 2-year flow top width. Typically, the 2-year top width is equal to or slightly wider than the BFW. WDFW has created a regression equation used for estimating BFW that can be found in Appendix C of the 2013 WCDG and shall only be used as a check to determine what a reasonable measurement is on streams within the limitations of that equation. Additional guidance will be provided in future revisions to the *Hydraulics Manual*.

It is not always evident where the influence of an undersized structure ends. On a low gradient system that has a high headwater at the crossing, the backwater during high flow events can extend upstream for hundreds of feet and result in an artificially wide BFW measurement. Once the existing conditions model is created the bankfull measurement locations should be checked to confirm they are outside the influence of the existing structure. If the BFW measurements are determined to be within the influence of the structure, additional site visits are required for reevaluating BFW measurements.

7-2.3 *Watershed and Land Cover*

Understanding the past, current, and potential future conditions of a watershed is important for the long-term success of a project.

Historical and current aerial photographs should be examined to determine what type of land cover the watershed has now and how that has changed over time. Verifying whether the system is in an urban setting, within an urban growth area, or in an active forest will also help determine what the land cover could look like in the future and may increase the design flows expected during the design life and create the need for a larger structure. Understanding of how the watershed has changed over time will help the Stream Designer create a crossing that will be successful.

If a watershed has a high potential for future forest fires or has been recently affected by a forest fire, this shall be documented and taken into consideration when determining the final structure size.

7-2.4 *Geology and Soils*

The soil types in the drainage basin not only assist the Stream Designer in understanding what is happening at the crossing, but also can impact the calculated hydrology at the site location if a continuous simulation method, such as MGSFlood, is utilized to determine flow rates.

The surrounding geology will have an impact on lateral migration and may influence where a new crossing is placed. It may also influence sediment load and size distribution in the channel. Generalized soil types may be found in Soil Surveys produced by the National Resources Conservation Service. Surficial geology maps are also useful in determining soil information.

7-2.5 *Fluvial Geomorphology*

Fluvial geomorphology is an integral part of determining where the crossing should be placed, how the stream or river should be aligned, and where the stream or river may end up in the future and is a primary determinant of the appropriate design of the channel. The channel should be examined to determine if there are signs of lateral and vertical stability or instability and how the stream may be impacted in the future. Delineation of channel migration zones should be investigated (and may be required by local jurisdictions). The potential for channel avulsion should also be assessed.

7-2.5.1 Channel Geometry

Streams have often been straightened or moved, resulting in shorter crossings that are perpendicular to the roadway. Roadway as-builts and old ROW plans are good sources for determining what the crossing looked like prior to roadway construction. Old aerials may give a good indication of the channel alignment over time, depending on tree cover. LiDAR, if available, is also a good resource to provide insight into general down-valley slopes and help identify grade breaks beyond the limits of the survey. LiDAR can also identify relic channel features, such as side channels, scroll bars, avulsions, and alluvial fans.

Many WSDOT roads were built at the edge of stream and river valleys. As a result, it is not uncommon for the reach through the roadway prism to be within a transition zone between an upstream reach and a downstream reach. Oftentimes, this leads to a historic slope steeper than the adjacent reaches. Culvert crossings at roadways can serve as grade controls, which have been in place in some instances for many years and may have had an effect on the channel upstream and downstream of the crossing. Having a good understanding of sediment supply and general transport regime with and without the existing crossing within the system is important in determining the long-term potential for channel slope change over time.

The channel slope and changes in the channel slope should be documented, both in the reference reach and near the culvert. These slopes shall be measured in the field or determined by survey data.

The channel shape, changes in vegetation, cross section break lines, and other well-defined features should be noted, as well as any low flow paths. It is important to verify that the survey matches what is in the field and represents the natural conditions in the hydraulic modeling.

7-2.5.2 Potential for Aggradation, Incision, and Headcutting

Note channel conditions within the reference reach. Look for the potential for aggradation/degradation within the channel, and note the channel planform and any channel incision.

Dams or undersized culverts within the drainage system can also have a lasting impact on the creek. A dam or undersized culvert upstream may cause deposition of sediment in the upper reach and starve the sediment load transported downstream, resulting in degradation downstream. This may affect the BFW and/or create a perched culvert. Likewise, a dam or undersized culvert lower in the system may cause a sediment supply issue affecting the gradient.

Upstream hillslope and/or channel instability or watersheds that have large areas of disturbed land can create a potential for large sediment pulses or aggradation at the crossing. A structure should be designed to accommodate any expected excessive sediment input to avoid becoming a maintenance problem in the future.

The specialty report shall note whether or not aggradation, degradation, or headcutting is a risk in the future and how the design will accommodate these risks.

7-2.5.3 Floodplain Flow Paths

Determine whether there is a mapped floodplain for the water body that the highway is crossing or an adjacent water body (i.e., the crossing may be in the mapped floodplain of a larger river). Also describe whether any floodplains exist and how the flows move through these floodplains.

Anticipated changes to floodplains as a result of any structure change or grading shall also be discussed, even if it is not required for a permit. The HQ Hydraulics Section will determine whether or not the changes to the floodplain are significant. If the changes are deemed significant, then the PEO will need to communicate those changes to the local jurisdiction. In some instances, this may require a FEMA map revision.

7-2.5.4 Channel Migration

A description of any past channel migration and potential future channel migration shall be documented in the specialty report. LiDAR and past aerial photographs should be utilized to determine where the channel has been in the past, if available.

7-2.5.5 Existing Large Woody Material and Channel Complexity Features

LWM within the reference reach and near the crossing shall be documented, as well as the potential for future LWM recruitment. The channel type (Montgomery and Buffington 1993) and any key features such as LWM, boulders, and bedrock outcrops that are creating channel complexity or influencing channel alignment shall be noted as well as the capability of the system to move wood if future conditions provided a stream buffer that could recruit LWM.

7-2.5.6 Sediment

Sediment size determination in the reference reach is typically done through either Wolman pebble counts (or other method as approved by the HQ Hydraulics Section shown to produce similar results) or grab samples, depending on the size of the streambed material. If a grab sample is used, the sample size needs to be large enough to produce accurate results. Guidance on sample size can be found in scientific literature.

The sediment sampled should be within the reference reach. Note any large, naturally occurring material that is on-site and include the notation within the design documentation. In some cases, large, unnatural material or large deposits not transported by the current flow regime may be shaping the current stream conditions including elements from previous or upstream streambank stabilization and scour protection efforts. While it may not be accurate to include this angular rock or other streambank stabilizing material in the pebble counts, making note of it may be useful for understanding the reach conditions and what the stream is capable of mobilizing.

Understanding the sediment supply in the system is critical to being able to determine the correct size material to be placed back into the stream. If a system is sediment starved, it may be necessary to provide material that is coarser than the adjacent reaches to avoid channel incision. If a system has a healthy sediment supply, it may make sense to place material that is mobile and matches the sediment in the adjacent reach.

7-2.6 *Hydrology*

If the hydrology at a site is estimated incorrectly, this can lead to underestimating or overestimating the required size for the structure's span, incorrect scour elevations and depth estimates, incorrect channel shape, and incorrect LWM sizing and anchoring requirements.

Additional information about hydrology can be found in Chapter 2. Justification for the chosen methodology being the most appropriate is required for all projects, including if the USGS regression equation is used. In many instances, the USGS regression equation may be the best available information, but this shall be confirmed through modeling, site conditions, maintenance history, and engineering judgment. The standard error for the USGS regression equation is quite high in some areas and it may be necessary to adjust the flows based on these standard errors. Other methodologies, such as the basin transfer method or HSPF may be more appropriate. In urban areas, hydrology models that include future build out conditions may be available for use.

7-3 Hydraulic Analysis

Model outputs are required as part of the specialty report and must be used to verify that the minimum proposed structure size meets the appropriate WACs, WDFW's 2013 WCDG, and this chapter. WSDOT requires the use of SRH-2D unless otherwise approved by the HQ Hydraulics Section. Additional guidance will be provided in future revisions to the *Hydraulics Manual*.

7-3.1 *Manning's n*

Special care shall be taken to determine Manning's n. In addition to the typical charts and tables available for Manning's selection, there are several equations that are valid for gravel bed systems that predict a Manning's n value. The selection process used for determining what the Manning's values are for the system shall be documented in the specialty report. Additional information on Manning's n can also be found in Chapter 4 and Appendix 4A.

7-3.2 *Boundary Conditions*

Boundary Conditions should be set for normal depth unless other information is known. In cases where there is tidal or flood influence from another water body, both a normal depth condition and a backwater condition shall be analyzed. For freeboard, the backwater condition is conservative and for scour the normal depth condition is conservative. The boundary conditions used for each scenario and the reasoning for them shall be documented in the specialty report.

7-4 Design

All WSDOT crossings for fish bearing waters must meet WAC 220-660, at a minimum. In WRIAs 1 through 23, the design must also meet the requirements of the 2013 Federal Court Injunction for Fish Passage.

The process that is required for WSDOT design projects is described in the sections that follow and summarized in Appendix 7A. These sections only cover the Bridge Design and Stream Simulation Design methods; other methods may be appropriate but must be approved by the HQ Hydraulics Section prior to use.

The design flow for WSDOT projects is the 100-year, unless it is determined on approval by the HQ Hydraulics Section that another lower event would cause a more extreme case than the 100-year. For example, if the project is within the floodplain and the entire area is flooded, then a different storm event or boundary condition may be more appropriate (i.e., 10-year backwater on larger water body for backwater with the 100-year storm on the smaller water body).

All the supporting calculations/information for the design process below shall be included in the specialty report.

7-4.1 *Constraints*

Constraints are infrastructure or land ownership issues that interfere with natural stream processes and need to be identified as soon as possible. Constraints can be both man-made and natural and, when encountered, should be discussed with stakeholders early in the design process to prevent project delays in the future if not all parties agree on whether a constraint exists or may resolvable within the scope of a project. There may be design constraints other than those covered in this section.

7-4.1.1 Infrastructure

Infrastructure can include adjacent culverts/bridges, pipelines, buildings, water intakes/diversions, groundwater wells, and roadways as well as other infrastructure types not listed here. Infrastructure that is a design constraint can be owned by WSDOT or by other parties.

7-4.1.2 Environmental Impacts

Environmental impacts should be considered when completing a stream design. If meeting the design methodology causes a large environmental footprint (i.e., if a roadway that needs to be raised next to a wetland or stream grading would need to be extended for a great distance), discussions with WDFW and the Tribes should occur to determine what the best design is to move forward and whether mitigation may be used in lieu of meeting requirements/recommendations.

7-4.1.3 Grade Separation

Many culverts have been in place for a long time and the stream has adapted around them. Culverts may have been historically placed at a grade break in the channel that is dissimilar to the upstream and downstream reach. If there is a large grade separation between the upstream reach and the downstream reach, it may be necessary to allow for a natural channel regrade, producing a steeper reach with an overcoarsened channel. As much information as possible should be obtained about historical conditions and the cause of the grade break and discussions with WDFW and the Tribes should occur to determine the best solution for the project.

7-4.1.4 Cultural Resources

Impacts to cultural resources should be considered when completing a stream design. If meeting the requirements and recommendations for the project would have an impact on cultural resources, WDFW and the Tribes should be consulted to determine the way to proceed.

7-4.2 *Channel Alignment*

Typically, it is favorable to keep the alignment in its current location unless the investigations done during the data gathering phase show that it should be relocated. Another reason to realign the crossing would be to eliminate the crossing all together if the channel realignment is done.

It is not always possible to cross a roadway at an ideal angle or avoid sharp bends leading into or out of a structure. The total length of covered stream should be considered and the maximum angle of a bridge structure to centerline of a roadway per the Bridge *Design Manual*, if a bridge structure is used. While the HQ Hydraulics Section does not typically recommend a structure type or layout, it is important for the Stream Designer to know what this constraint is and keep it in mind while designing the layout to make an efficient crossing. As a result of the crossing angle, if armoring is determined necessary, see Section 7-4.10.

Channel sinuosity and curve radii must match what would be expected in the reference reach, and a channel must not be artificially lengthened by increasing sinuosity beyond what would be expected to decrease slope. Meanders extended unnaturally to obtain length will not be stable. Conversely, channel sinuosity must not be unreasonably reduced or eliminated in the interest of shortening the structure span.

If a channel needs to be realigned, it must be done so in a way that does not increase the slope significantly or create an erosion risk. In the case of slope, WSDOT uses the stream simulation recommendation from WDFW's 2013 WCDG of a slope no steeper than 125 percent of the upstream reach (or downstream if it is deemed that the downstream reach is more appropriate). In systems where the slope is low gradient (i.e., less than 1 percent), exceeding the slope limit while still meeting this criterion may be permissible but must be approved by the HQ Hydraulics Section. If it is not practicable to meet the slope constraint, approval by the HQ Hydraulics Section is required.

If allowing for natural regrade is determined undesirable, the Stream Designer must evaluate the long-term degradation, scour, potential equilibrium slopes, and whether a larger structure will be required as a result of the channel regrade. Lateral migration during the process of the regrade should be considered and appropriate countermeasures must be implemented to protect banks from destabilization as a result of construction. Refer to Chapter 4 for additional guidance.

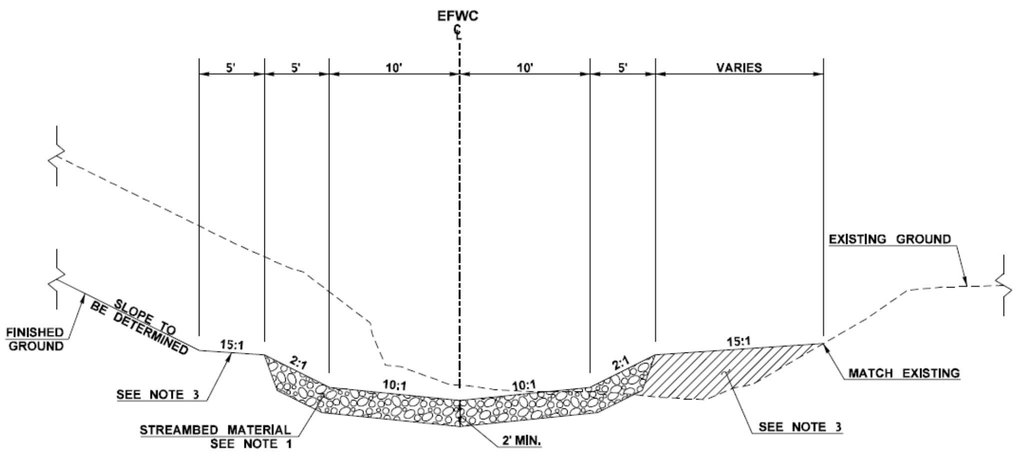
If regrade is determined not to be desirable, the reach must be designed to be stable. This may cause the project to be permitted as a fish passage improvement structure (see Section 7-5.2) and require long-term maintenance and monitoring. The streambed material decision tree found in Appendix 7A may help the Stream Designer determine whether or not to allow for channel regrade.

7-4.3 *Channel Cross Section*

The channel cross section should mimic that of the reference reach, while keeping construction methodologies in mind. If a system is highly modified (i.e., an agricultural ditch) and the grading for structure replacement is minimal, it may be appropriate to match the adjacent reach instead. For highly modified systems, contact the HQ Hydraulics Section for assistance.

Cross section lengths should be rounded to the nearest tenth of a foot. Slope should be rounded to the nearest 0.5:1. Example plans and plan requirements can be found in WSDOT's *Plans Preparation Manual*. An example cross section is illustrated in Figure 7-2.

Figure 7-2 Final Design Cross Section



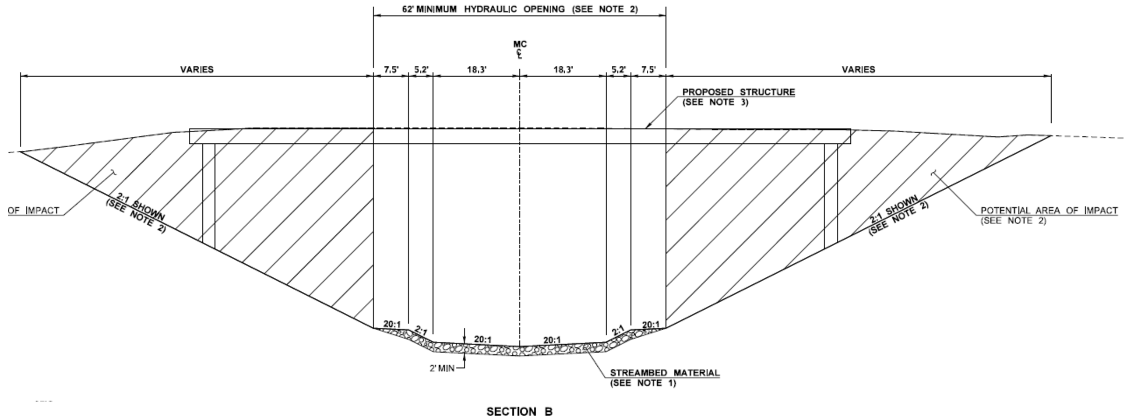
Flows within the channel cross section must mimic those in the reference reach. For example, if the active channel is overtopped at less than a 2-year event, the channel should behave the same through the design reach.

7-4.4 *Hydraulic Opening*

For the purposes of this chapter, the minimum structure width recommended by the specialty report is defined as the minimum hydraulic opening. This is needed to make a distinction between what is required to meet WAC and this chapter versus what may be installed at project completion. The minimum span of a structure shall be the hydraulic opening; however, the actual structure width determination is made by Region or the Bridge and Structures Office unless there is a hydraulic reason to place upper limitations on size. Any required scour protection shall not encroach within the hydraulic opening unless it is placed below the total scour elevation.

For preliminary plans, prior to the structure type being known, it is recommended that 2:1 cut slopes with a note that "grading limits to be based on final structure size, type and location" are shown unless it is known that the structure will be buried. This lets the reviewers know that the structure type is undetermined while showing the potential impact areas. Cross sections should clearly depict where the minimum opening is, as shown in Figure 7-3.

Figure 7-3 Minimum Hydraulic Opening



There are three methods for determining the minimum hydraulic opening: stream simulation, confined bridge, and unconfined bridge. However, the process used for confined bridge is the same as stream simulation. All methods are dependent on the floodplain utilization ration (FUR), which determines how confined a stream is. The minimum hydraulic opening is determined from Equation 7-1 (2013 WCDG, Equation 3.2), unless otherwise approved by the HQ Hydraulics Section.

$$W_{HYO} = 1.2{*}W_{bf} + 2 \text{ feet} \qquad (7\text{-}1)$$

Where

W_{HYO}= Width of hydraulic opening

W_{bf}= BFW

The minimum width of the hydraulic opening is to be taken vertically through the entire structure. If a round or arch structure is used, additional width/height may be necessary to maintain the opening through the anticipated scour/required freeboard.

The design flood for temporary bridges that will be in water for one season or less shall use the 25-year flow event for the design flood. For temporary bridges that will be in water for more than one season, the 100-year flow shall be used for the design flood.

7-4.4.1 Floodplain Utilization Ratio

The FUR needs to be calculated using existing conditions. The FUR is the width of the floodplain relative to the main channel. To determine the FUR for WSDOT designs, compare the 100-year water surface width from the model output to either the available BFW information or, if BFW is not available, the 2-year top width. To determine what the FUR is through the upstream reach, it is recommended that the existing structure be removed from the model.

A FUR larger than 3.0 is considered an unconfined system, while a FUR less than 3.0 is considered confined. If the system is unconfined, the unconfined bridge design method applies. If the system is confined, either the confined bridge design method or the stream simulation design method applies. More explanation of the FUR can be found in the 2013 WCDG. For areas that are tidally influenced, see Section 7-4.4.5.

7-4.4.2 Unconfined Systems

An unconfined system has a FUR of greater than 3.0. In these situations, the velocity ratio, which is defined as the velocity at the thalweg of the main channel through the structure divided by the velocity of the main channel immediately upstream of the structure if the roadway fill were to be removed entirely, is used to determine structure size. The velocity ratio shall be close to 1, which means that the ratio when rounded to the nearest tenth shall be 1.1 or less at the 100-year event. In some low velocity cases, a ratio of more than 1.1 may be allowable if the increase in velocity ratio does not result in bed coarsening, increased scour, significantly increased backwater, or negative biological/geomorphological effects. The HQ Hydraulics Section must approve in these instances.

If an existing structure is being replaced by a new structure, a velocity ratio of more than 1.1 may be acceptable. In this case, the existing structure should not have evidence of significant erosion, scour, or other performance issues. The HQ Hydraulics Section must approve in these instances.

For preliminary design, the Stream Designer is to assume vertical walls for the edge of structure while determining the minimum hydraulic opening in the hydraulic model. Once the final structure size has been determined by others, the model shall be updated to reflect the updated structure. Additional width may be required in instances where lateral migration is a concern.

7-4.4.3 Confined Systems

For confined systems, the BFW plus a factor of safety is recommended. In the case of WSDOT crossings, minimum structure width shall not be less than what is given by Equation 7-1 unless otherwise approved by the HQ Hydraulics Section. In many cases, this width is appropriate. In some cases, a wider structure may be more appropriate.

Additional width is required if the following apply:

- The structure is creating an excessive backwater.
- The velocities through the structure differ greatly from the adjacent reach.
- Aggradation is expected.
- Lateral migration is expected throughout the system.
- The Stream Designer has reason to believe additional width is needed.

7-4.4.4 Tidally Influenced Systems

For tidally influenced systems follow at a minimum Appendix D from the 2013 WCDG. Additional guidance will be provided in future revisions to the *Hydraulics Manual*.

7-4.4.5 Climate Resilience

WSDOT uses climate science and tools to evaluate the influence climate change has on projects throughout the state of Washington. This is done through the use of the best available science and working with the Climate Impacts Group and stakeholders' groups. Contact HQ Hydraulics for guidance on incorporating climate resiliency on projects. Additional guidance will be provided in future revisions to the *Hydraulics Manual*.

Climate resilience should also include the future risk of forest fire. If the watershed is located in an area that has a high potential for future forest fires, additional structure width and height may be warranted to accommodate this risk.

7-4.5 Freeboard

A structure that does not have adequate freeboard can require repeat maintenance activities and a more robust structure and could have an increased scour depth.

A minimum of 3 feet of freeboard above the 100-year water surface elevation is required on all structures greater than 20 feet and on all bridge structures unless otherwise approved by the HQ Hydraulics Section for reasons discussed below. The Stream Designer shall also confirm local ordinance requirements are met and any necessary permit conditions are satisfied.

The freeboard required on all buried structures unless otherwise approved by the HQ Hydraulics Section are listed in Figure 7-4.

Figure 7-4 Freeboard Requirements on Buried Structures

Structure Bankfull Width	Required Freeboard
Less than 8-foot bankfull width	1-foot above 100-year flow event
8- to 15-foot bankfull width	2 feet above 100-year flow event
Greater than 15-foot bankfull width	3 feet above 100-year flow event (bridge)

In areas that are tidally influenced, the impacts of 2 feet of sea level rise shall be considered for the project.

The required minimum freeboard shall be maintained across the entire minimum hydraulic opening, as shown in Figures 7-5 and 7-6. Additional consideration should be given to maximize freeboard for increased internal clearance for access and/or animal crossings, constructability, fill height, etc. If aggradation is expected to occur, additional freeboard shall be given above the 100-year equal to the anticipated aggradation.

Figure 7-5 Freeboard Requirements on Box Structures/Vertical Abutments

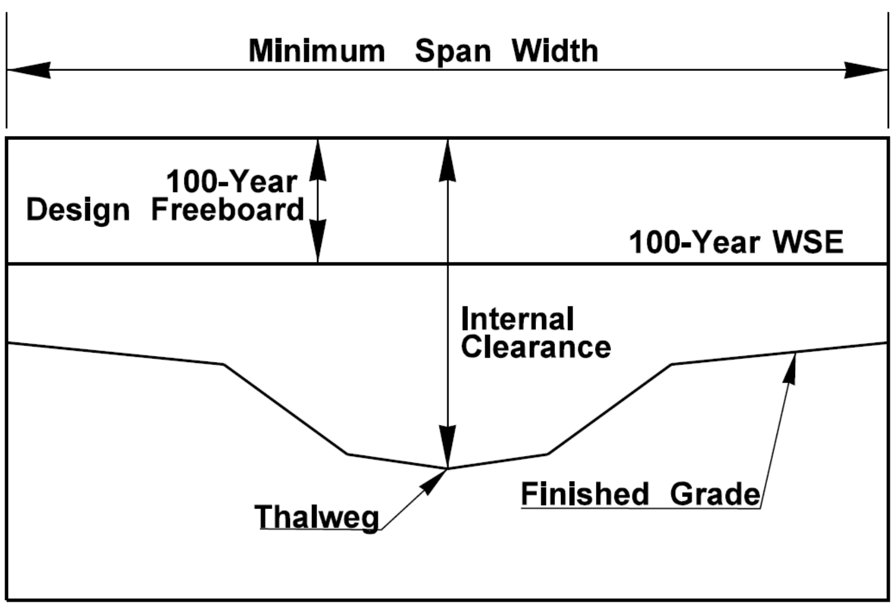
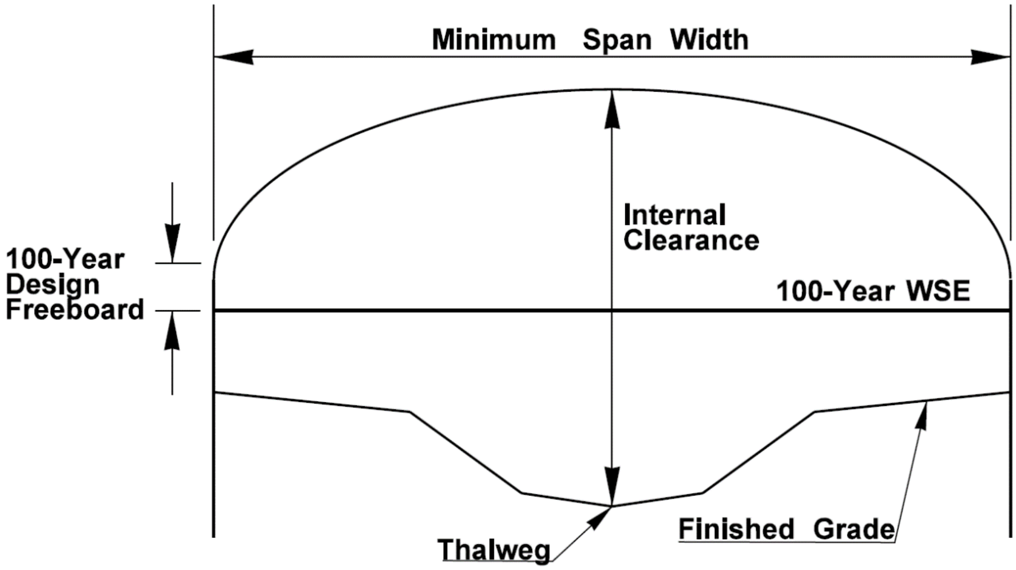


Figure 7-6 Freeboard Requirements on Arch Structures



The design flood for temporary bridges that will be in water for one season or less shall use the 25-year flow event for the design flood. For temporary bridges that will be in water for more than one season, the 100-year flow shall be used for the design flood. The freeboard required for temporary bridges shall be 1 foot above the design flood water surface elevation, at a minimum. Debris loading shall be evaluated for the system and the freeboard increased if additional clearance for debris is necessary.

7-4.6 *Buried Structures*

Buried structures for WSDOT projects can follow either the bridge design or stream simulation design criteria. If a buried structure is used, a few additional criteria apply.

If a structure length is more than 10 times its width, then the structure width shall be increased by 30 percent to allow the channel to meander, per the WCDG.

The WCDG and WAC require that all stream simulation culverts be countersunk a minimum of 30 percent and a maximum of 50 percent, but not less than 2 feet overall. Alternative depths of culvert fill may be acceptable with engineering justification that considers channel degradation, aggradation, and total scour. Scour analyses are typically considered acceptable engineering justification.

Buried structures shall be countersunk a minimum of 2 feet below total scour at the design flood, regardless of span width. If this requirement cannot be met, approval from the HQ Hydraulics Section is required. It is understood that four-sided structures are created in whole foot increments, so if the countersink is slightly below 2 feet, contact the HQ Hydraulics Section to verify if additional depth is required.

In some cases, constructability is easier if the structure is placed flat or the Stream Designer may recommend the structure be placed at a different slope than the streambed. Buried structures may be placed at a different slope than the prevailing stream gradient so long as the minimum freeboard is met throughout the structure, the minimum required countersink is met throughout the structure, and justification is provided and approved by the HQ Hydraulics Section. In some cases, this may require a slightly taller structure. The reasoning for placing the culvert at a different slope shall be described in the specialty report.

7-4.7 *Sediment*

WAC dictates allowable sediment sizes in a fish bearing stream. Stream simulation design aims to mimic natural conditions to the extent possible, but sometimes stream conditions have been altered, reaches have been sediment starved, or adjacent infrastructure (constraints) do not allow for bed mobility into adjacent reaches.

Apply the stream simulation requirement of a D_{50} that is within 20 percent of the reference reach unless constraints prevent this. A Streambed Material Decision Tree to further assist stream designers in determining which methodology to use for streambed sediment sizing in these special cases, is shown in Appendix 7A.

For sediment sizing, WSDOT uses the Modified Critical Shear Stress Approach, as described in Appendix E from the 2008 U.S. Forest Service (USFS) Guidelines for all systems under 4 percent and the Unit-Discharge Bed Design as described by the 2013 WCDG for systems greater than 4 percent. A system is considered stable if the D_{84} is stable at the design flow event.

7-4.7.1 No Constraints

As previously described, apply the stream simulation requirement of a D_{50} that is within 20 percent of the reference reach unless prevented by constraints. Most systems fall into this scenario. The design process for sediment sizing under these conditions is to match the reference reach material to the extent possible using the materials available from WSDOT's *Standard Specifications*.

Stability of the bed mix shall still be evaluated and documented in the report.

7-4.7.2 Constraints

If there are constraints in the systems, as described in Section 7-1, that could have an impact on the stream design, the risk of the stream not being stable will need to be evaluated.

In some cases, a bed design based on the pebble count from the existing reference reach will meet the requirements for stability. The existing pebble count will need to first be evaluated for stability, using the appropriate methodology from Section 7-4.7. If the D_{84} is not stable at the design flood, then a risk assessment will need to be conducted to determine the next steps. The HQ Hydraulics Section and RHE shall be a part of the risk assessment process.

7-4.7.2.1 Risk Assessment

To complete a risk assessment for the site, the constraints must be identified and what the potential impact to those constraints would be if natural processes were to occur. If the constraints are private or public infrastructure not owned by WSDOT, the owners of the infrastructure should be consulted. The Streambed Material Decision Tree in Appendix 7A can be helpful in determining the level of risk; however, the ultimate decision on constraints and risks to constraints is made by the project team.

If it is determined that the project is high risk and cannot be allowed to regrade, a roughened channel must be constructed. A roughened channel is designed to be completely non-deformable up to the design discharge. If a roughened channel is built, any habitat features must be installed at the time of construction, as they are unlikely to form themselves. A roughened channel will likely have additional permit requirements (and possibly long-term commitments) associated with it.

If a project is considered medium risk, an alternative analysis needs to be conducted. The Stream Designer needs to describe the constraint, describe the impact of meeting the requirements for sediment size, identify and evaluate any alternatives, and describe the preferred alternative. When describing the preferred alternative, the Stream Designer must also describe how the preferred alternative reduces the risk to an acceptable level and what potential impact to fish life this alternative may have. In cases where coarser sediment is necessary on a medium-risk project, an overcoarsened channel with habitat complexity features may be constructed. This channel is subject to agreements between WSDOT and permitting agencies. An overcoarsened channel has a $D_{84,}$ which is stable at the Design Flood.

If a project is determined to be low risk, then the bed material should match the pebble count in the reference reach and the process described in Section 7-4.7.1 applies.

7-4.7.3 Coarse Bands

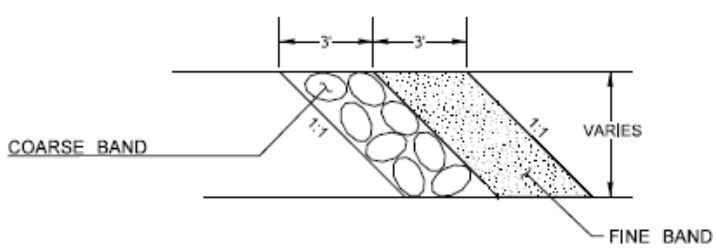
Coarse bands are bands of material that are coarser than the overall bed design material. They are meant to keep the stream centered in the culvert, should be partially deformable, and are not intended to be grade control. As a result of project monitoring and repair, it was determined that the use of a fine band of material upstream of a coarse band can help seal the streambed mix. Fine bands consisting of Streambed Fine Sediment, a natural or manufactured sand, meeting the grading requirements in Figure 7-7 shall be placed upstream of all coarse bands.

Figure 7-7 Fine Band Grading

Sieve Size	Percent Passing
No. 4	99 – 100
No. 10	46 – 86
No. 40	26 – 40
No. 200	10 – 20

The typical profile shape that WSDOT uses for Coarse and Fine Bands can be seen in Figure 7-8. More information on coarse bands, including spacing, can be found in the 2013 WCDG.

Figure 7-8 Coarse Band Profile Shape



Coarse bands are required within all structures that are four sided and that have a stream slope of 2 percent or less. Coarse bands are recommended within all structures that have a stream slope between 2 and 4 percent. Coarse bands are typically sized for the D_{84} to be stable at the 100-year flow event and shall not have material that is larger than twice the D_{100} of the design bed mix.

It may be necessary to have bands or clusters of coarser material beneath/inside of structures to help promote channel cross shape stability and channel complexity that are outside of what is recommended above. In these cases, the Stream Designer must use engineering judgement to determine what this will look like. See Section 7-4.10 for channel complexity. A Fine band may be required to be placed along side of the Coarse band to aide in streambed flow after construction. Contact the HQ Hydraulics Section for additional guidance on the use of Fine bands.

7-4.8 *Total Scour*

All structures shall be designed for total scour, as defined by HEC-18, regardless of structure span. All four-sided buried structures shall be countersunk a minimum of 2 feet below the total scour depth at the design flood and shall be countersunk deep enough for the bottom to not become exposed during the check flood. Foundation depth for three-sided buried structures/traditional bridge structures with abutments and piers shall be determined by the bridge and geotechnical office. Methodology used for determining total scour shall follow the methods described in HEC-18.

The design flood for temporary bridges that will be in water for one season or less shall use the 25-year flow event. For temporary bridges that will be in water for more than one season, the 100-year flow shall be used for the design flood.

7-4.9 *Lateral Migration*

All structures shall be designed to account for the lateral migration expected over the life of the structure. The Stream Designer shall document in the specialty report whether there is a high or low risk of the stream migrating to each pier and/or abutment and whether any preventative countermeasures or increase in structure size are recommended. In some cases, countermeasures may only be required if the structural element in question is not designed below the full depth of scour; this should be noted if it is the case and requires approval from the HQ Hydraulics Section, HQ Bridge Section, and HQ Geotechnical Section. If preventative countermeasures are necessary for embankment protection, this also is required to be described in the report.

7-4.10 *Channel Complexity*

Chapter 10 covers the requirements for channel complexity when LWM is used.

7-4.10.1 Outside of Structure

Channel complexity within the channel is most often accomplished by LWM and/or boulder clusters, depending on what is appropriate for the system.

If used in the system, boulder clusters should be sized large enough to remain stable, be placed in a way that they promote localized scour/pool development, do not create a low flow barrier risk, and engage in the active channel. Additional guidance will be provided in future revisions to the *Hydraulics Manual*. In addition to being stable during flow events, consideration should be given for the stream's location and whether vandalism could be an issue. If the location is in an area where there may be human activity, larger, heavier boulders may help keep the structures in place.

7-4.10.2 Inside/Under Structures

Mimicking bank structure inside of a structure or under a bridge is difficult when plants typically cannot grow, and the streambed material is designed to match that in adjacent reaches, which is often not stable at higher flow events. The lack of root structure at the edge of the bankfull channel and the instability of the material being placed can create a situation where the channel shape deteriorates over time. Aggradation inside of the structure can also cause the channel to lose its shape over time.

WSDOT has used coarse bands/coarse band barbs, boulder clusters, boulder cluster barbs, and lateral coarse bands. If boulders are used, the same recommendations as outside of the structure apply. Additional guidance will be provided in future revisions to the *Hydraulics Manual*.

7-4.10.3 Construction Recommendations

A channel takes a few large flows to have habitat elements form. In cases where a fish barrier is replaced, if these habitat elements are not formed during construction, the first migration of fish may be left with a long, straight channel that makes passage difficult. Leaving scour pools at the LWM and other complexity elements at locations where a pool would naturally form is recommended. A low flow pilot channel is also recommended to be installed that connects the habitat complexity elements immediately after construction.

7-4.11 *Preventative Countermeasures*

Preventative countermeasures are not always avoidable, whether it is to protect the structure itself or to protect a roadway adjacent to a water body. When a preventative countermeasure is necessary, the specialty report shall document the risk and rationale for the protection, any current evidence of erosion, and the countermeasure design. The ISPG, HEC-23, and Chapter 10 provide additional guidance on the implementation of countermeasures. The least amount of bank protection necessary to withstand the design flood is what should be installed and, if possible, buried away from the stream edge. For new structures, preventative countermeasures shall not encroach within the minimum hydraulic opening unless they are located below the total scour depth.

It is sometimes possible in low energy systems to provide the necessary protection through the use of planting and soil stabilization countermeasures.

7-4.12 *Landscaping/Planting*

There is guidance for planting near streams located in WSDOT's *Roadside Manual* Chapter 830, that the landscape architect will follow for all projects located near streams. It is also beneficial for the Stream Designer to review this chapter. The Stream Designer shall collaborate with the landscape architect and provide input on the need for bank stabilization countermeasures, habitat complexity, and any planting that needs to be done prior to the first storm event of the year. Typically, the planting windows for WSDOT projects that do not install irrigation are October 1 to March 1 west of the Cascade Crest and October 1 to November 15 east of the Cascade Crest, per the WSDOT *Standard Specifications for Road, Bridge, and Municipal Construction*. If planting needs to occur before the end of these windows for stability reasons, the contract will need to be updated to reflect the timeline.

7-4.13 *Determining Crossing Design Methodology for Documentation*

The three most used design methodologies by WSDOT from WDFW's 2013 WCDG are the Unconfined Bridge Methodology, Confined Bridge Methodology, and Stream Simulation Methodology. For all unconfined systems, the design methodology shall be described as unconfined bridge. For all confined systems over 20 feet, those expecting 1 foot or more of channel regrade, or slopes that are outside of the slope ratio, the methodology shall be described as confined bridge unless otherwise approved by the HQ Hydraulics Section. For all structures under 20 feet that do not fall into the categories described for unconfined bridge or confined bridge, the design methodology shall be stream simulation unless otherwise approved. If a different methodology was approved by the HQ Hydraulics Section, the design process shall be documented as the process that was approved. See Section 7-5 for some other available methods.

7-5 Other Design Methods

It is recognized that not all stream crossings will be able to meet stream simulation or either bridge design methodologies. As described in Section 7-4, there are other available design methodologies that can be accepted on a case-by-case basis with the approval of the HQ Hydraulics Section. This section will briefly describe some of the other methodology available.

Some of these design methodologies may need to include project objectives with performance measures, inspection schedules, maintenance triggers, and a contingency plan should the project fail to meet performance measures with permitting applications.

7-5.1 *No-Slope Design*

No-slope design recommendations can be found in the 2013 WCDG and the WAC. The no-slope designs are typically performed on BFWs of less than 10 feet, low gradients (less than 3 percent), and short culvert lengths (less than 75 feet). This design methodology is not preferred because it has a higher risk of becoming a barrier in the future, does not give the stream much room for natural processes, and has a lower capacity than stream simulation culverts and bridges.

7-5.2 *Fish Improvement Structures*

Fish improvement structures are any structures that facilitate the passage of fish either through or around the fish barrier that do not necessarily mimic natural channel processes. Structures such as roughened channels, roughened rock ramps, structure retrofit designs, and hydraulic culvert designs are examples of fish improvement structures. Fish improvement structures are only allowed by prior approval from the HQ Hydraulics Section. Additional information about roughened channels, roughened rock ramps, and structural retrofits are included below. Other fish improvement structures exist but are not covered here.

A fish improvement structure may be necessary to facilitate fish passage through an existing structure, allow for a transition between a newly constructed fish-passable structure and an upstream fishway, or as a means of grade control when deemed necessary. All fish improvement structures must meet WAC 220-660-200.

7-5.2.1 Roughened Channel Design Methodology

A roughened channel is a constructed channel with a streambed material and configuration designed to be non-deformable up to the design discharge. A roughened channel can help dissipate energy from an adjacent fishway into a newly constructed channel or may be necessary to prevent a channel from degrading over time.

7-5.2.2 Roughened Rock Ramp Design Methodology

Roughened rock ramps are similar to roughened channels except a roughened rock ramp uses large boulders to dissipate energy.

7-5.2.3 Structure Retrofit Design Methodology

An existing structure that currently does not provide fish passage can be authorized to remain in place until the end of its useful life by retrofitting the culvert to make it fish passable. It must be demonstrated that the culvert will comply with WAC 220-660-200(11). It is unlikely that a structure retrofits will be allowed within Water Resource Inventory Areas 1 through 23 due to the culvert injunction.

7-6 Temporary Stream Diversions

Temporary stream diversions shall be designed following the methodology described in Chapter 3 using the flow rates determined by this section. All other temporary culvert designs should follow the requirements of Chapter 3. Under most circumstances, the design and configuration of temporary culverts for streams is left to the Contractor to determine. This allows for the contractor to be able to create the most efficient work plan for their construction method. If the PEO wishes to design the temporary culverts, the reason shall be discussed with the HQ Hydraulics Section, and approval will be required.

For Design-Build Projects, the design and flow rate are determined by the Design-Builder based on the requirements of project permits.

For Design-Bid-Build projects on fish bearing streams, the HQ Hydraulics Section calculates the flow rates necessary for temporary culverts and that value is part of the special provisions. A conceptual level plan is typically required for permits, but no plans for the temporary culvert system should be put into the final plan set and should not be documented in the specialty report, unless otherwise approved.

Temporary culverts for streams shall be designed for the following storm events:

- **Single Season:** For a temporary culvert expected to be in place for a single fish window, the design flow rate shall be, at a minimum, equal to the expected 50 percent exceedance flow rate during the window the temporary culvert is in place with a contingency plan that shall be in place in two hours or less to bring the system to meet the expected 10 percent exceedance flow rate during the window the temporary culvert is in place. Determining the expected flow rates during the window the temporary culvert is in place can be done through Stream Gage Data (if available) or through an MGSFlood Seasonal Flow analysis (western Washington only). The flows can also be measured in the previous fish window years to get a base flow followed by an analysis for a 2-year storm based on rainfall for that fish window. If there is no data to calculate the flows during the construction window, then the expected 2-year flow rate shall be used for the design flow (contingency not necessary in this case) unless the PEO can justify a different flow if approved by the HQ Hydraulics Section.
- **Multiple Season:** The flow rate used for a temporary stream bypass expected to remain in place through a winter is to be the 10-year flow event as determined by the same hydrologic methodologies as for the 2-year event.

The design flood for temporary structures over water bodies shall be determined on a case-by-case basis by the HQ Hydraulics Section.

7-7 Monitoring

In September 2015, as part of the U.S. v. WA culvert injunction, state agencies and Tribal nations agreed upon and finalized a set of Monitoring Implementation Guidelines. Those guidelines are the basis of WSDOT's current fish passage monitoring plan. Some elements of the monitoring plan apply to all statewide fish passage projects, not just those within the case area. Some projects have monitoring requirements as part of a state or federal permit. The monitoring plan, based on the agreed upon guidelines, provides protocols that can be applied to those special monitoring requirements and will ensure a consistent and efficient process.

There are three basic types of monitoring inspections:

Post-Construction Compliance Inspection - WSDOT evaluates all fish passage projects to ensure they are constructed as designed and permitted. Sites are also evaluated for their ability to pass fish using WDFW barrier assessment methods.

Overwinter Inspection – WSDOT inspects sites corrected under the injunction after the first full winter to evaluate the impact of high seasonal flows on fish passage at the new structure.

Long-Term Evaluations - Sites corrected under the injunction are evaluated 5 and 10 years after construction to determine if they still provide fish passage and to determine if the structures still conform to the fish passage standards under which they were constructed.

The results of the monitoring effort are summarized each year in the Fish Passage Annual Report, which can be found on the WSDOT Fish Passage Website (www.wsdot.wa.gov/Projects/FishPassage/default.htm). WSDOT uses the information from the monitoring efforts to work with WDFW and Tribes to improve upon the design and construction processes and will update this chapter as needed to reflect current practices and best available science.

7-8 Additional Resources

The Stream Designer may find the following manuals helpful for additional information:

- HEC-17: Highways in the River Environment - Floodplains, Extreme Events, Risk, and Resilience
- HEC-18: Evaluating Scour at Bridges
- HEC-20: Stream Stability at Highway Structures Fourth Edition
- HEC-23: Bridge Scour and Stream Instability Countermeasures Experience, Selection, and Design Guidance Third Edition, Volumes 1 and 2
- HEC-25: Highways in the Coastal Environment, Volumes 1 and 2
- 2013 WDFW WCDG
- 2008 USFS Manual Stream Simulation: An Ecological Approach to Providing Passage for Aquatic Organisms at Road-Stream Crossing
- WDFW ISPG

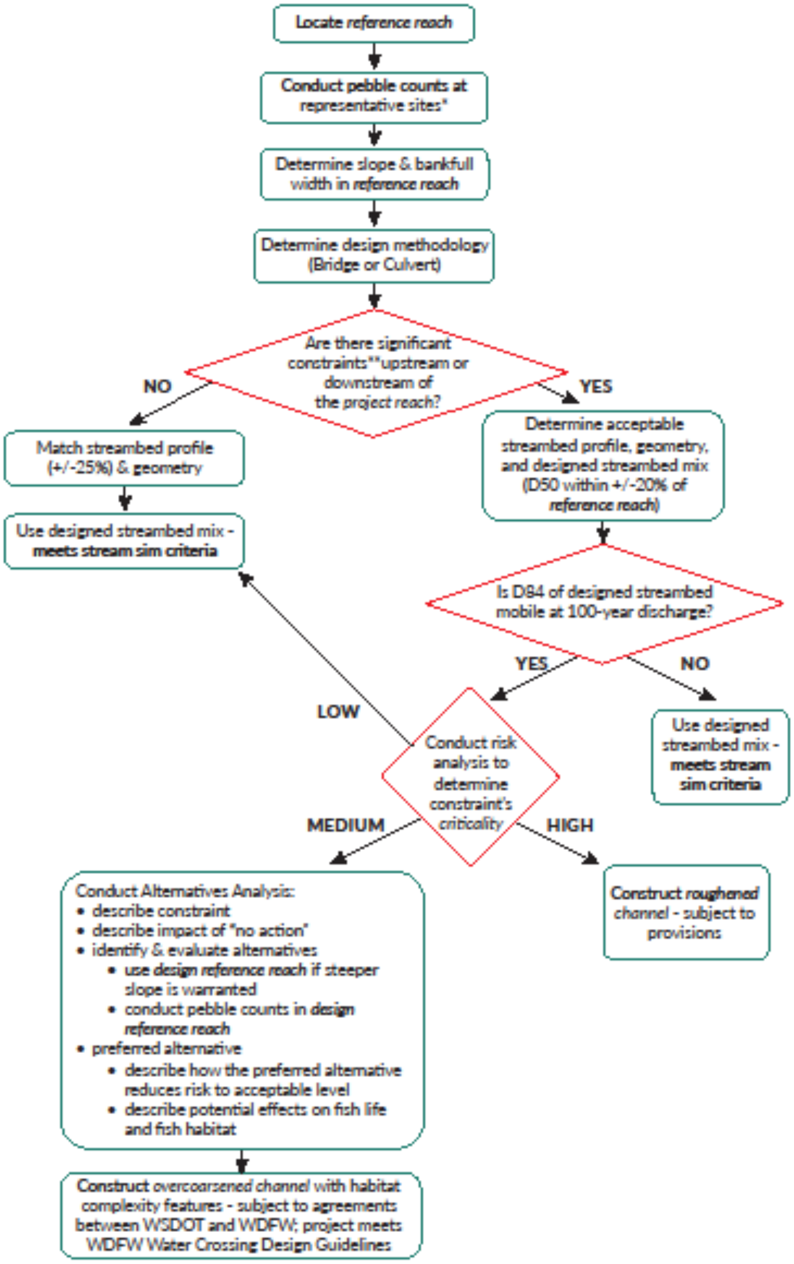
7-9 Appendices

Appendix 7A	Streambed Material Decision Tree
Appendix 7B	Design Methodology Requirements for Bridges and Stream Simulation Culverts

WSDOT

OCTOBER 2016

Streambed Material Decision Tree



This document is intended to guide fish passage restoration design in cases where there are site constraints that are either too costly to resolve, or would take too long to resolve. In these cases, the regraded reach may be steeper than the initially identified reference reach. The reach assessment is an essential part of the process, but this document's scope is limited to the decisions that affect the design of streambed materials which may be larger than what would normally be indicated by stream simulation-based design.

* Select representative sites that:
- are away from recent or chronic sediment sources
- are in riffles
- avoid pools
- outside of the influence of man-made or natural structures such as LWM, eroding banks

** Constraints on channel regrading (including but not limited to):
- upstream/downstream infrastructure: culverts/bridges, pipelines, etc.
- water intakes/diversions
- groundwater wells
- upstream/downstream wetlands/habitat
- non-WSDOT property
- roadway geometry

DEFINITIONS

Criticality: the combination of probability of an effect to a structure, and the level of impact that the effect would have; see example chart on back.

Designed streambed mix: sediment size distribution that uses pebble counts from the reference reach for the D50 and D84, and an even, designed distribution of sizes for finer classes (USFS, 2008)

Design reference reach: is a reach of stream, preferably within the same watershed, that is relatively stable

Overcoarsened Channel: a constructed channel with a median particle size that is greater than 20% larger than the median particle size of the design reference reach; is deformable at discharges below the 100-year discharge.

Project reach: the segment of stream in which the fish passage is located

Reference reach: A stable segment of stream with consistent slope, geometry, planform, and sediment load that represents, to the best available knowledge, background condition of the project reach. (Rosgen, D.H., 1989)

Roughened Channel: a constructed channel with streambed material and configuration designed to be non-deformable up to the design discharge

Stable stream: A stream, over time (in the present climate), that transports the flows and sediment produced by its watershed in such a manner that the dimension, pattern and profile are maintained without either aggrading, nor degrading (Rosgen, 1996)

		Impact			
		Minor	Moderate	Major	Extreme
Risk	Unlikely	Low	Low	Medium	Medium
	Moderate	Low	Medium	Medium	High
	Likely	Medium	Medium	High	High
	Very likely	Medium	High	High	High

Table 1. Criticality Matrix. Risk is the probability of an effect on a constraint. Impact is the level of effect that damage to the constraint would have.

REFERENCES

Barnard, R. J., J. Johnson, P. Brooks, K. M. Bates, B. Heiner, J. P. Klavas, D.C. Ponder, P.D. Smith, and P. D. Powers (2013), Water Crossings Design Guidelines, Washington Department of Fish and Wildlife, Olympia, Washington.

Rosgen, D.H., 1989. The Reference Reach: A Blueprint for Natural Channel Design. Engineering Approaches to Ecosystem Restoration: pp. 1009-1016.

Rosgen, D.L. (1996). Applied River Morphology. Wildland Hydrology Books, Pagosa Springs, Colo

U.S. Forest Service Stream-Simulation Working Group, 2008. STREAM SIMULATION: An Ecological Approach To Providing Passage for Aquatic Organisms at Road-Stream Crossings, National Technology and Development Program, San Dimas, CA 91773.

WSDOT

BRIDGE AND LARGE BOTTOMLESS CULVERT DESIGN METHODOLOGY

Stream Crossing Element	Goals	Summary of relevant Washington Administrative Code (WAC)	Summary of relevant WDFW Water Crossing Guidelines	Supplemental Guidance
Bankfull/Bed Width	Determine accurate bankfull width relative to site conditions. Design teams will reach agreement in the field where possible. If hydraulic modeling is necessary, meet after to discuss results.	A person must measure at least 3 widths that describe prevailing conditions at straight channel sections and outside the influence of any culvert, bridge, or other artificial or unique channel constriction [220-660-190(3)(e)]	Page 222–243. (Appendix C) Provides recommended methods to determine bankfull width.	Bankfull in highly modified (urban/agricultural) determined by hydraulic modelling, reference reach or comparative analysis. See also WSDOT Hydraulic Manual, Chapter 7 Fish Passage.
Channel Slope/ Gradient	For culvert, the slope of the bed in the project reach is within 25% of the prevailing slope upstream and downstream.	The slope of the bed inside a stream-simulation culvert must not exceed the slope of the upstream channel by more than twenty-five percent. [220-660-190(6)(a)(iv)] If the channel is heavily degraded, the slope should be that of a stable channel that would fit within the geomorphic context of the reach. [220-660-190(3)(c)(ii)]	Page 87. If channel is considered unconfined, channel gradient is indirectly accounted for in the velocity ratio. Where the velocity ratio is defined by the average velocity within the main channel of the proposed crossing divided by the average velocity in the main channel of the unobstructed river channel. For confined channels, there is no guidance for acceptable channel gradients.	Slope ratio greater than 1.25 or more than 1' of uncontrolled regrade needs formal reach analysis. In low gradient systems provide explanation of analysis if gradient is outside stream ratio. See also WSDOT Hydraulic Manual, Chapter 7 Fish Passage.
Countersink/ Scour	Culvert bottom or bridge foundation does not become exposed for life of structure and substrate size is similar to adjacent channel.	The bridge design must minimize the need for scour protection. Where mid-channel piers are necessary, design them so no additional scour protection is required. If scour protection is unavoidable, the design must minimize the scour protection to the amount needed to protect piers and abutments. The design must specify the size and placement of the scour protection so it withstands expected peak flows. [220-660-190 (4)(g)]	Page 70–72. Follow AASHTO and FHWA Guidelines. Prevent or limit local scour and coarsening of the stream substrate.	WSDOT designs bridges for the 500-year event. See also WSDOT Hydraulic Manual, Chapter 7 Fish Passage.
Channel Geometry	Continuity of channel planform and cross-section maintained throughout reach; address channel complexity, large woody material, and meander bends.	Must design water crossing structures in fish-bearing streams to allow fish to move freely through them at all flows when fish are expected to move. All water crossings must retain upstream and downstream connection in order to maintain expected channel processes. These processes include the movement and distribution of wood and sediment and shifting channel patterns. Water crossings that are too small in relation to the stream can block or alter these processes, although some encroachment of the flood plain and channel migration zone will be approved when it can be shown that such encroachment has minimal impacts to fish life and habitat that supports fish life. [WAC 220-660-190(2)(a)]	Page 72–73. The stream channel created or restored near the bridge should have a gradient and cross-section similar to the existing morphology upstream and downstream adjacent channel.	See also WSDOT Hydraulic Manual, Chapter 7 Fish Passage.
Floodplain Continuity	Constructed channel mimics adjacent floodplain habitat conditions and allows for floodplain connectivity.	All water crossings must retain upstream and downstream connection in order to maintain expected channel processes. These processes include the movement and distribution of wood and sediment and shifting channel patterns. Some encroach is allowed as long as proven to have minimal impacts to fish life and habitat [220-660-190(2)(a)]. A bridge over a watercourse with an active flood plain must be designed to prevent a significant increase in the main channel average velocity. The bridge is defined as the main bridge span(s) plus flood plain relief structures and approach road overtopping. This velocity must be determined at the 100-year flood flow or the design flood flow approved by the department. The significance threshold should be determined by considering bed coarsening, scour, backwater, flood plain flow, and related biological and geomorphological effects typically evaluated in a reach analysis. [220-660-190(4)(c)]	Page 70–72, 78–89. Allow continued down-valley flow of water on the floodplain. The bridge/culvert design must comply with legislation governing development within floodplains.	If the ratio V2/V1 is less than 1.1, no additional justification is needed. If V2/V1 is greater than 1.1, must explain how there is no significant effect. V2 = design velocity V1 = existing velocity
Freeboard	Maintain structural integrity of the crossing for the duration of the design life.	The design must have at least three feet of clearance between the bottom of the bridge structure and the water surface at the 100-year peak flow unless engineering justification shows a lower clearance will allow the free passage of anticipated debris. [220-660-190 (4) (f)]	Page 15, 81. Culverts shall be installed to an approved design to maintain structural integrity to the 100-year peak flow with consideration of the debris loading likely to be encountered. A table of estimating clearance is provided, although are not based on hydraulic modeling or empirical studies and therefore should be used caution.	See also WSDOT Hydraulic Manual, Chapter 7 Fish Passage. Additional justification possible when recommended freeboard is not achievable.
Substrate	Channel substrate mimics reference reach.	The water crossing design must provide unimpeded passage for all species of adult and juvenile fishes. Passage is assumed when there are no barriers due to behavioral impediments, excessive water slope, drop or velocity, shallow flow, lack of surface flow, uncharacteristically coarse bed material, and other related conditions. [220-660-190(2)(a)], [220-660-190(3)(a)]	Page 44–52, 80. A reference reach approach to sizing sediment is preferred. Substrate should be designed to address bed stability at high flows and must be well-graded to prevent loss of significant surface flow.	See also WSDOT Hydraulic Manual, Chapter 7 Fish Passage.
Structure Span	Crossing width (span) allows for geomorphic processes to occur including 100 year flood flows; Minimize the need for scour protection; maintain structural integrity for the duration of the design life.	The bridge must pass water, ice, large wood and associated woody material, and sediment likely to move under the bridge during the 100-year flood flows or the design flood flow approved by the department. The waterward face of all bridge elements must be landward of the Ordinary High Water Line (OHWL), except for mid-channel piers and protection required at the toe of embankment in confined channels. The span must be sized to prevent a significant increase in the main channel average velocity. The significance threshold should be determined by considering bed coarsening, scour, backwater, flood plain flow, and related biological and geomorphological effects. The span must account for channel migration during the bridge's lifespan. If there are levees or other infrastructure that constrains bridge design, WDFW may approve a shorter bridge span than would otherwise be required. [220-660-190(4)]	Page 70, 83–90 Existing bridges with a good performance rating can be replaced in kind. Confined channels, distance between bridge abutments should be bankfull width plus a safety factor. Unconfined channels with flood plain and overbank flow should be designed such that the velocity in the main channel under the bridge should be close to the prevailing velocity in the main channel of the river.	See also WSDOT Hydraulic Manual, Chapter 7 Fish Passage.
Coarse Bands	For culverts: help form structure and maintain gradient; to keep the thalweg off the culvert wall, not intended to be grade control or rigid structures that do not deform over time.	N/A	Page 42–46 Helps form channel complexity where LWM can't be installed. Less important in steep streams.	Use for channel slopes less than 2%. Evaluate for slopes between 2% to 4%. Use material between the D84 and two times the D100 of the streambed design mix.
Crossing Length	Minimize confined length of channel and riparian impacts, increase width for long crossings. Skew also needs to be considered - crossing should use skew to avoid abrupt bends leading to the culvert inlet and from the culvert outlet.	N/A	Page 70, 83–84.	See also WSDOT Hydraulic Manual, Chapter 7 Fish Passage.
Floodplain Utilization Ratio (FUR)	To determine if unconfined bridge design criteria are adequate for the bridge or buried structure.	N/A	N/A	Measure FUR outside the influence of any crossing structures.
Streambank Protection/ stabilization	Minimize armoring (use of rip rap or concrete) and use bio-engineering techniques where appropriate.	Any proposed bank hardening must include: (i) An analysis performed by a qualified professional assessing the level of risk to existing buildings, roads, or services being threatened by the erosion; (ii) Technical rationale specific to the project design, such as a reach and site assessment (iii) Evidence of erosion and/or slope instability to warrant the work. Any bank hardening must protect fish life and habitat by using the least-impacting technically feasible alternative. The common alternatives below are in order from most to the least preferred: (i) No action – Natural channel processes to occur; (ii) Biotechnical techniques; (iii) Combination of biotechnical and structural techniques; and (iv) Structural techniques Streambank stabilization should be limited to the least amount needed to protect eroding banks. The project must be designed to withstand the maximum selected design flow. Use natural materials whenever feasible, including large wood and vegetation; protect existing spawning and rearing habitat. [WAC 220-660-130]	N/A	See Integrated Streambank Protection Guidelines
Hydrology	Correlate to watershed conditions and land use, while avoiding over-engineered channels and banks.	N/A	Page 101. Page 282-287 (Appendix G) Design Flows for Fish Passage	Address potential effects of extreme events (e.g., 500-year); use of best available science.

Washington Department of Fish and Wildlife, 2016, Incorporating Climate Change into Design of Water Crossing Structures

WSDOT

STREAM SIMULATION CULVERT DESIGN METHODOLOGY

Stream Crossing Element	Goals	Summary of relevant Washington Administrative Code (WAC)	Summary of relevant WDFW Water Crossing Guidelines	Supplemental Guidance
Bankfull/Bed Width	Determine accurate bankfull width relative to site conditions. Design teams will reach agreement in the field where possible. If hydraulic modeling is necessary, meet after to discuss results.	A person must measure at least 3 widths that describe prevailing conditions at straight channel sections and outside the influence of any culvert, bridge, or other artificial or unique channel constriction. [220-660-190(3)(e)]	Page 222–243. (Appendix C) Provides recommended methods to determine bankfull width.	Bankfull in highly modified (urban/agricultural) determined by hydraulic modelling, reference reach or comparative analysis. See also WSDOT Hydraulic Manual, Chapter 7 Fish Passage.
Culvert Gradient	The culvert is set at an elevation below total scour and provides adequate freeboard.	The culvert gradient must be set at the prevailing stream gradient unless an alternative slope is approved by the department. [220-660-190 (6)(a)(iii)]	p. 34. While no specific minimum slope ratio is suggested, the goal is to place the bed in the culvert at the same gradient as the stream – not to over- or under-steepen it.	In cases where placing the culvert at the same gradient as the stream would cause constructability issues, placing the culvert at a zero slope is acceptable as aslong as the necessary embedment depth adn freeboard are met and the engineering justification is provided.
Channel Slope/ Gradient	The slope of the bed inside the culvert is within 25% of the slope of the upstream channel.	The slope of the bed inside a stream-simulation culvert must not exceed the slope of the upstream channel by more than twenty-five percent. [220-660-190(6)(a)(iv)] If the channel is heavily degraded, the slope should be that of a stable channel that would fit within the geomorphic context of the reach. [220-660-190(3)(c)(ii)]	Page 32–34. The slope of the bed inside a stream-simulation culvert must not exceed the slope of the upstream channel by more than twenty-five percent. ($S_{culvert}/S_{upstream\ ch}$) < 1.25	Slope ratio greater than 1.25 or more than 1' of uncontrolled regrade needs formal reach analysis. In low gradient systems, provide explanation if designed gradient is outside slope ratio. See also WSDOT Hydraulic Manual, Chapter 7 Fish Passage.
Channel Geometry	Continuity of channel shape maintained throughout reach [channel complexity].	All water crossings must retain upstream and downstream connection in order to maintain expected channel processes. WAC 220-660-190(2)(a)	Page 172–73. The natural channel cross-section and the cross-section constructed through the crossing should be the same (at least up to bank full) so that material that is moving in the natural channel will also pass through the constructed channel in the crossing.	See WSDOT Hydraulics Manual, Chapter 7 supplement
Countersink/ Scour	Culvert bottom does not become exposed for life of structure and substrate size is similar to adjacent channel.	Must be countersunk a minimum of 30% and a maximum of 50% of the culvert rise, but not less than two feet. Alternative depths of culvert fill may be accepted with engineering justification [220-660-190 (6)(a)(v)].	Page 207. 30% to 50%, not less than 2 feet unless justified by analysis.	WSDOT uses 100-year scour depth plus 2 feet. See WSDOT Hydraulics Manual, Chapter 7 supplement.
Cross Section	Adjacent channel shape is continuous through crossing.	If the channel is heavily degraded, the cross section must match expected stream measurements in order to limit main channel velocity and scour to prevailing conditions. [220-660-190(3)(c)(ii)]	Page 37–43, 53–64, 207–208. Bed cross section should be similar to the adjacent stream cross section.	
Floodplain Continuity	Constructed channel mimics adjacent channel habitat conditions.	Fish must be able to move freely at all flows when fish are expected to move. All water crossings must retain upstream and downstream channel processes. Floodplain encroachments may be approved if it can be shown that there are minimal impacts to fish life and habitat [220-660-190 (2) (a)]	N/A	
Freeboard	Crossing provides unimpeded passage of fish, 100-year flood flows, LWD, and sediment.	N/A	Page 15. Culverts shall be installed to an approved design to maintain structural integrity to the 100-year peak flow with consideration of the debris loading likely to be encountered. A table of estimating clearance is provided, although are not based on hydraulic modeling or empirical studies and therefore should be used caution.	See also WSDOT Hydraulic Manual, Chapter 7 Fish Passage.
Substrate	Channel substrate mimics reference reach.	D50 must be +/- 20% of the D50 of the reference reach. The department may approve exceptions if the proposed alternative sediment is appropriate for the circumstances. [220-660-190 (6) (vi)].	Page 44–52. A reference reach approach to sizing sediment is preferred. Substrate should be designed to address bed stability at high flows and must be well-graded to prevent loss of significant surface flow.	Streambed Material Decision Tree & WSDOT Hydraulic Manual, Ch. 7
Culvert size	Culvert opening should be wide enough to maintain water and sediment transport continuity.	Bed width inside a culvert may be calculated by using any published stream simulation design methodology approved by the department, or may be determined on a case-by-case basis with an approved alternative plan that includes project objectives, inspection, maintenance, and contingency components. [220-660-190 (6)(a)(ii)]	Page 37–40. Typically culvert bed is, 1.2*BFW+2 (in alluvial systems), note examples of exceptions for deviating. The structure span should span the calculated bed width.	
Coarse Bands	Help form structure and maintain gradient; to keep the thalweg off the culvert wall, not intended to be grade control or rigid structures that do not deform over time.	N/A	Page 29, 42–44. Coarse bands should be used on channel slopes less than 4%.	Use for channel slopes less than 2%. Evaluate for slopes between 2% to 4%. Use material between the D84 and two times the D100 of the streambed design mix.
Crossing Length	Minimize confined length of channel and riparian impacts, increase width for long crossings. Skew also needs to be considered - crossing should use skew to avoid abrupt bends leading to the culvert inlet and from the culvert outlet.	N/A	Page 70, 83–84. Bridge length in this section should be referred as "bridge span." See "structure span" in cell H13.	See also WSDOT Hydraulic Manual, Chapter 7 Fish Passage.
Floodplain Utilization Ratio (FUR)	Determine if a channel is confined (FUR < 3) or unconfined (FUR > 3). Look for frequent out of bank flows and/or high flows away from channel.	N/A	Page 19, 36, 75. FUR < 3 indicates a confined channel where a culvert is better suited. FUR is defined as the flood-prone (FPW) width divided by the bankfull width (BFW).	When FUR>3, use unconfined bridge method for minimum channel span. Measure FUR outside the influence of any crossing structures.
Streambank Protection/ stabilization	Minimize armoring (use of rip rap or concrete) and use bio-engineering techniques where appropriate.	Any proposed bank hardening must include: (i) An analysis performed by a qualified professional assessing the level of risk to existing buildings, roads, or services being threatened by the erosion; (ii) Technical rationale specific to the project design, such as a reach and site assessment (iii) Evidence of erosion and/or slope instability to warrant the work. Any bank hardening must protect fish life and habitat by using the least-impacting technically feasible alternative. The common alternatives below are in order from most to the least preferred: (i) No action – Natural channel processes to occur; (ii) Biotechnical techniques; (iii) Combination of biotechnical and structural techniques; and (iv) Structural techniques - Streambank stabilization should be limited to the least amount needed to protect eroding banks. The project must be designed to withstand the maximum selected design flow. Use natural materials whenever feasible, including large wood and vegetation; protect existing spawning and rearing habitat. [WAC 220-660-130]	N/A	
Hydrology/ Design Flows	Develop design flows that accurately reflect watershed conditions, including future conditions	N/A	Page 101. Page 282–287 (Appendix G) Design Flows for Fish Passage	Address potential effects of extreme events (e.g., 500-year); use of best available science.

Washington Department of Fish and Wildlife, 2016, Incorporating Climate Change into Design of Water Crossing Structures

19-01-0038

Chapter 8 Pipe Classifications and Materials

8-1 Introduction

WSDOT utilizes a number of different types of pipe for highway construction activities. To simplify contract plan and specification preparation, pipes have been grouped into five primary categories: drain pipe, underdrain pipe, culvert pipe, storm sewer pipe, and sanitary sewer pipe. Each category is intended to serve specific purposes and is described further in Section 8-2.

Within each pipe classifications there are several types of pipe materials, each with unique characteristics used in different conditions. Pipe material selection includes hydraulic characteristics, site conditions, geologic conditions, corrosion resistance, safety considerations, and cost. Section 8-3 provides a detailed discussion of the different pipe materials that are generally used in WSDOT design.

The type of material that is appropriate for a project is dependent on several factors including, but not limited to, pipe strength and corrosion and abrasion potential (Sections 8-4, 8-5, and 8-6); fill height (Section 8-12); the required pipe size, debris passage, and necessary end treatments (Chapter 3); and ease of fish passage (Chapter 7). Except for sizing the pipe, end treatments, and fish passage, each of these issues is further discussed in this chapter along with guidelines to assist the PEO in selecting the appropriate pipe material for a project site and application (Section 8-4).

This chapter also provides additional information about joining pipe materials (Section 8-7), use of pipe anchors (Section 8-8), acceptable forms of pipe rehabilitation (Section 8-9), design and installation techniques for pipe (Section 8-10), and abandoned pipe guidelines (Section 8-11).

Pipe producers follow specifications (ASTM, AASHTO, American Water Works Association) covering the manufacture of pipes and parameters such as cell class, material strength, internal diameter, loadings, and wall thickness. When these standards are referenced, the current year standards shall apply.

Pipe materials and installation methods shall conform with WSDOT's *Standard Specifications* and *Standard Plans* whenever possible. Other specifications may be used when the *Standard Specifications* and *Standard Plans* are not applicable.

8-2 Pipe Classifications

This section examines the five primary categories of pipes utilized in WSDOT projects: drain pipe, underdrain pipe, culvert pipe, storm sewer pipe, and sanitary sewer pipe.

8-2.1 *Drain Pipe*

Drain pipe is small-diameter pipe (usually less than 24-inch diameter) used to convey roadway runoff or groundwater away from the roadway profile. Drain pipe is not allowed to cross under the roadway profile and is intended for use in easily accessible locations should it become necessary to maintain or replace the pipe. The minimum design life expectancy is 25 years and no protective treatment is required.

Typical drain pipe applications include simple slope drains and small diameter “tight lines” used to connect underdrain pipe to storm sewers. Slope drains generally consist of one or two inlets with a pipe conveying roadway runoff down a fill slope. These drain pipes are relatively easy to install and are often replaced when roadway widening or embankment slope grading occurs. Slope drains are most critical during the first few years after installation, until the slope embankment and vegetation have had a chance to stabilize.

Drain pipe smaller than 12 inches in diameter can withstand fill heights of 30 feet or more without experiencing structural failure. All of the materials listed in WSDOT’s *Standard Specifications* are adequate under these conditions. For drain pipe applications utilizing pipe diameters 12 inches or larger, or with fill heights greater than 30 feet, the PEO shall specify only those materials listed in both the *Standard Specifications* and the fill height tables in Section 8-12.

8-2.2 *Underdrain Pipe*

Underdrain pipe is small diameter perforated pipe intended to intercept groundwater and convey it away from areas such as roadbeds or retaining walls. Typical underdrain applications utilize 6- to 8-inch-diameter pipe, but larger diameters can be specified. The minimum design life expectancy is 25 years, and no protective treatment is required. The *Standard Specifications* lists applicable materials for underdrain pipe.

Underdrain pipe is generally used in conjunction with well-draining backfill material and a construction geotextile. Details regarding the various applications of underdrain pipe are described in WSDOT’s *Design Manual*, the WSDOT CADD Detail Library, and the *Standard Plans*. The hydraulic design of underdrain pipe is discussed in Chapter 6.

8-2.3 *Culvert Pipe*

A culvert is a conduit under a roadway or embankment used to maintain flow from a natural channel or drainage ditch. Culverts are generally more difficult to replace than drain pipe, especially when located under high fills or major highways. Because of this, a minimum design life expectancy of 50 years is required for all culverts. Metal culvert pipes require a protective coating at some locations. Details are described in Section 8-5.3.1.

The maximum and minimum fill heights over a pipe material are provided in Section 8-12. For materials or sizes not provided in Section 8-12, contact the HQ Hydraulics Section or review the *Standard Specifications*.

The hydraulic design of culverts is discussed in Chapter 3. In addition to the hydraulic constraints of a location, the final decision regarding the appropriate culvert size may be governed by fish passage requirements, as discussed in Chapter 7.

Culvert shapes, sizes, and applications can vary substantially from one location to another. Listed below is a discussion of the various types of culverts that may appear on a typical contract.

8-2.3.1 Circular and Schedule Culvert Pipe

Circular culvert pipe measuring 12 to 48 inches in diameter is designated as “schedule pipe” and shall be selected unless a pipe material is excluded for engineering reasons. The pipe schedule table listed in Section 7-02 of the *Standard Specifications*, includes the structurally suitable pipe alternates available for a given culvert diameter and fill height. Additionally, Figures 8-8b, 8-9b, and 8-10b provide the PEO with a list of pipe alternatives and protective treatment depending on the corrosion zone. All schedule pipe shall be installed in accordance with Section 8-10.4.

Schedule culvert pipe shall be specified as “Schedule _____ Culv. Pipe ____ in Diam.” on the contract plan sheets. Schedule pipe must be treated with the same protective coatings as other culvert pipe.

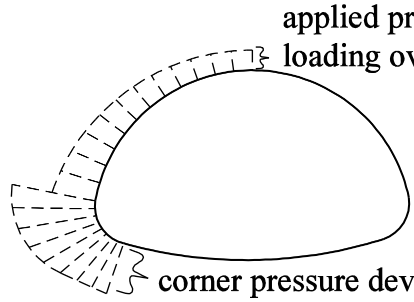
The type of material for circular culvert pipe measuring 54 to 120 inches in diameter shall be designated on the plan sheets. The structure notes sheet should include any acceptable alternate material for that particular installation. A schedule table for these large sizes has not been developed due to their limited use. Also, structural, hydraulic, or aesthetic issues may control the type of material to be used at a site, and a specific design for each type of material available is necessary.

8-2.3.2 Pipe Arches

Pipe arches, sometimes referred to as “squash pipe,” are circular culverts that have been reshaped into a structure with a circular top and a flat, wide bottom. For a given vertical dimension, pipe arches provide a larger hydraulic opening than a circular pipe. This can be useful in situations with minimal vertical clearances. Pipe arches also tend to be more effective than circular pipe in low flow conditions (such as fish passage flows) because pipe arches provide a majority of their hydraulic opening near the bottom of the structure, resulting in lower velocities and more of the main channel being spanned.

The primary disadvantage to using pipe arches is that the fill height range is somewhat limited. Due to the shape of the structure, significant corner pressures are developed in the haunch area as shown in Figure 8-1. The ability of the backfill to withstand the corner pressure near the haunches tends to be the limiting factor in pipe arch design and is demonstrated in the fill height tables shown in Section 8-12.

Figure 8-1 Typical Soil Pressure Surrounding a Pipe Arch



8-2.3.3 Structural Plate Culverts

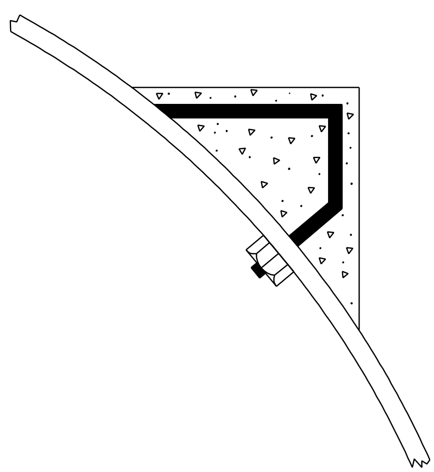
Structural plate culverts are steel or aluminum structures delivered to the project site as unassembled plates of material and bolted together. Structural plate culverts are typically large diameter—from 10 to 40 feet or more—and are available in a number of different shapes including circular, pipe arch, elliptical, and bottomless arch with footings. These structures are designed to span the main channel of a stream and are a viable option when fish passage is a concern.

The material requirements for structural plate culverts are described in the *Standard Specifications*. Aluminum structural plate culverts can be used anywhere in the state, regardless of the corrosion zone. Steel structure plate culverts are not permitted in salt water or Corrosion Zone III, as described in Section 8-4. The protective coatings described in Section 8-5.3.1 shall not be specified for use on these types of culverts because the coatings interfere with the bolted seam process.

To compensate for the lack of protective treatment, structural plate furnished in galvanized steel shall be specified with 1.5 ounces per square foot of galvanized coating on each plate surface (typical galvanized culvert pipe is manufactured with 1 ounce per square foot of galvanized coating on each pipe surface). The PEO of structural plate culverts may also add extra plate thickness to the bottom plates to compensate for corrosion and abrasion in high-risk areas. Increasing the gauge thickness in this manner can provide a service life of 50 years or more for a small cost increase.

Longitudinal or circumferential stiffeners may be added to prevent excessive deflection due to dead and/or live loads on larger structural plate culverts. Circumferential stiffeners are usually metal ribs bolted to the outside of the culvert. Longitudinal stiffeners may be metal or reinforced concrete thrust beams, as shown in Figure 8-2. The thrust beams are added to the structure prior to backfill. Concrete thrust beams provide circumferential and longitudinal stiffening and a solid vertical surface for soil pressures to act on; the solid surface also facilitates backfilling.

Figure 8-2 Concrete Thrust Beams Used as Longitudinal Stiffeners



Another method for diminishing loads placed on large-span culverts is to construct a reinforced concrete distribution slab over the top of the backfill above the culvert. The distribution slab is used in low-cover applications and distributes live loads into the soil column adjacent to the culvert. The HQ Hydraulics Section should be consulted to assist in the design of this type of structure.

8-2.3.4 Private Road Approach and Driveway Culverts

The requirements for culverts placed under private road approaches and driveways are less stringent than the requirements for culverts placed under roadways. Private road approaches and driveway culverts are off of the main line of the highway, so minimal hazard is presented to the traveling public if a failure occurs. Also, it is difficult to provide a minimum 2-foot cover over the top of these culverts. Therefore, private road approaches and driveway culverts can be specified without the protective treatments described in Section 8-5.3.1, and the minimum fill heights listed in Section 8-12 can be reduced to 1 foot (0.3 m). Concrete pipe of the class described in Fill Height Figure 8-16 shall be specified for fill heights less than 1 foot. The PEO shall follow the same recommendations for material and design life as noted in Section 8-2.1.

The PEO is cautioned that structural failure may occur on some private road approaches or driveways if the right combination of fill height, live load, soil conditions, and pipe material are present. If live loads approaching AASHTO HS 25 loading will consistently be traveling over the culvert and if the fill height is less than 2 feet, only pipes meeting the minimum fill height described in Fill Height Figure 8-16 shall be specified.

8-2.3.5 Concrete Box Culverts

Concrete box culverts are generally constructed of precast reinforced concrete, though some older ones may be cast-in-place. They have two configurations—monolithic (one-piece box) and split box. These structures are available in various spans and rises and can be used with varying cover, including no cover. Skew angles can be incorporated into the design and precast wing walls, headwalls, and aprons are available.

All precast box culverts shall be installed in accordance with the manufacturer's recommendations. Design and submittal requirements are listed in the *Standard Specifications*. For extending or new construction of cast-in-place box culverts, contact the HQ Hydraulics Section.

The dimensions and reinforcement requirements for precast box culverts are described by AASHTO. AASHTO M 259 describes precast box culverts with fill heights ranging from 2 to <20 feet. Refer to Section 8-12.2 for additional guidance on the use of concrete structures in shallow cover applications. If a precast box culvert is specified on a contract, the appropriate AASHTO specification should be referenced, along with a statement requiring the contractor to submit engineering calculations demonstrating that the box culvert meets the particular requirements of the AASHTO specification.

8-2.3.6 Three-Sided Concrete Box Culverts

Three-sided concrete box culverts refer to either rectangular or arch-shaped structures that are precast with reinforced concrete. The structures are generally supported by concrete footings, but can be fabricated with a full floor section, if necessary. When footings are used, the footing slope shall not be greater than 4 percent in the direction parallel to the channel.

The structures are well suited for low cover applications where a wide hydraulic opening must be provided. They can be specified with as little as zero cover and span lengths up to <20 feet. It is possible to utilize structures with greater span lengths, but the design for those structures must be coordinated with the Bridge and Structures Office. The structures can be installed quickly, often within one to two days, which can significantly decrease road closures or traffic delays. In addition to the hydraulic opening required, a location must be evaluated for suitability of the foundation material, footing type and size, and scour potential. The HQ Hydraulics Section should be contacted to perform the necessary scour analysis.

8-2.4 *Storm Sewer Pipe*

A storm sewer is defined as two or more inlet structures, connected by pipe for the purpose of collecting pavement drainage. Storm sewers are usually placed under pavement in urbanized areas and, for this reason, are costly to replace. The minimum design life of a storm sewer pipe is 50 years.

The pipe schedule table in the *Standard Specifications* lists all of the structurally suitable pipe alternates available for a given culvert diameter and fill height. Additionally, Figures 8-8b, 8-9b, and 8-10b provide the PEO with a list of pipe alternatives and protective treatments depending on the corrosion zone. All schedule pipe shall be installed in accordance with Section 8-10.4.

All storm sewer pipes, unless indicated otherwise on the plans, must be pressure tested. Pressure testing indicates the presence of leaking seams or joints or other structural failures that may have occurred during the manufacturing or installation of the pipe. The *Standard Specifications* describes three types of pressure tests that are available. The contractor generally has the option of choosing which pressure test to perform. Pressure tests include the following:

- **Exfiltration:** The section of pipe to be tested is filled with water, and an apparatus is connected to the upper end of the pipe so that an additional 6 feet of water column is placed on the test section. The pipe leakage is measured and must be less than the allowable leakage described in the *Standard Specifications*.
- **Infiltration:** This test is intended for situations where the groundwater table is above the crown of the upper end of the pipe test section. Once the pipe has been installed, water leaking into the pipe is collected and measured and must be less than the allowable leakage rate described in the *Standard Specifications*.
- **Low Pressure Air:** The section of pipe to be tested is plugged on both ends and compressed air is added until the pipe reaches a certain pressure. The test consists of measuring the time required for the pressure in the test section to drop approximately 1 psi (7 kilopascals). The measured time must be equal to or greater than the required time described in the *Standard Specifications*.

Metal storm sewer pipe requires the same protective coating to resist corrosion as culvert pipe. In addition, ungasketed helical seam metal pipes may require coatings to enable the pipe to pass one of the pressure tests described above. For example, Treatment 1, as described in Section 8-5.3.1, is needed to satisfy the pressure test for an ungasketed helical lock seam pipe. Gasketed helical lock seams and welded and remetallized seams are tight enough to pass the pressure test without a coating but may still require a coating for corrosion purposes in some areas of the state. Pipe used for storm sewers must be compatible with the structural fill height tables for maximum and minimum amounts of cover shown in Section 8-12.

8-2.5 *Sanitary Sewer Pipe*

Sanitary sewers and side sewers consist of pipes and manholes intended to carry either domestic or industrial sanitary wastewater. Any sanitary sewer work on WSDOT projects will likely consist of replacement or relocation of existing sanitary sewers for a municipal sewer system. Therefore, the pipe materials will be in accordance with the requirements of the local health department, sewer district, and the *Standard Specifications*.

8-3 Pipe Materials

Various types of pipe material are available for each classification described in Section 8-2. Each type of material has unique properties for structural design, corrosion/abrasion resistance, and hydraulic characteristics, which are further discussed in this section to assist the PEO in selecting the appropriate pipe materials.

Several pipe materials are acceptable to WSDOT, depending on the pipe classification (see the *Standard Specifications*). WSDOT's policy is to allow and encourage all schedule pipe alternates that will function properly at a reasonable cost.

If one or more of the schedule pipe alternates at any location are not satisfactory, or if the project has been designed for a specific pipe material, the schedule alternate or alternates shall be so stated on the plans, usually on the structure note sheet. Pipe materials shall conform to the *Hydraulics Manual*, the *Standard Specifications*, and the *Standard Plans*.

Justification for not providing a pipe material, as limited by the allowable fill heights, corrosion zones, soil resistivity, and the limitations of pH for steel and aluminum pipe shall be justified in the hydraulic report (Appendix 1B) and within the PS&E. Cost will not normally be a sufficient reason except in large structures such as box culverts or structural plate pipes. Frequently, structural requirements may have more control over acceptable material than hydraulic requirements.

When drain, culvert, or sewer pipe is being constructed for the benefit of cities or counties as part of the reconstruction of their facilities and they request a certain type of pipe, the PEO may specify a particular type without alternates; however, the city or county must submit a letter stating their justification. Existing culverts should be extended with the same pipe material and no alternates are required.

8-3.1 *Concrete Pipe*

8-3.1.1 Concrete Drain Pipe

Concrete drain pipe is non-reinforced. The strength requirements for concrete drain pipe are less than the strength requirements for other types of concrete pipe. Also, concrete drain pipe can be installed without the use of O-ring gaskets or mortar, which tends to permit water movement into and out of the joints.

8-3.1.2 Concrete Underdrain Pipe

Concrete underdrain pipe is no longer used. Additional guidance will be provided in future revisions to the *Hydraulics Manual*.

8-3.1.3 Concrete Culvert, Storm, and Sanitary Sewer Pipe

Concrete culvert, storm, and sanitary sewer pipe can be either plain or reinforced. Plain concrete pipe does not include steel reinforcing. Reinforced concrete pipe is available in Classes I through V. The amount of reinforcement in the pipe increases as the class designation increases. Correspondingly, the structural capacity of the pipe also increases. Due to its lack of strength, Class I reinforced concrete pipe is rarely used and is not listed in the fill height tables of Section 8-12.

The reinforcement placed in concrete pipe can be either circular or elliptical in shape. Elliptically designed reinforcing steel is positioned for tensile loading near the inside of the barrel at the crown and invert, and at the outside of the barrel at the springline. As shown in

Figure 8-14, a vertical line drawn through the crown and invert is referred to as the minor axis of reinforcement. The minor axis of reinforcement will be clearly marked by the manufacturer; the pipe must be handled and installed with the axis placed in the vertical position.

Concrete joints utilize rubber O-ring gaskets, allowing the pipe to meet the pressure-testing requirements for storm sewer applications. The joints, however, do not have any tensile strength and in some cases can pull apart, as discussed in Section 8-7. For this reason, concrete pipe is not recommended for use on grades over 10 percent without the use of pipe anchors, as discussed in Section 8-8.

Concrete pipe is permitted anywhere in the state, regardless of corrosion zone, pH, or resistivity. It has a smooth interior surface, which gives it a relatively low Manning's roughness coefficient (Appendix 4A). The maximum fill height for concrete pipe is limited to about 30 feet or less. However, concrete pipe is structurally superior for carrying wheel loads with shallow cover. For installations with less than 2 feet of cover, concrete pipe is an acceptable alternative. Fill Height Figure 8-16 lists the class of pipe that should be specified under these conditions.

Concrete is classified as a rigid pipe, which means that applied loads are resisted primarily by the strength of the pipe material, with some additional support given by the strength of the surrounding bedding and backfill. Additional information regarding the structural behavior of rigid pipes is discussed in Section 8-10.3. During the installation process, pipe should be uniformly supported to prevent point load concentrations from occurring along the barrel or at the joints.

Potential difficulties during installation include the weight of concrete pipe and, for sanitary sewer applications, hydrogen sulfide buildup. The PEO shall follow the recommendations of the local sewer district or municipality when deciding if concrete pipe is an acceptable alternate at a given location.

8-3.2 *Metal Pipe – General*

Metal pipe is available in galvanized steel, aluminized steel, or aluminum alloy. All three types of material can be produced with helical corrugations, annular corrugations, or as spiral rib pipe.

Metal pipe is classified as a flexible pipe, which means that applied loads are resisted primarily by the strength of the bedding and backfill surrounding the pipe, with some additional support given by the pipe material itself. Because of the dependence upon bedding strength and backfill material, it is critical that metal pipe be installed in accordance with the requirements of Section 8-10.4 to ensure proper performance.

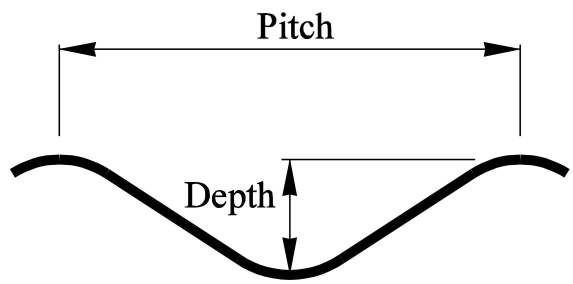
Metal pipe is available in a wide range of sizes and shapes and, depending on the type of material corrugation configuration, can be used with fill heights up to 100 feet (30 m) or more. Metal pipe is susceptible to both corrosion and abrasion; methods for limiting these issues are covered in Sections 8-5.3 and 8-6.

8-3.2.1 Helical Corrugations

Most metal pipe produced today is helically wound, where the corrugations are spiraled along the flow line. The seam for this type of pipe is continuous, and also runs helically along the pipe. The seam can be either an ungasketed lock seam (not pressure testable) or it could be gasketed lock seams (pressure-testable seams). If ungasketed lock seam pipe is used in storm sewer applications, it is generally necessary to coat the pipe with Treatment 1 (Section 8-5.3.1) for the pipe to pass the pressure testing requirements.

Helically wound corrugations are available in several standard sizes, including 2⅔-inch pitch by ½-inch depth, 3-inch by 1-inch, and 5-inch by 1-inch. Corrugation sizes are available in several different gauge thicknesses, depending on the pipe diameter and the fill height. Larger corrugation sizes are utilized as the pipe diameter exceeds about 60 inches. A typical corrugation section is shown in Figure 8-3.

Figure 8-3 Typical Corrugation Section



As a result of the helical manufacturing process, the Manning's roughness coefficient for smaller diameter—24 inches or less—metal pipe approaches the Manning's roughness coefficient for smooth wall pipe materials, such as concrete and thermoplastic pipe. This similarity will generally allow metal pipe to be specified as an alternative to smooth wall pipe without increasing the diameter. However, in situations where small changes in the headwater or head loss through a system are critical, or where the pipe diameter is greater than 24 inches, the PEO shall use the Manning's roughness coefficient specified in Appendix 4A to determine if a larger diameter metal pipe alternate is required.

8-3.2.2 Annular Corrugations

Metal pipe can be produced with annular corrugations, where the corrugations are perpendicular to the flow line of the pipe. The seams for this type of pipe are both circumferential and longitudinal and are joined by rivets. The Manning's roughness coefficient for all annularly corrugated metal pipes is specified in Appendix 4A. The fill heights shown in Section 8-12 apply to both helical and annular corrugated metal pipe.

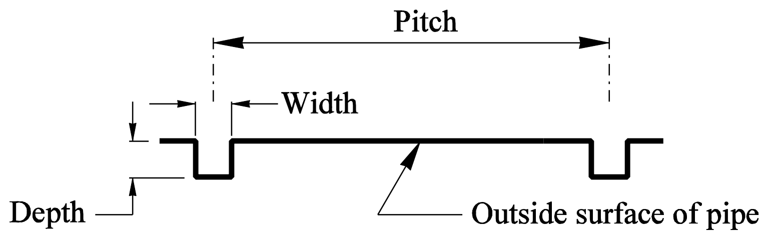
The typical corrugation section shown in Figure 8-3 is the same for annular corrugations, except that annular corrugations are available only in 2⅔-inch by ½-inch and 3-inch by 1-inch sizes.

8-3.2.3 Spiral Rib

Spiral rib pipe utilizes the same manufacturing process as helically wound pipe but, instead of using a standard corrugation pitch and depth, spiral rib pipe is comprised of rectangular ribs between flat wall areas. A typical spiral rib section is shown in Figure 8-4. Two profile configurations are available: ¾-inch width by ¾-inch depth by 7½-inch pitch or 1-inch by 1-inch by 11-inch. The seams for spiral rib pipe are either ungasketed lock seams for non-pressure testable applications or gasketed lock seam for pressure testable applications. If ungasketed lock seam pipe is used in storm sewer applications, it is generally necessary to coat the pipe with protective Treatment 1 (Section 8-5.3.1) for the pipe to pass the pressure-testing requirements.

The primary advantage of spiral rib pipe is that the rectangular rib configuration provides a hydraulically smooth pipe surface for all diameters, with a Manning's roughness coefficient specified in Appendix 4A.

Figure 8-4 Typical Spiral Rib Section



8-3.2.4 Galvanized Steel

Galvanized steel consists of corrugated or spiral rib steel pipe with 1 oz.ft^2 of galvanized coating on each surface of the pipe. Plain galvanized steel pipe is the least durable pipe from a corrosion standpoint and is not permitted when the pH is less than 5 or greater than 8.5 or if the soil resistivity is less than 1,000 ohm-cm. Galvanized steel pipe will, however, meet the required 50-year life expectancy for culvert and storm sewers installed in Corrosion Zone I, as described in Section 8-4. In more corrosive environments, such as Corrosion Zone II or III described in Section 8-4, galvanized steel pipe must be treated with a protective coating for the pipe to attain the required 50-year service life.

8-3.2.5 Aluminized Steel

Aluminized steel consists of corrugated or spiral rib steel pipe with an aluminum protective coating applied both inside and out. The aluminized coating is more resistant to corrosion than galvanized steel pipe and is considered to meet the 50-year life expectancy in both Corrosion Zone I and II without the use of protective coatings. Aluminized steel is not permitted when the pH is less than 5 or greater than 8.5 or if the soil resistivity is less than 1,000 ohm-cm.

8-3.2.6 Aluminum Alloy

Aluminum alloy (aluminum) consists of corrugated or spiral rib pipe and has been shown to be more resistant to corrosion than either galvanized or aluminized steel. When aluminum is exposed to water and air, an oxide layer forms on the metal surface, creating a barrier between the corrosive environment and the pipe surface. As long as this barrier is allowed to form, and is not disturbed once it forms, aluminum pipe will function well.

Aluminum meets the 50-year life expectancy for both Corrosion Zone I and II. It can also be used in Corrosion Zone III, provided that the pH is between 4 and 9; the resistivity is 500 ohm-cm or greater; and the pipe is backfilled with clean, well-draining, granular material. The backfill specified in Section 8-10.4 will meet this requirement.

Aluminum is not recommended when backfill material has a high clay content, because the backfill material can prevent oxygen from getting to the pipe surface and consequently, the protective oxide layer will not form. For the same reason, it is generally not recommended that aluminum pipe be coated with the protective treatments discussed in Section 8-5.3.1.

8-3.2.7 Ductile Iron Pipe

Ductile iron pipe is an extremely strong, durable pipe primarily designed for use in high-pressure water distribution and sanitary sewer systems. Ductile iron pipe is acceptable for culvert and storm sewers use; it is more expensive but is useful for shallow cover and deep installations. Ductile iron pipe is acceptable with as little as 0.5 feet of cover in most installation. Deep fill heights are available from manufactures and concurrence with the HQ Hydraulics Section. Joints systems for Ductile iron pipe include push-on, mechanical, or flanged. Depending on the type of joint, the pipe may be plain end, grooved, or flanged.

8-3.3 *Thermoplastic Pipe – General*

Thermoplastic is a term used to describe a number of different types of pipes including corrugated polyethylene (PE), high-density polyethylene (HDPE), polypropylene (PP), and polyvinyl chloride (PVC). These pipes are allowed for use in drain, underdrain, culvert, storm sewer, and sanitary sewer applications, although not all types of thermoplastic pipe are allowed for use in all applications. The PEO must reference the appropriate section of the *Standard Specifications* to determine the allowable thermoplastic pipe for a given application.

Thermoplastic pipe is classified as a flexible pipe, which means that applied loads are resisted primarily by the strength of the bedding and backfill surrounding the pipe, with some additional support given by the pipe material itself. Because of the dependence upon the strength of the bedding and backfill material, it is critical that thermoplastic pipe be installed in accordance with the requirements of Section 8-10.4 to ensure proper performance.

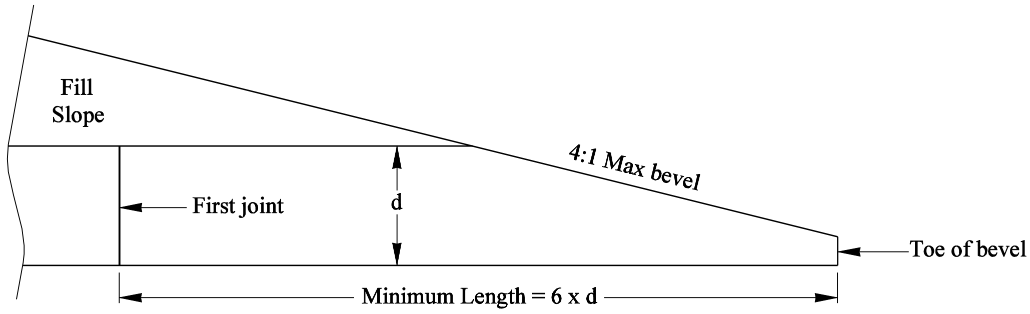
The physical properties of thermoplastic pipe are such that the pipe is resistant to both pH and resistivity. As a result, thermoplastic pipe is an acceptable alternative in all three corrosion zones statewide, and no protective treatment is required. Laboratory testing indicates that the resistance of thermoplastic pipe to abrasive bed loads is equal to or greater than that of other types of pipe material. However, because thermoplastic pipe cannot be structurally reinforced, it is not recommended for severely abrasive conditions as described in Figure 8-11.

The weight of thermoplastic pipe is light when compared to other pipe alternatives. This can simplify pipe handling because large equipment may not be necessary during installation. However, the light pipe weight can lead to soil or water flotation problems in the trench, requiring additional effort to secure the line and grade of the pipe. The allowable fill height and diameter range for thermoplastic pipe is somewhat limited. This may preclude thermoplastic pipe being specified for use in some situations.

Any exposed end of thermoplastic pipe used for culvert or storm sewer applications shall be beveled to match the surrounding embankment or ditch slope. The ends shall be beveled no flatter than 4H:1V, as a loss of structural integrity tends to occur after that point. It also becomes difficult to adequately secure the end of the pipe to the ground.

The minimum length of a section of beveled pipe shall be at least 6 times the diameter of the pipe, measured from the toe of the bevel to the first joint under the fill slope (see Figure 8-5). This distance into the fill slope will provide enough cover over the top of the pipe to counteract typical hydraulic uplift forces that may occur. For thermoplastic pipe 30 inches in diameter and larger, a Standard Plan B-75.20-02 headwall shall be used in conjunction with a beveled end.

Figure 8-5 Minimum Length for Thermoplastic Pipe Beveled Ends



8-3.3.1 Corrugated PE for Drains and Underdrains

Corrugated PE used for drains and underdrains is a single-wall pipe, corrugated inside and outside. It is available in diameters up to 10 inches. This type of pipe is extremely flexible and can be manipulated easily on the job site should it become necessary to bypass obstructions during installation. (See Chapter 3 for treating the exposed end for flotation.)

8-3.3.2 PVC Drain and Underdrain Pipe

PVC drain and underdrain pipe is a solid wall pipe with a smooth interior and exterior. It is available in diameters up to 8 inches. This type of pipe is typically delivered to the job site in 20-foot lengths and has a significant amount of longitudinal beam strength. This characteristic is useful when placing the pipe at a continuous grade but can also make it more difficult to bypass obstructions during installation. (See Chapter 3 for treating the exposed end for flotation.)

8-3.3.3 Corrugated PE Culvert and Storm Sewer Pipe

Corrugated PE used for culverts and storm sewers is double-walled, with a corrugated outer wall and a smooth interior. This type of pipe can be used under all state highways, subject to the fill height and diameter limits described in Section 8-12 and the *Standard Specifications*.

The primary difference between PE used for culvert applications and PE used for storm sewer applications is the type of joint specified. In culvert applications, the joint is not completely watertight and may allow an insignificant amount of infiltration. The culvert joint will prevent soils from migrating out of the pipe zone and is intended to be similar in performance to the coupling band and gasket required for metal pipe. If a culvert is to be installed where a combination of a high water table and fine-grained soils near the trench are expected, it is recommended that the joint used for storm sewer applications be specified. The storm sewer joint will eliminate the possibility of soil migration out of the pipe zone and will provide an improved connection between sections of pipe.

In storm sewer applications, all joints must be capable of passing WSDOT's pressure test requirements. Because of this requirement, the allowable pipe diameter for storm sewer applications may possibly be less than the allowable diameter for culvert applications. The PEO shall consult WSDOT's Qualified Products List for the current maximum allowable pipe diameter for both applications. Corrugated PE is a petroleum-based product and may ignite under certain conditions. If maintenance practices such as ditch or field burning are anticipated near the inlet or outlet of a pipe, PE shall not be allowed as a pipe alternate.

8-3.3.4 Solid Wall PVC Culvert, Storm, and Sanitary Sewer Pipe

Solid wall PVC culvert, storm, and sanitary sewer pipe is a solid wall pipe with a smooth interior and exterior. This type of pipe can be used under all state highways, subject to the fill height and diameter limits described in Section 8-12 and the *Standard Specifications*. This type of pipe is used primarily in water line and sanitary sewer applications but may occasionally be used for culverts or storm sewers. The only joint available for this type of PVC pipe is a watertight joint conforming to the requirements of the *Standard Specifications*.

8-3.3.5 Profile Wall PVC Culvert and Storm Sewer Pipe

Profile wall PVC culvert and storm sewer pipe consists of pipe with an essentially smooth waterway wall braced circumferentially or spirally with projections or ribs, as shown in Figure 8-6. The pipe may have an open profile, where the ribs are exposed, or the pipe may have a closed profile, where the ribs are enclosed in an outer wall. This pipe can be used under all state highways, subject to the fill height and diameter limits described in Section 8-12 and the *Standard Specifications*. The only joint available for profile wall PVC culvert and storm sewer pipe is a watertight joint conforming to the requirements of the *Standard Specifications*.

Figure 8-6 Typical Profile Wall PVC Cross Sections

8-3.3.6 Polypropylene Culvert and Storm Sewer Pipe

PP pipe is similar in style to corrugated PE pipe; the difference is in the compounds used to produce the pipe. The pipe is either double walled (corrugated inside and outside) or triple walled (smooth inside and out) with a corrugated inner wall. The joint systems are bell and spigot and are soil and water tight.

The compounds used in this pipe produce a much stiffer profile making it a good choice for storm and sanitary sewer applications where line and grade may be critical. It is also highly resistant to corrosive materials and abrasion. It is costlier than normal corrugated PE pipe.

8-3.3.7 Steel Rib Reinforced Polyethylene Culvert and Storm Sewer Pipe

Steel rib reinforced PE pipe has a fairly thin wall profile; the inner wall is smooth, and the outer wall has ribs that are steel encased in PE. This profile creates a lightweight, strong, corrosion- and abrasion-resistant pipe. Gasketed joints are made by bell and spigot connections in smaller diameters, and a welded or electrofusion joint creates a watertight connection in larger diameters.

8-3.3.8 Solid Wall HDPE

Solid wall HDPE pipe is used primarily for trenchless applications but occasionally this type of pipe is used for specific applications including bridge drainage, drains or outlet locations on very steep slopes, waterline installations, and sanitary sewer lines. Solid wall HDPE pipe is often an economical choice for deep fill applications or shallow cover down to 0.5 feet. This type of pipe is engineered to provide balanced properties for strength, toughness, flexibility, wear resistance, chemical resistance, and durability.

The pipe may be joined using many conventional methods, but the preferred method is by heat fusion. Properly joined, the joints provide a leak proof connection that is as strong as the pipe itself. There are a wide variety of grades and cell classifications for this pipe; contact HQ Hydraulics Section for specific pipe information.

8-4 Pipe Corrosion Zones and Pipe Alternate Selection

Once a PEO has determined the pipe classification needed for an application, the next step is to ensure the pipe durability will extend for the entire design life. Pipe durability can be evaluated by determining the corrosion and abrasion potential of a given site and then choosing the appropriate pipe material and protective treatment for that location.

To simplify this process, Washington State has been divided into three corrosion zones, based upon the general corrosive characteristics of that particular zone. A map delineating the three zones is shown in Figure 8-7. A flow chart and corresponding acceptable pipe alternate list have been developed for each of the corrosion zones and are shown in Figures 8-8a through 8-10b. The flow charts and pipe alternate lists summarize the information discussed in Figure 8-5 related to corrosion, pH, resistivity, and protective treatments and can be used to develop acceptable pipe alternates for a given location.

The flow charts and pipe alternate lists do not account for abrasion, as bed loads moving through pipes can quickly remove asphalt coatings applied for corrosion protection. If abrasion is expected to be significant at a given site, the guidelines discussed in Figure 8-11 shall be followed.

When selecting a pipe alternative, the PEO should consider the degree of difficulty that will be encountered in replacing a pipe at a future date. Drain pipes are relatively shallow and are readily replaced. Culverts tend to have greater depth of cover and pass under the highway alignment making them more difficult to replace. Storm sewers are generally utilized in congested urban areas with significant pavement cover, high traffic use, and a multitude of other buried utilities in the same vicinity. For these reasons, storm sewers are generally considered to be the most expensive and most difficult to replace and should have a long design life.

When special circumstances exist (i.e., extremely high fills or extremely expensive structure excavation) the PEO should use good engineering judgment to justify the cost effectiveness of a more expensive pipe option or a higher standard of protective treatment than is recommended on the figures in this section.

8-4.1 *Corrosion Zone I*

With the exceptions noted below, Corrosion Zone 1 encompasses most of eastern Washington and is considered the least corrosive part of the state. Plain galvanized steel, untreated aluminized steel, aluminum alloy, thermoplastic, and concrete pipe may all be used in Corrosion Zone I. (See Figures 8-8a and 8-8b for a complete listing of acceptable pipe alternates for culvert and storm sewer applications.) Treatment 1, 2, or 5 (Section 8-5.3) is required for all storm sewers if the seams are not pressure testable (ungasketed lock seam).

Parts of eastern Washington that are not within Corrosion Zone I are categorized as Corrosion Zone II:

- Okanogan Valley
- Pend Oreille Valley
- Disautel – Nespelem Vicinity

8-4.2 *Corrosion Zone II*

Most of western Washington, with the exceptions noted below, along with the three areas of eastern Washington identified above make up Corrosion Zone II. This is an area of moderate corrosion activity. Generally, Treatment 2 is the minimum needed to provide corrosion protection for galvanized steel culverts and storm sewers. Untreated aluminized steel, aluminum alloy, thermoplastic, and concrete pipe may be used in Corrosion Zone II. (See Figures 8-9a and 8-9b for a complete listing of acceptable pipe alternates for culvert and storm sewer applications.)

Parts of western Washington that are not within Corrosion Zone II, are placed into Corrosion Zone III:

1. Whatcom County Lowlands, described by the following:
    a. State Route (SR) 542 from its origin in Bellingham to the junction of SR 9.
    b. SR 9 from the junction of SR 542 to the international boundary.
    c. All other roads/areas lying northerly and westerly of the above routes.
2. Lower Nisqually Valley
3. Low-lying roadways in the Puget Sound basin and coastal areas subjected to the influence of saltwater bays, marshes, and tide flats. As a general guideline, this should include areas with elevations less than 20 feet above the average high tide elevation. Along the Pacific coast and the Straits of Juan de Fuca, areas within 300 to 600 feet of the edge of the average high tide can be influenced by salt spray and should be classified as Corrosion Zone III. However, this influence can vary significantly, depending on the roadway elevation and the presence of protective bluffs or vegetation. In these situations, the PEO is encouraged to evaluate existing pipes near the project to determine the most appropriate corrosion zone designation.

8-4.3 *Corrosion Zone III*

The severely corrosive areas identified above make up Corrosion Zone III. Concrete and thermoplastic pipe are allowed for use in this zone without protective treatments. Aluminum alloy is permitted only as described in Section 8-3. (See Figures 8-10a and 8-10b for a complete listing of all acceptable pipe alternates for culvert and storm sewer applications.)

Figure 8-7 Washington State Corrosion Zones

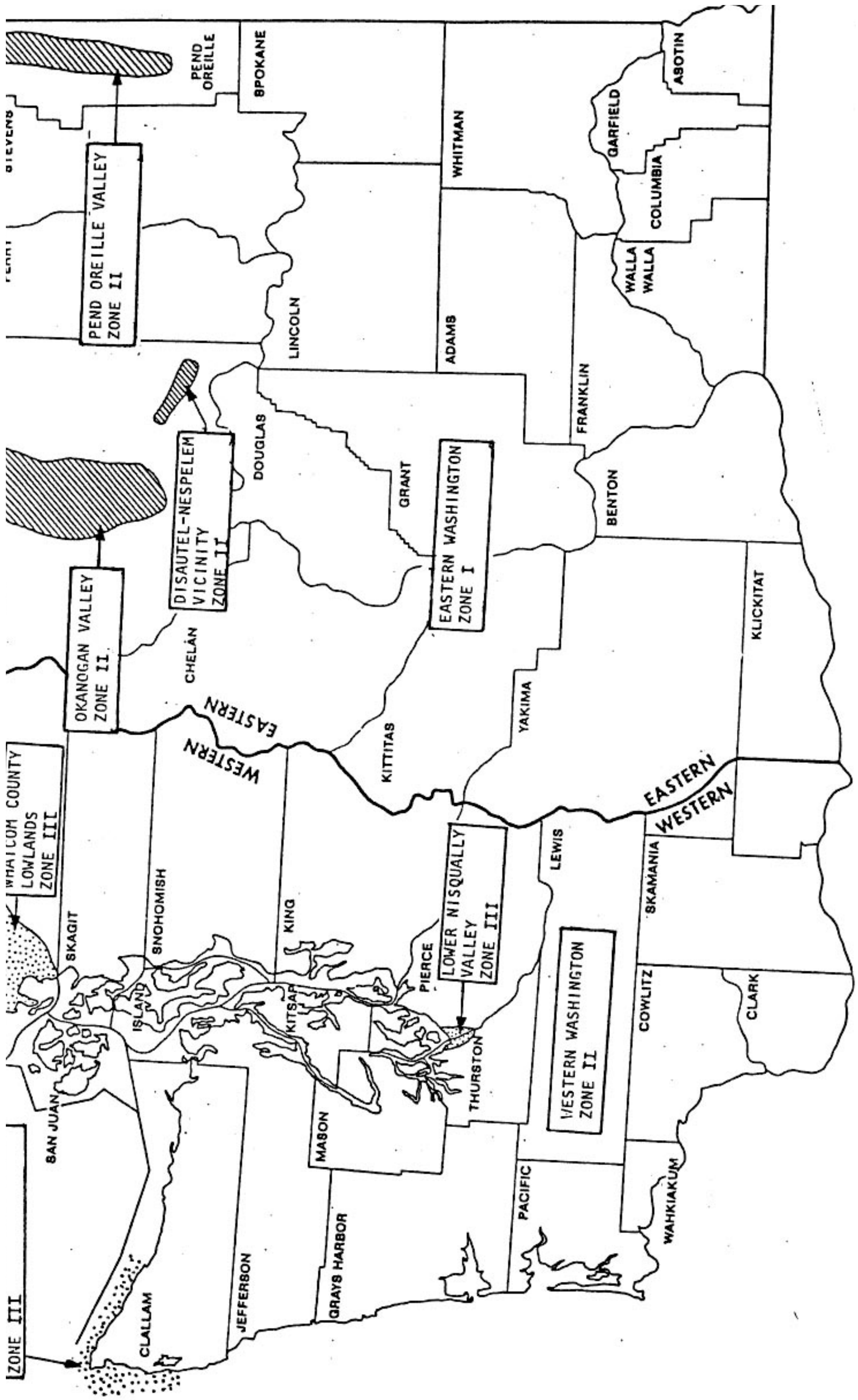


Figure 8-8a Corrosion Zone I:
Flowchart of Acceptable Pipe Alternates and Protective Treatments

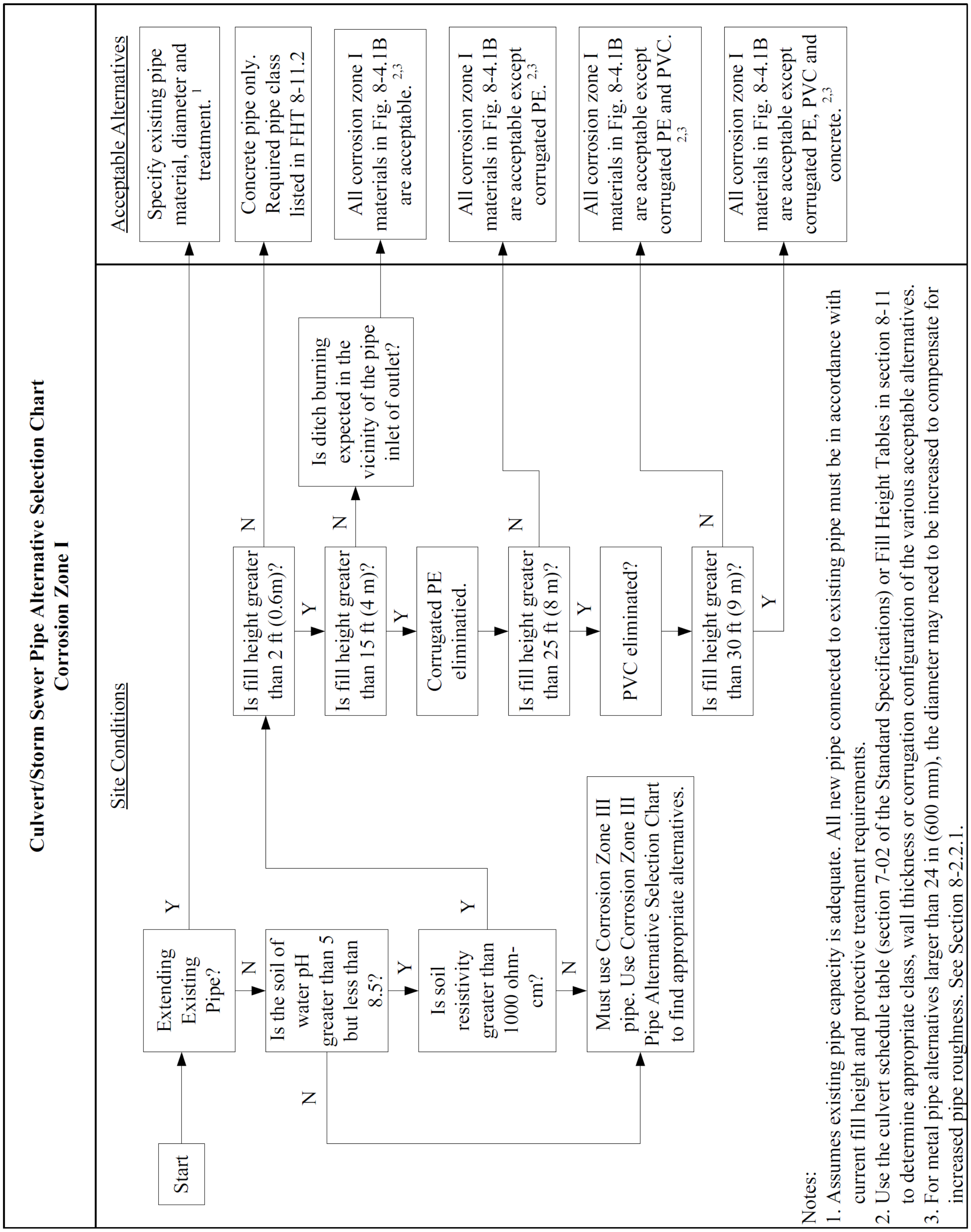


Figure 8-8b Corrosion Zone I:
Acceptable Pipe Alternates and Protective Treatments

Culverts	Storm Sewers

Schedule Pipe:
Schedule _____culvert pipe
 	
If Schedule pipe not selected, then:
 	
Concrete:
• Plain concrete culvert pipe
    – Cl___ reinforced concrete culvert pipe
PVC:
• Solid wall PVC culvert pipe
• Profile wall PVC culvert pipe
Polyethylene
• Corrugated PE culvert pipe
• HDPE pipe
Polypropylene Culvert pipe
A. Steel
    • Plain galvanized steel culvert pipe
    • Plain aluminized steel culvert pipe
Aluminum:
• Plain aluminum culvert pipe
	Concrete:
	• Plain concrete storm sewer pipe
	• Cl.___ reinforced concrete storm sewer pipe
	PVC:
	• Solid wall PVC storm sewer pipe
	• Profile wall PVC storm sewer pipe
	Polyethylene:
	• Corrugated PE storm sewer pipe
	• HDPE pipe
	Polypropylene Storm Sewer Pipe
	Steel:
	• Plain galvanized steel storm sewer pipe with gasketed or welded and remetallized seams
	• Treatment 1, 2, or 5 galvanized steel storm sewer pipe
	• Plain aluminized steel storm sewer pipe with gasketed or welded and remetallized seams
	• Treatment 1, 2, or 5 aluminized steel storm sewer pipe
	Steel Spiral Rib:
	• Plain galvanized steel spiral rib storm sewer pipe with gasketed or welded and remetallized seams
	• Treatment 1, 2, or 5 galvanized steel spiral rib storm sewer pipe
	• Plain aluminized steel spiral rib storm sewer with gasketed or welded or welded and remetallized seams
	• Treatment 1, 2 or 5 aluminum steel spiral rib storm sewer pipe
	Aluminum:
	• Plain aluminum spiral rib storm sewer pipe with gasketed seams
	• Treatment 1, 2, or 5 aluminum storm sewer pipe.
	Aluminum Spiral Rib:
	• Plain aluminum spiral rib storm sewer pipe with gasketed seams
	• Treatment 1, 2, or 5 aluminum spiral rib storm sewer pipe

Figure 8-9a Corrosion Zone II:
Flowchart of Acceptable Pipe Alternates and Protective Treatments

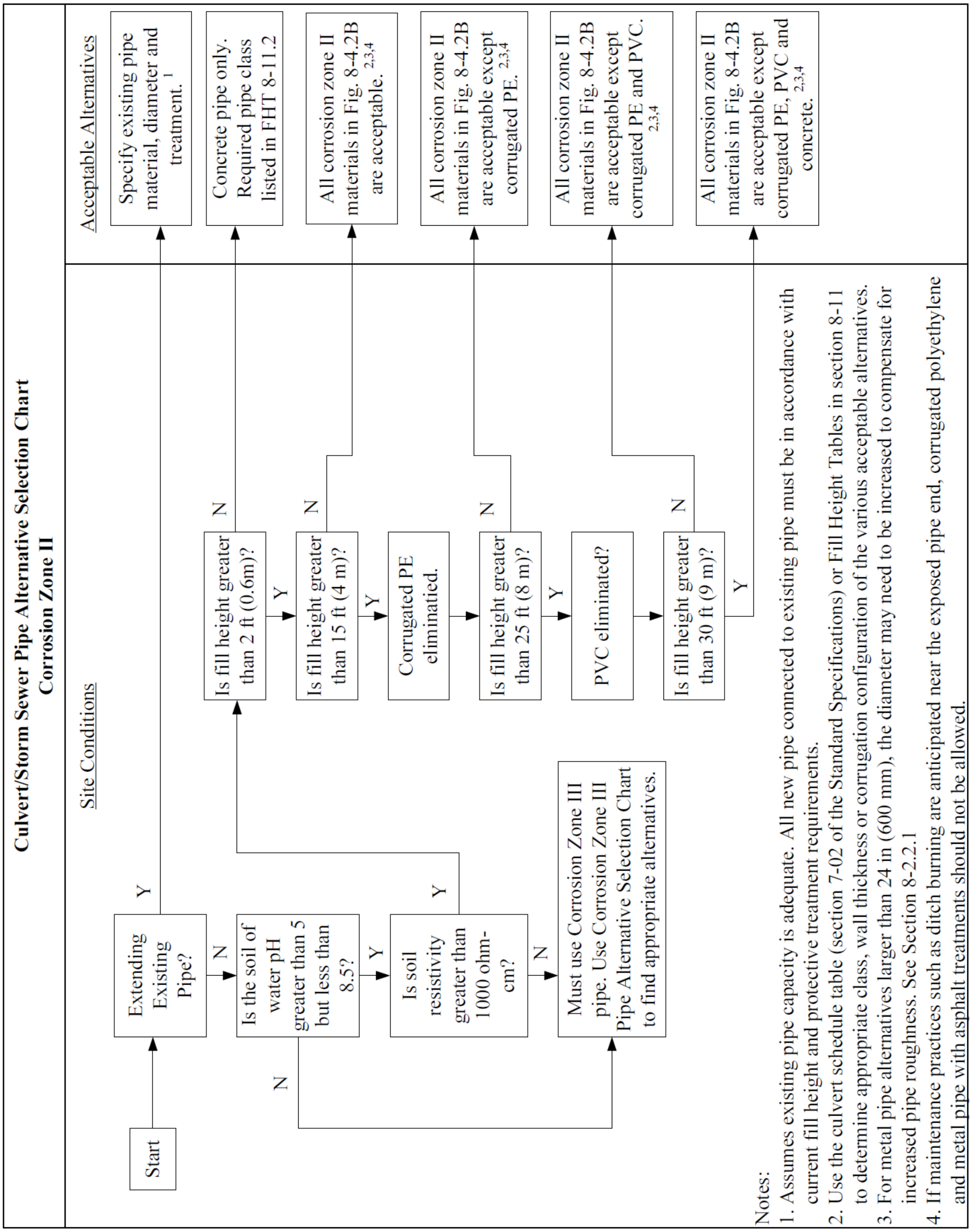


Figure 8-9b Corrosion Zone II:
Acceptable Pipe Alternates and Protective Treatments

Culverts	Storm Sewers
Schedule Pipe: Schedule _____culvert pipe Galvanized steel alternate shall have Treatment 2 If Schedule pipe not selected, then: **Concrete:** • Plain concrete culvert pipe • Cl___ reinforced concrete culvert pipe **PVC:** • Solid wall PVC culvert pipe • Profile wall PVC culvert pipe **Polyethylene:** • Corrugated PE culvert pipe • HDPE pipe **Polypropylene culvert pipe** **Steel** • Treatment 2 galvanized steel culvert pipe • Plain aluminized steel culvert pipe **Aluminum:** • Plain aluminum culvert pipe	**Concrete:** • Plain concrete storm sewer pipe • Cl.___ reinforced concrete storm sewer pipe **PVC:** • Solid wall PVC storm sewer pipe • Profile wall PVC storm sewer pipe **Polyethylene:** • Corrugated PE storm sewer pipe • HDPE pipe **Polypropylene Storm Sewer pipe** **Steel:** • Treatment 1, 2, or 5 galvanized steel storm sewer pipe • Treatment 1, 2, or 5 galvanized steel storm sewer pipe with gasketed or welded and remetallized seams • Plain aluminized steel spiral rib storm sewer pipe with gasketed or welded and remetallized seams • Treatment 1, 2, or 5 aluminized steel storm sewer pipe **Steel Spiral Rib:** • Treatment 1, 2, or 5 galvanized steel spiral rib storm sewer pipe • Treatment 1, 2, or 5 galvanized steel spiral rib storm sewer pipe with gasketed or welded and remetallized seams • Plain aluminized steel spiral rib storm sewer with gasketed or welded or welded and remetallized seams • Treatment 1, 2, or 5 aluminum steel spiral rib Storm sewer pipe **Aluminum:** • Plain aluminum storm sewer pipe with gasketed seams • Treatment 1, 2, or 5 aluminum storm sewer pipe **Aluminum Spiral Rib:** • Plain aluminum spiral rib storm sewer pipe with gasketed seams • Treatment 1, 2, or 5 aluminum spiral rib storm sewer pipe

Figure 8-10a Corrosion Zone III:
Flowchart of Acceptable Pipe Alternates and Protective Treatments

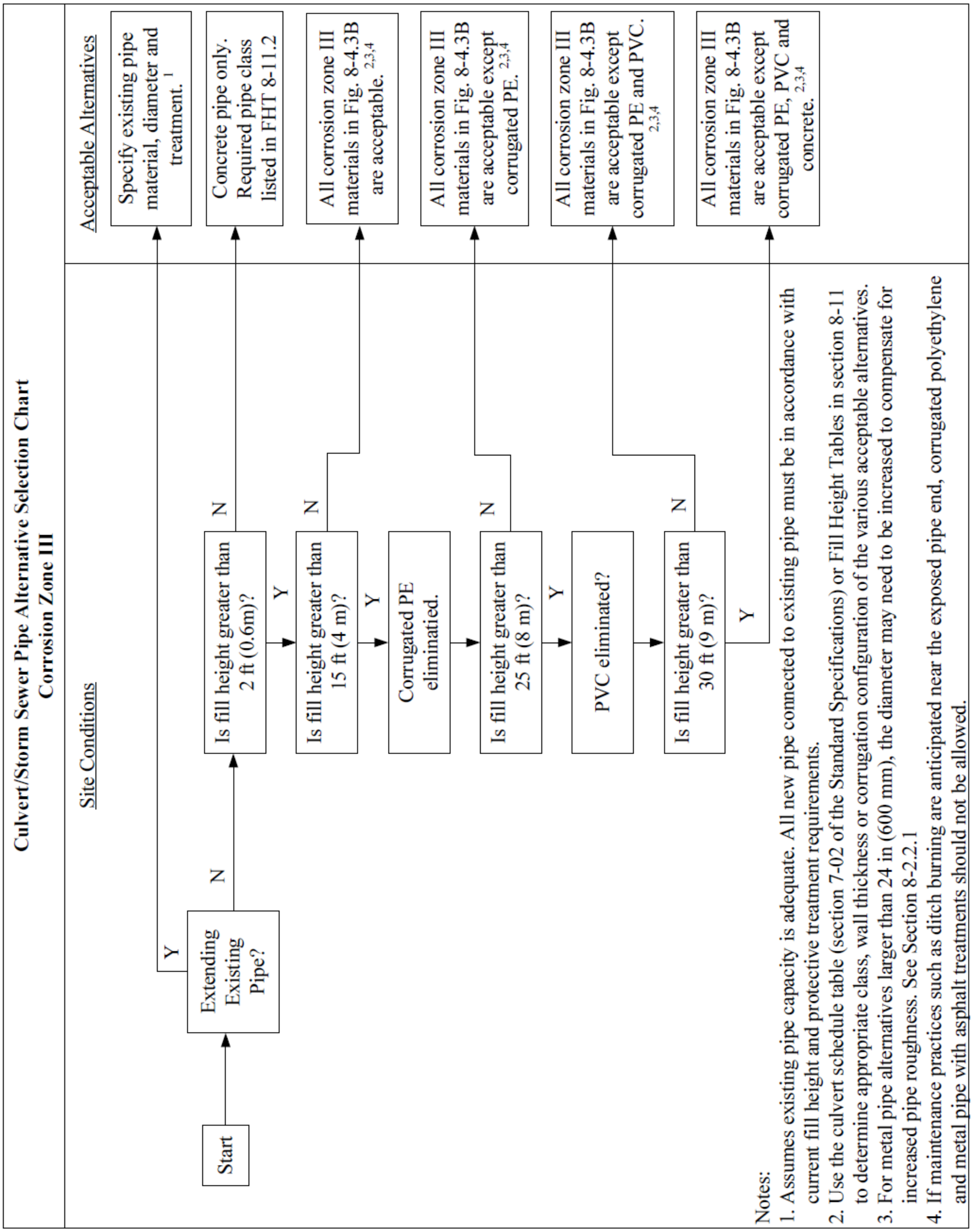


Figure 8-10b Corrosion Zone III:
Acceptable Pipe Alternates and Protective Treatments

Culverts	Storm Sewers
Schedule Pipe: Schedule _____culvert pipe _____in. diam. If Schedule pipe not selected, then: **Concrete:** • Plain concrete culvert pipe • Cl___ reinforced concrete culvert pipe **PVC:** • Solid wall PVC culvert pipe • Profile wall PVC culvert pipe **Polyethylene** • Corrugated PE culvert pipe • HDPE pipe **Polypropylene Culvert Pipe** **Aluminum:** • Plain aluminum culvert pipe	**Concrete:** • Plain concrete storm sewer pipe • Cl.___ reinforced concrete storm sewer pipe **PVC:** • Solid wall PVC storm sewer pipe • Profile wall PVC storm sewer pipe **Polyethylene:** • Corrugated PE storm sewer pipe • HDPE pipe **Polypropylene Storm Sewer pipe** **Aluminum:** • Plain aluminum storm sewer pipe with gasketed seams **Aluminum Spiral Rib:** • Plain aluminum spiral rib storm sewer pipe with gasketed seams

8-5 Corrosion

Corrosion is the destructive attack on a material by a chemical or electrochemical reaction with the surrounding environment. Corrosion is generally limited to metal pipes, and the parameters that tend to have the most significant influence on the corrosion potential for a site is the soil or water pH and the soil resistivity.

8-5.1 *pH*

The pH is a measurement of the relative acidity of a given substance. The pH scale ranges from 1 to 14, with 1 being extremely acidic, 7 being neutral, and 14 being extremely basic. The closer a pH value is to 7, the less potential the pipe has for corroding. When the pH is less than 5 or greater than 8.5, the site will be considered unsuitable and only Corrosion Zone III pipes, as discussed in Section 8-4.3, are acceptable.

The total number of pH tests required for a project will vary depending on different parameters, including the type of structures to be placed, the corrosion history of the site, and the project length and location. The general criteria listed below serves as minimum guidelines for determining the appropriate number of tests for a project.

1. **Size and importance of the drainage structure:** A project comprised of large culverts or storm sewers under an interstate or other major arterial warrant testing at each culvert or storm sewer location, while a project comprised of small culverts under a secondary highway may only need a few tests for the entire length of the project.
2. **Corrosion history of the project location:** A site in an area of the state with a high corrosion potential would warrant more tests than a site in an area of the state with a low corrosion potential.

3. **Distance of the project:** Longer projects tend to pass through several different soil types and geologic conditions, increasing the likelihood of variable pH readings. Tests should be taken at each major change in soil type or topography, or in some cases, at each proposed culvert location. Backfill material that is not native to the site and that will be placed around metal pipe should also be tested.
4. **Initial testing results:** If initial pH tests indicate that the values are close to or outside of the acceptable range of 5 to 8.5, or if the values vary considerably from location to location, additional testing may be appropriate.

8-5.2 *Resistivity*

Resistivity is the measure of the ability of soil or water to pass electric current. The lower the resistivity value, the easier it is for the soil or water to pass current, resulting in increased corrosion potential. If the resistivity is less than 1,000 ohm-cm for a location, then Corrosion Zone III pipe materials are the only acceptable alternates. Resistivity test are usually performed in conjunction with pH tests, and the criteria for frequency of pH testing shall apply to resistivity testing as well.

8-5.3 *Corrosion Control Methods*

8-5.3.1 Protective Treatments

Metal pipe, depending on the material and the geographical location, may require a protective asphalt coating for corrosion resistance throughout the pipe design life. As a general guideline, research has shown that asphalt coatings can typically add 15 to 35 years of life to metal pipes. Listed below are three different protective asphalt treatments available for use. The material specifications for the protective asphalt treatments are described in the *Standard Specifications*.

Treatment 1: Coated uniformly inside and out with asphalt. This treatment will protect the soil side of the pipe from corrosion but will only protect the waterside of the pipe from corrosion in environments that have little or no bed load moving through the pipe. Most culverts and storm sewers experience some degree of bed load, whether it is native upstream material or roadway sanding debris. The abrasive characteristics of the bed load can remove the asphalt coating relatively quickly, eliminating any corrosion resistance benefit. Consequently, this treatment is rarely specified.

As an alternative to Treatment 1 – Corrugated steel pipe may be coated on both sides with a polymer coating conforming to AASHTO M 246. The coating shall be a minimum of 10 mils thick and be composed of PE and acrylic acid copolymer.

Treatment 2: Coated uniformly inside and out with asphalt and with an asphalt paved invert. This treatment differs from Treatment 1 in that the invert of the pipe is paved with asphalt. Normal water levels within a pipe encompass about 40 percent of the pipe circumference, and this is where most of the corrosion takes place. The inside coating of the pipe above the normal watermark is not usually attacked by corrosion. Below the normal watermark, the protective coating suffers from wet and dry cycles and is also exposed to abrasion. For these reasons, the bottom 40 percent of the pipe is most critical and, therefore, paved with asphalt.

As an alternative to Treatment 2 – Corrugated steel pipe may be coated on both sides with a polymer coating conforming to AASHTO M 246. The coating shall be a minimum of 10 mils thick and be composed of PE and acrylic acid copolymer.

Treatment 3: No longer available.

Treatment 4: No longer available.

Treatment 5: Coated uniformly inside and out with asphalt and a 100 percent periphery inside spun asphalt lining. This treatment coats the entire inside circumference of the pipe with a thick layer of asphalt, covering the inside corrugations and creating a hydraulically smooth (see Manning's value in Appendix 4A) interior. The coating also provides invert protection similar to Treatment 2. Treatment 5 can be used on ungasketed lock seam pipe to seal the seam and allow the pipe to pass a pressure test in storm sewer applications.

Treatment 6: No longer available.

The protective treatments, when required, shall be placed on circular pipe and pipe arch culverts. Structural plate pipes do not require protective treatment, as described in Section 8-2.3.3. Protective treatments are not allowed for culverts placed in fish-bearing streams. This may preclude the use of metal culverts in some applications.

The treatments specified in this section are the standard minimum applications, which are adequate for a large majority of installations; however, a more stringent treatment may be used at the PEO's discretion. When unusual abrasive or corrosive conditions are anticipated, and it is difficult to determine which treatment would be adequate, it is recommended that either the HQ Materials Laboratory or HQ Hydraulics Section be consulted.

8-5.3.2 Increased Gauge Thickness

As an alternative to asphalt protective treatments, the thickness of corrugated steel pipes can be increased to compensate for loss of metal due to corrosion or abrasion. A methodology has been developed by the California Transportation Department (Caltrans) to estimate the expected service life of untreated corrugated steel pipes. The method utilizes pH, resistivity, and pipe thickness and is based on data taken from hundreds of culverts throughout California. Copies of the design charts for this method can be obtained from the HQ Hydraulics Section.

8-6 Abrasion

Abrasion is the wearing away of pipe material by water carrying sands, gravels, and rocks. All types of pipe material are subject to abrasion and can experience structural failure around the pipe invert if not adequately protected. Four abrasion levels have been developed to assist the PEO in quantifying the abrasion potential of a site. The abrasion levels are identified in Figure 8-11.

The abrasion level descriptions are intended to serve as general guidance only; not all of the criteria listed for a particular abrasion level need to be present to justify placing a site at that level. Included with each abrasion level description are guidelines for providing additional invert protection. The PEO is encouraged to use those guidelines in conjunction with the abrasion history of a site to achieve the desired design life of a pipe.

Sampling streambed materials is generally not necessary, but visual examination and documentation of the size of the materials in the streambed and the average stream slopes will give the PEO guidance on the expected level of abrasion. Where existing culverts are in place in the same drainage, the condition of the inverts should also be used as guidance. The stream velocity shall be based on typical flows, such as a 6-month event, and not a 10- or 50-year event. This is because most of the abrasion will occur during those smaller events.

In streams with significant bed loads, placing culverts on flat grades can encourage bed load deposition within the culvert. This can substantially decrease the hydraulic capacity of a culvert, ultimately leading to plugging or potential roadway overtopping on the upstream side of the culvert. As a standard practice, culvert diameters shall be increased two or more standard sizes over the required hydraulic opening in situations where abrasion and bed load concerns have been identified.

Figure 8-11 Pipe Abrasion Levels

Abrasion Level	General Site Characteristics	Recommended Invert Protection
Non-Abrasive	• Little or no bed load • Slope less than 1% • Velocities less than 3 feet per second (ft/s)	Generally, most pipes may be used under these circumstances, if a protective treatment is deemed necessary for metal pipes, any of the protective treatments specified in Section 8-5.3.1 would be adequate.
Low Abrasive	• Minor bed loads of sands, silts, and clays • Slopes 1% to 2% • Velocities less than 6 ft/s	For metal pipes, an additional gauge thickness may be specified if existing pipes in the vicinity show susceptibility to abrasion, or any of the protective treatments specified in Section 8-5.3.1 would be adequate.
Moderate Abrasive	• Moderate bed loads of sands and gravels, with stone sizes up to about 3 inches • Slopes 2% to 4% • Velocities from 6 to 15 ft/s	Metal pipes shall be specified with asphalt paved inverts and the pipe thickness shall be increased one or two standard gauges. The PEO may want to consider a concrete-lined alternative. Concrete pipe and box culverts shall be specified with an increased wall thickness or an increased concrete compressive strength. Thermoplastic pipe may be used without additional treatments.
Severe Abrasive	• Heavy bed loads of sands, gravel, and rocks, with stones sizes up to 12 inches or larger • Slopes steeper than 4% • Velocities greater than 15 ft/s	Asphalt protective treatments will have short life expectancies, sometimes lasting only a few months to a few years. Metal pipe thickness shall be increased at least two standard gauges, or the pipe invert shall be lined with concrete. Box culverts shall be specified with an increased wall thickness or an increased concrete compressive strength. Sacrificial metal pipe exhibits better abrasion characteristics than metal or concrete. However, it generally cannot be reinforced to provide additional invert protection and is not recommended in this condition.

8-7 Pipe Joints

Culverts, storm sewers, and sanitary sewers require the use of gasketed or fused joints to restrict the amount of leakage into or out of the pipe. The type of gasket material varies, depending on the pipe application and the type of pipe material being used. The *Standard Plans* and *Standard Specifications* should be consulted for specific descriptions of the types of joints, coupling bands, and gaskets for the various types of pipe material.

Corrugated metal pipe joints incorporate the use of a metal coupling band and neoprene gasket that strap on around the outside of the two sections of pipe to be joined. This joint provides a positive connection between the pipe sections and is capable of withstanding significant tensile forces. These joints work well in culvert applications, but usually do not meet the pressure test requirements for storm sewer applications.

Concrete pipe joints incorporate the use of a rubber O-ring gasket and are held together by friction and the weight of the pipe. Precautions must be taken when concrete pipe is placed on grades greater than 10 percent or in fills where significant settlement is expected, because it is possible for the joints to pull apart. Outlets to concrete pipe must be properly protected from erosion because a small amount of undermining could cause the end section of pipe to disjoin, ultimately leading to failure of the entire pipe system. Concrete joints, because of the O-ring gasket, function well in culvert applications and also consistently pass the pressure testing requirements for storm sewers.

Thermoplastic pipe joints vary; some are similar in performance to either the corrugated metal pipe joint or the concrete pipe joint described above, while others are completely watertight and as strong as the pipe itself. The following joint types are available for thermoplastic pipe:

- Integral, gasketed bell ends that positively connect to the spigot end
- Slip-on bell ends connected with O-ring gaskets on the spigot end
- Strap-on corrugated coupling bands
- Snap together, or threaded, bell and spigot connections
- Butt fusion welded or electrofusion coupling
- Mechanical or flanged

All types of joints have demonstrated adequate pull-apart resistance and can generally be used on most highway or embankment slopes.

8-8 Pipe Anchors

Pipe anchor installation is rare and usually occurs when a pipe or half pipe is replaced aboveground on a very steep (15 to 20 percent grade) or highly erosive slope. In these cases, the pipe diameter is relatively small (10 inches or smaller). Continuous PE tubing may be used without the need for anchors since there are no joints in the pipe. On larger pipes, HDPE pipe with fused joints may be used without the use of pipe anchors. For further design guidance, contact the HQ Hydraulics Section.

8-8.1 *Thrust Blocks*

Thrust blocks should be designed to help stabilize fittings (tees, valves, bends, etc.) of water mains or pressure mains from movement by increasing the soil bearing area. The key to sizing a thrust block is a correct determination of the soil bearing value. These values can range from less than one thousand pounds per square foot for soft soils to many thousands of pounds per square foot for hard rock. A correctly sized thrust block will also fail unless the block is placed against undisturbed soil with the face of the block perpendicular to the direction of and centered on the line of the action of the thrust. (See Standard Plan B-90.50-00, Concrete Thrust Block, for details on placement and sizing of a thrust block for various fittings.)

8-9 Pipe Rehabilitation – Trenchless Technology

Pipes that have deteriorated over time due to either corrosion or abrasion can significantly affect the structural integrity of the roadway embankment. Once identified, these pipes should be replaced or repaired in a timely manner, as failure of the pipe could ultimately result in failure of the roadway. The PEO will have two options for deteriorated pipes: rehabilitation or replacement.

The most common option for a deteriorated pipe is to remove the existing culvert and replace it with a new one. This method generally requires that all or part of the roadway be closed down for a given amount of time. This may or may not be feasible due to many factors, including the location and importance of the roadway, size of the pipe structure involved, depth of the fill and width of the workable roadway prism. This type of construction has become increasingly difficult on interstates and other high average daily traffic roadways.

For locations where replacing the pipe is not feasible, it may be possible to use rehabilitation methods to restore the structural integrity of the pipe system, with minimal impact to roadway traffic. These method are referred to as trenchless technology due to minimal trenching needed.

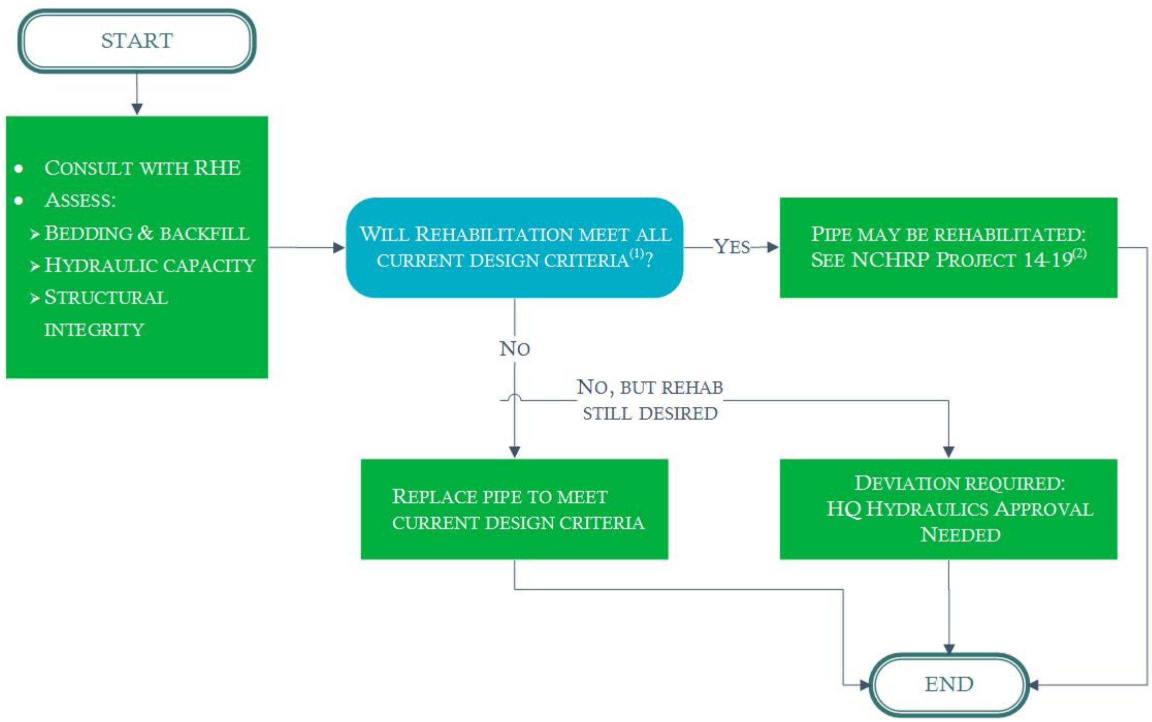
The following sections describe suggested methods for repairing existing pipes. Prior to selecting a trenchless technology method, the PEO shall investigate the feasibility of a pipe being rehabilitated to provide a long-term fix. The investigation shall include, at a minimum:

- **Evaluation of the pipe bedding and backfill conditions:** The pipe bedding and backfill shall be evaluated to determine if the existing conditions meet current design criteria. For example, if the existing pipe has cracked, water may have leaked through the pipe wall and caused erosion of the bedding material. In this case, the void spaces may need to be grouted between the backfill and the host pipe prior to rehabilitation.
- **Analysis of the hydraulic capacity of pipe:** The hydraulic capacity of a rehabilitated pipe shall be analyzed using the same criteria required for a new pipe. This includes a complete basin analysis as the contributing area may have changed since the original pipe was designed. Also, many trenchless technologies involve methods that reduce the diameter of the host pipe. Structural integrity of the pipe shall be analyzed to determine if the pipe can tolerate a trenchless technology.
- **Evaluation of the structural integrity of the pipe:** The structural integrity of the pipe shall be evaluated to determine if the host pipe is strong enough to tolerate the trenchless technology. This will involve contacting the HQ Hydraulics Section for guidance on inspecting the pipe and developing a risk assessment. The vendors providing the trenchless technology should also be consulted for determining the minimum structural requirements of the pipe.

If this analysis indicates that rehabilitating the pipe using trenchless methods will meet all current design criteria, then the pipe may be rehabilitated. Refer to NCHRP Project 14-19 (onlinepubs.trb.org/onlinepubs/project14-19) for additional guidance on rehabilitation methods for various pipe types. If the analysis indicates that the rehabilitated pipe will not meet current design criteria, then it must be replaced with one that does, or a deviation must be received from the HQ Hydraulics Section. See Figure 8-12.

Note: Additional guidance will be provided in future revisions to the *Hydraulics Manual*.

Figure 8-12 Replace or Rehabilitate Decision Tree



(1)See Chapter 3, Chapter 6, or other applicable chapter.
(2)http://onlinepubs.trb.org/onlinepubs/project14-19/index.html

8-9.1 *Trenchless Techniques for Pipe Replacement*

A number of rehabilitation methods are available that can restore structural integrity to the pipe system while minimally affecting roadway traffic. As the name implies, these methods involve minimal trenching along with the ability to retrofit or completely replace a pipe without digging up the pipe.

Various types of liners can retrofit the pipe interior, providing structural support. One of these techniques involves pulling a folded HDPE pipe through the existing (host) pipe. The liner pipe is then inflated with hot air or water so it molds itself to the host pipe, sealing cracks and creating a new pipe within a pipe. Another technique uses the same method, but the liner is made of a felt material impregnated with resins.

- **Sliplining** is a technique that involves inserting a full round pipe with a smaller diameter into the host pipe and then filling the space between the two pipes with grout.

- **Pipe bursting** is a technique where a pneumatically operated device moves through the host pipe, bursting it into pieces. Attached to the device is a pipe string, usually thermally fused HDPE. Using this method and depending on the soil type, the new pipe may be a larger diameter than the pipe being burst.
- **Tunneling**, while typically much more expensive than the other methods, may be the only feasible option for placing large-diameter pipes under interstates or major arterials.
- **Horizontal directional drilling (HDD)** is a technique that uses guided drilling for creating an arc profile. This technique can be used for drilling long distances such as under rivers, lagoons, or highly urbanized areas. The process involves three main stages: drilling a pilot hole, pilot hole enlargement, and pullback installation of the carrier pipe.
- **Pipe jacking or ramming** is probably most commonly used method. Pipe diameters less than 48 inches can be jacked both economically and easily. Pipe diameters to 144 inches are possible; however, the complexity and cost increase with the diameter of the pipe.

Protective treatment is not required on smooth-walled steel pipe used for jacking installations; however, jacked pipes require extra wall thickness to accommodate the expected jacking stresses.

A full hydraulic analysis must be done on pipes to be rehabilitated or replaced to be sure they are hydraulically adequate. Any type of liner reduces the diameter of the pipe, thus reducing capacity; however, the improved efficiency of the new liner may or may not compensate for the lost capacity.

8-10 Pipe Design

8-10.1 *Categories of Structural Materials: Rigid or Flexible*

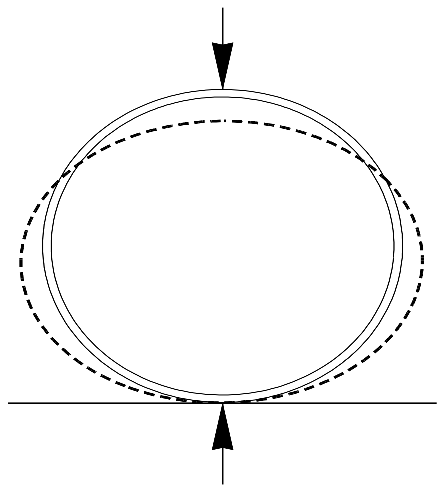
Based upon material type, pipes can be divided into two broad structural categories: flexible and rigid. Flexible pipes have little structural bending strength. The material they are made of, such as corrugated metal or thermoplastic, can be flexed or distorted significantly without cracking. Flexible pipes depend on support from the backfill to resist bending. Rigid pipes are stiff and do not deflect appreciably. The material they are made of, such as concrete, provides the primary resistance to bending.

8-10.2 *Structural Behavior of Flexible Pipes*

A flexible pipe is a composite structure made up of the pipe barrel and the surrounding soil. The barrel and the soil are both vital elements to the structural performance of the pipe. Flexible pipe has relatively little bending stiffness or bedding strength on its own. As loads are applied to the pipe, the pipe attempts to deflect. In the case of round pipe, the vertical diameter decreases and the horizontal diameter increases, as shown in Figure 8-13. When adequate soil support and backfill material are well compacted around the pipe, the increase in the horizontal diameter of the pipe is resisted by the lateral soil pressure. The result is a relatively uniform radial pressure around the pipe, which creates a compressive force in the pipe walls called thrust. To ensure that a stable soil envelope around the pipe is attained during construction, follow the guidelines in Section 8-10.4 for backfill and installation.

As vertical loads are applied, a flexible culvert attempts to deflect. The vertical diameter decreases while the horizontal diameter increases. Soil pressures resist the increase in horizontal diameter. The thrust can be calculated, based on the diameter of the pipe and the load placed on the top of the pipe, and is then used as a parameter in the structural design of the pipe.

Figure 8-13 Deflection of Flexible Pipes

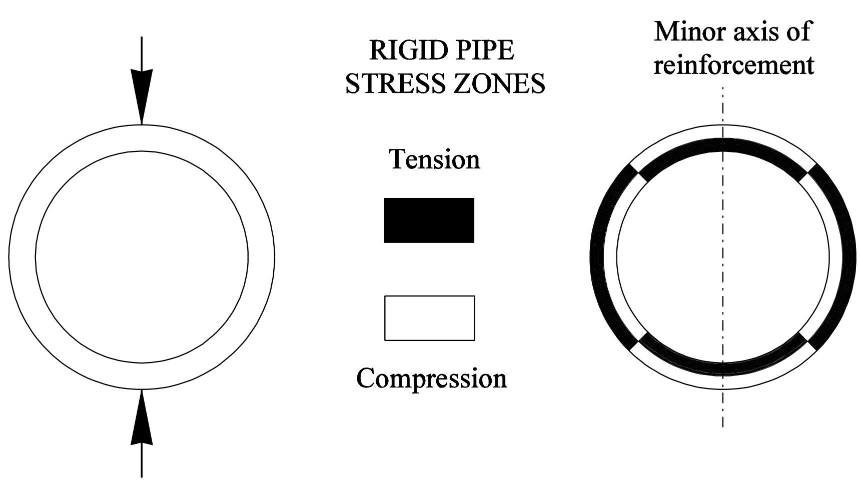


The flexibility of a pipe also allows for some bend in the horizontal when designing the pipe layout. The PEO shall limit the bend to a maximum of 1.5 degrees. This same allowable bend does not apply to pipe profiles, which shall be designed to be straight. When bends occur in the profile, "bellies" form that cause sediment to accumulate.

8-10.3 *Structural Behavior of Rigid Pipes*

The load carrying capability of rigid pipes is essentially provided by the structural strength of the pipe itself, with some additional support given by the surrounding bedding and backfill. When vertical loads are applied to a rigid pipe, zones of compression and tension are created as illustrated in Figure 8-14. Reinforcing steel can be added to the tension zones to increase the tensile strength of concrete pipe. The minor axis for elliptical reinforcement is discussed in Section 8-3.1.

Figure 8-14 Zones of Tension and Compression in Rigid Pipes



Rigid pipe is stiffer than the surrounding soil and it carries a substantial portion of the applied load. Shear stress in the haunch area can be critical for heavily loaded rigid pipe on hard foundations, especially if the haunch support is inadequate. Standard Plan B-55.20-02 and the *Standard Specifications* describe the backfill material requirements and installation procedures required for placing the various types of pipe materials. The fill height tables for concrete pipe shown in Section 8-12 were developed assuming that those requirements were followed during installation.

8-10.4 *Foundations, Bedding, and Backfill*

A foundation capable of providing uniform and stable support is important for both flexible and rigid pipes. The foundation must be able to uniformly support the pipe at the proposed grade and elevation without concentrating the load along the pipe. Establishing a suitable foundation requires removal and replacement of any hard spots or soft spots that would result in load concentration along the pipe.

Bedding is needed to level out any irregularities in the foundation and to ensure adequate compaction of the backfill material. (See the *Standard Plans* for Pipe Zone Bedding and Backfill and the *Standard Specifications* Backfilling for guidelines.) Any trenching conditions not described in the *Standard Plans* or *Standard Specifications* require approval from the HQ Hydraulics Section.

When using flexible pipes, the bedding should be shaped to provide support under the haunches of the pipe. When using rigid pipe, the bedding should be shaped to provide uniform support under the haunches and also shaped to provide clearance for the bell ends on bell and spigot type pipe. The importance of proper backfill for flexible and rigid pipe is discussed in Sections 8-10.2 and 8-10.3, respectively.

The bedding and backfill must also be installed properly to prevent piping from occurring. Piping is a term used to describe the movement of water around and along the outside of a pipe, washing away backfill material that supports the pipe. Piping is primarily a concern in culvert applications, where water at the culvert inlet can saturate the embankment and move into the pipe zone. Piping can be prevented through the use of headwalls, dikes, or plugs. Headwalls are described in Chapter 3 and dikes and plugs are discussed in the *Standard Specifications*.

To simplify measurement and payment during construction, all costs associated with furnishing and installing the bedding and backfill material within the pipe zone are included in the unit contract price of the pipe.

8-11 Abandoned Pipe Guidelines

Abandoned pipes shall be removed. However, if it is not practical to remove the pipe, the pipe can be abandoned in place and the pipe ends plugged as specified in the *Standard Specifications*. All pipes shall be evaluated prior to abandonment by either the project PEO, RHE, or HQ Hydraulics Section to determine what potential hazards are associated with pipe failure. If a pipe failure could cause a collapse of the roadway prism, the pipe shall either be removed or completely filled with a controlled density fill (CDF) that meets the requirements per the *Standard Specifications*.

8-12 Structural Analysis and Fill Height Tables

The HQ Hydraulics Section, using currently accepted design methodologies, has performed a structural analysis for the various types of pipe material available. The results are shown in the fill height tables at the end of this section. The fill height tables demonstrate the maximum and minimum amounts of cover that can be placed over an existing or new pipe, assuming that the pipe is installed in accordance with WSDOT specifications. All culverts, storm sewers, and sanitary sewers shall be installed within the limitations shown in the fill height tables.

The PEO shall specify the same wall thickness or class of material for the entire length of a given pipe, and that will be based on the most critical load configuration experienced by any part of the pipe. This will negate the necessity of removing structurally inadequate pipe sections at some point in the future should roadway widening occur. Additionally, when selecting corrugated pipe, the PEO should review all of the tables in Section 8-12.3 and select the most efficient corrugation thickness for the pipe diameter. For fill heights in excess of 100 feet, special designs by the HQ Hydraulics Section will be required.

8-12.1 *Pipe Cover*

Pipe systems shall be designed to provide at least 2 feet of cover over the pipe, measured from the outside diameter of the pipe to the bottom of pavement. This measurement does not include any asphalt or concrete paving above the top course. This depth provides adequate structural distribution of the live load and also allows a significant number of pipe alternatives to be specified on a contract. Unless the contract plans specify a specific pipe material, the PEO shall plan for the schedule pipe fill heights as described in the *Standard Specifications*. If there is no possibility of a wheel load over the pipe, a PEO may request using non-scheduled pipe with approval from the HQ Hydraulics Section.

During construction, more restrictive fill heights are required, and are specified in the *Standard Specifications*. The restrictive fill heights are intended to protect pipe from construction loads that can exceed typical highway design loads.

8-12.2 *Shallow Cover Installation*

In some cases, it is not possible to lower a pipe profile to obtain the necessary minimum cover. In those cases, pipe of the class shown in Fill Height Figure 8-32 may be specified. Included in that table are typical pipe wall thicknesses for a given diameter. The pipe wall thickness must be taken into consideration in low cover applications. Justification must also be included in the hydraulic report describing why it was not possible to lower the pipe profile to obtain the preferred 2 feet of cover.

In addition to circular pipe, concrete box culverts and concrete arches are also available for use in shallow cover installations. For three-sided or box concrete culverts, the PEO must verify that the shallow cover will still provide HS 25 loading. Other options include ductile iron pipe, plain steel pipe, PP pipe, or the placement of a concrete distribution slab. The PEO should consult with either the Region Hydraulics Office/Contact or the HQ Hydraulics Section for additional guidance on the use of these structures in this application.

8-12.3 *Fill Height Tables*

Figure 8-15 Concrete Pipe

Pipe Diameter in.	Maximum Cover in Feet				
	Plain AASHTO M 86	Class II AASHTO M 170	Class III AASHTO M 170	Class IV AASHTO M 170	Class V AASHTO M 170
12	18	10	14	21	26
18	18	11	14	22	28
24	16	11	15	22	28
30	--	11	15	23	29
36	--	11	15	23	29
48	--	12	15	23	29
60	--	12	16	24	30
72	--	12	16	24	30
84	--	12	16	24	30

Notes:

-- = not applicable

Minimum cover is 2 feet.

Figure 8-16 Concrete Pipe for Shallow Cover Installations

Pipe Diameter in.	Pipe Wall Thick in.	Minimum Cover in Feet			
		Plain AASHTO M 86	Class III AASHTO M 170	Class IV AASHTO M 170	Class V AASHTO M 170
12	2	1.5	1.5	1.0	0.5
18	2.5	1.5	1.5	1.0	0.5
24	3	1.5	1.5	1.0	0.5
30	3.5	1.5	1.5	1.0	0.5
36	4	1.5	1.5	1.0	0.5
48	5	--	1.5	1.0	0.5
60	6	--	1.5	1.0	0.5
72	7	--	1.5	1.0	0.5
84	8	--	1.5	1.0	0.5

Notes:

-- = not applicable

in. = inch

Figure 8-17 Corrugated Steel Pipe: 2⅔ in. × ½ in. Corrugations – AASHTO M 36

Pipe Diameter in.	Maximum Cover in Feet				
	0.064 in. 16 ga	**0.079 in. 14 ga**	**0.109 in. 12 ga**	**0.138 in. 10 ga**	**0.168 in. 8 ga**
12	100	100	100	100	--
18	100	100	100	100	--
24	98	100	100	100	100
30	78	98	100	100	100
36[1]	65	81	100	100	100
42[1]	56	70	98	100	100
48[1]	49	61	86	100	100
54[1]	--	54	76	98	100
60[1]	--	--	68	88	100
66[1]	--	--	--	80	98
72[1]	--	--	--	73	90
78[1]	--	--	--	--	80
84[1]	--	--	--	--	69

Notes:
-- = not applicable
ga = gallon
in. = inch
Minimum cover is 2 feet.
[1]The PEO should consider the most efficient corrugation for the pipe diameter.

Figure 8-18 Corrugated Steel Pipe: 3 in. × 1 in. Corrugations – AASHTO M 36

Pipe Diameter in.	Maximum Cover in Feet				
	0.064 in. 16 ga	**0.079 in. 14 ga**	**0.109 in. 12 ga**	**0.138 in. 10 ga**	**0.168 in. 8 ga**
36	75	94	100	100	100
42	64	80	100	100	100
48	56	70	99	100	100
54	50	62	88	100	100
60	45	56	79	100	100
66	41	51	72	92	100
72	37	47	66	84	100
78	34	43	60	78	95
84	32	40	56	72	89
90	30	37	52	67	83
96	--	35	49	63	77
102	--	33	46	59	73
108	--	--	44	56	69
114	--	--	41	53	65
120	--	--	39	50	62

Notes:
-- = not applicable
ga = gallon
in. = inch
Minimum cover is 2 feet.

Figure 8-19 Corrugated Steel Pipe: 5 in. × 1 in. Corrugations – AASHTO M 36

Pipe Diameter in.	Maximum Cover in Feet				
	0.064 in. 16 ga	0.079 in. 14 ga	0.109 in. 12 ga	0.138 in. 10 ga	0.168 in. 8 ga
30	80	100	100	100	100
36	67	83	100	100	100
42	57	71	100	100	100
48	50	62	88	100	100
54	44	55	78	100	100
60	40	50	70	90	100
66	36	45	64	82	100
72	33	41	58	75	92
78	31	38	54	69	85
84	28	35	50	64	79
90	26	33	47	60	73
96	--	31	44	56	69

Notes:

-- = not applicable
ga = gallon
in. = inch
Minimum cover is 2 feet.

Figure 8-20 Corrugated Steel Structural Plate Circular Pipe: 6 in. × 2 in. Corrugations

Pipe Diameter in.	Minimum Cover ft.	Maximum Cover in Feet						
		0.111 in. 12 ga	0.140 in. 10 ga	0.170 in. 8 ga	0.188 in. 7 ga	0.218 in. 5 ga	0.249 in. 3 ga	0.280 in. 1 ga
60	2	42	63	83	92	100	100	100
72	2	35	53	69	79	94	100	100
84	2	30	45	59	67	81	95	100
96	2	27	40	52	59	71	84	92
108	2	23	35	46	53	64	75	81
120	2	21	31	42	47	57	67	74
132	2	19	29	37	42	52	61	66
144	2	18	26	37	40	47	56	61
156	2	16	24	31	36	43	52	56
168	2	15	22	30	33	41	48	53
180	2	14	20	28	31	38	44	49
192	2	--	19	26	30	35	42	46
204	3	--	18	24	28	33	40	43
216	3	--	--	23	26	31	37	41
228	3	--	--	--	25	30	35	39
240	3	--	--	--	23	29	33	37

Notes:

-- = not applicable
ga = gallon
in. = inch
6 in. × 2 in. corrugations require field assembly for multiplate; diameter is too large to ship in full section.

Figure 8-21 Corrugated Steel Pipe Arch: 2⅔ in. × ½ in. Corrugations – AASHTO M 36

Span × Rise in. × in.	Min Corner Radius in.	Thickness		Minimum Cover Feet	Maximum Cover in Feet for Soil Bearing Capacity of:	
		in.	Gauge		2 tons/ft^2	3 tons/ft^2
17 × 13	3	0.064	16 ga	2	12	18
21 × 15	3	0.064	16 ga	2	10	14
24 × 18	3	0.064	16 ga	2	7	13
28 × 20	3	0.064	16 ga	2	5	11
35 × 24	3	0.064	16 ga	2.5	NS	7
42 × 29	3.5	0.064	16 ga	2.5	NS	7
49 × 33	4	0.079	14 ga	2.5	NS	6
57 × 38	5	0.109	12 ga	2.5	NS	8
64 × 43	6	0.109	12 ga	2.5	NS	9
71 × 47	7	0.138	10 ga	2	NS	10
77 × 52	8	0.168	8 ga	2	5	10
83 × 57	9	0.168	8 ga	2	5	10

Notes:
ft^2 = square feet
ga = gallon
in. = inch
NS = Not Suitable

Figure 8-22 Corrugated Steel Pipe Arch: 3 in. × 1 in. Corrugations – AASHTO M 36

Span × Rise in. × in.	Corner Radius in.	Thickness		Minimum Cover Feet	Maximum Cover in Feet for Soil Bearing Capacity of:	
		in.	Gauge		2 tons/ft^2	3 tons/ft^2
40 × 31	5	0.079	14 ga	2.5	8	12
46 × 36	6	0.079	14 ga	2	8	13
53 × 41	7	0.079	14 ga	2	8	13
60 × 46	8	0.079	14 ga	2	8	13
66 × 51	9	0.079	14 ga	2	9	13
73 × 55	12	0.079	14 ga	2	11	16
81 × 59	14	0.079	14 ga	2	11	17
87 × 63	14	0.079	14 ga	2	10	16
95 × 67	16	0.079	14 ga	2	11	17
103 × 71	16	0.109	12 ga	2	10	15
112 × 75	18	0.109	12 ga	2	10	16
117 × 79	18	0.109	12 ga	2	10	15
128 × 83	18	0.138	10 ga	2	9	14
137 × 87	18	0.138	10 ga	2	8	13
142 × 91	18	0.168	10 ga	2	7	12

Notes:
ft^2 = square feet
ga = gallon
in. = inch

Figure 8-23 Corrugated Steel Structural Plate Pipe Arch: 6 in. × 2 in. Corrugations

Span × Rise ft.-in. × ft.-in.	Corner Radius in.	Thickness		2 TSF Soil Bearing Capacity		3 TSF Soil Bearing Capacity	
		in.	Gauge	Min. Cover ft.	Max. Cover ft.	Min. Cover ft.	Max. Cover ft.
6 – 1 × 4 - 7	18	0.111	12 ga	2	16	2	24
7 – 0 × 5 - 1	18	0.111	12 ga	2	14	2	21
7 – 11 × 5 - 7	18	0.111	12 ga	2	13	2	19
8 – 10 × 6 - 1	18	0.111	12 ga	2	11	2	17
9 – 9 × 6 - 7	18	0.111	12 ga	2	10	2	15
10 – 11 × 7 - 1	18	0.111	12 ga	2	9	2	14
11 – 10 × 7 - 7	18	0.111	12 ga	2	7	2	13
12 – 10 × 8 - 4	18	0.111	12 ga	2.5	6	2	12
13 – 3 × 9 - 4	31	0.111	12 ga	2	13	2	17(1)
14 – 2 × 9 - 10	31	0.111	12 ga	2	12	2	16(1)
15 – 4 × 10 - 4	31	0.140	10 ga	2	11	2	15(1)
16 – 3 × 10 - 10	31	0.140	10 ga	2	11	2	14(1)
17 – 2 × 11 - 4	31	0.140	10 ga	2.5	10	2.5	13(1)
18 – 1 × 11 - 10	31	0.168	8 ga	2.5	10	2.5	12(1)
19 – 3 × 12 - 4	31	0.168	8 ga	2.5	9	2.5	13

Notes:

ft. = feet
ga = gallon
in. = inch
TSF = tons per square foot
(1)Fill limited by the seam strength of the bolts. Additional sizes are available. Contact the OSC Hydraulics Office for more information.

Figure 8-24 Aluminum Pipe: 2⅔ in. × ½ in. Corrugations – AASHTO M 196

Pipe Diameter in.	Maximum Cover in Feet				
	0.060 in. (16 ga)	0.075 in. (14 ga)	0.105 in. (12 ga)	0.135 in. (10 ga)	0.164 in. (8 ga)
12	100	100	--	--	--
18	75	94	100	--	--
24	56	71	99	--	--
30	--	56	79	--	--
36	--	47	66	85	--
42	--	--	56	73	--
48	--	--	49	63	78
54	--	--	43	56	69
60	--	--	--	50	62
66	--	--	--	--	56
72	--	--	--	--	45

Notes:

-- = not applicable
in. = inch
ga = gallon
Minimum cover is 2 feet.

Figure 8-25 Aluminum Pipe: 3 in. × 1 in. Corrugations – AASHTO M 196

Pipe Diameter in.	Maximum Cover in Feet				
	0.060 in. (16 ga)	0.075 in. (14 ga)	0.105 in. (12 ga)	0.135 in. (10 ga)	0.164 in. (8 ga)
36	43	65	76	98	--
42	36	46	65	84	--
48	32	40	57	73	90
54	28	35	50	65	80
60	--	32	45	58	72
66	--	28	41	53	65
72	--	26	37	48	59
78	--	24	34	44	55
84	--	--	31	41	51
90	--	--	29	38	47
96	--	--	27	36	44
102	--	--	--	33	41
108	--	--	--	31	39
114	--	--	--	--	37
120	--	--	--	--	35

Notes:
-- = not applicable
in. = inch
ga = gallon
Minimum cover is 2 feet.

Figure 8-26 Aluminum Structural Plate: 9 in. × 2 in. Corrugations with Galvanized Steel Bolts

Pipe Diameter in.	Maximum Cover in Feet						
	0.100 in.	0.125 in.	0.150 in.	0.175 in.	0.200 in.	0.225 in.	0.250 in.
60	31	45	60	70	81	92	100
72	25	37	50	58	67	77	86
84	22	32	42	50	58	66	73
96	19	28	37	44	50	57	64
108	17	25	33	39	45	51	57
120	15	22	30	35	40	46	51
132	14	20	27	32	37	42	47
144	12	18	25	29	33	38	43
156	--	17	23	27	31	35	39
168	--	--	31	25	29	33	36
180	--	--	--	23	27	30	34

Notes:
-- = not applicable
in. = inch
Minimum cover is 2 feet.

Figure 8-27 Aluminum Pipe Arch: 2⅔ in. × ½ in. Corrugations – AASHTO M 196

Span × Rise in. × in.	Corner Radius in.	Thickness in.	Thickness Gauge	Minimum Cover Feet	Maximum Cover in Feet for Soil Bearing Capacity of: 2 tons/ft²	3 tons/ft²
17 × 13	3	0.060	16 ga	2	12	18
21 × 15	3	0.060	16 ga	2	10	14
24 × 18	3	0.060	16 ga	2	7	13
28 × 20	3	0.075	14 ga	2	5	11
35 × 24	3	0.075	14 ga	2.5	NS	7
42 × 29	3.5	0.105	12 ga	2.5	NS	7
49 × 33	4	0.105	12 ga	2.5	NS	6
57 × 38	5	0.135	10 ga	2.5	NS	8
64 × 43	6	0.135	10 ga	2.5	NS	9
71 × 47	7	0.164	8 ga	2	NS	10

Notes:
ft^2 = square feet
ga = gallon
in. = inch
NS = Not Suitable

Figure 8-28 Aluminum Pipe Arch: 3 in. × 1 in. Corrugations – AASHTO M 196

Span × Rise in. × in.	Corner Radius in.	Thickness in.	Thickness Gauge	Minimum Cover Feet	Maximum Cover in Feet for Soil Bearing Capacity of: 2 tons/ft²	3 tons/ft²
40 × 31	5	0.075	14 ga	2.5	8	12
46 × 36	6	0.075	14 ga	2	8	13
53 × 41	7	0.075	14 ga	2	8	13
60 × 46	8	0.075	14 ga	2	8	13
66 × 51	9	0.060	14 ga	2	9	13
73 × 55	12	0.075	14 ga	2	11	16
81 × 59	14	0.105	12 ga	2	11	17
87 × 63	14	0.105	12 ga	2	10	16
95 × 67	16	0.105	12 ga	2	11	17
103 × 71	16	0.135	10 ga	2	10	15
112 × 75	18	0.164	8 ga	2	10	16

Notes:
ft^2 = square feet
ga = gallon
in. = inch

Figure 8-29 Aluminum Structural Plate Pipe Arch:
9 in. × 2⅔ in. Corrugations, ¼ in. Steel Bolts, 4 Bolts/Corrugation

	Span × Rise ft-in. × ft-in.	Corner Radius in.	Min. Gauge Thickness in.	Min. Cover Feet	Maximum Cover(1) in Feet forSoil Bearing Capacity 2 tons/ft^2	3 tons/ft^2
a	5 – 11 × 5 – 5	31.8	0.100	2	24(2)	24(2)
b	6 – 11 × 5 –9	31.8	0.100	2	22(2)	22(2)
c	7 – 3 × 5 – 11	31.8	0.100	2	20(2)	20(2)
d	7 – 9 × 6 – 0	31.8	0.100	2	28(2)	18(2)
e	8 – 5 × 6 – 3	31.8	0.100	2	17(2)	17(2)
f	9 – 3 × 6 – 5	31.8	0.100	2	15(2)	15(2)
g	10 – 3 × 6 – 9	31.8	0.100	2	14(2)	14(2)
h	10 – 9 × 6 – 10	31.8	0.100	2	13(2)	13(2)
i	11 – 5 × 7 –1	31.8	0.100	2	12(2)	12(2)
j	12 – 7 × 7 – 5	31.8	0.125	2	14	16(2)
k	12 – 11 × 7 – 6	31.8	0.150	2	13	14(2)
l	13 – 1 × 8 – 2	31.8	0.150	2	13	18(2)
m	13 – 11 × 8 – 5	31.8	0.150	2	12	17(2)
n	14 – 8 × 9 – 8	31.8	0.175	2	12	18
o	15 – 4 × 10 – 0	31.8	0.175	2	11	17
p	16 – 1 × 10 – 4	31.8	0.200	2	10	16
q	16 – 9 × 10 – 8	31.8	0.200	2.17	10	15
r	17 – 3 × 11 – 0	31.8	0.225	2.25	10	15
s	18 – 0 × 11 – 4	31.8	0.255	2.25	9	14
t	18 – 8 × 11 – 8	31.8	0.250	2.33	9	14

Notes:

in. = inch

ft^2 = square feet

(1)Additional sizes and varying cover heights are available, depending on gauge thickness and reinforcement spacing. Contact the HQ Hydraulics Section for more information.

(2)Fill limited by the seam strength of the bolts.

Figure 8-30 Steel and Aluminized Steel Spiral Rib Pipe:
¾ × 1 × 11½ in. or ¾ × ¾ × 7½ in. Corrugations – AASHTO M 36

Diameter in.	Maximum Cover in Feet		
	0.064 in. 16 ga	0.079 in. 14 ga	0.109 in. 12 ga
18	50	72	--
24	50	72	100
30	41	58	97
36	34	48	81
42	29	41	69
48	26	36	61
54	21	32	54
60	19	29	49

Notes:
-- = not applicable
ga = gallon
in. = inch
Minimum cover is 2 feet.

Figure 8-31 Aluminum Alloy Spiral Rib Pipe:
¾ × 1 × 11½ in. or ¾ × ¾ × 7½ in. Corrugations – AASHTO M 196

Diameter in.	Maximum Cover in Feet			
	0.060 in. 16 ga	0.075 in. 14 ga	0.105 in. 12 ga	0.135 10 ga
12	35	50	--	--
18	34	49	--	--
24	25	36	63	82
30	19	28	50	65
36	15	24	41	54
42	--	19	35	46
48	--	17	30	40
54	--	14	27	35
60	--	12	24	30

Notes:
-- = not applicable
ga = gallon
in. = inch
Minimum cover is 2 feet.

Figure 8-32 Thermoplastic and Ductile Iron Pipe

Solid Wall PVC	Profile Wall PVC	Corrugated Polyethylene
ASTM D 3034 SDR 35 3 in. to 15 in. diameter ASTM F 679 Type 1 18 in. to 48 in. diameter	AASHTO M 304 or ASTM F 794 Series 46 4 in. to 48 in. diameter	AASHTO M 294 Type S 12 in. to 60 in. diameter
25 feet max, 2 feet min All diameters	25 feet max, 2 feet min All diameters	25 feet max, 2 feet min All diameters
HDPE	**Polypropylene**	**Ductile Iron Pipe**
Std Spec 9-05.23	Std Spec 9-05.24 12 in. to 60 in. diameter	Std Spec 9-05.13 12 in. to 48 in. diameter
25 feet max, 0.5 feet min. All diameters	25 feet max, 1-foot min. All diameters	25 feet max, 0.5-foot min All diameters

Notes:
in. = inch

Chapter 9 Highway Rest Areas

9-1 Introduction

Contact the HQ Hydraulics Section for design guidance.

Chapter 10 Large Woody Material

10-1 Introduction

Large woody material (LWM), also known as large woody debris (LWD), plays a critical role in many Washington streams through its influence on aquatic habitat and stream geomorphic processes. In many forested streams, wood is a fundamental driver of fluvial geomorphology—the shape of the stream channel and how it changes over time. The quantity, size, and function of LWM in many of these stream systems have been altered through decades of timber harvesting, channel clearing, snag removal, and human alteration to stream channels and riparian zones, resulting in changes to stream channel form and function and the degradation of aquatic habitat. Restoration of in-stream LWM has therefore become a common restoration practice in Washington State and throughout the Pacific Northwest. Placement of LWM can achieve a variety of physical and biological benefits to stream morphology and aquatic habitat. LWM projects can be used to directly provide habitat cover, complexity, and natural levels of streambank stability, or may provide indirect benefits through their influence on pool development, sediment trapping, hydraulic roughness, lateral channel dynamics, and maintenance of channel bedform.

This chapter provides guidance on designing projects that use LWM in streams or rivers. Section 10-2 gives an overview of the design process, including reach assessments (Section 10-3), recreational safety considerations (Section 10-4), and developing and understanding clear project objectives (Section 10-5). Design criteria, including using mobile wood, are discussed in Section 10-6 through 10-8. Section 10-9 provides guidance on inspection and maintenance, and Section 10-10 provides a list of references used in the *Hydraulics Manual*.

Over the past century and beyond, the role of LWM in forming and maintaining stream habitat was not understood or was largely ignored. As settlement and development increased, so did the removal of LWM and boulders from the state's waterways. Past logging practices often removed trees to the edge of the stream, limiting future wood input to the stream. In many cases, streams were also cleared of wood for conveyance or fish migration. Over time, these and other activities resulted in depletion of aquatic habitat and channel-forming processes in many streams. The removal of in-stream LWM has dramatically altered channel form, and how LWM, sediment, and fish moved through the river system. LWM can be used effectively to provide infrastructure protection as well as aquatic habitat.

Since processes associated with LWM have been impaired, aquatic habitat restoration activities are an important method for reintroducing the necessary structure to stream channels. Frequently, the best approach for habitat restoration is to mimic natural conditions to which salmon have adapted. Natural wood loading conditions can help provide a reference to guide quantities, sizes, and placement of LWM as a component of restoration. This approach is most effective when the adjacent riparian forest also mimics natural conditions (or is on a trajectory to reach these conditions) so instream wood recruitment and other riparian processes can be maintained.

Windthrow emulation duplicates delivery of wood to the stream by the uprooting of trees or groups of trees during a windstorm. Trees delivered by windthrow may have only part of the tree in the active channel, often with some of the trunk still on the streambank. The weight of the log on the bank increases the stability and reduces downstream movement. In addition, one or more logs can be placed on top of another, so the weight of the top log pins the lower log. Complex placements with multiple logs with interlocking pieces of wood provide better habitat and mimic wood accumulation over time.

Channel migration in alluvial stream valleys is the principal mechanism of wood recruitment to channels. Numerous studies have shown that erosion rates in areas with mature timber are half or lower the rate in areas with small trees or pasture (Abbe and Brooks 2011; U.S. Bureau of Reclamation [USBR] and USACE 2016). LWM can be a significant factor in reducing erosion rates, though isolated key pieces can locally increase rates. Logjams can also trigger channel avulsions which can then result in large inputs of LWM. Engineered logjam (ELJ) projects have been proven effective in limiting channel migration and in improving channel alignment at bridge crossings.

10-1.1 *Purpose and Need*

Aquatic habitat enhancement and restoration is an important environmental stewardship function in all work within riverine corridors, including eliminating fish passage barriers at stream crossings of the state highway system (see Chapter 7). Fish barriers have functioned to hold stream grade, so replacing these barriers can trigger channel incision. Wood placement in reconstructed channels reduces the risk of future channel incision by improving sediment storage and flow complexity. The addition of LWM for bank stabilization that contains rock can be self-mitigating (determined on a case-by-case basis). Incorporating LWM into bank stability and scour protection projects as sustainable habitat features is encouraged.

The purpose of this guidance is to assist in determining when LWM is appropriate so these features can be incorporated into design at project initiation rather than during a redesign as a response to comments from Tribes and other stakeholders or permitting agencies.

10-1.2 *Guidance for Emergency Large Woody Material Placement*

Generally, failure of a culvert system or a bank requires rapid response to stabilize and prevent additional damage to WSDOT facilities and to restore a safe travel corridor. In these cases, regional maintenance staff likely need to act without the benefit of a reach assessment and a new engineering design to replace damaged facilities. Maintenance staff are left to stabilize or restore the site to the previous design specifications, in likely adverse environmental conditions. In as much as engineering judgement calls are needed during such situations, LWM placement during emergency repairs should be done only in consultation with the HQ Hydraulics Section. The maintenance or project office in charge of emergency repairs must also consult with WDFW and the appropriate tribal contacts for the area. Typically, emergency actions still require permits from regulatory agencies, and those permits may be conditioned with mitigation requirements. In these cases, LWM placement should be included as a mitigation element for aquatic habitat impacts.

10-1.3 *Design Oversight*

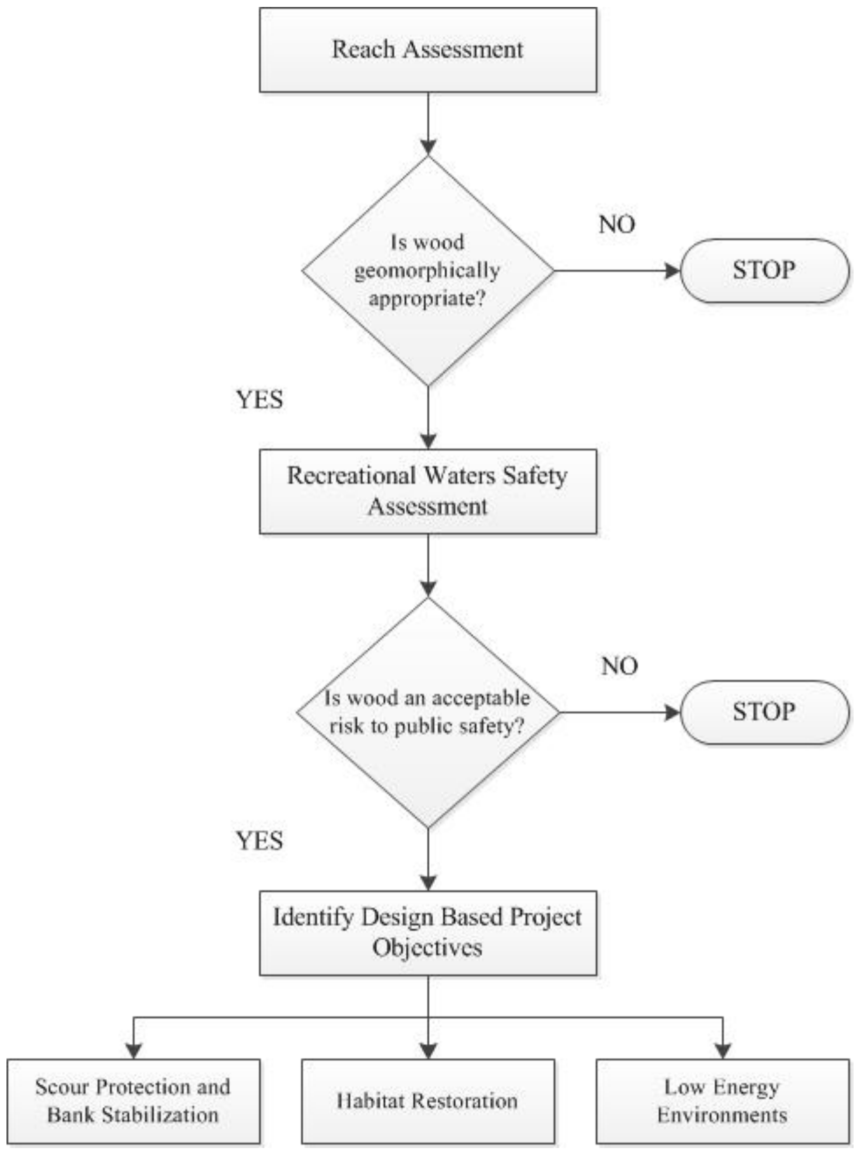
The project designs including LWM or ELJs require expertise in hydrology, hydraulics, geomorphology, riparian ecology, biology, and civil engineering. Because of the risks involved, all LWM placements in bank protection and stream restoration projects shall be designed under the supervision of the HQ Hydraulics Section, as described in Chapter 1. All LWM placement within the 100-year flood elevation must be approved by HQ Hydraulics.

10-2 Design Process

Design of LWM structures and placement shall follow a geomorphic and ecological assessment of the watershed and a similar, more detailed assessment of the river reach or site to be treated, including an analysis of existing conditions and anticipated responses related to stability. The LWM design process is a multistep process shown in Figure 10-1:

- A reach assessment is prepared to describe the geomorphic and habitat conditions of the site, the constraints, and the existing LWM in the system and to determine that the use of LWM is suitable for the site conditions.
- A recreational water safety assessment is made to identify potential risks to the public and provide guidance to reduce potential risks.
- The design-based project objectives are identified.
- The design is created using general and project-specific design criteria.

Figure 10-1 LWM Design Process



Note: Lower energy environments are areas outside the BFW, including wetlands and floodplains.

10-3 Reach Assessments

A reach assessment is required for all WSDOT projects that incorporate LWM. A reach assessment is a scalable report and, based on the conditions at a site, may range from a few paragraphs in the Hydraulic Design Report to a stand-alone report. The level of effort for the reach assessment will be determined by the HQ Hydraulics Section. Reach assessments provide important geomorphic and habitat information that is critical to the successful design of LWM projects.

A reach assessment should follow the ISPG outline (WDFW 2002) and characterize the project site conditions and the larger representative reach of the channel and the watershed. In addition to identifying problems at a site and possible solutions, the reach assessment should include the following:

- A description of LWM found at the project site and within the representative reach including the LWM's likely sources and functions in the channel.
- A discussion of the potential for LWM to be recruited: bank erosion, mass wasting, windthrow, etc.
- A discussion of the ability of the water course to transport LWM to the project site.

NCHRP's *Effects of Debris on Bridge Pier Scour* (Lagasse et al. 2010) and the FHWA's (Debris Control Structures: Evaluation and Countermeasures (HEC-9) provide thorough discussions of the recruitment and transport of LWM.

Finally, the reach assessment should determine if the use of LWM is suited to the conditions found at the project site. The following locations and conditions require additional analysis beyond the typical reach assessment LWM placement:

- Channels that have a history and/or a near-future likelihood of debris torrents and other mass-wasting activity.
- Locations upstream and within 50 feet of permanent culverts or bridges unless LWM is incorporated and designed as a protective project element.
- Locations within or under culverts or bridges.
- Confined channels where the valley floor width is less than twice the BFW.
- Alluvial streams with a gradient of more than 2 percent.
- Non-alluvial streams with a gradient of more than 4 percent.

The NRCS's National Engineering Handbook (Technical Supplement 14J: Use of LWM for habitat and bank protection) provides additional discussion of the limitations on using LWM. The National Large Wood Manual, produced by USBR and ERDC (2016), provides similar discussion.

10-4 Recreational Waters Safety Assessment

Like a reach assessment, a recreational waters safety assessment is a scalable report that, based on the unique conditions at a site that, may range from a few paragraphs in the Hydraulic Design Report to a stand-alone report. The assessment should identify the water body, the likely recreational activities that could occur at the site or in the project reach, and the risks or hazards that LWM may pose to recreational users and determine if LWM can be used with an acceptable level of risk. This type of assessment is often required by the Washington State Department of Natural Resources for aquatic lands use permits and should include an inventory of nearby public access points, such as WDFW and USFS boating access sites. A review of regional paddling guidebooks will also help identify recreational water use. The American Whitewater Association (www.americanwhitewater.org) has a searchable database of recreational river runs.

The following types of water bodies are considered "recreational" by WSDOT for the purposes of this guidance:

- All rivers designated as "Wild and Scenic" rivers.
- All rivers and streams designated as navigational waters by the U.S. Coast Guard.
- All rivers and streams within State Parks, National Parks, National Monuments, National Recreation Areas, and Wilderness Areas.
- Rivers, streams, and other water bodies known to local law enforcement, fire departments, and other river rescue organizations to receive heavy recreational (boating/swimming) use. These organizations can be very helpful in determining the degree of recreational use and relative hazard.
- All streams with a BFW greater than 30 feet.

LWM may present risks to recreational users and these risks should be considered in the assessment and later in the planning and design phases of project development. In general, for channels with recreational boating/floating activities:

- LWM placement in confined channels should be limited to grade control on the streambed and not structures obstructing flow.
- LWM structures shall not be placed where there is poor visibility from upstream.
- LWM structures shall not be put in channels that do not allow for circumnavigation.
- Larger LWM structures shall not be constructed downstream of, or within 100 feet upstream of boat ramps.

Basic engineering standards require consideration of safety and risk and that, ultimately, design decisions regarding the use of LWM in recreational waters must be left to the HQ Hydraulics Section. The methods and assumptions used for the recreational water safety assessment analysis will be fully documented in the project's Hydraulic Design Report.

10-5 Project Objectives

A type of LWM structure or placement should be selected using similar criteria employed for selecting any approach for stream stabilization or habitat rehabilitation:

- LWM structure or placement should address the dominant erosion processes operating on the site.
- Key habitat deficiencies (lack of pools, cover, woody substrate) should be addressed.
- Completed project should function in harmony with the anticipated future geomorphic response of the reach (e.g. erosive reaches should incorporate the potential for erosion and consider increasing overburden or anchoring forces; transport reaches should evaluate the sediment balance within the reach and determine whether LWM would be beneficial to the sediment balance; depositional reaches should consider if accumulation rates will negatively impact the structure or encourage lateral channel migration, etc.).
- Risks to safety for recreational use of the completed project are minimized.

FHWA has published several references that can aide in the selection of appropriate structures for scour and bank protection: Bridge Scour and Stream Instability Countermeasures Experience, Selection, and Design Guidance (HEC-23) and two companion documents— Evaluating Scour at Bridges (HEC-18) and Stream Stability at Highway Structures (HEC-20).

The Washington State Aquatic Guidelines Program has published ISPGs and Stream Habitat Restoration Guidelines, which provide more detailed guidance for using LWM. In addition, the NRCS's National Engineering Handbook Technical Supplement 14J (2007) and the National Large Wood Manual (USBR and ERDC 2016) provide similar discussion.

The balance of this chapter provides general design criteria that apply to all LWM projects and more project-specific criteria related to using LWM in bridge scour and streambank protection projects, stream habitat restoration projects, and floodplain habitat enhancement projects. In addition, Appendices 10A, 10B, and 10C provide photographs and illustrations of typical LWM configurations as well as a brief narrative on its application and limitations.

10-6 General Design Criteria

The following sections provide design criteria that apply to all LWM projects. The criteria cover:

- Design life
- Wood selection,
- Design flow,
- Stability and anchoring
- Scour
- FEMA floodplains and floodways

10-6.1 *Design Life*

One of the key elements in any project design is identifying the design life. Projects that include LWM are no different; however, LWM decays over time. The project objectives need to be considered when selecting LWM as a design element. LWM used to protect banks or redirect flow to protect critical infrastructure are usually intended to be functional for an extended period of time. LWM used primarily for habitat may have a considerable shorter design life as it is anticipated that the riparian corridor will contribute LWM in the future. LWM can last indefinitely if it remains wet or is buried in substrate that is frequently saturated (e.g., the stream banks).

LWM varies by species in its durability and decay-resistant properties. Decay is also directly linked to the size of wood used—the larger it is the longer it will last. It is unlikely that deciduous wood would last for more than 5 or 10 years. Cottonwood and alder, even in the large sizes needed for installations along major rivers, are the most rapidly decaying tree species. While maple will also decay fairly quickly, it is more durable than the other deciduous tree species; water saturated maple may last 10 to 20 years. For maximum longevity, it is best to use decay-resistant coniferous species whenever possible. Well-designed LWM structures can last 50 years or longer.

Of the conifers, hemlock is poorly suited because of its rapid decay rate. While very durable, Sitka spruce and western red cedar have low densities (i.e., are more buoyant) and require more anchoring that other softwoods.

Douglas fir has excellent durability, especially when maintained in a saturated condition; it is also the most abundant of the commercially managed softwoods. Douglas fir will generally survive for at least 25 to 50 years. Such longevity puts this species within the normal estimates of the functional design lifetime expected for conventional riverbank stabilization installations. (Johnson and Stypula 1993). Cedar has the most longevity of any Northwest species but is more susceptible to mechanical damage.

The longevity of any wood will be greatly enhanced if it remains fully saturated (i.e., waterlogged). The maximum decay rate occurs with alternate wetting and drying, or consistently damp condition, rather than full saturation. Repetitive wetting and drying of LWM structures can shorten their life span. Logs that are buried or submerged in fresh water can last for decades or even centuries. Consequently, LWM structural elements should be placed as low as possible, preferably in locations where they remain submerged. This is also preferable for habitat logs.

10-6.2 *Wood Selection*

Both the strength and relative buoyancy of logs is determined chiefly by wood density. The physical characteristics of various tree species are presented in Figure 10-2. The denser the wood used in the structure, the more strength and resiliency the structure has. Conifers are generally specified as preferable for use in LWM structures due to the following factors:

- Density and resultant strength
- Relative uniformity of trunk shape (which makes them easier to construct with than deciduous species)
- Large ratio between the trunk diameter at breast height (DBH) and root wad diameter (roots are shallow and radiate from the stem)

Of the conifer species that occur and are readily available in the Pacific Northwest, Douglas fir has the highest density and the best geometric properties for LWM structures (Figure 10-2). Other conifers such as western red cedar and Sitka spruce have lower specific gravities and strengths (Figure 10-2). These species can be used for cribbing structural members but only used as posts if large enough to exceed strength requirements. Deciduous species generally have lower densities and should only be used for non-structural elements of LWM structures. As described previously, the longevity of any wood will be greatly enhanced if it remains fully saturated (i.e., waterlogged). The Stream Designer should use species best suited for the project location and objectives.

Figure 10-2 Physical Characteristics of Woods Found in the Pacific Northwest

Common Name	Genus	Species	Green Wood (moisture content ~ 30%)			Dry Wood (moisture content ~ 12%)		
			Specific Gravity(1)	Modulus of Rupture N/m^2	Modulus of Elasticity N/m^2	Specific Gravity(1)	Modulus of Rupture N/m^2	Modulus of Elasticity N/m^2
Subalpine Fir	*Abies*	*lasiocarpa*	0.31	3.40E+07	7.20E+06	0.32	5.90E+07	8.90E+06
Western Red Cedar	*Thuja*	*plicata*	0.31	3.59E+07	6.50E+06	0.32	5.17E+07	7.70E+06
Black Cottonwood	*Populus*	*trichocarpa*	0.31	3.40E+07	7.40E+06	0.35	5.90E+07	8.80E+06
Engelmann Spruce	*Picea*	*engelmannii*	0.33	3.20E+07	7.10E+06	0.35	6.40E+07	8.90E+06
Grand Fir	*Abies*	*grandis*	0.35	4.00E+07	8.60E+06	0.37	6.10E+07	1.08E+07
Sitka Spruce	*Picea*	*sitchensis*	0.37	3.90E+07	7.40E+06	0.40	7.00E+07	1.08E+07
Ponderosa Pine	*Pinus*	*ponderosa*	0.38	3.50E+07	6.90E+06	0.40	6.50E+07	8.90E+06
Red Alder	*Alnus*	*rubra*	0.37	4.50E+07	8.10E+06	0.41	6.80E+07	9.50E+06
Silver Fir	*Abies*	*amabilis*	0.40	4.40E+07	9.80E+06	0.43	7.30E+07	1.19E+07
Yellow Cedar	*Chamaecyparis*	*nootkatensis*	0.42	4.40E+07	7.90E+06	0.44	7.70E+07	9.80E+06
Mountain Hemlock	*Tsuga*	*mertensiana*	0.42	4.30E+07	7.20E+06	0.45	7.90E+07	9.20E+06
Western Hemlock	*Tsuga*	*heterophylla*	0.42	4.60E+07	9.00E+06	0.45	7.80E+07	1.13E+07
Big Leaf Maple	*Acer*	*macrophyllu*	0.44	5.10E+07	7.60E+06	0.48	7.40E+07	1.00E+07
Douglas Fir	*Pseudotsuga*	*menziesii*	0.45	5.30E+07	1.08E+07	0.48	8.50E+07	1.34E+07

Notes:

N/m^2 = newton per square meter

(1)Specific gravity computed from oven-dry weight (0% moisture) and volume at 12% moisture content.

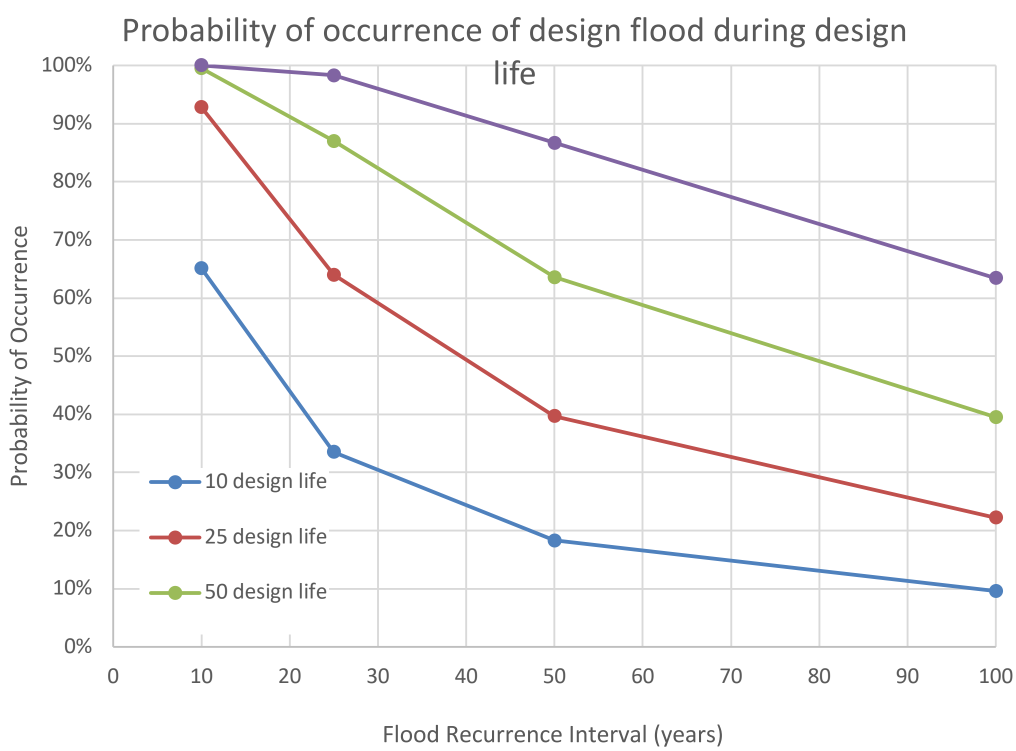
10-6.3 Design Flow

When designing LWM placement, several flows must be considered. Because most LWM bank stabilization and flow-directing structures are intended to function over a long project design life (50 years or longer), design flows equivalent to the 100-year recurrence flood must be used to estimate depth and channel velocity to estimate buoyancy and drag loads on LWM to ensure that they do not become mobilized during extreme floods to the detriment of the projector other facilities. However, wood for habitat should be placed in the channel to interact with water at low flow conditions.

Although LWM for habitat projects may have a shorter design life, to reduce risks to WSDOT and other infrastructure and property, the 100-year recurrence flood flow shall be used for stability and scour analyses. Climate resiliency should also be considered as current science suggests both the magnitude and frequency of peak flows are expected to increase (WDFW 2016). The mean annual discharge or more frequent flows should be considered for the purpose of placing LWM in the channel so that it regularly interacts with the low flow channel to enhance or create habitat. Mobile woody material (MWM; see Section 10-8) may use a lower recurrence interval design flow, based on habitat objectives.

Figure 10-3 shows that for a project design life, a design flow of the same recurrence has about a 63 percent chance of occurring during the project life, regardless of the flow. It also shows that the likelihood of a project experiencing a design flood increases somewhat as the recurrence interval increases.

Figure 10-3 Design Flow Risks Occurring During Project Life



Note:
Probability of a single exceedance over design life: P = 1 - (1 - 1/RI)^N

As described in Chapter 2, design flows can be determined from gauge data (preferred), regional regression analyses, or hydrologic models (e.g., MGSFlood). The USGS StreamStats website has links to gauge- and regression-based flow data.

10-6.4 *Stability and Anchoring*

A force balance analysis will identify the potential for incipient motion of LWM. The ultimate mobility of the wood will then depend on the stream's ability to transport the wood based on flow depth and power and riparian features such as established trees that will resist wood transport (mobility resistance).

10-6.4.1 Incipient Motion

LWM is subjected to a combination of hydrodynamic, frictional, and gravitational forces that act either on the LWM or on its anchors. The principle forces are listed below:

- Vertical buoyancy force acting on the LWM and transferred to its anchors
- Horizontal fluid drag force acting on the LWM and transferred to the anchors
- Horizontal fluid drag force acting directly on the anchors
- Vertical lift force acting directly on the anchors
- Immersed weight of the anchor (if boulders are used as anchors)
- Frictional forces at the base of the anchor that resist sliding (if boulders are used as anchors) or being pulled out (if posts or pilings are used as anchors)

At a site where the objective is primarily habitat enhancement, it is preferable to not have artificial anchors for LWM, but this must be carefully considered. LWM can, if sized and positioned correctly, be "self-ballasting" during the design flow. This means enough mass of the wood is above water to counteract the buoyant and drag forces of the wood below water. In addition, a mobility analysis/risk analysis (see below) should be conducted to show that the wood, if mobilized, would not move a significant distance, and/or that there is little or no risk to property or infrastructure downstream.

There are numerous techniques for anchoring LWM. In order of preference, here are some commonly used anchoring techniques:

- Natural existing vegetation
- Self-ballasting
- Wood ballast
- Soil ballast
- Wood piles/racking
- Boulder ballast
- Earth anchors
- Boulder anchors
- Dolosse-timber or log jacks
- Deadman anchors

LWM can be attached to anchors with rope (less common), steel chain, steel cable, rebar pins, or threaded bolts and nuts. Generally speaking, the fewer components in the anchoring system, the better. This is true not only because there are fewer connection points to fail, but also there are fewer nonnatural elements entered into the stream system. USBR (2014) provides extensive guidance on and examples of anchoring systems.

Wherever possible, redundant anchoring systems should be used. Examples of this include combining pilings or anchors with bank overburden partially burying the LWM in the bank. Anchoring systems should be designed with an appropriate factor of safety to account for uncertainty and risk, where the factor of safety is defined as the ratio of the resisting forces divided by the driving forces. WSDOT generally uses factors of safety of 1.5, higher if there is greater uncertainty in force balance calculations and if the wood mobility could pose a high threat to infrastructure. The 100-year discharge is used as the design flow. More frequent design flows may be used if the wood function is primarily for habitat. The HQ Hydraulics Section must be consulted for projects proposing design flows more frequent than the 100-year flow.

The USBR (2014) has developed guidance on selecting safety factors to use for each of the forces described previously (*Large Woody Material – Risk Based Design Guidelines*) that considers the risks to public safety and property damage. A design that proposes factors of safety less than 1.5 shall be coordinated with and approved by the HQ Hydraulics Section.

There are numerous guidance documents dealing with the stability analysis equations for estimating these forces. A description of applicable equations and their use can be found in NRCS (2007) and D'Aoust's (2000) *Large Woody Debris Fish Habitat Structure Performance and Ballasting Requirements* (1991). More recently, the USFS has published *Computational Design Tool for Evaluating the Stability of Large Wood Structures* (Rafferty 2016), which is the accepted reference for such calculations. Other methods may be acceptable upon review by the HQ Hydraulics Section.

The buoyancy force Factor of Safety calculation is based on the Equation 10-1:

$$\mathrm{FOS}_{\mathrm{buoyancy}} = \mathrm{F_D}/\mathrm{F_U} \qquad (10\text{-}1)$$

Where:

F_D = total downward force

F_U = total upward force

And where:

$F_D = W_O + W_{anchor}$

And:

W_O = weight of overburden

W_{anchor} = weight of anchor

And where:

$F_U = B_{root} + {}_{Bbole}$

And:

B_{root} = buoyancy of rootwad

B_{bole} = buoyancy of log bole

Appendix 10A contains the parameters and equations for calculating weight and buoyancy of the objects in an LWM structure. Note that this is just a framework and that the specific design of a structure may necessitate inclusion of calculations for logs that interact with each other (e.g., a structure with a footer log and a rack log). More complex structures will require multiple interrelated FOS calculations.

The FOS_{drag} (same as USBR's $FOS_{sliding}$), is based on:

$$FOS_{drag} = F_f/F_{Dr} \tag{10-2}$$

Where:

F_f = total friction force

F_{dr} = total drag force

And where:

$F_f = -(F_D - F_U)*C_{rl}$ riverbed-log friction coefficient

And:

C_{rl} = riverbed-log friction coefficient

And where:

$F_{Dr} = C_{dr}(y/g)*(v)\text{^}2*(A_{rtwd})\text{^}0.5$

And:

C_{dr} = unitless drag coefficient

y = specific weight of water

g = gravitational acceleration

v = computed water velocity

A_{rtwd} = projected area of rootwad

Moment force is not typically a concern for LWM structures in Washington streams, since the structures are usually long in the direction of flow, narrow in the direction perpendicular to flow, and not very tall (USBR 2014). Nonetheless, the LWM spreadsheet tool calculates the moment forces. See Appendix 10A for more information. The methods and assumptions used for stability analysis will be fully documented in the project's Hydraulic Design Report.

10-6.4.2 Mobility Analysis

By default, the risk associated with movement is equated with incipient motion—essentially equating failure with any movement of placed wood. However, there are cases when considering the risk of LWM mobility, once moved, can help achieve project objectives. This is primarily when the project objective is exclusively habitat restoration or enhancement. Many natural stream corridors also have riparian trees and other features that may resist transporting wood downstream, especially in smaller streams where the wood is large relative to the flow depth.

In such cases, an LWM mobility analysis may be conducted that assesses the likelihood of LWM movement in a stream reach as well as the potential impact to property and infrastructure. Currently there is no well-established methodology for conducting such an analysis, but certain references may be helpful (Braudrick and Grant 2000; Kramer and Wohl 2016; Ruiz-Villanueva et al. 2016). The HQ Hydraulics Section will review and approve any mobility analysis. It is helpful to contact the HQ Hydraulics Section before beginning the mobility analysis work.

10-6.5 *Scour*

Scour is the principal failure mechanism of many in-stream structures, such as bridge piers, abutments, rock revetments, levees, and flood walls. It is also a primary threat to LWM structures, from simple log weirs to large ELJs. Scour at LWM placements creates important habitat features but can also cause undesirable movement or destabilization of logs and/or streambanks. LWM placements must be designed to accommodate anticipated scour conditions, particularly if the LWM is for habitat objectives. The destabilizing effects of scour can be minimized by substantial embedment of rack logs in the streambank; this can be done in a way that assures continued engagement of the wood with low flows. LWM shall be located so that it does not create scour that could undermine bridge members (e.g., piers, abutments) or road embankments. Bioengineering techniques should be considered whenever the bank opposite the LWM is made of fill or is unconsolidated natural material, and the LWM is expected to direct flow toward the opposite bank.

Reliable methods for estimating scour at LWM placements have not yet been developed in either the engineering or the scientific communities. In some cases, equations developed for bridge piers and abutments have been used to predict scour, but these are overly conservative for gravel bed streams found in much of Washington and may not accurately represent the unique geometry of LWM. Scour analysis for LWM projects will therefore often rely heavily on engineering judgment and lessons learned from practical experience. It is always worthwhile to measure residual pool depths (the difference in depth or bed elevation between a pool and the downstream riffle crest) in a project reach to get minimum estimates (during flood flows these pools may deepen). The methods and assumptions used for this analysis will be fully documented in the project's Hydraulic Design Report.

10-6.6 *FEMA Floodplain and Floodways*

A FEMA floodway is the portion of a floodplain that is designated to carry the majority of flood flows through a particular area. Floodways are often intensively regulated by the local community and FEMA. The regulations often restrict or prevent additional fill being placed in the floodway to prevent worsening flood conditions due to development. To enforce this, many local communities have enacted "Zero Rise" flood regulations. This means that a project proponent shall demonstrate through hydrologic and hydraulic modeling that their project will not increase flood elevations.

10-6.7 *Recreational Safety*

It is recognized that river recreation, including swimming, boating, and fishing, carry varying degrees of risk. The level of risk is influenced by many factors, including the person's level of experience, skill, and judgment, and conditions in the watercourse, such as depth, turbulence, velocity, temperature, bank form (steep banks or beach), and in-stream elements, such as LWM.

Given that the planning-level recreational waters safety assessment (Section 10-4), indicated that LWM would be an acceptable risk, LWM may still present residual risks to recreational users and these risks should be considered in design:

- LWM structures shall not be constructed in confined channels except as grade control on the streambed and not obstructing the channel.
- LWM structures shall be placed where there is good visibility from upstream (50 feet or three BFWs, whichever is larger).
- LWM structures shall not be put in channels that do not allow for circumnavigation. Locations that include features such as gravel bars allow recreational users to land, walk around, and avoid the LWM structures.
- Larger LWM structures, such as ELJs, shall not be placed on the outside of a meander bend where the curve ("tortuosity") of the bend is less than 3 using the formula Rc/W<3, where Rc is the radius of the meander curve, and W is the BFW in the upstream riffle.

- Larger LWM structures shall not be constructed in close proximity downstream from boat ramps (100 feet or three BFWs, whichever is larger).
- Signage should be addressed on a case-by-case basis, particularly where upstream visibility is limited due to meandering channels, etc.

In addition to the safety considerations regarding placement of LWM structures, LWM structures should be designed to limit flow-through characteristics by including an impermeable core to prevent "straining." Straining is a phenomenon by which swift water flowing through an LWM structure tends to draw floating objects toward and into it. The denser the core of the structure, the less this tends to occur.

At sites with heavy recreational use, public notification and involvement may be desired to minimize the risks of LWM structures. Public notification should be handled on a case-by-case basis depending on the size and complexity of the project and the degree of public use of the water body. The public involvement procedures under the National Environmental Policy Act and the State Environmental Policy Act should be used as the primary mechanism for informing the public about WSDOT LWM projects.

Guidance for these processes can be found in the *Environmental Manual* M 31-11, Chapter 400. Additional guidance for public involvement can be found in WSDOT's *Design Manual*.

10-7 Project-Specific Design Criteria

10-7.1 *Bridge Scour and Bank Stabilization*

Bridge scour repair and bank stabilization are important preservation functions. These activities preserve the infrastructure, protect the public investment, provide that bridges and highways function properly for their design life, and protect the safety of the traveling public. Bridge scour consists of the undermining of bridge piers, abutments, and other structural components by the erosive forces of rivers. Bank scour may occur as part of bridge scour or independently at other locations along the highway embankment. As a result, bridge scour repairs, scour countermeasures, and bank stabilization inherently involve in-water work.

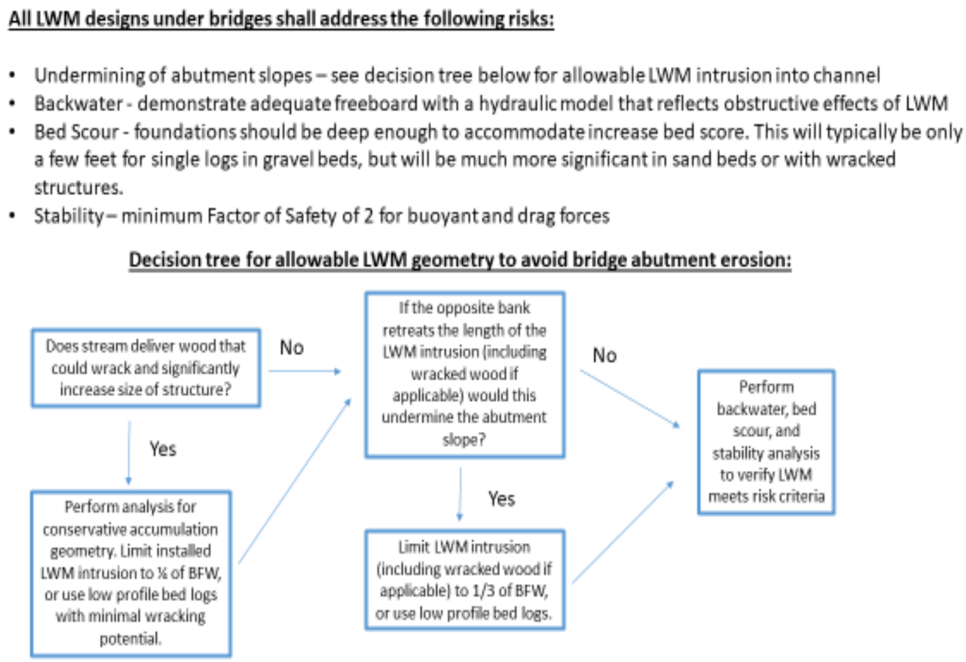
Because of the high impact that damage to bridge infrastructure can have, we must minimize the risks associated with incorporating LWM into projects, particularly if LWM is considered adjacent to or under a bridge. Figure 10-5 shows a flow chart for consideration of LWM adjacent or under a bridge. Note that placement of anchored LWM within the limits of the 100-year flood under a bridge is excluded. Mobile wood (designed for incipient motion at less than the 100-year flood; see Section 10-8) may be used under a bridge. Outside the limits of the 100-year flood, LWM and other habitat features may be considered (Section 10-7.2).

Public safety concerns for recreational users also pose additional risk in utilization of LWM. This is particularly true with regard to bridges for three reasons:

- Loading of LWM on bridge piers can place immense force against the structure that can increase the likelihood of damage or failure. If a bridge is also experiencing scour problems, then these risks can mutually reinforce the effects, dramatically increasing threat to the structure and the safety of the traveling public.
- Bridges often present preexisting obstructions to flow (such as piers, abutments, etc.), that affect various aspects of flow and sediment dynamics including velocity, flow directions, and backwater effects.

- Bridges located at the intersection of highways and rivers and highways adjacent to rivers often present the easiest way for the public to access the river for boat launches, fishing and swimming access, trails, etc. The public is naturally drawn to these highway/river interfaces; thus, public safety concerns are heightened.

Figure 10-4 Decision Tree for Consideration of LWM Under or Adjacent to Bridges



To ensure for adequate public safety and the stability of ELJ and other LWM structures for bridge scour projects, it must be emphasized that design shall be coordinated through the HQ Hydraulics Section (Chapter 1). The project objective, and the surrounding infrastructure, must be considered. Where LWM is to be incorporated into bank stability design, the decay and degradation of the wood over time must be considered. Where needed, bank stabilization measures should contain redundancies (such as traditional "hard" structural measures).

Appendix 10B provides photographs and brief narratives of various types of LWM installations, While the primary intent of the appendix is as a guideline for siting and structure design, it may also help define parameters for permit conditions and for carrying out due diligence with regard to public safety concerns expressed by some recreational river users. In addition, resources such as the ISPG and HEC-23 are available to help guide selection of appropriate bridge scour and bank instability countermeasures.

Many LWM structures are gravity-based, meaning that they rely on the weight of the structures and overburden to remain stable. Structures can also be stabilized using vertical elements such as driven piles or excavated vertical and batter (inclined) posts (Abbe and Brooks 2011). These structures rely on passive earth pressure and skin friction acting on vertical timbers. These structures can also include horizontal elements such beams or cribbing. Cabling or chain can be used to secure horizontal logs to structural piles or posts. Large and complex LWM designs are generally better suited to larger streams (greater than 30 feet BFW). This includes structures such as high crib walls, flow deflection jams, apex bar jams, and dolotimbers (concrete dolo and timber assemblage (Abbe and Brooks 2011). More sophisticated engineering, geomorphic, and hydraulic analyses are necessary to achieve stability and desired function for complex designs in larger streams. Single logs will have minimal effect on the larger streams. Additionally, large streams are more likely to be used by recreational users for swimming, rafting, and boating. Potential impacts to recreational users should be included in the design process. These more complex structures include ELJs, which are structures that:

- Are modeled after logjams that are formed by natural riverine processes.
- Extend both below predicted scour depth and above the bankfull water surface, similar to natural logjams.
- Can be designed either as a gravity structure, a piling anchored structure, or a combination of both depending on site conditions and intended function.
- Consist of 10 or more logs and are designed to be multiple layers of logs high. In plan view, these are usually configured in a triangular, square, fan, or crescent shape.
- Are designed to redirect flow for streambank protection and stability, similar to the function of traditional groins or spur dikes, but with added advantage that ELJ deflectors allow you to establish a riparian buffer between the road and river channel.

For WSDOT to use these large, complex designs, the HQ Hydraulics Section need to be involved early in the process and represented on the design team. Due to the specialty nature of these projects, this work may be contracted out to a consultant. In this case, the primary role of the PEO would be to provide informed comments on consultant work products. Consultant contracts shall be written and managed by the HQ Hydraulics Section.

10-7.2 *Stream Habitat Restoration*

WSDOT often performs stream habitat restoration to reconstruct stream corridors through new bridges or culverts. Stream habitat restoration may also occur in road widening or realignment projects or as an element of wetland or aquatic habitat mitigation projects. Permitting agencies will often require WSDOT to incorporate LWM into these projects as sustainable habitat features. These features increase the channel complexity and diversity of habitat necessary to support a healthy aquatic ecosystem.

The concept of stream restoration refers to returning degraded ecosystems to a more stable, healthier condition. In some systems this includes allowance for processes such as channel migration. All crossing designs should not just consider flow conveyance, but also the passage of sediment and wood. Many streams have been severely impacted by land clearing and urbanization, resulting in changes to their hydrologic and sediment regimes, loss of streambank vegetation, and channel alterations. Restoration upstream of crossings can help to reduce risks by capturing mobile wood that might otherwise cause blockages. Restoration also can be instrumental in preventing channel incision through a new crossing.

Stream restoration activities include the following:

- Constructing channels with the appropriate planform, grade, width, and depth, and channel substrate, as discussed in Chapters 4 and 7.
- Constructing overbank and floodplain areas, where appropriate.
- Decision Tree for Consideration of LWM Under or Adjacent to Bridges
- Stabilizing the channel banks and disturbed floodplain and upland areas with revegetation and bioengineering according to WSDOT's *Roadside Manual* M 25-30.

LWM is typically used to provide the habitat and geomorphic functions associated with key pieces. Key pieces are logs that are large enough to persist and influence hydraulics and bed topography in a stream through a wide range of flow conditions and provide the following functions, either directly or indirectly:

- Creation of stable obstructions that capture organic debris and form logjams
- Pool formation
- Eddy creation and flow complexity
- Deposition of finer sediments to create substrate diversity
- Enhance hyporheic flow by locally increasing hydraulic head
- Cover for aquatic organisms
- Woody substrate for invertebrates and other aquatic species
- Accumulation of mobile wood and other organic debris
- Help activate side channels with flood flows

WSDOT may install LWM to provide these functions where infrastructure or land use limits natural delivery of LWM, or where replanted riparian zones are not expected to deliver LWM for many decades. Note that all vegetation to be cleared on a site must be evaluated for use for habitat purposes and so used if determined to be acceptable quality.

Reconstructed channels near WSDOT infrastructure require a level of predictability that will often limit the ability to place wood in a fully natural manner. In these cases, wood will be placed with anchoring systems that emulate natural key piece functions while limiting wood movement and hydraulic effects that would threaten public safety, infrastructure, or other resources.

LWM can enhance stream stability by dissipating energy, reducing basal shear stress, deflecting erosive forces, and encouraging deposition of bed material. LWM may also be strategically placed to improve the stability and to facilitate establishment of the designed channel banks and bed.

10-7.3 *Habitat Design Process*

The LWM habitat design process is multistepped. Assuming that a reach assessment and the recreational water safety assessments indicate LWM is suitable for a project site, the next steps are listed below:

- Determine the BFW, depth, and gradient
- Identify grain size distribution of the streambed
- Identify the characteristics of the key pieces
- Identify the quantity of key pieces
- Configure the key pieces

The BFW is a determining factor identifying the size and number of key pieces that should be used. As described in Chapter 7, the WDFW's WCDG (Appendix 10C) describes in detail the procedures for determining BFW.

The following sections provide narratives of key piece characteristics, quantities, and configurations. Appendix 10A works though an example of the design process for a western Washington fish passage project.

10-7.3.1 Key Piece Characteristics

Key pieces must be logs with sufficient structural integrity to resist decay, abrasion, and breakage. Although conifers are strongly preferred due to their higher resistance to decay, deciduous species may be considered if they naturally act as key pieces in the riparian community in the project area. Roots and bark shall be retained to the extent practicable to maximize habitat values. Rootwads significantly improve the stability and habitat benefits of key pieces and should not be cut or broken off (e.g., Abbe and Montgomery 1996; Abbe and Brooks 2011). To be as effective as possible, rootwads must not be cut or broken off. Logs should arrive at the staging area with the rootwad fully intact.

The size of key pieces shall be sufficient to provide the mass needed for persistence and habitat formation. This is generally defined by the DBH, measured at a height of 4.5 feet above ground for standing trees. Figure 10-5 provides typical DBH of key pieces for various ranges of BFWs.

Figure 10-5 Bankfull Widths - Minimum Volume of Logs for Key Pieces

Bankfull Width (feet)	Minimum Volume (cubic yards)	Diameter at Breast Height 25-foot log (inches)
0 to 16	1.3	15
17 to 33	3.3	22
34 to 49	7.8	33
50 to 66	11.8	40
67 to 98	12.8	41
99 to 164	13.7	42
164 to 328	14.0	43

Source: Fox and Bolton 2007

10-7.3.2 Target Quantities of Key Pieces

Projects should seek to place key pieces in a manner that emulates natural delivery by bank erosion, wind throw, and landslides. Fox and Bolton (2007) measured several parameters of wood in streams of various widths and in various environments. Because this is the most detailed study of LWM in Washington, the *Hydraulics Manual* uses it as a reference. Additionally, when LWM is being used to emulate habitat functions in a newly created reach of stream, the 75th percentile of key piece density found by Fox and Bolton (2007) will be set as the target quantity. This was identified by the authors of that study to compensate for cumulative deficits of wood loading due to development. Figure 10-6 shows the target number of key pieces per 100 feet of stream for each of the categories of streams.

To account for portions of the channel where infrastructure limits LWM placement (e.g., under a bridge or in a culvert), a higher density may be needed in some channel segments to achieve the target density for the entire restored segment.

Density targets assume the LWM will be engaged with in-stream flows so that it functions to create habitat such as pools, low velocity refugia, cover, capture sediment, or grade control. To best achieve these functions LWM should be placed within the low flow channel and must be stable at the design discharge. In some settings some excavation or other means of stabilization such as batter piles or rock ballast may be necessary.

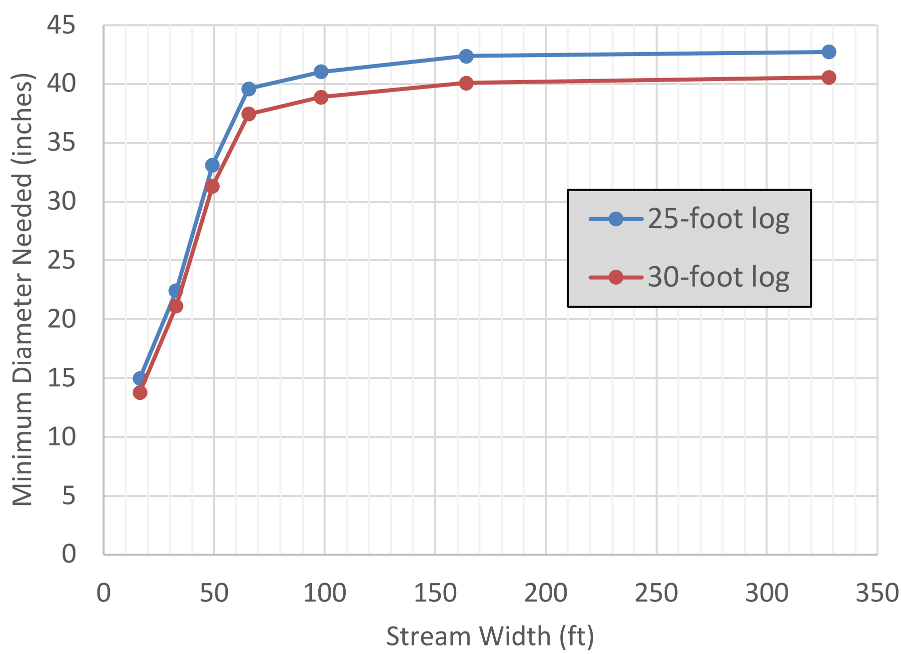
Figure 10-6 Target Number of Key Pieces of LWM

Region	Bankfull Width (feet)	75th Percentile[1]
Western Washington	0 to 33	3.3
	34 to 330	1.2
Alpine	0 to 49	1.2
	50 to 164	0.3
Douglas Fir/Ponderosa Pine Zone	0 to 98	0.6

Source: Fox and Bolton 2007.
Note:
(1)Number of key pieces per 100 feet of stream.

In addition, size of key pieces is important. Figure 10-5 indicates the volume of key pieces needed for each of several BFW classes; this data was used to create a guide for sizing key pieces, in the chart below. Sizing is based on using Douglas Fir (other species may have slightly different results [Figure 10-7]). The single log method of Rafferty (2016) was used to determine the DBH using total volume of wood from Fox and Bolton (2007). This can be used to size the diameter of other log lengths not presented here.

Figure 10-7 Guide for Sizing Key Pieces



Sources: Fox and Bolton 2007; Rafferty (2016).

10-7.3.3 Configuration

The configuration of LWM will depend on the project objectives. Configuration of LWM for bank protection is different than that for aquatic or floodplain habitat enhancement. To provide the best certainty for fish habitat, mimicking natural configurations and spatial organizations known to foster adaptations by salmonids is recommended. For example, see Fox (2003) and Abbe and Montgomery (1996).

10-7.3.3.1 Large Woody Material for Bank Stabilization/Protection

In most fish passage and stream restoration projects, there is a need to protect newly constructed streambanks composed of unconsolidated fill, until revegetation provides enough root strength. Logs with rootwads still attached can be used to absorb energy from high flows, break up turbulence, and deflect momentum of the water away from the streambank. The size of wood, elevation of placement, angle of placement, and height of structure are all site-specific elements that depend on channel geometry and anticipated depth and shear stress of the design flow. Additionally, use of LWM, rather than rock, is often a permit condition.

Numerous guidance documents are available to assist in determining configuration of LWM for streambank stabilization. These include the ISPG (WDFW 2002), NRCS (2007), and USBR and ERDC (2016). Some examples of configuration can be seen in Appendix 10B.

10-7.3.3.2 Large Woody Material for Aquatic Habitat Enhancement

Before laying out the LWM design for aquatic habitat enhancement, it is important to have some understanding of the species that utilize the stream and what habitat features the design will provide. The Stream Designer needs to know what kind of fish and what kind of habitat is needed and how the channel has been impacted by the loss of functional wood. For example, many channels experience incision or downcutting after wood is removed, which can impact culverts and bridges. Thus, restoring functional wood is not simply just for habitat, but can be important in protecting infrastructure. In addition to the resources in the following paragraphs, the RHE and the HQ Environmental Services Office resource specialists are available to assist. The Stream Designer should consider the following:

1. Is the stream fish bearing?

   The Washington State Department of Natural Resources Forest Practices Application Mapping Tool identifies fish bearing streams. It is helpful to determine fish species in the reach since different species have different habitat preferences or needs. The WDFW SalmonScape web mapping tool identifies the presence of various salmonid species.

2. What is the habitat-limiting factor that the project would address?

   Common limiting factors in Washington's waterways include: water quality (temperature, sediment), stream flow, in-stream structure and complexity, pool size and/or frequency, spawning habitat, overwinter habitat, rearing habitat, and interaction with floodplain. Assessments identifying the limiting factors for a stream or basin have been completed for about half of Washington's watersheds in accordance with the 1998 Washington State Watershed Management Act. Links to studies and reports for each Water Resources Inventory Area can be found at Ecology's website: ecology.wa.gov.

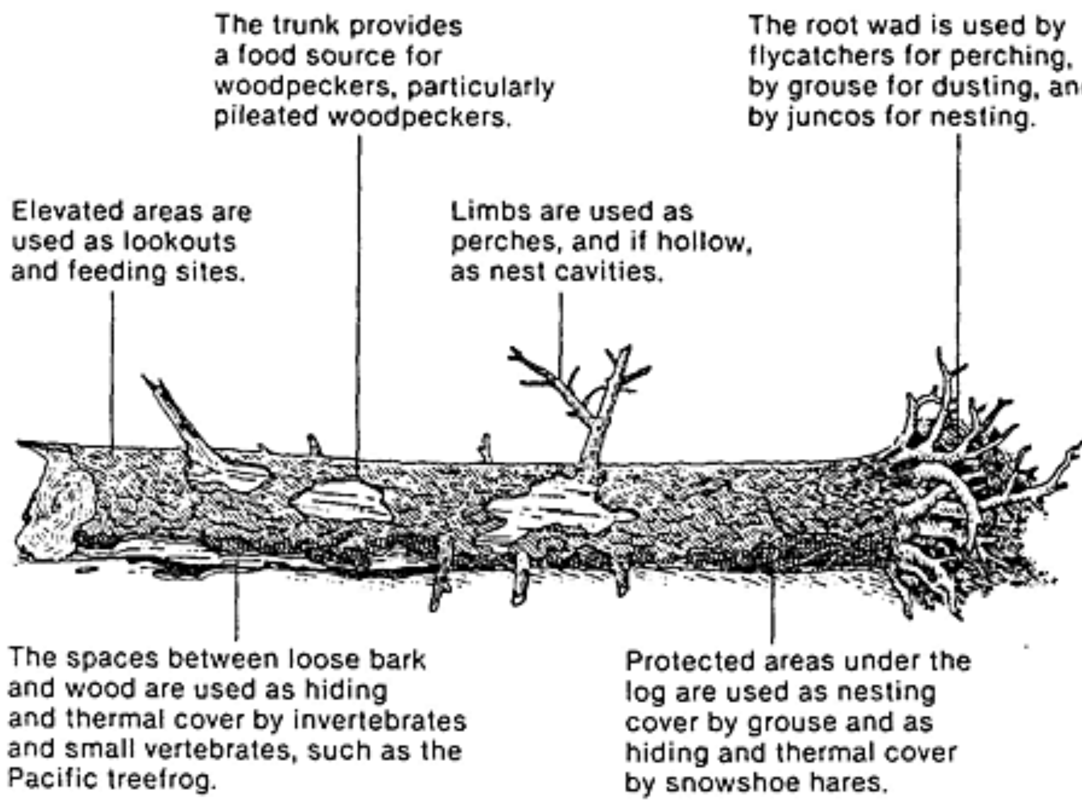
Knowing the species life history and habitat needs, as well as an understanding of the stream system, helps identify an appropriate LWM configuration. For example, LWM located at the outer limits of the bankfull channel may provide high flow refuge but provide little rearing habitat or summer thermal refugia as it may be well away from the active low flow channel. Conversely, LWM placements low in the channel to enhance low flow habitat values may not provide high flow refuge.

Whenever possible, a tree with a rootwad attached should have the rootwad placed in the active channel. The roots create excellent hiding habitat for juvenile fish. The roots also add to the stability of the structure by maintaining contact with the stream bottom over a wider range of stream flows. Appendix 10C provides some typical LWM layouts that are used commonly for stream restoration projects.

10-7.3.3.3 Floodplain and Wetland (Low Energy) Environments.

Dead and down woody materials are important components of wildlife habitats in western forests (Figure 10-9). These materials furnish cover and serve as sites for feeding, reproducing, and resting for many wildlife species. LWM can be placed in low energy aquatic environments such as wetlands and floodplain fringes where flooding is so shallow and slow moving that the LWM cannot be mobilized.

Figure 10-8 Habitat Benefits of LWM in Low Energy Environments



Source: Bartels et al. 1985.

10-8 Mobile Woody Material

Clearing riparian areas for construction access will often result in the accumulation of downed woody material. This material is commonly left in slash piles or disposed of by the construction contractor. Woody debris is an important component of aquatic and terrestrial habitats with many crucial ecological functions: habitat for organisms, energy flow, and nutrient cycling. Consequently, permitting agencies are increasingly requiring redistribution of this material as MWM within the stream corridor after construction is completed. The following sections describe the transport of MWM and guidelines for its placement.

10-8.1 *Introduction*

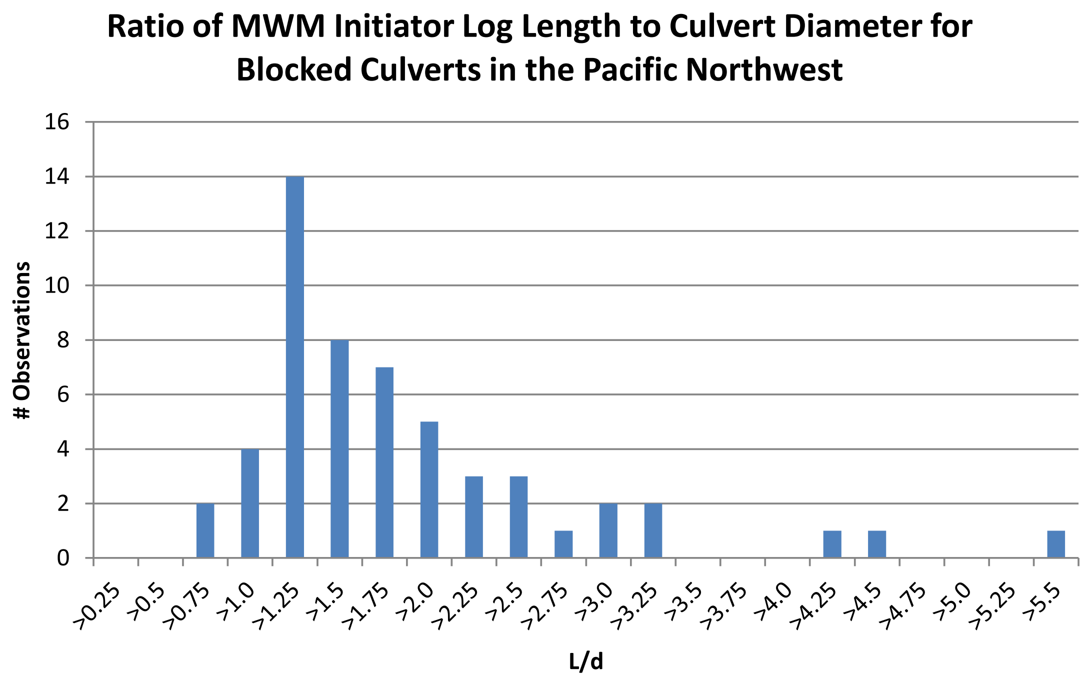
MWM is defined as meeting the minimum criteria for LWM as per WAC 220-660-220[1]—larger than 4 inches in diameter and 6 feet in length—while not meeting the size criteria for stable LWM key pieces, as defined in the *Hydraulics Manual*.

Studies on the transport of MWM in streams in the Pacific Northwest and Northern California emphasize the differences between two distinct wood transport regimes: uncongested and congested (Braudrick et al. 1997). During uncongested transport, individual logs move without piece-to-piece interactions and generally occupy less than 10 percent of the active channel area. In congested transport, logs move together as a single coordinated mass or “raft” and can occupy more than 33 percent of the active channel area. Congested wood transport can result in stream channel blockages due to its large effective size relative to its individual members and can result in channel migration, bank erosion, and blockages of downstream road-stream crossings. Congested wood transport is relatively rare; most accumulations of MWM tend to break apart and the pieces move individually (e.g., Diehl and Bryan 1993).

Studies of MWM blockages at culverts in small streams indicate that the plugging of culverts by MWM is typically initiated by one or more “initiator pieces” lodging across the culvert inlet during high flows (Furniss et al. 1998; Flanagan 2005; Figure 10-10). The point of contact with the edge of the culvert barrel then becomes a nucleation site for the continued accumulation of finer material – both wood and sediment. Wood accumulating over multiple floods will eventually result in diminished culvert capacity or complete blockage. Only 3.7 percent (2 out of 54) of initiator pieces in plugged culverts had lengths that were between 75 and 100 percent of the culvert width, and in both of those instances the initiator pieces had substantial root wads attached that had lodged themselves on the barrel edges of the culverts. This implies that if MWM is to be sized so that downstream culvert clogging is to be minimized, then individual logs with root wads should be no longer than 75 percent of the downstream culvert diameter and MWM without root wads should be no longer than 100 percent of the downstream culvert diameter. The use of MWM must be evaluated on a site-specific basis – the degree of mobility with the riparian corridor, the amount of natural wood recruitment, and the distance to the next downstream culvert are all factors. The HQ Hydraulics Section shall approve the placement and use of MWM.

An additional study (Flanagan 2003) indicates that 99.5 percent of fluvially transported pieces of MWM through low-order channels are shorter than the BFW of the stream.

Figure 10-9 Ratio of MWM initiator log length to culvert diameter



Source: Flanagan 2005.

10-8.2 *Design Criteria*

This section provides design criteria for using MWM to improve ecologic functions in the riparian corridor while minimizing downstream disturbances that could lead to property damage and tort liability.

- MWM should be placed as “racking” material in front of stable logjams.
- MWM can be placed on top of stable logjams to improve revegetation.
- MWM should be placed in a riparian area cleared of trees (in the case of a constructed channel) between the edge of the active stream channel or floodway and the 100-year flood elevation.
- MWM should be distributed in consultation with the HQ Hydraulics Section throughout the impacted project area within the stream corridor.
- The MWM should be distributed at a wide range of elevations in the impacted area to prevent mass mobilization of MWM in a single high flow event.
- If there is a culvert or bridge less than 500 feet downstream, the length of each piece of MWM should be less than 50 percent of the effective culvert or bridge opening width if the MWM has an intact rootwad or less than 75 percent of the width if the rootwad is removed.

In some cases, the clearing limits of a constructed channel may extend further above the 100-year water surface. Downed woody material can also be placed in those areas for habitat purposes, in accordance with landscape plans; however, it is not expected that it could mobilize.

10-9 Inspection and Maintenance

LWM structures need to be inspected and maintained. As wooded members decay, they lose strength and may ultimately fail and then be transported by the stream. LWM may also capture MWM transported from upstream in which the accumulation of wood becomes a hazard either by redirecting flow or constricting the channel. Although LWM used for fish passage projects is intended to mimic natural channel wood, it may also be used to provide bank protection or bank stability and needs to be inspected to ensure it provides the function intended and does not become mobilized or present a risk to infrastructure. Therefore, it is necessary to develop a site-specific inspection and maintenance plan as part of each project.

- LWM projects shall be inspected by lead design personnel prior to completion of the project and demobilization of the contractor to verify that the LWM was installed in accordance with the plans. Because pieces of wood are irregular, field adjustments may be necessary.
- LWM projects shall be inspected after the first significant flood (2 year or greater) or 1 year, whichever is sooner, to verify that the LWM is functioning as it was intended to function.
- LWM projects shall be inspected every 5 years of service or more frequently if identified by maintenance staff for a performance issue. The LWM should be examined for rot, and the anchoring system (if used) should be inspected for pullout, corrosion, abrasion, or breakage.
- After 10 years of service, LWM projects shall be inspected and a brief memo report shall document the condition of the LWM and the establishment of native vegetation. The report shall recommend the need and frequency of future inspections, as well as any long-term maintenance, replacement, or abandonment activities that needed to be programed into the budget.

If a maintenance or repair need is identified, the RHE shall coordinate with the HQ Hydraulics Section to determine an appropriate course of action to repair, modify, replace, or abandon the LWM. Additional guidance will be provided in future revisions to the *Hydraulics Manual*.

10-10 Appendices

Appendix 10A	LWM Design Process Examples for Fish Passage Projects in Washing
Appendix 10B	LWM Structure Examples
Appendix 10C	Typical LWM Installations for Stream Habitat Restoration

Appendix 10A LWM Design Process Examples for Fish Passage Projects in Washing

This appendix presents an example of LWM design for a fish passage project in western Washington. The example illustrates the typical design process used for LWM placement at WSDOT projects, including identifying project objectives for LWM, assessing reach conditions and recreational uses, developing the LWM layout, and analyzing LWM stability.

10A-1 Project Objectives for LWM

This project will replace an existing box culvert with a bridge that meets fish passage criteria. Replacing the culvert will require reconstruction of about 450 feet of stream channel to realign the crossing and provide stable tie-ins upstream and downstream (Figure 10A-1). Project objectives for LWM include:

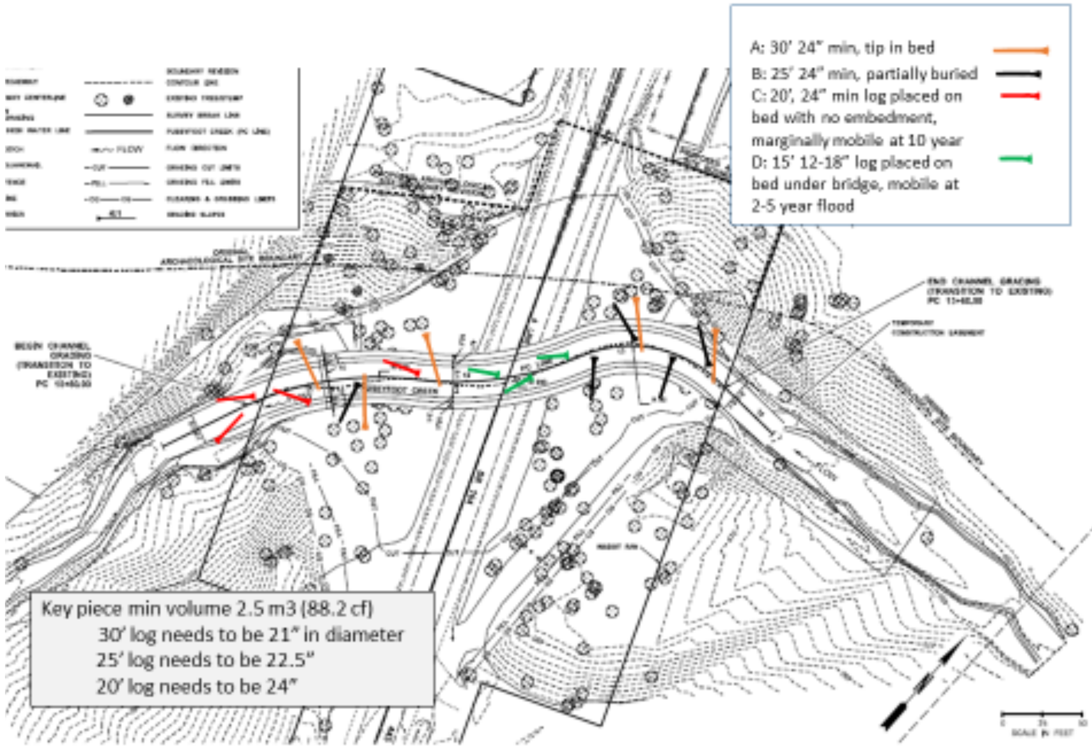
- Install key pieces of LWM in the reconstructed channel to provide aquatic habitat and geomorphic functions while the stream corridor recovers from construction. These functions include pool formation, flow complexity, enhanced hyporheic flow, cover, woody substrate, and recruitment of wood and organic debris.
- Place LWM to mimic natural wood loads, at or near the 75-percentile key-piece density level found by Fox and Bolton (2007) in similar natural streams in the region. This 75-percentile density level is often recommended in reconstructed stream segments in western Washington where natural recruitment of LWM is limited.
- Provide habitat mitigation and flow deflection along the toe of an armored bank at the culvert inlet.
- Anchor LWM as needed to improve stability and minimize risks to infrastructure.

These are typical objectives for fish passage projects. Objectives for bank stabilization projects will generally place more emphasis on reducing erosive forces and providing habitat mitigation.

10A-2 Reach Assessment

A reach assessment was performed to characterize the geomorphic and habitat functions of LWM in this system, and to identify any unique risks. The stream is moderately confined with a BFW of 29 feet and a 0.5 percent gradient. The channel upstream of the culvert has been channelized and flows past commercial development along the right bank that limits delivery of large wood. Road crossings limit the transport of LWM from upstream reaches. Riparian conditions are generally much better downstream of the culvert, with a mature forest that readily delivers LWM to the channel. Existing clusters of one- to three-logs create pool and side channel habitat.

Figure 10A-1 Example Layout of Large Woody Material



10A-3 Recreational Use and Land Use Constraints

This reach does not see significant recreational use and is not large enough for boating. The nearest public access point is a city park about 800 feet upstream. The channel upstream of the bridge is confined by a levee protecting businesses along the right bank, so wood placements should avoid increasing erosion risks on this bank.

10A-4 LWM Layout and Configuration

The design of LWM will usually start with a conceptual plan-view layout of logs that meets the project objectives and avoids constraints. Figure 10A-1 shows the resulting layout of LWM for this project. The project will place 14 key pieces, meeting the minimum volume specified in Figure 10-5, within 450 feet of reconstructed stream channel, similar to the 75-percentile density level of 3.35 key pieces per 100 feet identified by Fox and Bolton (2007). Logs were distributed throughout the reconstructed channel to provide continuous habitat, with more complex placements at locations where risks to infrastructure are lower.

Two clusters of three logs will be placed in areas downstream of the culvert where there are few constraints that would limit use of complex structures. These structures mimic LWM accumulations typically found in smaller streams and consist of a footer log placed in the bed parallel to the bank and held in place by two rack logs with stems buried in the bank. A third three-log structure will be placed on the left bank upstream of the culvert where a high bank allows good anchoring for stability.

Single logs will be placed along the reconstructed banks to improve the distribution of habitat, particularly in areas like the highway median where more complex structures might incur more risk to bridge supports. These logs will be placed with stems embedded in the bank and the root in the stream to mimic a tree undercut by erosion and dislodged by wind.

Six additional logs will be embedded along the toe of the armored right bank at the bridge inlet to improve erosion resistance and aquatic habitat. These six logs are intended to improve bank armor, and therefore do not count towards the density needed to meet habitat and geomorphic objectives for restoration of the reconstructed channel.

LWM will not be installed in selected portions of the restored channel due to site-specific constraints. This includes areas directly under or adjacent to the bridges where LWM accumulation could block the bridge opening.

10A-5 Stability Analysis and Anchor Design

A stability analysis was performed to confirm the log structures will be adequately anchored to resist buoyant and drag/sliding forces generated during the 100-year design flood. Force balances were calculated in the vertical direction for buoyant forces and the horizontal/downstream direction for sliding forces. Anchors were then sized so they would in combination with overburden weight provide design safety factors that exceed 2.0. Moments were also calculated to confirm logs will not rotate.

Figure 10A-2 illustrates the free body diagram and stability calculations performed using a spreadsheet developed by WSDOT's Hydraulics Section for a single log with stem buried in the bank. We assumed the log stem will be embedded in a trench that is backfilled with coarse alluvial material. Buoyant forces will be resisted by the weight of the alluvial material placed on top of the log. We assumed all overburden soil, anchors, and logs will be fully submerged during the 100-year flood. The safety factor for vertical forces is then given by:

$$\text{FSvertical} = (\text{Submerged Overburden Weight} + \text{Anchor Force})/(\text{Net Buoyancy of Log})$$

For moments each force was assumed to act at its centroid distance from the buried tip of the rack log, assuming the log could rotate upward about this pivot point. The structure will be stable if the downward moments generated by overburden and anchors are larger than the upward moments generated by the buoyant forces.

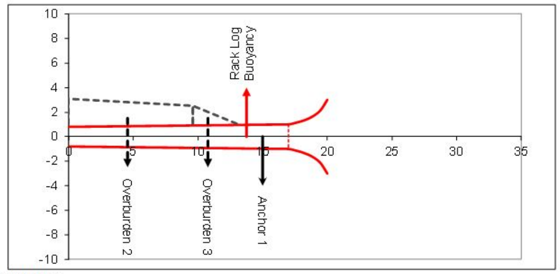
In this case there is not sufficient overburden to provide a factor of safety of 2.0, so an additional anchor force of 1,550 pounds will be needed. This could either be the design pull-out force for a buried duckbill-type anchor, or the required submerged weight of anchor boulders cabled to the log stem.

Drag forces on the protruding rootwad will be resisted by the bearing strength of the soil surrounding the buried log stem. Project experience has shown that drag forces and moments will be adequately resisted if at least ⅔ of the total length of log is buried.

Figure 10A-2 Example Stability Calculations for a Single Bank Log

Force Balance for Type A Bank Log, Right Bank Median

Calculates buoyant and gravitational forces for trunk, rootwad, and up to three Overburden Elements
Buoyancy of footer log transferred to rack log; can input two anchors
Peforms moments balance about pivot at the tip of the log
Drag forces assumed to be resisted by embedment (2/3 length); not explicitly calculated
Input data in Gray Shaded Cells



By:	**Rob Schanz; corrected protrusion from bank**
Project:	**SR 8 EF Wildcat - Type A Right bank median log**
Date:	**4/12/2016: November 2015 Section 5+54**
LWM Spreadsheet Version:	**4-1**

Log and Anchor Data

Specific weight of water =	**62.4**	lbs/ft^3	
Bulk Specific Gravity of Wood =	**0.5**	dry	S_W, value from D'oust and Millar
Diameter of rootwad	**6**	feet	SR 508 Log dimensions
Length of rootwad section	**3**	feet	SR 508 Log dimensions, upper range
Diameter at trunk/root transition	**2**	feet	Max of 18-24" range
Length of log (not including root)	**17**	feet	Remainder of 20' log
Diameter at buried end of log	**1.6**	feet	SR 508 Log dimensions

	Specific Gravity
Overburden 1	1.3
Overburden 2	1.3
Overburden 3	1.3

Net downward force from anchor 1	**1550**	lbs
Distance from buried end/tip of log	**15**	feet
Net downward force from anchor 2	**0**	lbs; enter absolute value of force
Distance from buried end/tip of log	**0**	feet
Net buoyant force from footer log	**0**	No footer
Location from buried end/tip of log	**0**	feet

2.431517 Equivalent diameter (ft) for each of two submerged boulders

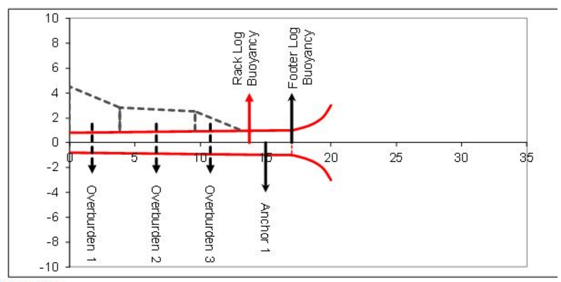
Net Vertical Force and Moment Balance

Weight of overburden and anchors =	***4308***	***lbs***	$F_D = W_O + W_{anchor}$
Net Buoyancy of rack and footer logs	***2193***	***lbs***	$F_U = B_{root} + B_{bole} + B_{footer}$
Net Force (positive upward) =	***-2115***	***lbs***	$F_D - F_U$
Safety Factor =	***2.0***		F_D/F_U; ***minimum 2.0 recommended***
Total Turning Moment (stable if negative)	***-8079***		***ft-lbs***

Figure 10A-3 Example Stability Calculations for a Rack Log in a Complex Structure

Force Balance for Type C Rack Log, Left Bank Outlet

Calculates buoyant and gravitational forces for trunk, rootwad, and up to three Overburden Elements
Buoyancy of footer log transferred to rack log; can input two anchors
Peforms moments balance about pivot at the tip of the log
Drag forces assumed to be resisted by embedment (2/3 length); not explicitly calculated
Input data in Gray Shaded Cells



By:	**Rob Schanz; corrected protrusion from bank**
Project:	**SR 8 East Fork Wildcat, Type C LB outlet**
Date:	**4/12/2016; November 2015 Section 4+64**
LWM Spreadsheet Version:	**4-1**

Log and Anchor Data

Specific weight of water =	**62.4**	lbs/ft^3	
Bulk Specific Gravity of Wood =	**0.5**	dry	S_W, value from D'oust and Millar
Diameter of rootwad	**6**	feet	SR 508 Log dimensions
Length of rootwad section	**3**	feet	SR 508 Log dimensions, upper range
Diameter at trunk/root transition	**2**	feet	Max LWM, 12-24"
Length of log (not including root)	**17**	feet	Remainder of 20' log
Diameter at buried end of log	**1.6**	feet	SR 508 Log dimensions

	Specific Gravity
Overburden 1	1.3
Overburden 2	1.3
Overburden 3	1.3

Net downward force from anchor 1	**3300**	lbs
Distance from buried end/tip of log	**15**	feet
Net downward force from anchor 2	**0**	lbs; enter absolute value of force
Distance from buried end/tip of log	**0**	feet
Net buoyant force from footer log	**1097**	lbs; Default set to net footer log buoyancy/number of rack logs
Location from buried end/tip of log	**17**	feet

3.128026 Equivalent diameter (ft) for each of two submerged boulders

Net Vertical Force and Moment Balance

Weight of overburden and anchors =	***6427***	***lbs***	$F_D = W_O + W_{anchor}$
Net Buoyancy of rack and footer logs	***3290***	***lbs***	$F_U = B_{root} + B_{bole} + B_{footer}$
Net Force (positive upward) =	***-3138***	***lbs***	$F_D - F_U$
Safety Factor =	***2.0***		F_D/F_U; ***minimum 2.0 recommended***
Total Turning Moment (stable if negative)	***-16342***		***ft-lbs***

The three-log structures require a more complex stability analysis that accounts for the transfer of forces between the footer log and the overlying rack logs. Figure 10A-3 illustrates the free body diagram and calculations for one of the rack logs in these structures. We assumed the footer log buoyancy will be transferred equally to each of the two rack logs. The factor of safety for vertical buoyancy forces for each rack log is then:

$$FOS_{buoyancy} = F_D/F_U$$

Where:

F_D = total downward force

F_U = total upward force

And where:

$F_D = W_O + W_{anchor}$

And:

W_O = weight of overburden

W_{anchor} = weight of anchor

And where:

$F_U = B_{root} + B_{bole}$

And:

B_{root} = buoyancy of rootwad

B_{bole} = buoyancy of log bole

This type of structure will often need more anchoring because of the additional buoyancy of the footer log. In this case a total anchor force of 3,300 pounds will be needed to obtain a safety factor of 2.0.

Figure A10-4 illustrates the sliding force calculations for the footer log. The footer log is subject to drag on the upstream face of the rootwad. This is resisted by friction forces generated by the net downward normal force transferred onto the footer log by the overlying rack logs. The factor of safety for sliding is then given by:

$$FOS_{drag} = F_f/F_{Dr}$$

Where:

F_f = total friction force

F_{dr} = total drag force

And where:

$F_f = -(F_D - F_U)*C_{rl}$ riverbed-log friction coefficient

And:

C_{rl} = riverbed-log friction coefficient

And where:

$F_{Dr} = C_{dr}(y/g)*(v)^{\wedge 2*}(A_{rtwd})^{\wedge 0.5}$

And:

C_{dr} = unitless drag coefficient

y = specific weight of water

g = gravitational acceleration

v = computed water velocity

A_{rtwd} = projected area of rootwad

The drag force was calculated using the 100-year velocity from the project Hydrologic Engineering Center's River Analysis System (HEC-RAS) model. This force was assumed to act on the projected area of the rootwad face perpendicular to flow. In this case, the anchor force needed to resist buoyant forces also provided a sufficient factor of safety for sliding forces.

The impacts of scour on structure stability were considered by burying the lower halves of rack log rootwads and most of the footer log in the streambed. These will be exposed by scour as the channel evolves to create the desired pool and cover habitat. Rack log stems and anchors will be embedded in the bank where they will not be exposed or undermined by scour.

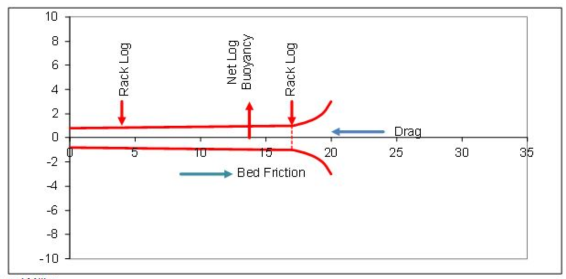
The project HEC-RAS model was used to simulate the effects of LWM on flood elevations. The effects of channel margin wood placements are usually simulated by increasing hydraulic roughness factors. The model demonstrated the LWM will not cause increases in 100-year flood elevations that would threaten the proposed bridge or violate local floodplain ordinances.

Figure 10A-4 Example Stability Calculations for Sliding Forces on a Footer Leg

Sliding Force Balance for Type C Footer Log, Left Bank Outlet
Calculates buoyant and gravitational forces for trunk, rootwad, and stream overburden
Can input 2 anchors (not including rack logs)
Calculates drag and resisting friction forces for flow perpendicular to the face of the rootwad

Input data in Gray Shaded Cells

By:	**Rob Schanz**
Project	**SR 8 East Fork Wildcat, Footer with 2 Rack logs**
Date:	**4/12/2016; November 2015 Section 4+64**
LWM Spreadsheet Version:	***4-1***



Log and Anchor Data

Specific weight of water =	**62.4**	lbs/ft^3	
Bulk Specific Gravity of Wood =	**0.5**	dry	$S_{W,}$ value from D'oust and Millar
Diameter of rootwad	**6**	feet	SR 508 Log dimensions
Length of rootwad section	**3**	feet	SR 508 Log dimensions, upper range
Diameter at trunk/root transition	**2**	feet	Max LWM, 12-24"
Length of log (not including root)	**17**	feet	remainder of 20' log
Diameter at buried end of log	**1.6**	feet	SR 508 Log dimensions
Net downward force from anchor 1	**0**	lbs; enter absolute value of force	
Distance from downstream tip of log	**0**	feet	
Net downward force from anchor 2	**0**	lbs; enter absolute value of force	
Distance from downstream tip of log	**0**	feet	
Number of Rack Logs	**2**		

Buoyant Forces Transferred to overlying Rack Logs:

Weight of overburden and anchors =	***0***	***lbs***	$F_D = W_O + W_{anchor}$
Net Buoyancy of Log =	***2193***	***lbs***	$F_U = B_{root} + B_{bole}$
Net Force (positive upward) =	***2193***	***lbs***	$F_D - F_U$

Drag and Friction Forces (x-Direction, acting on face of rootwad):

River Velocity perpendicular to root face =	**5.75**	fps 100 year velocity from 4/4/2016 HECRAS Section 464
Riverbed/log friction coefficient =	**0.87**	lbs/lbs, from Abbe's table 1.2 for gravel. Abbe's dissertation found 1.2 for real logs
Unitless Drag Coefficient	**1.2**	upper end of values reported in SHRG for blunt rootwad
Projected Area of root wad	28.3	square feet, circle
Drag Force	1087	lbs, = (1/2)* Cd * density of water * velocity^2 * Area
Total Normal Force	6275	lbs, Net of all downward Forces from Rack Logs
Friction Force	-5460	lbs, friction coefficient*F_{normal}
Net Force in X-Direction, Postive Downstream	***-4373***	***lbs***
Drag Safety Factor	***5.0***	

Appendix 10B LWM Structure Examples

10B-1 Self-ballasting Large Wood Structures

These structures are for habitat primarily but can be used to encourage natural processes to enhance a stream system, such as encouraging aggradation in a degraded system. A log of sufficient size, relative to the stream, and placed correctly, can be stable without anchors. Additionally, the design flow may be lower than the 100-year flow if site conditions permit.

Figure 10B-1 Self-ballasting Large Wood Structure, Swauk Creek, Kittitas County

10B-2 Rootwad Habitat Structures

As the name implies, these structures consist of logs with rootwads or series of logs with rootwads located to interact with the channel at low and high flows to provide habitat variability and structure in stream corridor.

Figure 10B-2 Rootwad Habitat Structures, Evans Creek, King County

10B-3 Wood Studded Revetments

As the name implies, wood studded revetments consist of a rock revetment studded with root wads to provide roughness, energy diffusion, and minor flow deflection.

Figure 10B-3 Wood Studded Revetments, Newaukum River, Lewis County

10B-4 Crib Walls

Crib walls are constructed with logs in a rectilinear array, with voids backfilled with mineral and/or organic soils. Wood or steel piles may be integrated for additional stability. They provide contiguous protection to the bank with a great deal of roughness and complexity. Crib walls are narrow in profile and minimize encroachment into the channel. They are especially useful in narrow channels/banks that cannot accommodate wider structures.

Figure 10B-4 Crib Wall with Wood Piles, Beaver Creek, Okanogan County

Figure 10B-5 Crib Wall with Steel Piles, Sauk River Side Channel

10B-5 Flow Deflection Jams

Flow deflection jams consist of a series of logs with attached root wads (key members) and often include large volumes of material. These are sometimes linked with revetments or crib wall structures where contiguous protection is desired.

Figure 10B-6 Flow Deflection Jams, Hoh River, 2004

10B-6 Apex Bar Jams

Apex bar jams are crescent or fan shaped structures constructed at the head of islands or gravel bars. Apex bar jams act to split and turn flows. Bars forming downstream of them tend to grow and become persistent. Apex bar jams recruit large volumes of additional wood. The potential for major changes in hydraulic and geomorphic functions resulting from wood recruitment is an important risk factor than must be considered in design.

Figure 10B-7 Apex Bar Jams, Hoh River, 2004

10B-7 Dolotimber

The use of Dolotimber structures, or other ballasted prefabricated LWM structure matrices. They may be considered in situations with extreme high flows and imminent danger to infrastructure. They offer excellent interstitial habitat and are extremely effective at reducing near-bank shear stress (Abbe and Brooks 2011).

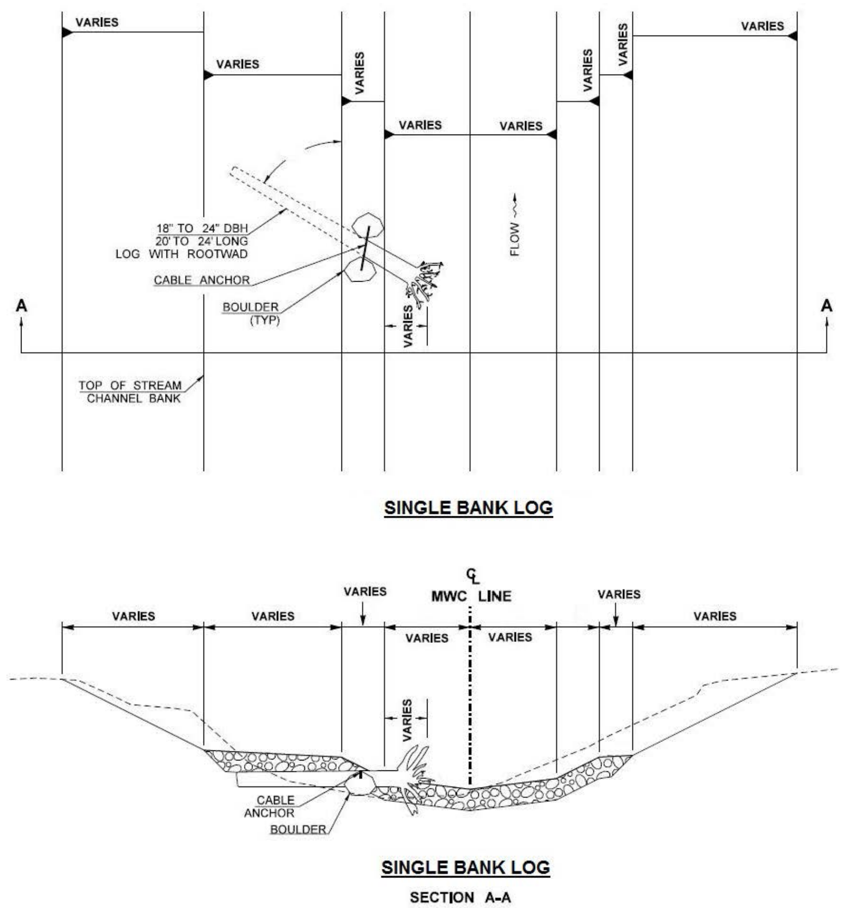
Figure 10B-8 Dolotimber Structures, Skagit River

Appendix 10C Typical LWM Installations for Stream Habitat Restoration

10C-1 Single Bank Log

This is the simplest and generally most stable type of LWM placement, consisting of a single log with the stem buried in the bank and the root wad partially embedded in the streambed. This type of placement creates localized pool habitat, cover, and woody substrate on the margins of the channel while having minimal impacts on channel hydraulics and erosion. With sufficient overburden this type of placement may not require additional anchoring, but boulder anchors can be used to increase stability in situations with shallow burial depths.

Figure 10C-1 Single Bank Log



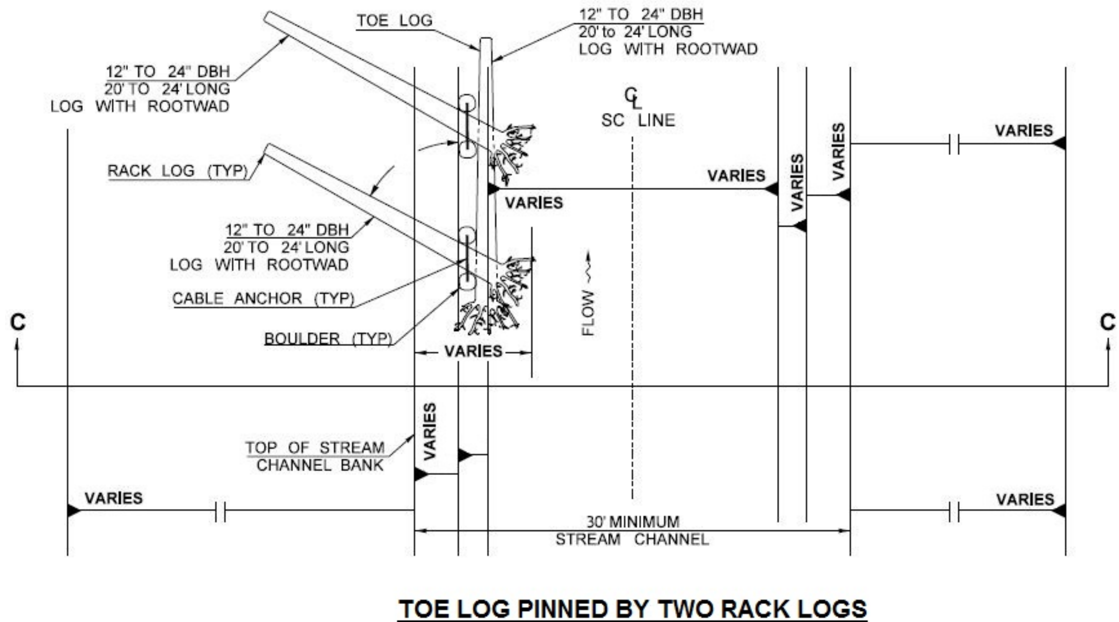
NOTES:

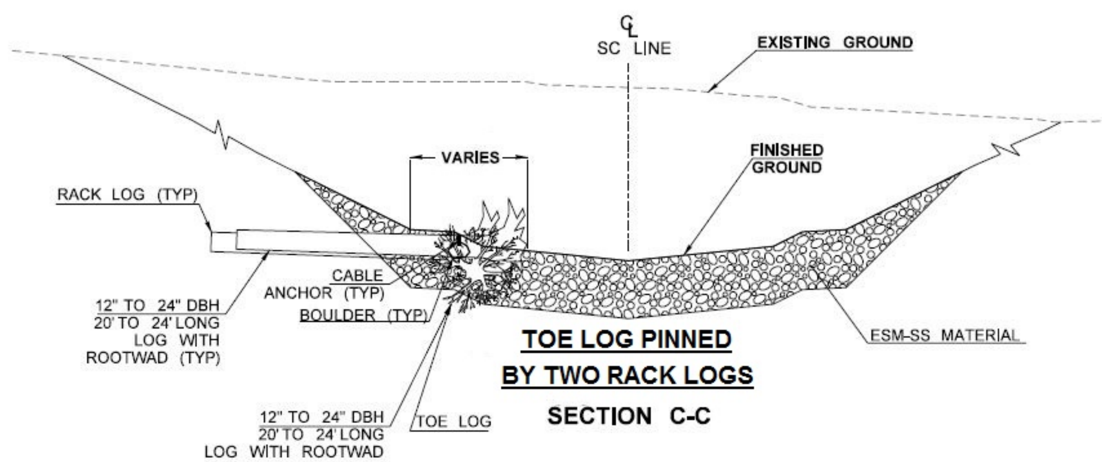
1. ROCK DIAMETER, CABLE SIZE/MATERIAL, AND ANCHORAGE ATTACHMENT DETAILS TO BE DETERMINED.
2. LOCATIONS AND ORIENTATION OF LARGE WOODY MATERIAL (LWM) AS SHOWN ON THIS SHEET ARE APPROXIMATE, FINAL LOCATIONS TO BE DETERMINED ON SITE BY THE ENGINEER.

10C-2 Toe Log Pinned by Two Rack Logs

This is a more complex placement that creates more habitat variability and greater contact with the streambed. It consists of a toe or footer log placed in the streambed parallel to the bank and pinned in place by two overlying rack logs that are buried in the bank. The LWM is anchored by burial of the rack logs in the streambank, but additional boulder anchors are generally needed to resist drag and buoyant forces exerted on the toe log.

Figure 10C-2 Toe Log Pinned by Two Rack Logs

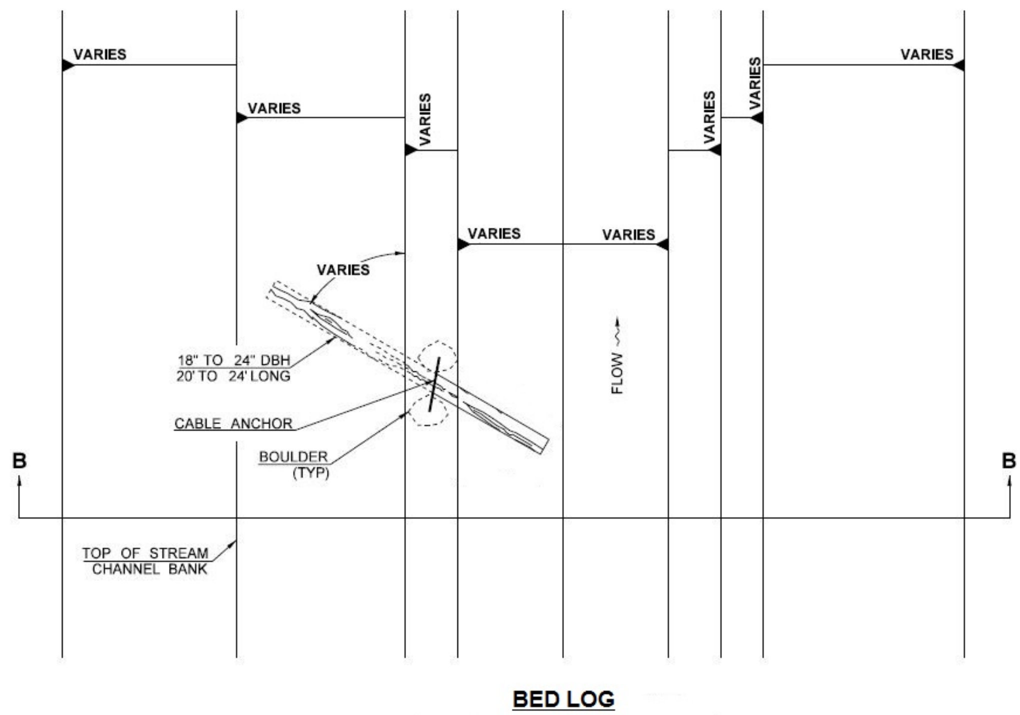


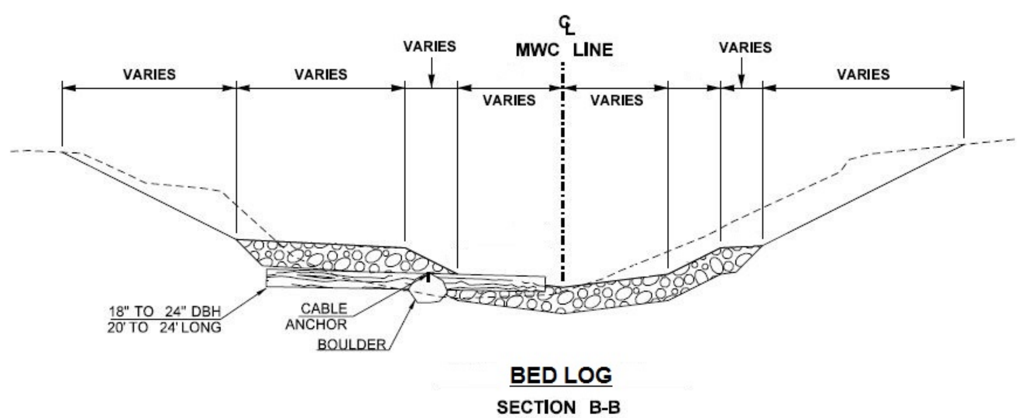


10C-3 Bed Log

This type of placement consists of a log without roots partially buried in the bed and extending out to the center of the channel. This low-profile placement of logs mimics tip-first delivery of logs to the stream by windthrow. These logs have high contact with the streambed and enhance streambed stability by encouraging sediment accumulation on the upstream side and flow deflection towards the center of the channel. A localized plunge pool may form on the downstream side of the log. The bed log is anchored by stem burial and boulders as needed.

Figure 10C-3 Bed Log





Glossary & Sources

Acronyms and Abbreviations

AASHTO	American Association of State Highway and Transportation Officials
AMC	antecedent moisture condition
ASTM	American Society for Testing and Materials
BFW	bankfull width
BMP	best management practice
cfs	cubic feet per second
CN	curve number
DBH	diameter at breast height
Ecology	Washington State Department of Ecology
ELJ	engineered logjam
ERDC	U.S. Army Engineer Research and Development Center
FEMA	Federal Emergency Management Agency
FHWA	Federal Highway Administration
ft/ft	feet/foot
ft/s	feet per second
FUR	floodplain utilization ratio
GIS	geographic information system
HDS	Hydraulic Design Series
HEC	Hydraulic Engineering Circular
HEC-RAS	Hydrologic Engineering Center's River Analysis System
HGL	hydraulic grade line
HQ	WSDOT Headquarters
HSPF	Hydrological Simulation Program-Fortran
HW/D	headwater/diameter
ISPG	Integrated Streambank Protection Guidelines
LW	large wood (also known as LWD or LWM)
LWD	large woody debris (also known as LW or LWM)
LWM	large woody material (also known as LWD or LW)

MDL	master deliverable list
mph	miles per hour
MRI	mean recurrence interval
MW	mobile wood (also known as MWM)
MWM	mobile woody material (also known as MW)
NCHRP	National Cooperative Highway Research Program
NRCS	Natural Resources Conservation Service
PEO	Project Engineer’s Office
PS&E	plans, specifications, and estimates
RHE	Region Hydraulics Engineer
ROW	right-of-way
SBUH	Santa Barbara Urban Hydrograph
SCS	Soil Conservation Service
SR	State Route
SRH-2D	Sedimentation and River Hydraulics – 2D Model
USACE	U.S. Army Corps of Engineers
USBR	U.S. Bureau of Reclamation
USDA	U.S. Department of Agriculture
USFS	U.S. Forest Service
USGS	U.S. Geological Survey
WAC	Washington Administrative Code
WCDG	Water Crossing Design Guidelines
WDFW	Washington Department of Fish and Wildlife
WSDOT	Washington State Department of Transportation

Main Glossary of Terms

A

access A means of entering or leaving a public road, street, or highway with respect to abutting property or another public road, street, or highway.

access point Any point that allows private or public entrance to or exit from the traveled way of a state highway, including “locked gate” access and maintenance access points.

approach An access point, other than a public road/street, that allows access to or from a limited access highway on the state highway system.

average daily traffic (ADT)
The total volume during a given time period (in whole days): greater than one day and less than one year, divided by the number of days in that time period.

B

bankfull width The bankfull channel is defined as the stage when water just begins to overflow into the active floodplain. In channels where there is no floodplain, it is the width of a stream or river at the dominant channel forming flow. For the purposes of this chapter, it refers to a section that is not highly influenced by man.

benefit/cost analysis A method of valuing a proposition by first monetizing all current expenditures to execute—cost—as well as the expected yields into the future—benefit, then dividing the total benefit by the total cost, thus providing a ratio. Alternatives may be rendered and compared in this fashion where, typically, a higher ratio is preferable, indicating a better return on investment.

bicycle Any device propelled solely by human power upon which a person or persons may ride, having two tandem wheels, either of which is 16 inches or more in diameter, or three wheels, any one of which is more than 20 inches in diameter.

bridge Any structure that is 20 feet or larger in span measured along the centerline of the roadway.

buried structures TBD

C

channel complexity The variation in physical channel components, which may include planform, longitudinal profile, cross-section, sediment distribution, etc.

channel width For the purposes of Chapter 7, channel width is used to describe bankfull width in a situation where the channel is highly influenced by man or heavily degraded conditions exist (WDFW, 2013)

check flood 500-year flood event

clear zone The total roadside border area, available for use by errant vehicles, starting at the edge of the traveled way and oriented from the outside or inside shoulder (in median applications) as applicable. This area may consist of a shoulder, a recoverable slope, a nonrecoverable slope, and/or a clear run-out area. The clear zone cannot contain a critical fill slope, fixed objects, or water deeper than 2 feet.

climate change vulnerability The risk a transportation facility will be impacted by the effects of climate change.

collector A context description of a roadway intended to provide a mix of access and mobility performance. Typically low speed, collecting traffic from local roads and connecting them with destination points or arterials. This term is used in multiple classification systems, but is most commonly associated with the *Functional Classification* System.

collector system Routes that primarily serve the more important intercounty, intracounty, and intraurban travel corridors; collect traffic from the system of local access roads and convey it to the arterial system; and on which, regardless of traffic volume, the predominant travel distances are shorter than on arterial routes (RCW 47.05.021).

consider To think carefully about, especially in order to make a decision. The decision to document a consideration is left to the discretion of the engineer.

countermeasure an action taken to counteract an existing or anticipated condition.

critical fill slope A slope on which a vehicle is likely to overturn. Slopes steeper than 3H:1V are considered critical fill slopes.

crossroad The minor roadway at an intersection. At a stop-controlled intersection, the crossroad has the stop.

curb section A roadway cross section with curb and sidewalk.

D

deliverable Any unique and verifiable product, result or capability to perform a service that must be produced to complete a process, phase, or project.

design approval Documented approval of the design at this early milestone locks in design policy for three years. Design approval becomes part of the Design Documentation Package (see *Design Manual* Chapter 300.)

design-bid-build The project delivery method where design and construction are sequential steps in the project development process (23 CFR 636.103).

design-build contract An agreement that provides for design and construction of improvements by a consultant/contractor team. The term encompasses design-build-maintain, design-build-operate, design-build-finance, and other contracts that include services in addition to design and construction. Franchise and concession agreements are included in the term if they provide for the franchisee or concessionaire to develop the project that is the subject of the agreement (23 CFR 636.103).

design-builder The firm, partnership, joint venture, or organization that contracts with WSDOT to perform the work.

design element Any component or feature associated with roadway design that becomes part of the final product. Examples include lane width, shoulder width, alignment, and clear zone (see *Design Manual* Chapter 1105.)

designer This term applies to WSDOT design personnel. Wherever "designer" appears in this manual, design-build personnel shall deem it to mean: Engineer of Record, Design Quality Assurance Manager, design-builder, or any other term used in the design-build contract to indicate design-build personnel responsible for the design elements of a design-build project, depending on the context of information being conveyed.

design flood 100-year flood event.

design reference reach A reach of stream, preferably within the same watershed, that is relatively stable.

desirable Design criteria that are recommended for inclusion in the design.

document (verb) The act of including a short note to the Design Documentation Package that explains a design decision.

driveway A vehicular access point that provides access to or from a public roadway.

E

easement A documented right, as a right of way, to use the property of another for designated purposes.

element An architectural or mechanical component or design feature of a space, site, or public right of way.

F

facility All or any portion of buildings, structures, improvements, elements, and pedestrian or vehicular routes located in a public right of way.

Federal Highway Administration (FHWA)
The division of the U.S. Department of Transportation with jurisdiction over the use of federal transportation funds for state highway and local road and street improvements.

final design Any design activities following preliminary design; expressly includes the preparation of final construction plans and detailed specifications for the performance of construction work (23 CFR 636.103). Final design is also defined by the fact that it occurs after NEPA/SEPA approval has been obtained.

floodplain utilization ratio (FUR)
The floodplain utilization ratio is the flood-prone width (100-year top width) divided by the bankfull width.

G

Geographic Information System (GIS)
A computerized geographic information system used to store, analyze, and map data. Data may be used with GIS if the data includes the Accumulated Route Mile (ARM) or State Route Milepost (SRMP) programs. Global Positioning System (GPS) technology provides a means of collecting data and is an alternative to ARM and SRMP. WSDOT's primary desktop tool to view and analyze GIS data is ArcGIS software. GIS is used to gather and analyze data to support the purpose and need as described in the Project Summary (http://wwwi.wsdot.wa.gov/gis/supportteam/default.asp).

geotextiles (nonwoven)
A sheet of continuous or staple fibers entangled randomly into a felt for needle-punched nonwovens and pressed and melted together at the fiber contact points for heat-bonded nonwovens. Nonwoven geotextiles tend to have low-to-medium strength and stiffness with high elongation at failure and relatively good drainage characteristics. The high elongation characteristic gives them superior ability to deform around stones and sticks.

geotextiles (woven) Slit polymer tapes, monofilament fibers, fibrillated yarns, or multifilament yarns simply woven into a mat. Woven geotextiles generally have relatively high strength and stiffness and, except for the monofilament wovens, relatively poor drainage characteristics.

H

highway A general term denoting a street, road, or public way for the purpose of vehicular travel, including the entire area within the right of way.

hydraulic opening The width perpendicular to the creek beneath the proposed structure that is necessary to convey the Design Flow.

I

intersection An at-grade access point connecting a state highway with a road or street duly established as a public road or public street by the local governmental entity.

Interstate System A network of routes designated by the state and the FHWA under terms of the federal-aid acts as being the most important to the development of a national system. The Interstate System is part of the principal arterial system.

J

justify Preparing a memo to the DDP identifying the reasons for the decision: a comparison of advantages and disadvantages of all options considered. A more rigorous effort than document.

K

key pieces Logs that are large enough to persist and influence hydraulics and bed topography in a stream through a wide range of flow conditions.

L

lane A strip of roadway used for a single line of vehicles.

lane width The lateral design width for a single lane, striped as shown in the *Standard Plans* and the *Standard Specifications*. The width of an existing lane is measured from the edge of traveled way to the center of the lane line or between the centers of adjacent lane lines.

lateral (storm sewer) These are typically the first inlets that contributes flow into a storm sewer system.

level of service (LOS) LOS is based on peak hour, except where noted. LOS assigns a rank (A – F) to facility sections based on traffic flow concepts like density, delay, and/or corresponding safety performance conditions. (See the Highway Capacity Manual and AASHTO's Geometric Design of Highways and Streets ["Green Book"] for further details.)

M

managing project delivery
A WSDOT management process for project delivery from team initiation through project closing.

median The portion of a divided highway separating vehicular traffic traveling in opposite directions

O

over-coarsened channel
A constructed channel with a median particle size that is greater than 20% larger than the median particle size of the design reference reach; is deformable at discharges below the 100-year discharge.

P

Plans, Specifications, and Estimates (PS&E)
The project development activity that follows Project Definition and culminates in the completion of contract-ready documents and the engineer's cost estimate.

preventative countermeasure
Structures or other management actions used to prevent erosion from damaging critical infrastructure.

project The Project Management Institute defines a project to be "a temporary endeavor undertaken to create a unique product or service."

project definition (see *Project Summary*)

Project Engineer This term applies to WSDOT personnel. Wherever "Project Engineer" appears in this manual, the design-builder shall deem it to mean "Engineer of Record."

project reach The segment of stream in which the project is located.

proposal The combination of projects/actions selected through the study process to meet a specific transportation system need.

purpose General project goals such as improve safety, enhance mobility, or enhance economic development.

R

reference reach A stable segment of stream with consistent slope, geometry, planform, and sediment load that represents, to the best available knowledge, background condition of the project reach.

Request for Proposal (RFP)
The document package issued by WSDOT requesting submittal of proposals for the project and providing information relevant to the preparation and submittal of proposals, including the instructions to proposers, contract documents, bidding procedures, and reference documents.

residual pool depth	The difference in depth or bed elevation between a pool and the downstream riffle crest.
right of way	A general term denoting land or interest therein, acquired for or designated for transportation purposes. More specifically, lands that have been dedicated for public transportation purposes or land in which WSDOT, a county, or a municipality owns the fee simple title, has an easement devoted to or required for use as a public road/street and appurtenant facilities, or has established ownership by prescriptive right.
road approach	An access point, other than a public road/street, that allows access to or from a limited access highway on the state highway system.
roadway	The portion of a highway, including shoulders.
roughened channel	A constructed channel with streambed material and configuration designed to be non-deformable up to the design discharge.
roundabout	A circular intersection at grade with yield control of all entering traffic, channelized approaches with raised splitter islands, counter-clockwise circulation, and appropriate geometric curvature to force travel speeds on the circulating roadway generally to less than 25 mph.

S

shoulder	The portion of the roadway contiguous with the traveled way, primarily for accommodation of stopped vehicles, emergency use, lateral support of the traveled way, and, where allowed, use by pedestrians and bicycles.
site	Parcel(s) of land bounded by a property line or a designated portion of a public right of way.
speed	The operations or target or posted speed of a roadway. There are three classifications of speed established: • ***Low speed*** is considered 35 mph and below. • ***Intermediate speed*** is considered 40-45 mph. • ***High speed*** is considered 50 mph and above.
stable stream	A stream, over time (in the present climate), that transports the flows and sediment produced by its watershed in such a manner that the dimension, pattern, and profile are maintained without either aggrading, nor degrading.
state highway system	All roads, streets, and highways designated as state routes in compliance with RCW 47.17.
stream designer	This term applies to WSDOT design personnel and is used to distinguish the work that is performed using Chapter 7 and 10 from the rest of the manual. Wherever "stream designer" appears in this manual, design-build personnel shall deem it to mean: Water Resources Engineer of Record, Design Quality Assurance Manager, design-builder, or any other term used in the design-build contract to indicate design-build personnel responsible for the design elements of a design-build project, depending on the context of information being conveyed.

stream simulation The design methodology outlined in the 2013 Water Crossing Design Guidelines defined as Stream Simulation.

streambed mix Sediment size distribution that uses pebble counts from the reference reach for the D50 and D84 and an even, designed distribution of sizes for finer classes (USFS, 2008)..

superelevation The rotation of the roadway cross section in such a manner as to overcome part of the centrifugal force that acts on a vehicle traversing a curve.

superelevation transition length
The length of highway needed to change the cross slope from normal crown or normal pavement slope to full superelevation.

T

thalweg Relates to the geometrics of natural or artificial water conveyance channels. More specifically, a thalweg delineates the line connecting the deepest points throughout any given point in a channel.

traveling public Motorists, motorcyclists, bicyclists, pedestrians, and pedestrians with disabilities.

trunk (storm sewer) The pipes that make up the storm sewer system that are not laterals.

U

urban area An area designated by the Washington State Department of Transportation (WSDOT) in cooperation with the Transportation Improvement Board and Regional Transportation Planning Organizations, subject to the approval of the FHWA.

urbanized area An urban area with a population of 50,000 or more.

W

Water Crossing Design Guidelines (2013 WCDG)
The 2013 Water Crossing Design Guidelines, as published by the Washington Department of Fish and Wildlife at https://wdfw.wa.gov/publications/01501/wdfw01501.pdf. This version of the document has been approved for use on WSDOT projects with exceptions as noted in Chapters 7 and 10. If a newer version of the document is published, the Hydraulics Section must approve of it prior to use.

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