Washington State
Department of Transportation

# Hydraulics Manual

**M 23-03.11**

April 2025

**Engineering and Regional Operations Hydraulics Office**

## ENGLISH

### Title VI Notice to Public

It is the Washington State Department of Transportation's (WSDOT) policy to assure that no person shall, on the grounds of race, color, national origin, as provided by Title VI of the Civil Rights Act of 1964, be excluded from participation in, be denied the benefits of, or be otherwise discriminated against under any of its programs and activities. Any person who believes his/her Title VI protection has been violated, may file a complaint with WSDOT's Office of Equity and Civil Rights (OECR). For additional information regarding Title VI complaint procedures and/or information regarding our non-discrimination obligations, please contact OECR's Title VI Coordinator at (360) 705-7090.

### Americans with Disabilities Act (ADA) Information

This material can be made available in an alternate format by emailing the Office of Equity and Civil Rights at wsdotada@wsdot.wa.gov or by calling toll free, 855-362-4ADA(4232). Persons who are deaf or hard of hearing may make a request by calling the Washington State Relay at 711.

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## ESPAÑOL

### Notificación de Titulo VI al Público

La política del Departamento de Transporte del Estado de Washington (Washington State Department of Transportation, WSDOT) es garantizar que ninguna persona, por motivos de raza, color u origen nacional, según lo dispuesto en el Título VI de la Ley de Derechos Civiles de 1964, sea excluida de la participación, se le nieguen los beneficios o se le discrimine de otro modo en cualquiera de sus programas y actividades. Cualquier persona que considere que se ha violado su protección del Título VI puede presentar una queja ante la Oficina de Equidad y Derechos Civiles (Office of Equity and Civil Rights, OECR) del WSDOT. Para obtener más información sobre los procedimientos de queja del Título VI o información sobre nuestras obligaciones contra la discriminación, comuníquese con el coordinador del Título VI de la OECR al (360) 705-7090.

### Información de la Ley sobre Estadounidenses con Discapacidades (ADA, por sus siglas en inglés)

Este material puede estar disponible en un formato alternativo al enviar un correo electrónico a la Oficina de Equidad y Derechos Civiles a wsdotada@wsdot.wa.gov o llamando a la línea sin cargo 855-362-4ADA(4232). Personas sordas o con discapacidad auditiva pueden solicitar la misma información llamando al Washington State Relay al 711.

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## 한국어 – KOREAN

### 제6조 관련 공지사항

워싱턴 주 교통부(WSDOT)는 1964년 민권법 타이틀 VI 규정에 따라, 누구도 인종, 피부색 또는 출신 국가를 근거로 본 부서의 모든 프로그램 및 활동에 대한 참여가 배제되거나 혜택이 거부되거나, 또는 달리 차별받지 않도록 하는 것을 정책으로 하고 있습니다. 타이틀 VI에 따른 그/그녀에 대한 보호 조항이 위반되었다고 생각된다면 누구든지 WSDOT의 평등 및 민권 사무국(OECR)에 민원을 제기할 수 있습니다. 타이틀 VI에 따른 민원 처리 절차에 관한 보다 자세한 정보 및/또는 본 부서의 차별금지 의무에 관한 정보를 원하신다면, (360) 705-7090으로 OECR의 타이틀 VI 담당자에게 연락해주십시오.

### 미국 장애인법(ADA) 정보

본 자료는 또한 평등 및 민권 사무국에 이메일 wsdotada@wsdot.wa.gov 을 보내시거나 무료 전화 855-362-4ADA(4232)로 연락하셔서 대체 형식으로 받아보실 수 있습니다. 청각 장애인은 워싱턴주 중계 711로 전화하여 요청하실 수 있습니다.

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## русский – RUSSIAN

### Раздел VI Общественное заявление

Политика Департамента транспорта штата Вашингтон (WSDOT) заключается в том, чтобы исключить любые случаи дискриминации по признаку расы, цвета кожи или национального происхождения, как это предусмотрено Разделом VI Закона о гражданских правах 1964 года, а также случаи недопущения участия, лишения льгот или другие формы дискриминации в рамках любой из своих программ и мероприятий. Любое лицо, которое считает, что его средства защиты в рамках раздела VI были нарушены, может подать жалобу в Ведомство по вопросам равенства и гражданских прав WSDOT (OECR). Для дополнительной информации о процедуре подачи жалобы на несоблюдение требований раздела VI, а также получения информации о наших обязательствах по борьбе с дискриминацией, пожалуйста, свяжитесь с координатором OECR по разделу VI по телефону (360) 705-7090.

### Закон США о защите прав граждан с ограниченными возможностями (ADA)

Эту информацию можно получить в альтернативном формате, отправив электронное письмо в Ведомство по вопросам равенства и гражданских прав по адресу wsdotada@wsdot.wa.gov или позвонив по бесплатному телефону 855-362-4ADA(4232). Глухие и слабослышащие лица могут сделать запрос, позвонив в специальную диспетчерскую службу штата Вашингтон по номеру 711.

**tiếng Việt – VIETNAMESE**

**Thông báo Khoản VI dành cho công chúng**

Chính sách của Sở Giao Thông Vận Tải Tiểu Bang Washington (WSDOT) là bảo đảm không để cho ai bị loại khỏi sự tham gia, bị từ khước quyền lợi, hoặc bị kỳ thị trong bất cứ chương trình hay hoạt động nào vì lý do chủng tộc, màu da, hoặc nguồn gốc quốc gia, theo như quy định trong Mục VI của Đạo Luật Dân Quyền năm 1964. Bất cứ ai tin rằng quyền bảo vệ trong Mục VI của họ bị vi phạm, đều có thể nộp đơn khiếu nại cho Văn Phòng Bảo Vệ Dân Quyền và Bình Đẳng (OECR) của WSDOT. Muốn biết thêm chi tiết liên quan đến thủ tục khiếu nại Mục VI và/hoặc chi tiết liên quan đến trách nhiệm không kỳ thị của chúng tôi, xin liên lạc với Phối Trí Viên Mục VI của OECR số (360) 705-7090.

**Thông tin về Đạo luật Người Mỹ tàn tật (Americans with Disabilities Act, ADA)**

Tài liệu này có thể thực hiện bằng một hình thức khác bằng cách email cho Văn Phòng Bảo Vệ Dân Quyền và Bình Đẳng wsdotada@wsdot.wa.gov hoặc gọi điện thoại miễn phí số, 855-362-4ADA(4232). Người điếc hoặc khiếm thính có thể yêu cầu bằng cách gọi cho Dịch vụ Tiếp âm Tiểu bang Washington theo số 711.

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**ARABIC – العَرَبِيّة**

**العنوان 6 إشعار للجمهور**

تتمثل سياسة وزارة النقل في ولاية واشنطن (WSDOT) في ضمان عدم استبعاد أي شخص، على أساس العرق أو اللون أو الأصل القومي من المشاركة في أي من برامجها وأنشطتها أو الحرمان من الفوائد المتاحة بموجبها أو التعرض للتمييز فيها بخلاف ذلك، كما هو منصوص عليه في الباب السادس من قانون الحقوق المدنية لعام 1964. ويمكن لأي شخص يعتقد أنه تم انتهاك حقوقه التي يكفلها الباب السادس تقديم شكوى إلى مكتب المساواة والحقوق المدنية (OECR) التابع لوزارة النقل في ولاية واشنطن. للحصول على معلومات إضافية بشأن إجراءات الشكاوى و/أو بشأن التزاماتنا بعدم التمييز بموجب الباب السادس، يرجى الاتصال بمنسق الباب السادس في مكتب المساواة والحقوق المدنية على الرقم 7090-705 (360).

**معلومات قانون الأمريكيين ذوي الإعاقة (ADA)**

يمكن توفير هذه المواد في تنسيق بديل عن طريق إرسال رسالة بريد إلكتروني إلى مكتب المساواة والحقوق المدنية على wsdotada@wsdot.wa.gov أو عن طريق الاتصال بالرقم المجاني: (4232) 855-362-4ADA. يمكن للأشخاص الصم أو ضعاف السمع تقديم طلب عن طريق الاتصال بخدمة Washington State Relay على الرق 711.

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**中文 – CHINESE**

《权利法案》Title VI公告

<華盛頓州交通部(WSDOT)政策規定，按照《1964 年民權法案》第六篇規定，確保無人因種族、膚色或國籍而被排除在WSDOT任何計畫和活動之外，被剝奪相關權益或以其他方式遭到歧視。如任何人認為其第六篇保護權益遭到侵犯，則可向WSDOT的公平和民權辦公室(OECR)提交投訴。如需關於第六篇投訴程式的更多資訊和/或關於我們非歧視義務的資訊，請聯絡OECR的第六篇協調員，電話(360) 705-7090。

《美国残疾人法案》(ADA)信息

可向公平和民權辦公室發送電子郵件wsdotada@wsdot.wa.gov或撥打免費電話 855-362-4ADA(4232)，以其他格式獲取此資料。听力丧失或听觉障碍人士可拨打711联系Washington州转接站。

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**Af-soomaaliga – SOMALI**

**Ciwaanka VI Ogeysiiska Dadweynaha**

Waa siyaasada Waaxda Gaadiidka Gobolka Washington (WSDOT) in la xaqiijiyo in aan qofna, ayadoo la cuskanaayo sababo la xariira isir, midab, ama wadanku kasoo jeedo, sida ku qoran Title VI (Qodobka VI) ee Sharciga Xaquuqda Madaniga ah ah oo soo baxay 1964, laga saarin ka qaybgalka, loo diidin faa'iidooyinka, ama si kale loogu takoorin barnaamijyadeeda iyo shaqooyinkeeda. Qof kasta oo aaminsan in difaaciisa Title VI la jebiyay, ayaa cabasho u gudbin kara Xafiiska Sinaanta iyo Xaquuqda Madaniga ah (OECR) ee WSDOT. Si aad u hesho xog dheeraad ah oo ku saabsan hanaannada cabashada Title VI iyo/ama xogta la xariirta waajibaadkeena ka caagan takoorka, fadlan la xariir Iskuduwaha Title VI ee OECR oo aad ka wacayso (360) 705-7090.

**Macluumaadka Xeerka Naafada Marykanka (ADA)**

Agabkaan ayaad ku heli kartaa qaab kale adoo iimeel u diraaya Xafiiska Sinaanta iyo Xaquuqda Madaniga ah oo aad ka helayso wsdotada@wsdot.wa.gov ama adoo wacaaya laynka bilaashka ah, 855-362-4ADA(4232). Dadka naafada maqalka ama maqalku ku adag yahay waxay ku codsan karaan wicitaanka Adeega Gudbinta Gobolka Washington 711.

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# Chapter 1 Design Policy

## 1-1 Introduction

This *Hydraulics Manual* provides policy for designing hydraulic features related to Washington State Department of Transportation (WSDOT) roadways including hydrology, culverts, open-channel flow, drainage collection and conveyance systems, water crossings, 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 also based on the Federal Highway Administration's (FHWA's) *Hydraulic Engineering Circulars* (HECs) and the American Association of State Highway and Transportation Officials (AAS*HTO)* *Drainage Manual*.

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* for general hydraulic design guidance. Design-build projects should also consult the *Design Manual* and the *Design-Build Manual*.

In addition to the guidance in the *Hydraulics Manual*, the hydraulic designer shall use good engineering judgment and be mindful of WSDOT's legal and ethical obligations concerning hydraulic issues. Drainage facilities must be designed to convey 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 that 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 Project Engineer's Office (PEO), Region Hydraulics Engineer (RHE), and State Hydraulics Office are defined. WSDOT has specific documentation requirements for a 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 design tools or software for most hydraulic features contained in the *Hydraulics Manual* is provided in Section 1-4. Section 1-5 describes the Complete Streets program and how it may affect some aspects of hydraulic design. Section 1-6 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. All drainage structure types, culverts, storm sewer, drainage, general pipe connections, and pipe locations must be verified and annotated by the PEO. Actual design work may be performed by the PEO, by another WSDOT office, or by a private consulting firm with engineering staff who are licensed in Washington State; 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 1-6. 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 preparation of hydraulic reports and plans, specifications, and estimates (PS&E) for all drainage facilities, assistance from the RHE and the State Hydraulics Office may be requested for any drainage facility design. The RHE and State Hydraulics Office offer technical assistance to PEOs 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, etc.).
3. Analysis of closed drainage basins and unusual or unique drainage conditions.
4. Upstream and downstream analysis to identify and evaluate potential impacts from the project on the hydraulic conveyance system near the project site. The analysis shall be divided into three sections:
    a) Review of resources
    b) Inspection of drainage conveyance systems in the site area
    c) Analysis of upstream effects
    d) Analysis of downstream effects

The roles and responsibilities of the RHE and State Hydraulics Office are outlined in Table 1-1. The State Hydraulics Office also takes primary responsibility for the following:

1. Design of habitat features and stream restoration elements.
2. Hydraulic analysis (one-dimensional [1D] and two-dimensional [2D]) and support for scour of water crossings.
3. Analysis of streambank erosion along roadways, river and stream lateral migration, the design of countermeasures for scour and stream instability, and environmental mitigation.
4. Floodplain studies, flood predictions, and special hydrological analysis (snowmelt estimates, storm frequency predictions, etc.).
5. Wind and wave analysis.
6. Technical support to local programs for hydraulic or bridge-related needs.
7. Providing the Washington State Attorney General's Office with technical assistance on hydraulic issues.
8. Updating information in the Hydraulics Manual periodically.

9. Providing technical information for the *Highway Runoff Manual* updates.
10. Maintaining WSDOT's Standard Plans; *Standard Specifications for Road, Bridge, and Municipal Construction* (Standard Specifications); and General Special Provisions (GSP) involving drainage-related items.
11. 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 State Hydraulics Office.
12. Reviewing and concurring with Type A hydraulic reports, unless otherwise delegated to the RHE by the State Hydraulics Office.
13. Providing the regions with technical assistance on hydraulic issues that are the primary responsibility of the PEO.
14. Providing basic hydrology and hydraulics training material to the regions. Either the RHE or State Hydraulics Office personnel can perform the actual training. (See the State Hydraulics Office on the WSDOT Hydraulics Training web page for information on course availability.)

## 1-3 Hydraulic Reports

The hydraulic report is intended to serve as a complete documented record containing the engineering justification for all drainage-, water crossing–, floodplain-, conveyance-, and stormwater-related installations and modifications that occur as a result of the project. A hydraulic report facilitates design review and assists in PS&E preparation. The hydraulic report shall be well written in the appropriate WSDOT template, 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 who 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 and is required on the title page of any hydraulic report created for WSDOT. See training information on the WSDOT Hydraulics Training web page.

A *Fish Passage and Stream Restoration Design* (FPSRD) certificate number is required for all authors and co-authors of any portion of a fish passage and stream restoration design specialty report. See Table 1-1 for a list of specialty reports and other requirements. An FPSRD certificate number is given to those who have viewed all the training modules and successfully passed the comprehensive exam. Additional information, training resources, and the point of contact for this training can be found on the WSDOT Hydraulics Training web page. As WSDOT updates the FPSRD training modules a re-certification number is also required. Any updates to this training will be posted on the WSDOT Hydraulics Training web page.

A scour analysis is required for all WSDOT projects or WSDOT-managed infrastructure associated with scour or that have a potential to be impacted by scour, such as water crossings, walls, roadway embankments, and other WSDOT infrastructure. A *WSDOT Scour Certification Record* number is required for all Stream Team members (defined in Chapter 7-1) that are conducting scour calculations, lateral migration, scour analysis, and reviews as part of or supporting specialty reports. See Table 1-1 for a list of specialty reports and other requirements. A *Scour Certification Record* certificate number is given to those who have viewed all the WSDOT Scour Training Workshops and FHWA Bridge Scour Workshop Recordings; completed National Highway Institute (NHI) Course 135046, *Stream Stability and Scour at Highway Bridges, and* NHI Course 135048, *Countermeasures Design for Bridge Scour and Stream Instability*; and successfully passed the comprehensive exam. Additional information, training resources, and the point of contact for this training can be found on the WSDOT Hydraulics Training web page. As WSDOT updates the Scour Training modules a re-certification number is also required. Any updates to this training will be posted on the WSDOT Hydraulics Training web page.

The following training courses are required to obtain a scour certification:

- FHWA Bridge Scour Workshop Recordings
- NHI Course 135046, *Stream Stability and Scour at Highway Bridges*
- NHI Course 135048, *Countermeasures Design for Bridge Scour and Stream Instability*
- WSDOT 2023 Scour training

SRH-2D hydraulic modeling training is required for all WSDOT projects or WSDOT-managed infrastructure that requires hydraulic modeling as part of the hydraulic design process. Hydraulic modelers are required to obtain a training certificate from NHI for attending Course 135095, *Two-Dimensional Hydraulic Modeling of Rivers at Highway Encroachments*. Other equivalent SRH-2D hydraulic modeling training requires approval by the State Hydraulics Office.

## 1-3.1 *Hydraulic Report Types*

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

**Table 1-1** Hydraulic Report Documentation

| Report Type | Description[b] | Concurrence[c] | | PE Stamp |
|---|---|---|---|---|
| | | RHE | State Hydraulics Office | |
| Stormwater and hydraulic assessment[a] | All projects shall complete a stormwater and hydraulics assessment to determine what type of stormwater and hydraulic design, documentation, and level of effort are needed for the project. Some questions for the PEO to answer as part of the stormwater and hydraulic assessment include:<br>• Does the project have existing stormwater and hydraulic deficiencies within the project limits? If so, assess and discuss the risk of the project not addressing these deficiencies.<br>• Does the project's impacts or modifications make existing stormwater and hydraulic conditions worse?<br>• Does the project's impacts or modifications create new stormwater and hydraulic issues that need to be addressed?<br>• Are there any stormwater retrofit opportunities within the project limits? | | | |
| Specialty report[d,l] | Projects with any of the following components:<br>• Culverts or buried structures greater than 48 inches in diameter or span<br>• Bridge drainage<br>• Fish passage[e]<br>• Bank protection<br>• Woody material (WM)[e]<br>• River structures (e.g., barbs, engineered log jams [ELJs], levees)[e]<br>• Channel realignment/modifications or restoration[e]<br>• Any fills in floodplain or floodway<br>• Pump stations<br>• Hydraulic connectivity zones<br>• Siphons<br>• Bridges<br>• Scour analysis (e.g., bridges, walls, roadway embankments, other WSDOT infrastructure)[f] | | ✓ | ✓[g] |

| Report Type | Description[b] | Concurrence[c] | | PE Stamp |
|---|---|---|---|---|
| | | RHE | State Hydraulics Office | |
| A[d,l] | Projects with any of the following components:<br>• Water quality treatment facility<br>• Flow control facility<br>• Storm sewer systems that discharge into a stormwater treatment or flow control facility<br>• Create, modify, or remove any existing or new BMP (full or partial treatment BMP)<br>• Fish passage stormwater treatment assessment for full or partial treatment[h]<br>• Region facilities projects[i] | ✓[i,j] | | ✓ |
| B[c,d,k] | Projects without Type A components and with any of the following components:<br>• Stormwater and non-fish passage culverts up to 48 inches in diameter[d]<br>• Storm sewer systems that do not discharge into a stormwater treatment or flow control facility<br>• Paving/safety restoration and preservation projects | ✓ | | ✓ |

***Notes:***

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

a. A stormwater and hydraulic assessment typically occurs just after project kickoff during design (project development). In some cases, a stormwater and hydraulic assessment may be as early as the predesign phase of a project.
b. Projects listed are examples. Projects not listed may still require a specialty report based on direction from the RHE.
c. In no case may the PEO provide concurrence on its own design.
d. State Hydraulic Office and the RHE shall be involved in developing the scope, budget, schedule, and/or the Request for Proposal for projects.
e. Fish passage projects shall be designed by a Stream Team, approved by the State Hydraulics Office, and consisting of a stream design engineer, geomorphologist, and biologist, who shall all co-author the specialty report and have received their FPSRD certifications.
f. Scour certification is required for stream design engineers, Geomorphologists, or any other team members conducting and reviewing scour calculations and analysis.
g. The PE stamp shall be either by the State Hydraulics Office or by a licensed engineer in Washington State and approved by the State Hydraulics Office.
h. All fish passage projects shall complete a stormwater assessment for the feasibility of full or partial stormwater treatment BMPs. See *Highway Runoff Manual* for more information.
i. Facilities designed by the RHE will have concurrence from the State Hydraulics Office.
j. The State Hydraulics Office may delegate 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.
k. A Hydraulic Design Concurrence memo is required by the RHE to the PEO to document that all comments have been addressed.
l. A Hydraulic Design Concurrence memo is required by the State Hydraulic Office to document that all comments have been addressed.

## 1-3.2 Preparing Hydraulic Documentation

The overall hydraulic design process is part of scoping, predesign, design, and construction. To allow the most efficient hydraulic report review and assessment, PEOs shall follow the hydraulic review process outlined in Section 1-6.

### 1-3.2.1 Type A and Type B Hydraulic Report Content and Outline

The hydraulic report checklist identifies the required subject matter that the Type A (and sometimes Type B) hydraulic report shall contain. 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 design decisions made for the design of the project. The report shall contain enough information to allow reproduction of the design in its entirety, but at the same time the report shall 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 Type A hydraulic report outline has been developed as a starting point. Use of the outline is mandatory; organizing reports in the outline format may expedite the review process. Because some regions have modified the outline to meet specific regional needs or requirements, PEOs shall contact their RHE to determine the correct outline before starting a report. Once the relevant outline is selected, PEOs shall read through the outline, determine which sections are applicable to the project, and delete those that are not. Either the RHE or the State Hydraulics Office can be contacted for assistance in preparing a Type A hydraulic report and for current updates to the Type A hydraulic report outline.

The detailed documentation of a Type B hydraulic report can vary greatly depending on the details of the project scope. Work with the RHE to determine the appropriate level of detail needed to document the hydraulic design decisions in a Type B hydraulic report.

The author shall not copy sections of the *Hydraulics Manual* or *Highway Runoff Manual* into the hydraulic report because it would add redundant information to the report. Instead, authors shall reference the relevant section and version in the hydraulic report narrative.

### 1-3.2.2 Specialty Report Content and Outline

Specialty reports shall consist of a preliminary hydraulic design (PHD) report and a final hydraulic design (FHD) report. The PHD report is created during the initial stages of project design, prior to the final design and construction phase. This report provides a preliminary analysis of the hydraulic considerations that will influence the design moving forward. The FHD report is the basis for the project's FHD approval and is used throughout the construction phase to ensure that hydraulic components function as intended and support the overall safety and functionality of the transportation infrastructure.

Both reports are critical in ensuring that WSDOT projects meet necessary hydrological requirements and WSDOT design policies. Report templates can be found on the WSDOT Hydraulics and Hydrology webpage (Hydraulics & hydrology | WSDOT).

### 1-3.2.3 Stormwater and Hydraulic Assessment Content

A stormwater and hydraulic assessment is required to be completed for every project. The purpose of the assessment is to identify if there is any drainage-, water crossing–, conveyance-, and stormwater-related work on the project so the level of effort and required hydraulic documentation can be discussed and planned for. The PEO shall conduct the assessment right after project kickoff and it may take the form of a general meeting between the PEO and the RHE or State Hydraulics Office. When the level of effort and required hydraulic documentation discussed during the assessment is determined to be very minor (e.g., a paver project), the assessment documentation could simply be the meeting notes or follow-up email between the PEO and RHE or State Hydraulics Office stating that there is no drainage-, water crossing–, conveyance-, or stormwater-related work on the project. If the level of effort and required hydraulic documentation discussed during the assessment appears to be significant, it is recommended that the PEO schedule regular check-in meetings with the RHE or State Hydraulics Office as the design progresses. See the hydraulic report checklist. The stormwater and hydraulic assessment deliverable would be the meeting notes.

One important outcome from the stormwater and hydraulic assessment is a discussion on the feasibility of dispersion and infiltration on the site to aid in the design of low-impact development (LID) BMPs. To determine the feasibility of LID BMPs, the PEO may need geotechnical information about site soils, infiltration rates, and seasonal high groundwater table elevations where potential stormwater BMP locations are along the project. After the stormwater and hydraulic assessment and if the project may construct or place stormwater BMPs, it is strongly recommended that the PEO issue a geotechnical soils investigation memorandum as early as possible. The PEO shall discuss these issues with the Region Materials Engineer (RME) or HQ Geotechnical Office in preparation of a geotechnical investigation memorandum. Issuing the geotechnical investigation memorandum early in the project development process will give enough time for the geotechnical investigation work to be completed so that the stormwater designs can be completed on time.

### 1-3.2.4 Deviations from the *Hydraulics Manual*

Deviations from the requirements in the *Hydraulics Manual* must clearly state why a deviation is necessary and document all the steps used in the analysis in a hydraulic deviation. Deviations from this manual require approval prior to submitting a hydraulic report for review. Requests for a deviation shall go through the RHE to the State Hydraulics Office engineering staff. A Hydraulic Deviation template is available on the WSDOT Hydraulics & hydrology website under the Tools, templates & links tab.

### 1-3.2.5 Design Tools and Software

The design tools and programs described in the *Hydraulics Manual* and in the *Highway Runoff Manual* shall be used whenever possible. To determine if software and/or a design tool is required, PEOs shall review Section 1-4 or check the expanded list on the State Hydraulics Office web page. If a PEO wishes to use a design tool or software other than those required, it must request concurrence during the 10 percent milestone timeline for the hydraulic design report through the RHE.

#### 1-3.2.6 Contract or Scope of Work for Hydraulic Support

Contact the RHE and/or State Hydraulics Office to review the contract or scope prior to hiring a consultant.

### 1-3.3 *Hydraulic Report Deliverables, Submittals, and Archiving*

It is important to understand the various stormwater and hydraulic deliverables produced for a given project. It is equally important to understand to whom to submit deliverables and when. Hydraulic reports have their own WSDOT document retention schedule so understanding the process for archiving these records is also discussed in this section.

#### 1-3.3.1 Hydraulic Report Deliverables for Design-Bid-Build Projects

Following Table 1-1, at a minimum, the PEO shall develop a stormwater and hydraulic assessment for each project and coordinate with RHE. In the scenario where there is a lot of stream work but little road work (like a fish barrier correction project), the PEO would need a stormwater and hydraulic assessment, a Type B hydraulic report, and a specialty report. For more complicated roadway improvement projects, the PEO would need a stormwater and hydraulic assessment, a Type A hydraulic report, and possibly a specialty report. The PEO shall work with the RHE or State Hydraulics Office to determine what type of hydraulic documentation is needed for the design-bid-build project during the stormwater and hydraulic assessment.

#### *1-3.3.1.1* Hydraulic Report Submittal Process for Design-Bid-Build Projects

##### 1-3.3.1.1.1 Specialty Report Submittals

The PEO shall coordinate with the State Hydraulics Office for the PHD and FHD report.

##### 1-3.3.1.1.2 Type A and Type B Hydraulic Report Submittals

The hydraulic report submittal process will vary based on the hydraulic report type. For a Type B hydraulic report for a design-bid-build project, because the drainage-, conveyance-, and stormwater-related work on the project is very limited, the PEO can work with the RHE or State Hydraulics Office to determine a submittal timeline for the Type B hydraulic report. For a Type A hydraulic report for a design-bid-build project, the submittal process is a little more defined. Below is a description of each Type A hydraulic report submittal and the approximate timing of each submittal:

a) **Preliminary Type A report:** This submittal shall occur during the project development phase after the stormwater and hydraulic assessment.

    a. Recommend this submittal after *Highway Runoff Manual* minimum requirements and Endangered Species Act (ESA) programmatic consultation stormwater requirements (if applicable) have been determined, draft threshold discharge area (TDA) delineations are complete, discharge locations have been identified, existing drainage issues within the project limits have been identified, existing stormwater drainage system has been mapped within the project limits, stormwater retrofitting requirements have been determined (if applicable), potential stormwater (*Highway Runoff Manual*) and hydraulic (*Hydraulics Manual*) deviations have been identified, and cursory review of possible stormwater connection utility discharge permits has been conducted.

b. Generally, this may occur around 30 percent project design.

c. The design PEO submits the Preliminary Hydraulic report Type A for review and comments to the RHE or State Hydraulics Office per Table 1-1.

b) **Intermediate Type A Hydraulic report:** This submittal shall occur before the start of the PS&E phase when all of the engineering has been completed.

a. Recommend this submittal when the stormwater and hydraulic design is complete; the final stormwater BMP type, size, and locations have been designed; the conveyance design is complete; the upstream and downstream analysis is complete; any stormwater (*Highway Runoff Manual*) and hydraulic (*Hydraulics Manual*) deviations have been approved; and draft BMP maintenance plans have been created.

b. The design PEO submits the intermediate Type A hydraulic report for review, comments, and concurrence to the RHE or State Hydraulics Office per Table 1-1.

c. This generally occurs around 60 percent project design.

d. If there are drainage-related addendums (changes) during the PS&E phase of the project, the PEO shall contact the engineer of the intermediate Type A hydraulic report to evaluate those addendums to determine if they affect the stormwater and hydraulic design and if those changes require an update to the intermediate Type A hydraulic report. Any changes need to be incorporated into the intermediate Type A hydraulic report and the report needs to be restamped.

e. The design PEO submits the revised intermediate Type A hydraulic report (because of addendums) for review, comments, and concurrence to the RHE or State Hydraulics Office per Table 1-1.

c) **Drainage-related change orders:** These submittals shall occur after the start of the construction phase of the project but before substantial completion.

a. Recommend this submittal when drainage-related change orders occur during the construction phase of the project.

i. For any drainage-related change orders that may affect the stormwater and hydraulics design, the construction office needs to contact the engineer of record who stamped the intermediate Type A hydraulic report so that those drainage-related change orders can be evaluated to determine if they affect any other parts of the stormwater and hydraulic design and if any redesign is required. If contacting the original engineer of record is not possible, the construction office can work with the RHE or State Hydraulics Office to determine if any changes need to be made to the stormwater and hydraulic design and intermediate Type A hydraulic report because of the drainage-related change order(s). Any drainage-related changes need to be worked into the overall stormwater and hydraulic design and the final Type A hydraulic report. In some cases where drainage-related change orders require significant changes, many things may need to be updated including TDA delineations, *Highway Runoff Manual* minimum requirements, the stormwater design documentation spreadsheet, the conveyance design, and any stormwater and hydraulic deviations previously approved. The engineer of record overseeing these new changes would need to stamp the final Type A hydraulic report to cover any changes as a result of the drainage related change orders.

b. The construction PEO submits the drainage-related change orders for review and comments to the RHE or State Hydraulics Office per Table 1-1.

c. If the drainage-related change orders, after consulting with the engineer of record and the RHE or State Hydraulics Office, do not require a change or there are no drainage-related change orders to the intermediate Type A hydraulic report, then the intermediate Type A hydraulic report can be renamed as the final Type A hydraulic report.

d) **Final Type A Hydraulic report:** This submittal shall occur after construction of the project has reached substantial completion.

a. Recommend this submittal after all drainage-related change order submittals (if any) have been approved and constructed, drainage-related change order submittal changes have been incorporated into the final Type A hydraulic report and all relevant sections of the final Type A hydraulic report have been updated, and as-built verification of stormwater and hydraulic features has occurred.

b. The construction PEO submits the final Type A hydraulic report for review, comments, and concurrence to the RHE or State Hydraulics Office per Table 1-1.

c. This generally occurs during the construction phase of the project but after substantial completion.

d. BMP maintenance plans shall be finalized along with the final Type A hydraulic report.

PEOs shall ensure that any electronic submittal is complete and is searchable. The PEO can use the hydraulic report checklist to help identify and schedule critical submittal dates.

### 1-3.3.2 Hydraulic Report Deliverables for Design-Build Projects

Projects using a design-build delivery method have a different hydraulic report submittal process from that described for the design-bid-build delivery method (see Section 1-3.3.1).

The PEO shall coordinate with the RHE or the State Hydraulics Office to determine the expected deliverable for the design-build project and coordinate on the completion of the Request for Proposals (RFP) Technical Requirements 2.30, Water Crossings.

The design PEO typically creates a conceptual Type A or Type B hydraulic report and completes the RFP Technical Requirements 2.14, Stormwater, in preparation for the procurement phase of the design-build process. Once the design-builder is selected and awarded the contract, the design-builder becomes the engineer of record and completes the stormwater and hydraulic design and Type A or Type B hydraulic report for the project. The PEO shall coordinate with the State Hydraulics Office for the PHD and FHD report deliverables to complete RFP Section 2.30

A conceptual Type A or Type B hydraulic report describes the conceptual stormwater and hydraulic designs for the project that are used for various purposes. The conceptual hydraulic report is used to show one possible pathway for the design-builder to reach compliance with the hydraulic design requirements for the project. More information regarding the conceptual hydraulic report and other details can be found in the *Design-Build Manual*.

The design PEO must work with the RHE or State Hydraulics Office to develop the conceptual Type A or Type B hydraulic report and to complete RFP Section 2.14 for the project. The PEO shall coordinate with the State Hydraulics Office for the PHD and FHD report deliverables to complete RFP Section 2.30.

#### *1-3.3.2.1* Hydraulic Report Submittal Process for Design-Build Projects

All submittals shall be in electronic format. All pages of all submittals shall be in searchable Portable Document Format (PDF). In addition to the searchable PDF document, submittals that include hidden information not visible in PDF format (such as calculations in the cells of a spreadsheet or drawing) shall be submitted in their original format (e.g., Word, Excel, InRoads) to facilitate WSDOT's full review and understanding of the basis and assumptions for calculations and other output

A conceptual Type A or Type B hydraulic report shall have the following items:

a) **Conceptual hydraulic report:** This submittal shall occur during the project development phase after the stormwater and hydraulic assessment but before finalizing the RFP.

    a. Recommend this submittal after *Highway Runoff Manual* minimum requirements and ESA programmatic consultation requirements (if applicable) have been determined, draft TDA delineations are complete, existing drainage issues have been identified, the existing stormwater drainage system has been identified, discharge locations have been identified, stormwater retrofitting requirements have been determined (if applicable), potential stormwater (*Highway Runoff Manual*) and hydraulic (*Hydraulics Manual*) deviations have been identified and received approval, and cursory review of possible stormwater connection utility discharge permits has been conducted.

b. Generally, this may occur around 30 percent project design.

c. The design PEO submits the conceptual hydraulic design report for review and comments to the RHE or State Hydraulics Office per Table 1-1.

b) **Design-builder's Preliminary Type A or Type B hydraulic report:** This submittal shall occur after the project has been awarded to a design-builder but before the first intermediate drainage design package.

a. This submittal shall provide draft designs and preliminary responses for the following issues:

i. Meet *Highway Runoff Manual* minimum requirements and ESA programmatic consultation requirements (if applicable)

ii. Provide draft TDA delineations

iii. Determine existing discharge locations within the project limits

iv. Determine existing drainage issues within the project limits

v. Map the existing stormwater drainage system within the project limits

vi. Determine stormwater retrofitting requirements (if applicable)

vii. Identified potential stormwater (*Highway Runoff Manual*) and hydraulic (*Hydraulics Manual*) deviations

viii. Identified possible stormwater connection utility discharge permits

ix. Any additional requirements per the RFP

b. The design-builder submits the Type A or B preliminary hydraulic report for review and comment to the WSDOT engineer (who sends it to the RHE or State Hydraulics Office per Table 1-1).

c) **Design-builder's intermediate hydraulic design packages:** These submittals shall occur after the design-builder's preliminary hydraulic report but before the design-builder's Type A or B intermediate hydraulic report.

a. The design-builder submits the Type A or B intermediate hydraulic design packages for review and comments to the WSDOT engineer (who sends it to the RHE or State Hydraulics Office per Table 1-1).

d) **Design-builder's Type A or Type B intermediate hydraulic report:** This submittal shall occur after the last design-builder's hydraulic design package but before the design-builder's Type A or B final hydraulic report.

  a. This submittal shall incorporate all of the hydraulic design packages into one coherent and complete stormwater and hydraulics design and Type A or B hydraulic report that shows how the project has addressed and is compliant with the mandatory standards and the RFP.
  b. The design-builder submits the Type A or B intermediate hydraulic report for review and comments to the WSDOT engineer (who sends it to the RHE or State Hydraulics Office per Table 1-1).

e) **Design-builder's Type A or Type B final hydraulic report:** This submittal shall occur after construction is complete on the project and after the as-built verification of stormwater and hydraulic features walk-through.

  a. This submittal shall incorporate any changes that occurred after the intermediate hydraulic report and generate one coherent and complete stormwater and hydraulics design and Type A or B hydraulic report that shows how the project has addressed and is compliant with the mandatory standards and the RFP.
  b. The design-builder submits the Type A or B final hydraulic report for review and comments to the WSDOT engineer (who sends it to the RHE or State Hydraulics Office per Table 1-1). The SHO or RHE shall issue a hydraulic report concurrence memo once all comments for the final hydraulic report have been resolved.
  c. BMP maintenance plans shall be finalized along with the Type A final hydraulic report.

**f) Specialty Reports:** The specialty report(s) shall describe the approach taken and the order of the calculations, including sections on the methodologies used (appropriateness and accuracy requirements), design decisions made, and resultant summaries. The calculations shall include electronic copies of the input and output from the supporting computer programs, spreadsheets, hand calculations, exhibits, and sketches. At a minimum, the calculations shall also include the following design calculation items:

  a) Word and PDF file;
  b) Excel files for figures in text;
  c) Long profile and long-term degradation;
  d) Pebble counts and sediment mobility calculations;
  e) Reference reach cross-section comparison figure;
  f) Others;
  g) Geographic information system (GIS) data;
  h) Field visit data including bankfull width (BFW), pebble count, and reference reach locations;
  i) Basin boundary;

j) Appendix files;
k) Large woody material (LWM) calculator;
l) Sediment size and mobility;
m) Manning's n roughness;
n) Excel files for model results at cross sections and profiles;
o) Scour calculations FHWA Toolbox Report and HYD files;
p) Scour countermeasure calculations FHWA Toolbox Report and HYD files;
q) Field visit photos;
r) Hydrology;
s) MGSFlood model if used;
t) Other hydrology models;
u) Hydraulic model;
v) SRH-2D model;
w) All input and output files;
x) Remove extraneous or working files/simulations: coverages and simulations shall be clearly named;
y) Coverages used for results reporting including observation lines and 1D centerline and cross section;
z) Special design features: design-builder shall include a brief narrative of design decisions or revisions, electronic files from design calculations, and justification;
aa) Design decision summaries;
bb) Technical specifications necessary for construction;
cc) Drainage maps showing the water crossing structures and all other illustrations necessary to support and clarify the design calculations. Electronic design drawings and maps, when printed, shall be on 11-by-17-inch pages;
dd) Channel section design;
ee) Streambed material sizing;
ff) Scour analysis;
gg) Scour analysis for streambed gravel sizing around LWM structures, if applicable;
hh) LWM buoyancy and anchoring calculations, if applicable; and
**ii)** Other applicable data or analysis.

PEOs and the design-builder shall ensure that any electronic submittal is complete and is searchable. The PEO can use the hydraulic report checklist to help identify and schedule critical submittal dates.

#### 1-3.3.3 Final Copies and Archiving

Upon receiving concurrence of a Type A or B hydraulic report, PEOs shall submit a searchable electronic copy of the Type A or B hydraulic report, which shall also include the concurrence letter, to the offices noted below. Electronic copies shall include the entire contents of the Type A or B hydraulic report (including the appendices files) in a PDF file.

1. For design-bid-build projects, send one PDF of the Type A or B intermediate hydraulic report to the Construction Office for reference during construction.
2. For design-bid-build projects, along with the concurrence letter, the PEO shall upload the Type A or Type B intermediate hydraulic report to the Enterprise Content Management (ECM) application along with the Design Decision Package (DDP) for archiving.
3. For design-bid-build projects, if any stormwater or hydraulic related change orders occur during the project's construction that affect a hydraulic feature's intended function, the Type A or Type B hydraulic report shall be revised to incorporate the changes. After a review of the revised hydraulic report following Table 1-1 and receiving a new concurrence letter, the revised hydraulic report and concurrence letter shall be combined into one final hydraulic report document (PDF) and uploaded to the EMC by the construction office or RHE before the construction project closeout. If no stormwater or hydraulic related change orders occurred during the construction phase of the project, the construction office or RHE can make a note of this in the ECM and can rename the Type A or B intermediate hydraulic report to the final Type A or Type B hydraulic report.
4. For water crossings documented in FHD reports, send one PDF to the Bridge Preservation Office.
5. For design-build projects, the Type A or B final hydraulic report and Specialty Reports shall be uploaded to the ECM application by the construction project office.

### 1-3.4 *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 *Design Manual*, Utilities Manual, and Local Agency Guidelines manual.

### 1-3.5 *Upstream and Downstream Analysis*

Conducting an upstream and downstream analysis as part of a Type A or B or specialty report identifies, evaluates, and documents the impacts and risks, if any, that a project will have on the drainage conveyance system, properties, and sensitive areas. All projects that propose to discharge stormwater from WSDOT ROW and meet the requirements below are required to provide an analysis as part of the hydraulic report;

see the hydraulic report outline for more information. For projects that require a flood risk assessment see additional guidance in Chapter 7.

- 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 either the upstream or 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 upstream or downstream of WSDOT's ROW
- Projects that alter existing hydrology or drainage

### 1-3.5.1 Upstream and Downstream Analysis for Type A and B 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, flood duration, water surface elevations (WSELs), bank erosion, channel erosion, and nutrient loading from the project site. The analysis is divided into three steps that follow sequentially:

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

### 1-3.5.2 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 include 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, GIS and light detecting and ranging (LiDAR) information, and any previously completed upstream or 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 and upstream to a point where any backwater conditions cease.

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.

### 1-3.5.3 Inspection of Drainage Conveyance System

The PEO must inspect the 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 the WSDOT ROW for a distance of at least 0.25 mile and upstream to a point where any backwater conditions cease. 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 or 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 upstream and downstream resources.

#### 1-3.5.4 Analysis of Upstream and Downstream Effects

This final step analyzes information gathered in the first two steps of the 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 upstream and 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* statewide map, the PEO must run extreme events (e.g., the 100-year storm event) and evaluate the impacts and stability of the conveyance system. The PEO will perform a risk assessment based on the extreme events showing impacts to the conveyance system and to downstream properties and sensitive areas.

PEOs will 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 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 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. Below are two examples:
    - Roadway runoff in a project’s TDA was sheet-flowing to the roadway side slopes in the pre-developed condition but is now being collected and conveyed to a stormwater detention pond in the post-developed condition. The detention pond’s emergency overflow usually discharges to the same location as the riser structure and overflow structure but sometimes discharges to a different location. In both scenarios, even though the detention pond will provide flow control for more frequent storm events (up to the 25-year for eastern Washington or 50-year for western Washington), the larger, less frequent storm events (100-year) may not have flow control. These scenarios need to be analyzed as part of the downstream analysis. Because the stormwater is now collected and conveyed to one or two discharge locations, there may be more flow at those discharge locations

than in the pre-developed condition. If a situation is encountered where downstream impacts will result 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.

### 1-3.6 *Existing Stormwater Drainage Conveyance System*

During the stormwater and hydraulic assessment, the existing stormwater drainage conveyance system (culverts, storm sewers, catch basins, manholes, inlets, grates, and ditches) shall be discussed to identify any needed repairs or replacements. If possible, it is strongly recommended that the PEO physically inspect the entire existing stormwater drainage conveyance system within the project limits, especially if adding new stormwater flows to it. There may be condition ratings for some of these existing stormwater features in Highway Activities Tracking System (HATS) or the Stormwater Features Inventory that may aid in determining the physical inspection requirements. Contact the State Hydraulics Office for culvert Level 1 and Level 2 inspection requirements and guidelines. See the 2020 AASHTO Culvert and Storm Drain System Inspection Guide for guidance on inspecting storm sewer, catch basins, manholes, inlets, grates, and ditches.

## 1-4 Storm Frequency Policy and Design Tools and Software

WSDOT policy regarding design storm frequency for 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. A more detailed discussion of MRI can be found in Chapter 2. New hydraulic structures shall also consider climate resilience for final design size by evaluating higher storm events. Consult the RHE and the State Hydraulics Office early for discussion and concurrence for climate-resilient designs.

For design of hydraulic features, the PEO shall review Section 1-3.2.5 for required design tools and software. The PEO shall work with the RHE to verify that the required design tools and software are used for design of hydraulic features.

If the PEO wants to use a design tool or hydraulic software that has not been approved by the State Hydraulics Office, the PEO shall provide a side-by-side comparison analysis showing the differences between the approved design tool or approved software and the proposed design tool or proposed software. The analysis shall be submitted to the RHE for review and approval. The approval of using an alternative design tool or alternative software shall be obtained before the intermediate hydraulic report can be submitted. Contact the RHE for additional guidance.

Table 1-2 presents a design reference chart and approved software.

**Table 1-2** Design Reference

| Type of Structure | Chapter Reference | Approved Software |
|---|---|---|
| Gutters | 5 | Inlet spreadsheet |
| Storm sewer inlets on longitudinal slope | 5 (MRI based on farthest downstream BMP or 10 year, whichever is greater) | Inlet spreadsheet |
| Storm sewer inlets on vertical curve sag/closed contour location | 5 (MRI based on farthest downstream BMP or 50, whichever is greater) | Sag spreadsheet |
| Storm sewers | 6[b] (MRI based on farthest downstream BMP or 25) | StormShed3G |
| Ditches | 4 | StormShed3G or *FHWA Hydraulic Toolbox* |
| Non-fish passage culverts[a] | 3 | HY-8, HEC-RAS, SRH-2D[C] |
| Temporary diversions[a] | 3 | StormShed3G, HY-8, HEC-RAS, SRH-2D[C] |
| Water crossings | 7 | SRH-2D[C] |
| Stormwater BMP | See the *Highway Runoff Manual* | |

***Notes:***

a. Coordinate with the RHE to determine the appropriate software to use and potential reports required.

b. When tying into existing systems, the hydrologic methods used shall be the rational method.

c. Use the model checklist found on WSDOT's Hydraulics & hydrology website under the Tools, templates & links tab.

## 1-5 Complete Streets

WSDOT projects involving Complete Streets are designed and operated to promote use and mobility for all users, including pedestrians, bicyclists, or transit riders. The program prioritizes comfortable, equitable network connectivity for all roadway users through close coordination with local partners and stakeholders. See the WSDOT *Design Manual* for additional information including the screening process to determine a project's need for the program.

Complete Streets or other active transportation design projects may cause changes to drainage structures or other hydraulic features beyond their basic requirements outlined in this manual; see Sections 5-4, 6-1 and 7-6.1 for additional information.

## 1-6 Hydraulic Design Schedule

Establishing a design schedule that includes the hydraulic components is critical to ensuring that the project's overall design and implementation proceed smoothly and efficiently. Hydraulic elements, such as drainage systems, culverts, stormwater management, and flood risk mitigation, can have significant impacts on various other aspects of a project, including environmental considerations, structural design, and compliance with regulations. By incorporating hydraulic design milestones early in the schedule, project teams can proactively assess how these components interact with and influence other design elements, identify potential conflicts, and make necessary adjustments to avoid delays or cost overruns. This integrated approach helps to ensure that the project is completed on time, meets regulatory requirements, and achieves its performance goals without unforeseen challenges arising from hydraulic issues.

### 1-6.1 *Milestones and Scheduling*

There are three primary types of hydraulic reports (see Section 1-3):

- Type A
- Type B
- Specialty

Schedule templates for these different types of hydraulic reports can be found online under Tools, Templates and Links on the WSDOT Hydraulics & hydrology website. Refer to *Design Manual* Section 800.03 and Exhibits 800-1 through 800-5 for an overview of the hydraulic design process. For additional guidance on schedule development please contact your RHE or the State Hydraulics Office.

# Chapter 2 Hydrology

## 2-1 Introduction

This chapter presents WSDOT's procedures and acceptable methodologies for hydraulics and hydrologic analyses for transportation hydraulic features. The procedures and methodologies presented in this chapter are based on a basic understanding of the science of hydrology and its principles. Additionally, the PEO and Stream Team (defined in Chapter 7-1) 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. However, documented reporting and high-water mark observations shall be used wherever possible to calibrate or validate the results of the below statistical and empirical methods. Where calculated results vary from on-site observations, further investigation may be required. The following methods are discussed in detail in subsequent sections of this chapter:

- Rational Method
- Santa Barbara Urban Hydrograph (SBUH) Method
- Continuous-Simulation Hydrologic Model (MGSFlood)
- Published streamflow record
- United States Geological Survey (USGS) regional regression equations
- Existing hydrologic studies
- Documented reporting

The PEO and Stream Team shall give serious consideration to documented testimony of long-time residents. Independent calculations shall be made to verify this type of reporting and observations. The information furnished by residents of the area shall include, but not be limited to, the following:

- Dates of past flood events
- High-water marks
- Amount of drift
- Any changes in the channel that may have occurred (i.e., streambed stability—is the channel widening, migrating, or meandering)
- Estimated streamflow velocity
- Description of flooding characteristics between normal flow to flood stage
- 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. However, caution shall be applied if the high-water marks are from a similar period (e.g., bathymetry/topography similar, flood event did not inundate nearby culverts or bridges causing backwater, there was not significant accumulation of debris, etc.). 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 and Stream Team accurate flood flow estimates. The State Hydraulics Office shall be contacted before a procedure other than those listed above is used in a hydrologic analysis.

The State Hydraulics Office and RHE 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 2-5 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 2-12.

## 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 Table 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 or the flow volume. It is a simple and commonly used 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.
- **Santa Barbara Urban Hydrograph (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 the State Hydraulics Office.

- **Published flow record:** This method shall be used whenever appropriate stream discharge gage data are available. This is a collection of data rather than a predictive analysis like the other methods listed. USGS, cities, counties, and other agencies gather stream discharge data on a regular basis. Collected data can be analyzed statistically to predict flood flows and are 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 no appropriate stream gage data are available. It is a set of regression equations that were developed using data from stream flow gaging 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 predominantly undeveloped basin areas. PEOs and Stream Teams shall 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 Federal Emergency Management Agency (FEMA) flood insurance studies, smaller urban drainages, citywide or countywide drainage master plans, and calibrated HSPF models. Often these values are accurate because they were developed from an in-depth analysis. Flood flow estimates can be derived from FEMA and other approved sources, including the State Hydraulics Office. Obtained data may be appropriate for culvert and bridge design.
- **Basin transfer of gage data with regional USGS equations:** When a project is located on an ungaged stream, but a stream is nearby with a substantial flow record, it is possible to extrapolate flows from one basin to the other, provided that certain criteria are met. The watersheds of the gaged and ungaged streams must have similar geology and soils, elevation range, vegetation, and canopy cover, and must be roughly the same size. The concept is simple (see Equation 2-1):

$$Q_{\text{ungaged}} = Q_{\text{gaged}}(A_{\text{ungaged}}/A_{\text{gaged}}) \qquad (2\text{-}1)$$

where:

$Q$ = discharge
$A$ = drainage area

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 on the WSDOT Hydraulics & Hydrology website.

The Flood Q Ratio Tool puts bounds on the ungaged site—it must be within 50 percent of the area of the gaged 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 that 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 ungaged basin.

**Table 2-1** Methods for Estimating Runoff Rates and Volumes

| Method | Assumptions | Data Needs |
|---|---|---|
| Rational | • Basins <200 acres<br>• Time of concentration <1 hour<br>• Storm duration less than or equal to concentration time<br>• Rainfall uniformly distributed in time and space<br>• Runoff is primarily overland flow<br>• Negligible channel storage (such as detention ponds, channels with significant volume, and floodplain storage) | • Time of concentration (minutes)<br>• Drainage area (acreage)<br>• Runoff coefficient (C values)<br>• Rainfall intensity (use m, n values to calculate inches/hour) |
| Santa Barbara Urban Hydrograph | • Rainfall uniformly distributed in time and space<br>• Runoff is based on surface flow<br>• Small to medium basins <1,000 acres<br>• Urban type area (pavement usually suffices)<br>• Regional storms (eastern Washington)[a]<br>• Short-duration storm for stormwater conveyance<br>• Long-duration storm for stormwater volume<br>• Type 1A storm (western Washington)[a] stormwater conveyance | • Curve number (CN values)<br>• Drainage area (acreage)<br>• Digital grid-based precipitation values in the WSDOT GIS, or National Oceanic and Atmospheric Administration Atlas or isopluvials |
| Continuous-simulation hydrologic model (western Washington) | • HSPF routine for stormwater BMPs for flow control facilities, such as detention and infiltration ponds, and water quality facilities, such as vegetated filter strips and bioswales,<br>• Elevations below 1,500 feet<br>• Seasonal flow for streams | • Drainage basin area (acreage)<br>• Land cover (impervious, vegetation), soils (hydrologic soil group, saturated)<br>• Slope<br>• Climatic region (mean annual precipitation) |
| Published flow record | • Basins with stream gage data<br>• Appropriate station and/or generalized skew coefficient relationship applied | • 10 or more years of gaged flood records (contact the State Hydraulics Office for additional guidance) |
| USGS regional regression equations | • Appropriate for culvert and bridge design<br>• Midsized and large basins<br>• Simple but lack accuracy of flow records for basins with more than 5% total impervious area | • 2016 regional equations<br>• Annual precipitation (inches)<br>• Drainage area (square miles)<br>• Area-weighted forest canopy (percent) |

| Method | Assumptions | Data Needs |
|---|---|---|
| Existing hydrologic studies | • Appropriate for culvert and bridge design<br>• Midsized and large watersheds<br>• Report accuracy varies so confirm level of accuracy with entity that the report derives from | • Available from FEMA or local floodplain or stormwater administrative agency—typically the city or county (however, this method is not used for culverts or bridges unless calibrated) |

***Notes:***

HSPF = Hydrological Simulation Program-Fortran.

a. The *Highway Runoff Manual* provides detailed guidance for design storms.

## 2-3 Drainage Basin

Drainage basins are the areas that contribute runoff to a point of interest such as catch basins, inlets, culverts, drainage ditches, and stormwater BMPs. These areas may include both on-site and off-site runoff and areas that extend outside of WSDOT ROW and beyond the project.

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

## 2-4 Site Basins

To determine the basin area, use the StreamStats web application, USGS quadrangle maps, or ArcMap/GIS Workbench data including LiDAR and NHD watersheds. These tools must be used with caution in urban areas and all subbasins shall 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 because of 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. In urban areas query the local stormwater management agency for their infrastructure maps. Some communities have data available in shapefile format or as PDFs; others may have a web-based parcel mapping tool that includes stormwater. Field verification of how the impervious areas and pervious areas are connected or disconnected to the flow paths may be required.

## 2-5 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: Section 2-5.1 focuses on calculating the impacts of snowmelt and Section 2-5.2 provides additional considerations for PEOs when evaluating the impacts of snowmelt in a project location.

### 2-5.1 *Calculating Snowmelt*

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 snowmelt impacts. The PEO shall apply the method described in this section and consult the RHE, the local Maintenance Office, the local 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 shall consider is whether snowmelt effects will impact a project. In particular, PEOs shall 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 State Hydraulics Office:

- Washington Climate Summaries
- Washington Snow Map

The following equation uses a factor of 5, developed from the energy budget equation by the United States Army Corps of Engineers (USACE), and available snow for eastern Washington cities to convert depth of snow to snow water equivalent. This amount, up to 1.5 inches, 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 shall be applied only when the average daily snow depth within the month at a project location meets or exceeds 2 inches:

$$Snow/Water\,equivalent = \frac{Average\ snow\ depth\ (maximum\ per\ month\ [inches/day])}{5} \qquad (2\text{-}2)$$

The snow/water equivalent added shall not be greater than 1.5 inches regardless of the results.

### 2-5.2 *Additional Considerations*

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

- **Roadside drainage:** During the design phase, consideration shall 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 shall be given to an alternate course of travel for runoff. This will help prevent the water ponding that sometimes occurs in certain areas because of snowmelt and rain not having an open area in which to drain off the roadway. This may require coordination with the WSDOT Maintenance Office.
- **Retention ponds:** When detention or retention ponds are located near the roadway, the emergency spillway shall be located outside of any snow storage areas that could block overflow passage, or an alternative flow route shall be designated. This may require coordination with the WSDOT Maintenance Office.

- **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 historical gage records. No corrections or adjustments need to be made to these hydrology methods for frozen ground or snowmelt. For smaller basins, the SBUH Method and Rational Method are 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 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.

## 2-6 Rational Method

This section presents a description of the Rational Method.

### 2-6.1 *General*

The Rational Method is used to estimate 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 2-3 and 2-4); 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, because the USGS regression equations will yield a more accurate flow estimate for that size of basin. The formula for the Rational Method is as follows:

$$Q = \frac{CIA}{K_C} \tag{2-3}$$

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 units

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

$$Q = \frac{I\Sigma CA}{K_C} \quad (2\text{-}4)$$

where:

$$\Sigma CA = C_1 x A_1 + C_2 x A_2 + \cdots + C_n x A_n$$

Hydrologic information calculated by the Rational Method shall be submitted as a calculation package within the hydraulic report using the spreadsheet found on WSDOT's hydraulics and hydrology webpage under tools, templates, and links or other similar forms approved by the State Hydraulics Office that best describe the project's hydraulic information.

This spreadsheet contains all the required input information and the resulting discharge. The description of each area shall 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.

### 2-6.2 *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 shall review Section 2-5.2, third bullet. In a high growth rate area, runoff factors shall 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 Table 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.

**Table 2-2** Runoff Coefficients for the Rational Method: 10-Year Return Frequency

| Cover Type | Flat | Rolling (2%–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 |

## 2-6.3 *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 water takes 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 shall be used for designing is the highest intensity that will occur with the entire basin contributing runoff to the flow rate location being studied. This may be a much lower intensity than the maximum intensity because of 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, which was developed by the Natural Resources Conservation Service (NRCS, formerly known as the Soil Conservation Service [SCS]), is described below. It is sensitive to slope, type of ground cover, and channel size. If the total time of concentration is less than 5 minutes, a minimum of 5 minutes shall be used as the duration (see Section 2-6.4 for details). Table 2-3 lists ground cover coefficients.

The time of concentration can be calculated as in Equations 2-5 and 2-6:

$$T_t = \frac{L}{K\sqrt{S}} = \frac{L^{1.5}}{K\sqrt{\Delta H}} \tag{2-5}$$

$$T_C = T_{t1} + T_{t2} + \cdots + T_{tn}$$

where:

$T_t$ = travel time of flow segment in minutes (2-6)
$T_c$ = time of concentration in minutes
L = length of segment in feet
ΔH = elevation change across segment in feet
K = ground cover coefficient in feet/minute
S = slope of segment ${}^{\Delta H}/_{\Delta L}$ in feet

**Table 2-3** Ground Cover Coefficients

| Type of Cover | Flow depth (inches) | K (feet/min.) |
|---|---|---|
| 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[a] | 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
a = these values are for RCP, coefficient must be adjusted for different materials
w/d = width/depth ratio

## 2-6.4 *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. Rainfall intensity is the average of the most intense period enveloped by the time of concentration and is not instantaneous rainfall. Rainfall intensity, duration, and frequency (IDF) curves can be used to estimate rainfall intensity. Regional IDF curves are available from the National Oceanic and Atmospheric Administration (NOAA). Curves for Washington State can be found on NOAA's Precipitation Frequency Data Server.

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:

- 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.

- 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 2-7 calculates rainfall intensity.

$$I = \frac{m}{(T_c)^n} \qquad (2\text{-}7)$$

where:

I = rainfall intensity in inches per hour
$T_c$ = time of concentration in minutes
m and n = coefficients in dimensionless units (Table 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 Table 2-4 are accurate from 5-minute durations to 1,440-minute durations (24 hours).

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

**Table 2-4** Index 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 | 3.41 | 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 |

# 2-7 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 shall not be used where groundwater flow can be a major contributor to the total flow.

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

Several commercially available computer programs include the SBUH Method. See Chapter 1.

## 2-7.1 *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 time of 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 2-8 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 using an isopluvial map that clearly identifies the location within the map contours (see Figure 2-1).

## 2-7.2 *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 shall 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 shall 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 can be made more accurate.

### 2-7.2.1 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,* 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 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 shall 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 shall 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.

#### 2-7.2.2 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 as described below:

- **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 5 days, and soil is near saturated or saturated

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

**Table 2-5** Total 5-Day Antecedent Rainfall

| Antecedent Moisture Condition | Dormant Season (inches) | Growing Season (inches) |
|---|---|---|
| I | Less than 0.5 | Less than 1.4 |
| II | 0.5–1.1 | 1.4–2.1 |
| 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 shall 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).

#### 2-7.2.3 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 water takes 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 2-6.3, 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 ($n_s$, which is a modified Manning's roughness coefficient) is used. These $n_s$ 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 2-8) to directly compute $T_t$:

$$T_t = (0.42\,(n_s L)^{0.8})/((P2)^{0.527}(S_o)^{0.4}) \qquad (2\text{-}8)$$

where:

$T_t$ = travel time (minutes)

$n_s$ = sheet flow Manning's coefficient

$L$ = flow length (feet)

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

$S_o$ = slope of hydraulic grade line (land slope, feet vertical/1 foot horizontal [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 2-9), 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 indicate that 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 bankfull 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:

$$V = (k)(S_o^{0.5}) \qquad (2\text{-}9)$$

where:
$V$ = velocity (feet per second $[ft/s]$)
$k$ = time of concentration velocity factor
$S_o$ = slope of flow path $(ft/ft)$

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

- Manning's kinematic solution shall not be used for sheet flow longer than 300 feet.
- The equations given here to calculate velocity were developed by empirical means; therefore, English units must be used for all input variables for the equation to yield a correct result. 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*.

## 2-8 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.

MGSFlood does have limitations that the PEO should understand before using the program, regarding the project location, conveyance design, and 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 shall be limited to areas where snowmelt is not usually a major contributor to floods or to the annual runoff volume. MGSFlood is not used for 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 State Hydraulics Office staff for assistance.

## 2-8.1 *Modeling Requirements*

MGSFlood shall 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 is further described in the sections below.

### 2-8.1.1 Precipitation Input

Two methods for transposing precipitation time series 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:

**Extended precipitation time series selection:** Uses a family of prescaled precipitation and evaporation time series. Extended precipitation time series regions (Figure 2-1) 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 shall be used for stormwater facility design for project sites.

**Precipitation station selection:** For project sites located outside the extended time series region, a second precipitation scaling method is used. The precipitation station selection outside extended precipitation time series regions is when a source gage is selected (Figure 2-2), and a single scaling factor is applied to transpose the hourly record from the source gage 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 2019), 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 State Hydraulics Office staff if assistance is needed in selecting the appropriate gage.

**Figure 2-1** Extended Precipitation Time Series Regions

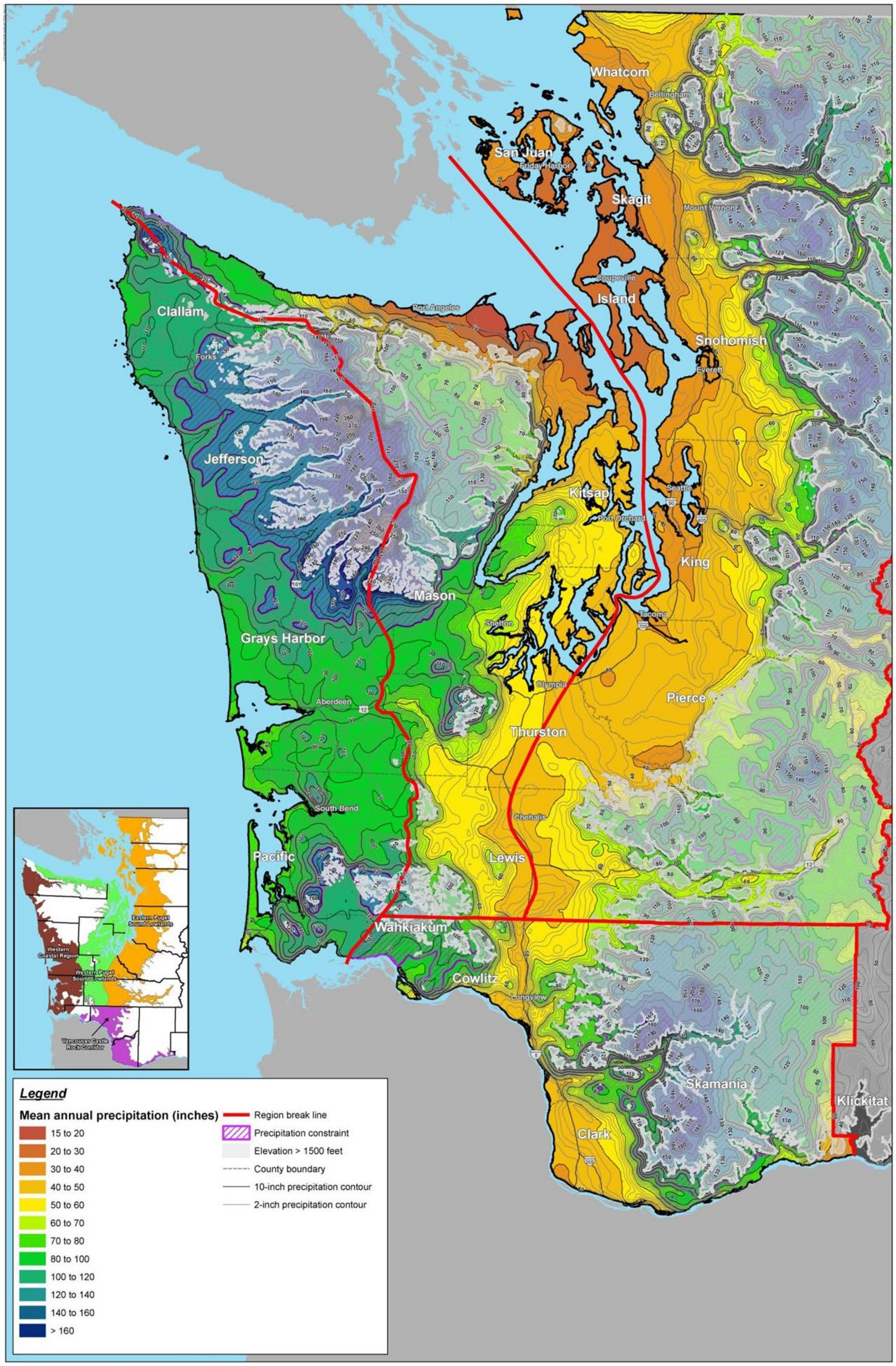
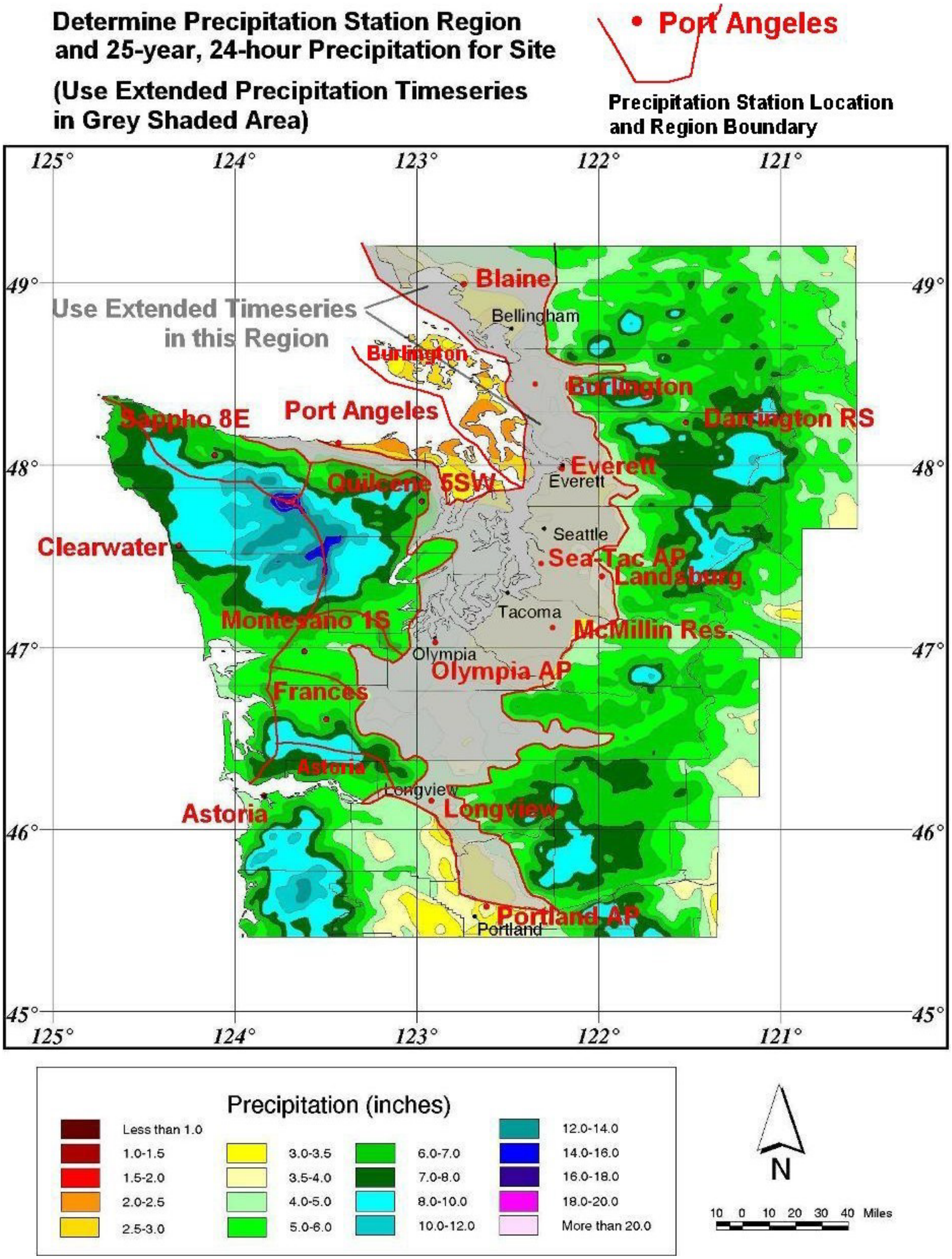


**Figure 2-2** Precipitation Station Selection outside Extended Precipitation Time Series Regions



#### 2-8.1.2 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 USGS). Mapping of soil types by NRCS is the most common source of soil/geologic information used in hydrologic analyses for stormwater facility design. Each soil type defined by 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 Table 2-6.

**Table 2-6** 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 Type B soils shall be classified as outwash. It is important to work with the HQ Materials Laboratory or a licensed geotechnical engineer to confirm that 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 given when including wetlands in continuous-simulation runoff modeling.

MGSFlood v4.56 and later uses the default HSPF parameters from Ecology's Western Washington Hydrology Model (WWHM), which includes slope groups. These are the default when MGSFlood opens and are labeled "Ecology Default" on the Parameter screen assessed from the Tools tab. The original MGSFlood default parameters are labeled "USGS Default." Projects created in the older versions of MGSFlood automatically open using the original parameters. Be sure to select "USGS Default" when using MGSFlood for stormwater system design per the *Highway Runoff Manual*.

## 2-9 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 stream flows are measured at a gaging site for several years. A statistical analysis, using the USGS Regression Peak FQ, is then performed on the measured flows to estimate the recurrence interval flows. USGS, Ecology, local and state municipalities, and several utility companies work together to maintain gaging sites throughout Washington State. Flood discharges for these gaging sites can be found in the following websites:

- StreamStats
- https://pubs.er.usgs.gov/publication/sir20165118
- Freshwater DataStream data map
- https://waterdata.usgs.gov/state/washington/

## 2-10 USGS Regression Equations

While measured flows provide the best data for design purposes, it is not practical to gage all rivers and streams in the state. USGS has developed a set of regression equations to calculate flows for drainage basins in the absence of a stream flow gage. The equations were developed by performing a regression analysis on stream flow gage records to determine which drainage basin parameters are most influential in determining peak runoff rates.

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 regression analysis divided the state into four 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 and percentage of the drainage basin that is in forest cover. The PEO and Stream Team can determine these variables through use of site maps, aerial photographs, and site inspections.

The PEO and Stream Team must be aware of the limitations of these equations. They were developed for natural rural basins. The equations can be used in urban ungaged areas with additional backup data (i.e., comparing results to the nearest gage data for calibration and sensitivity analysis, field inspection of high-water lines, and information from local maintenance). PEOs and Stream Team shall contact the State Hydraulics Office for further guidance. Also, any river that has a dam and reservoir in it shall not be analyzed with these equations. Finally, the PEO and Stream Team must keep in mind that, because of 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 PEOs and Stream Team shall use the mean value determined from the regression equations with no standard error or confidence interval. The PEO shall validate the calculated flow rate based on collected field data and site conditions. If the flows are too low or too high for that basin based on information that the PEO and Stream Team has collected, then the PEO and Stream Team may apply the standard error specific to the regression equation accordingly. The PEO and Stream Team shall consult the RHE or State Hydraulics Office for assistance.

StreamStats is another USGS tool that not only estimates peak flows but also can delineate the basin area and determine the mean annual precipitation as well as other basin characteristics.

## 2-11 Existing Hydrologic Studies

Existing hydrologic studies have been developed for many rivers in Washington State. FEMA has developed most of these studies. USACE and local agencies have developed other reports.

Many small and medium 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.

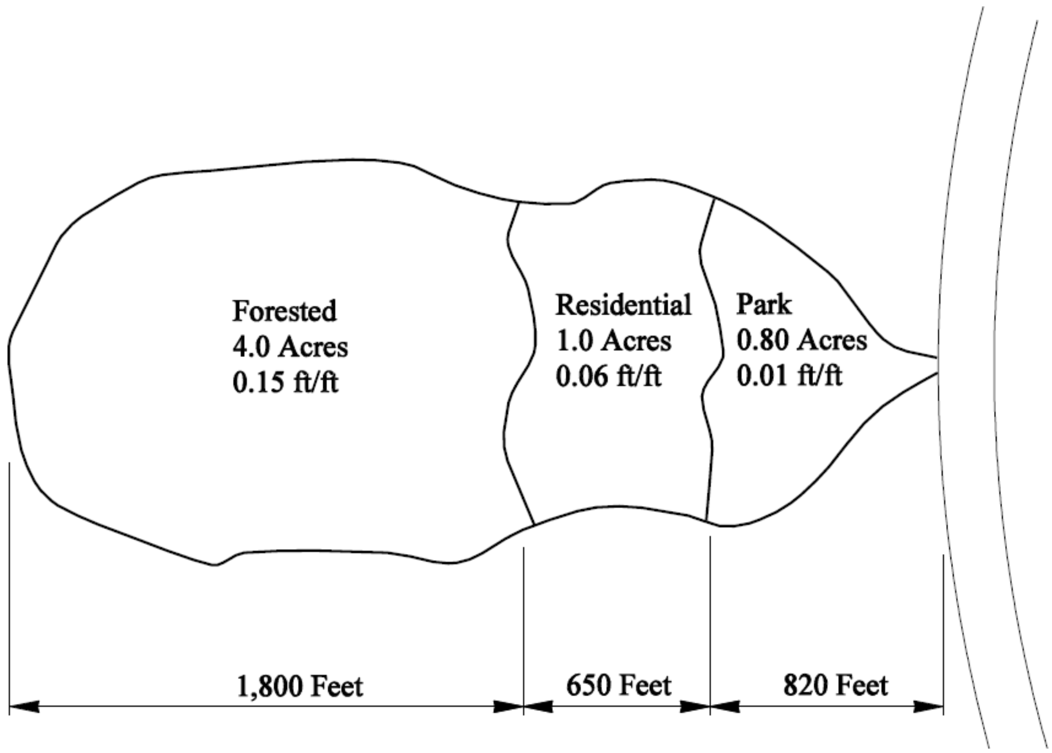
These reports are a good source of flow information because they were developed to analyze the flows during flooding conditions of a particular river or stream. The types of calculations used by the agency conducting the analysis may be more complex than the Rational Method or USGS regression equations. 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 existing hydrologic studies are available on the FEMA map service center website. The State Hydraulics Office shall be contacted for local agency reports. The State Hydraulics Office may also have basin planning documents or action plans that could contain flow rate information. These studies shall be used with caution as they may have been developed for a different purpose or may be outdated and therefore may not be transferable/applicable for the design of transportation infrastructure.

## 2-12 Examples

Compute the 25-year runoff for the Spokane watershed shown in Figure 2-3. 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 foot vertical per 1 foot horizontal (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 2-3** Rational Formula Example




$$T_c = \Sigma \frac{L}{K\sqrt{S}} = \frac{1800}{150\sqrt{0.15}} + \frac{650}{420\sqrt{0.06}} + \frac{820}{3900\sqrt{0.01}}$$

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

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

$$\Sigma \mathrm{CA} = 0.22(4.0\ acres) + 0.44(1.0\ acre) + 0.11(0.8\ acre) = 1.4\ \mathrm{acres}$$

$$Q = \frac{I(\Sigma CA)}{K_c} = \frac{(0.93)(1.4)}{1} = 1.31\ cfs$$

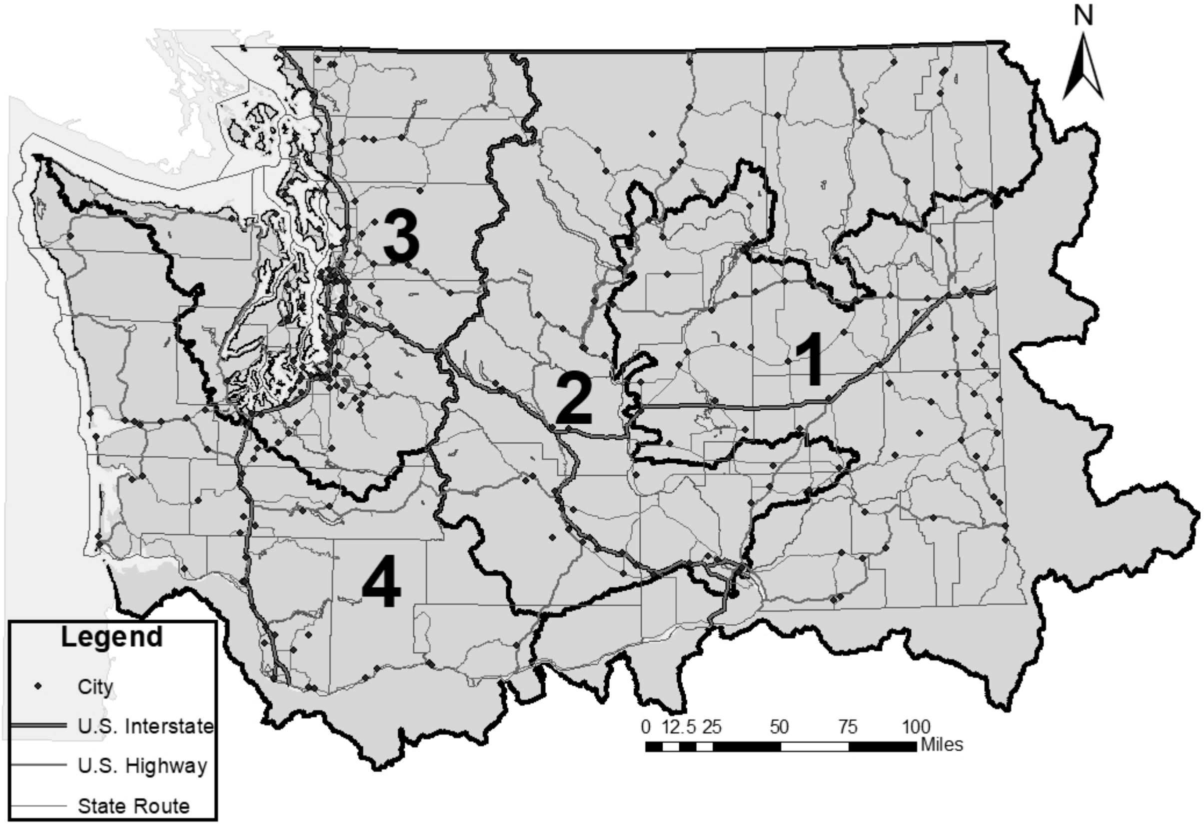
## 2-13 Appendices

| | |
|---|---|
| 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 mean annual precipitation maps for Washington are available in PDF format on WSDOT's hydraulics and hydrology webpage under tools, templates, and links or by using GIS Workbench. Contact your local GIS group for how to extract digital precipitation data using ArcMap.

# Appendix 2B USGS Regression Equation Zone Map



# 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 shall be taken into consideration. These factors can include the economy of alternative pipe materials and sizes, horizontal (H) and vertical (V) 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 from what 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 and require a Stream Team (defined in Chapter 7-1) approved by the State Hydraulics Office to complete the design.

The design policy in this chapter applies only to culverts with 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

This section describes culvert design documentation, including hydraulic reports, required field data, and engineering analysis.

### 3-2.1 *Hydraulic Design Reports*

The PEO shall collect field data and perform an engineering analysis as described in Sections 3-2.2 and 3-2.3, respectively. 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 alternatives based on site conditions. The decision regarding which type of pipe material is 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 alternatives.

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 State Hydraulics Office or a licensed engineer approved by the State Hydraulics Office, as outlined in Chapter 1.

In addition to standard culvert design, the State Hydraulics Office can assist in the design of any unique culvert installation. The requirements for these structures will vary, and the State Hydraulics Office shall 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:

- Topographic map showing the contours and the outline of the drainage area
- Description of drainage area ground cover
- Fish passage requirement, if applicable; see Chapter 7
- Soils investigation per WSDOT's *Design Manual*
- Proposed roadway profile and alignment in the vicinity of the culvert
- Proposed roadway cross section at the culvert
- Corrosion zone location, pH, and resistivity of the site
- 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 and anticipated sediment transport
- Other considerations discussed in Section 3-5

If an existing culvert 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. Table 3-1 is a general outline showing the information and field data requirements for a hydraulic report and specialty report.

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

**Table 3-1** Field Data Requirements for Hydraulic Reports and Specialty Reports

| Information and Field Data | New Culvert Site | Extending or Replacing | Specialty Report |
|---|---|---|---|
| Topographic survey | R | O | R |
| Ground cover description | R | O | R |
| Ground soil investigation | R | O | R |
| Proposed roadway profile and alignment | R | O | R |
| Proposed roadway cross section | R | O | R |
| Corrosion zone, pH, resistivity[a] | R[a] | O[a] | R[a] |
| Unique features | R | O | R |

***Notes:***

O = optional.

R = required.

a. Required only 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 1 inch on either side of the culvert. The PEO shall contact the State Hydraulics Office 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 the following information:

1. Culvert hydrology and hydraulic calculations, as described in Section 3-3 and Table 3-2.
2. Proposed roadway stationing of the culvert location.
3. Culvert length.
4. Culvert diameter. The minimum diameter of culvert pipes under a main roadway shall be 18 inches. Culvert pipe under roadway approaches (i.e., driveway) shall have a minimum diameter of 12 inches.
5. Culvert material.
6. Headwater depths, WSELs, 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 Table 3-2). For a more detailed diagnostic about what is required for a specialty report for water crossings, see Chapter 7.

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

| Engineering Analysis Item | New Culvert Site | Extending or Replacing | Specialty Report |
|---|---|---|---|
| Culvert hydraulic and hydrology calculations | R | O | R |
| Roadway stationing at culvert | R | R | R |
| Culvert and stream profile | R | O | R |
| Culvert length and size | R | R | R |
| Culvert material | R | R | R |
| Hydraulic details | R | O | R |
| Proposed roadway details | R | O | R |
| End treatment | R | R | R |
| Hydraulic features | R | O | R |
| Additional fill material added | R | R | 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 FHWA's Hydraulic Design Series (HDS) 5, *Hydraulic Design of Highway Culverts*. The State Hydraulics Office 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 measured along the centerline of the roadway is summarized in the steps below. Culvert spans more than 20 feet wide measured along the centerline of the roadway are considered bridges and any hydraulic design for bridges is the responsibility of the State Hydraulics Office; 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*

This section presents 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 State Hydraulics Office (see Chapter 7). Guidelines for temporary culverts are described further in Section 3-3.1.9. 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 Structures over 20 Feet

Once a structure exceeds 20 feet along the centerline of the roadway, it is defined as a bridge and all hydraulic analyses on bridges are the responsibility of the State Hydraulics Office (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. (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 State Hydraulics Office or a hydraulics engineer approved by the State Hydraulics Office; 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, and the WSDOT *Design Manual*, Chapter 800 discusses when the PEO and hydraulics engineer need to coordinate.

### 3-3.1.3 Alignment and Grade

Culverts shall 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 upstream 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 State Hydraulics Office for design assistance.

#### 3-3.1.5 Minimum Spacing

The use of multiple culvert openings is not allowed.

#### 3-3.1.6 Culvert Extension

Culvert extensions shall be done in-kind—using 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. The PEO shall follow the manufacturer's recommendations for joining pipe. For situations not listed, contact the RHE.

- Culvert pipe connections for dissimilar materials, when approved by the RHE, must follow Standard Plan B-60.20-02 of WSDOT's *Standard Plans*.
- 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, AASHTO M 259, M 273, and Standard Specification 6-02.3(28).

#### 3-3.1.7 Minimum Culvert Diameter

The minimum diameter of a culvert under a main roadway must be 18 inches. Culvert pipe under roadway approaches must have a minimum diameter of 12 inches. If replacing an existing culvert, the new culvert shall have at least the same or larger diameter as the existing culvert even if the hydraulic analysis shows that a smaller-diameter culvert would meet hydraulic design requirements in that location.

#### 3-3.1.8 Culvert Pipe at Walls and Foundations

Culvert pipes in the reinforcement zone of walls or the soil-bearing zone of foundations shall be coordinated with the geotechnical engineer.

#### 3-3.1.9 Temporary Diversions

Temporary diversions for non-fish-bearing streams or drainages that are 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 State Hydraulics Office or RHE approval. The design storm for multiple-season construction projects shall be a risk-based decision and shall be determined by the PEO in coordination with the RHE.

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 State Hydraulics Office calculates the flow rates necessary for temporary diversions and that value is part of the contract documents. A conceptual-level plan is required for permits, but no plans for the temporary diversion system shall be put into the final plan set and shall not be documented in the specialty report, unless otherwise approved.

Temporary diversions for fish-bearing streams shall be designed for the following storm events:

- **Single season:** For a temporary diversion 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 when the temporary diversion is in place with a contingency plan that shall be in place within 2 hours or less to bring the system to meet the expected 10 percent exceedance flow rate during the window when the temporary diversion is in place. The expected flow rates during the window when the temporary diversion is in place can be determined 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 are 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 State Hydraulics Office.
- **Multiple season:** A gravity bypass is required if the stream diversion is expected to remain in place over the winter; pump bypasses will not be allowed. The culvert shall be the lesser of the size required to pass the 25-year flow event or that required to meet the existing culvert capacity. The length of the stream bypass contained within a culvert shall not be longer than the existing culvert unless otherwise approved by the State Hydraulics Office. Fish passage shall not be decreased from the existing conditions as evaluated by the *Fish Passage Inventory, Assessment, and Prioritization Manual.*

The design flood for temporary structures over water bodies shall be determined by the State Hydraulics Office.

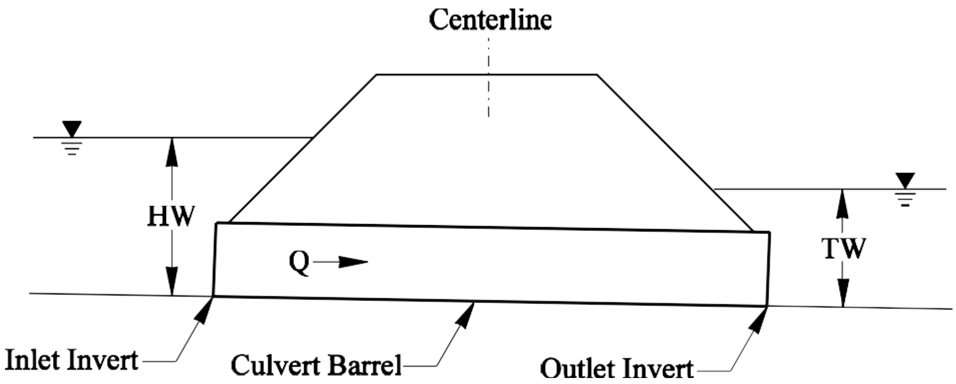
## 3-3.2 *Allowable Headwater*

This section presents hydraulic design criteria for allowable headwater for circular and box culverts, pipe arches, and bottomless culverts.

### 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 3-1. See the Main Glossary of Terms for definitions.

**Figure 3-1** Headwater and Tailwater Diagram




Limiting the amount of headwater during a design flow can be beneficial for several reasons. The potential for debris clogging reduces 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 hazard area, the floodway and floodplain requirements for that local jurisdiction may govern the allowable amount of headwater. In this situation, the PEO shall contact the State Hydraulics Office for additional guidance. Additional information is included in Section 4-7.

#### 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 (HW/D <1.25). HW/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 HW/D ratio of 1.25 must be discussed with the State Hydraulics Office and, if approved by the RHE, included as a narrative in the hydraulic report.

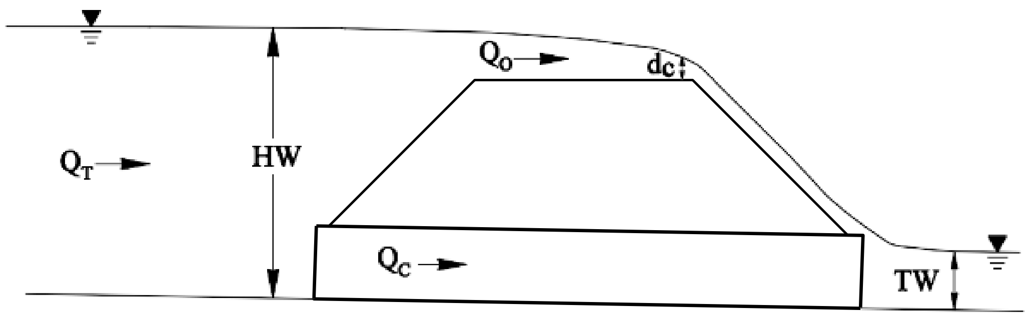
The headwater that occurs during the 100-year flow event must also be investigated. Two sets of criteria exist for the allowable headwater during the 100-year flow event, depending on the type of roadway over the culvert:

1. If the culvert is under an interstate or major state route that must be kept open during major flood events, the culvert shall 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, the culvert shall 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 because of 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 3-2. 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 State Hydraulics Office shall be contacted for further design guidance. Additionally, the PEO shall verify that the adjacent ditch does not overtop and transport runoff, causing damage to either public or private infrastructure.

**Figure 3-2** Roadway Overtopping

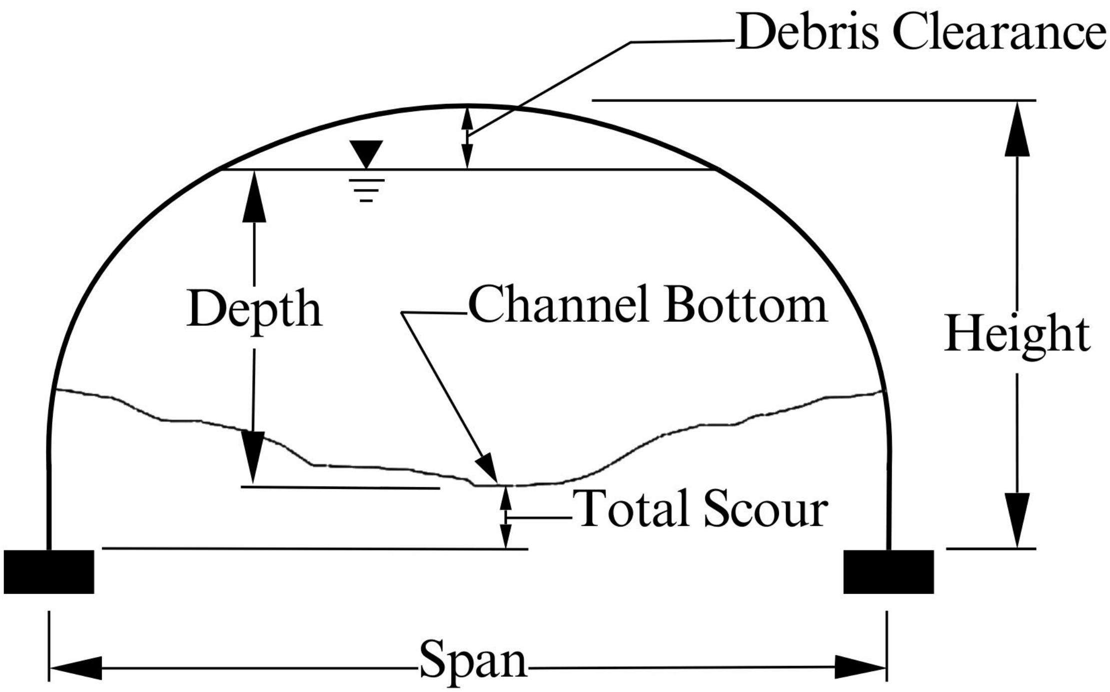



### 3-3.2.3 Allowable Headwater for Bottomless Culverts

Bottomless culverts with footings shall be designed such that 1 foot of debris clearance from the water surface to the culvert crown is provided during the 25-year flow event (see Figure 3-3). In many instances, bottomless culverts function similarly to bridges. They usually 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 1-foot criterion will alleviate many of these potential problems. Bottomless culverts shall also be designed such that the 100-year flow 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. A scour analysis shall be conducted for the footing.

**Figure 3-3** 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 3-1 above. 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 HDS-5. Generally, one of three conditions will exist downstream of the culvert and the tailwater can be determined as described below:

- 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.
- 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.
- 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 State Hydraulics Office 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 feet per second (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 because 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, must be investigated when it becomes necessary to modify the outlet velocity. Velocities less than 3 ft/s shall require a deviation from the State Hydraulics Office, thus needing approval from the RHE. Velocities more than 10 ft/s must be discussed with the RHE for potential solutions and final design exception approval by the RHE.

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. The PEO must consult with the RHE to determine the appropriate culvert size for this application.

If velocities exceed about 10 ft/s, 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.

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

### 3-3.6 *Culvert Hydraulic Calculations Form*

Approval from RHE is required when using 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 shall use one of several commercially available culvert design software programs. FHWA has developed a culvert design program named HY-8 that uses 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 free software distribution. It is available by contacting either the RHE or the State Hydraulics Office at the following link.

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 types of end treatments are 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.

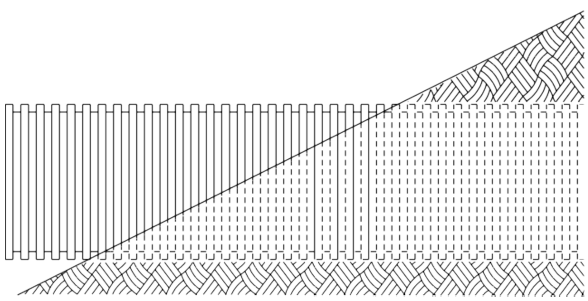
## 3-4.1 *Projecting Ends*

A projecting end is a treatment where the culvert is allowed to protrude out of the embankment (see Figure 3-4). 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 because the pipe consists of a complete ring structure out to the culvert end.

Projecting ends have several disadvantages. 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 shall be used only 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 3-4** Projecting End



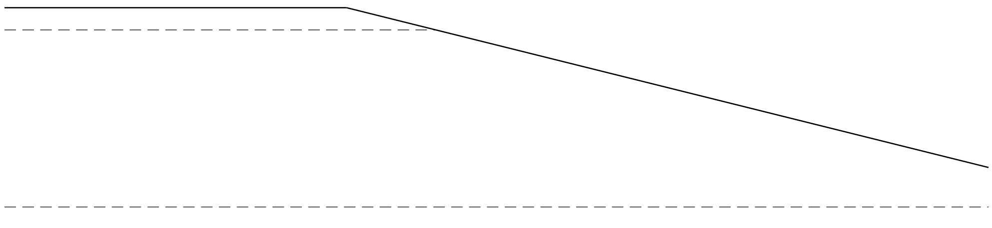
Metal culverts exceeding 6 feet in diameter but less than 10 feet in diameter, and all thermoplastic culverts, must be installed with a beveled end and a concrete headwall or slope collar as described in Sections 3-4.2 and 3-4.4. Concrete pipe will not experience buoyancy problems and can be projected 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 *Mitered End Sections*

A mitered 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 because of 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 3-5. 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.

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 State Hydraulics Office can assist in the design of special end treatments.

**Figure 3-5** 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 because they serve to blend the culvert end into the finished embankment slope.

Flared end sections are used only on circular pipe or pipe arches. The acceptable size ranges for flared ends and other details are shown on Standard Plan B-70.60-01 for Flared End Sections. Flared ends are generally constructed out of steel and aluminum and shall 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.

A flared end section is usually the most feasible option in smaller pipe sizes and shall 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 flared end sections.

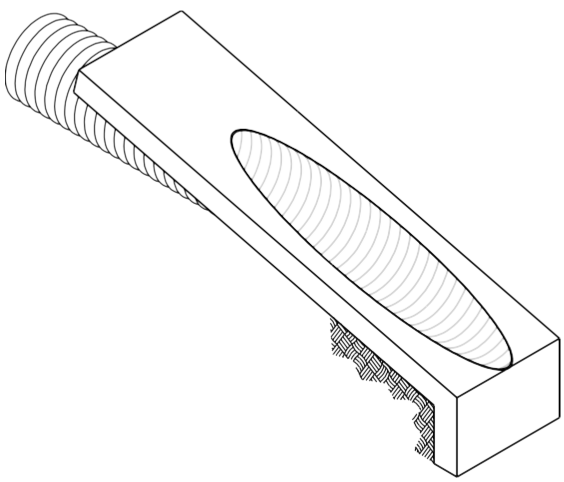
The undesirable safety properties of flared end sections generally prohibit their use in the clear zone for all but the smallest diameters (see WSDOT's *Design Manual* for culvert design). A flared end section is made of light-gage metal and, because of the overall width of the structure, it is not possible 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*

A headwall is a concrete frame poured around a beveled culvert end. It provides structural support to the culvert, eliminates the tendency for buoyancy and provides inlet and outlet protection. A headwall is a required end treatment for all culverts that range in size from 4 to 10 feet. Contact the RHE for direction on headwalls required for culverts smaller than 4 feet. Headwalls shall be used on all thermoplastic culverts, 30 inches in diameter and larger. A typical headwall is shown on *Standard Plans* B-75.20-03 or in Figure 3-6. 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 alternative solution to safety bars, such as increasing the culvert size or providing guardrail protection around the culvert end.

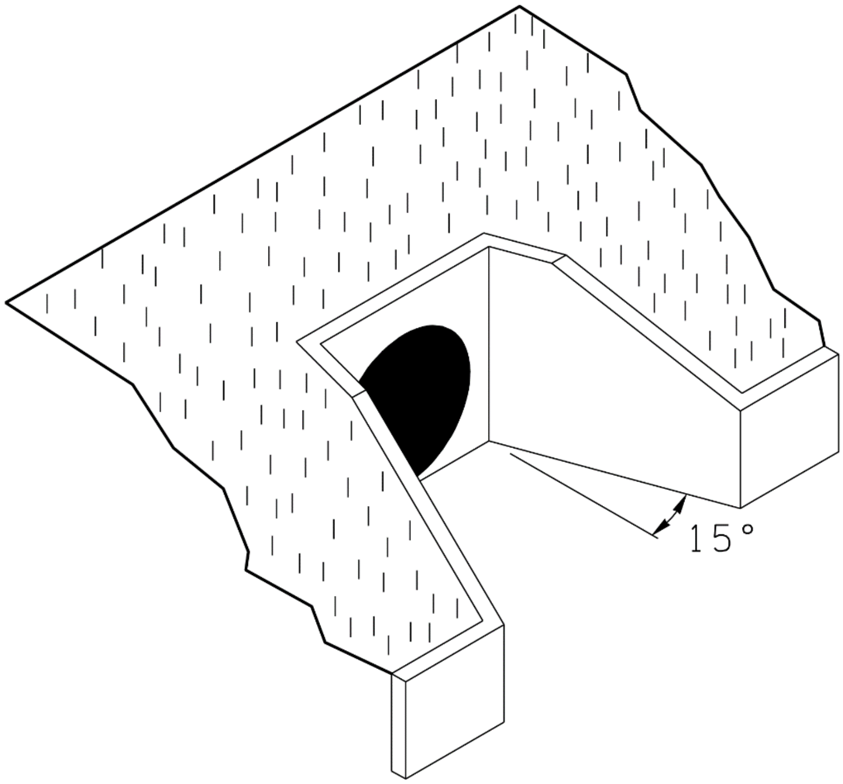
**Figure 3-6** Headwall



### 3-4.5 *Wing Walls and Aprons*

Buried structures greater than 10 feet long require wing walls. Wing walls and aprons are required with reinforced concrete box culverts and other types of buried structures. Wing walls shall be designed in accordance with Section 8 of the *Bridge Design Manual*. In lieu of using wing walls, box culvert extensions may be acceptable if site conditions are suitable and the State Hydraulics Office approves. Wing walls may also be modified for use on circular culverts in areas of severe scour problems (Figure 3-7). 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 State Hydraulics Office can assist in this design.

**Figure 3-7** 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. Contact the RHE for guidance when considering using 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 (more than 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 RHE shall be contacted. Also, HDS-5 contains a significant amount of information related to the design of improved inlets.

### 3-4.7 Energy Dissipators

The PEO shall use an energy dissipator for all outlets . Energy dissipators can be quite simple or very complex, depending on site conditions. Debris and maintenance problems shall be considered when designing energy dissipators.

Energy dissipators include:

- Rock-protected outlets

Rock is frequently placed around the outlet end of culverts to protect against the erosive action of the water (Figure 3-8). The material size at the outlet is dependent on the outlet velocity as determined using a full flow analysis as noted in Table 3-3. The limits of this protection would cover an area that would be vulnerable to scour holes. As an alternative to using Figure 3-8 and Table 3-3, the Hydraulic Toolbox calculator, which can be downloaded from FHWA's website, can be used to determine the area of the scour protection and the size of the rock. A granular filter or geotextile must be placed between rock and ground (see Figure 3-8). Section 4-6.2.2 provides guidance for selection of filter type and required calculations. The calculation results need to be included in the hydraulic report. (See Section 3-4.5 for details on wing walls and aprons.)

**Figure 3-8** Rock-Protected Outlet

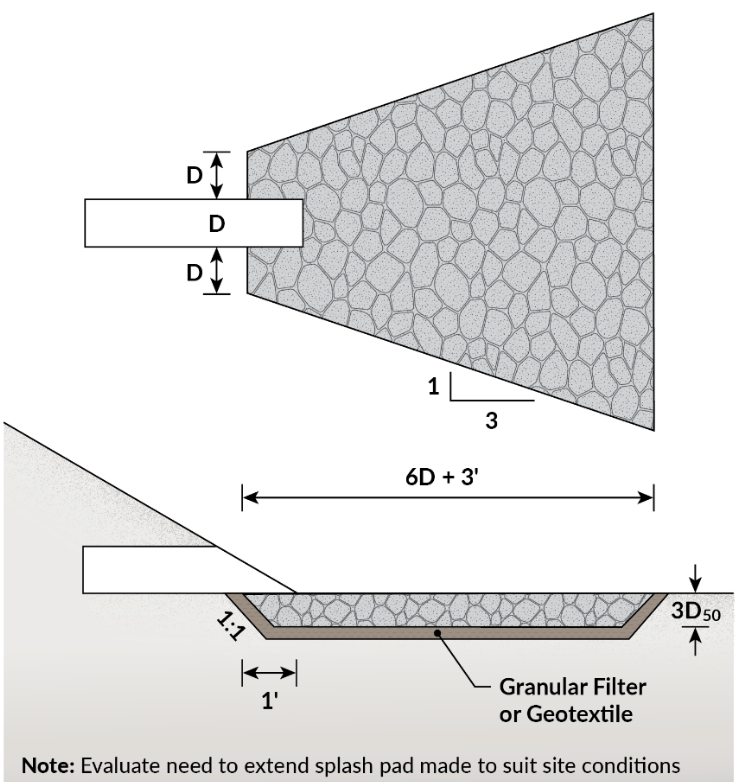


**Table 3-3** Outlet Protection Material Size

| Outlet Velocity (ft/s) | Material |
|---|---|
| Up to 7 | Quarry spalls |
| 7–10 | Rock for erosion and scour protection (RESP) Class A |
| 10–15 | RESP Class B |
| >15 | RESP Class C |

***Note:***

The outlet velocities are based on full flow calculations. The PEO shall provide a filter such as geotextile or a granular filter between the rock protection and the existing ground. The gradation of the existing ground or base soil should be known to size the filter. See Section 4-6.2 for guidance on selection of filter type and required calculations.

- 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. The State Hydraulics Office shall 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 shall 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. The PEO must consult with the RHE for guidance on potential culvert debris issues.

## 3-5 Miscellaneous Culvert Design Considerations

This section presents miscellaneous culvert design considerations, including multiple culvert openings, camber, horizontal and vertical angle points, upstream ponding, and siphons.

### 3-5.1 *Multiple Culvert Openings*

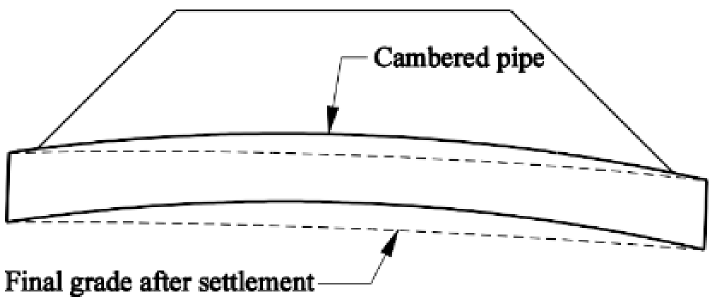
The use of multiple culvert openings is not allowed for a single water crossing.

### 3-5.2 *Camber*

When a culvert is installed under moderate to high fills 30 to 60 feet or higher, greater settlement of the fill may occur under the center of the roadway than at the sides. This occurs because at the culvert ends there is little fill while 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 3-9. 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 3-9** Camber under High Fills



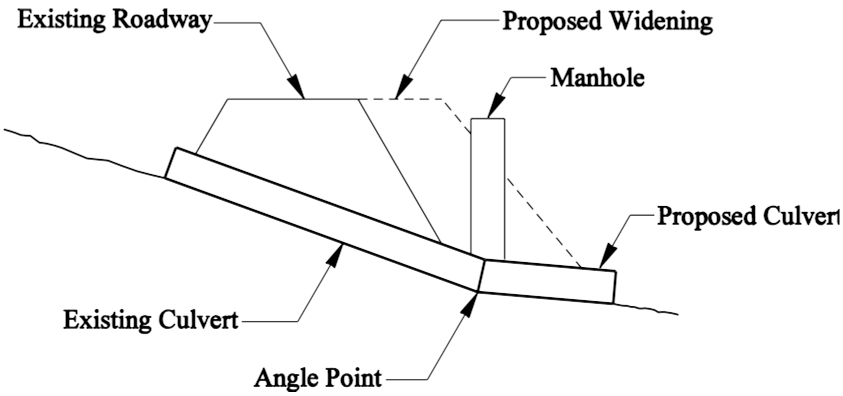

### 3-5.3 *Horizontal and Vertical Angle Points*

The slope of a culvert shall 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, a manhole shall be incorporated into the design at the angle point, as shown in Figure 3-10. The PEO shall contact the RHE regarding the incorporation of a manhole. 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 3-10** 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 using flood-routing concepts similar to designing a stormwater detention pond, but that methodology is beyond the scope of the *Hydraulics Manual*. Because the need for this type of culvert design is rare, the RHE shall be contacted for further assistance.

### 3-5.5 *Miscellaneous Design Considerations: Siphons*

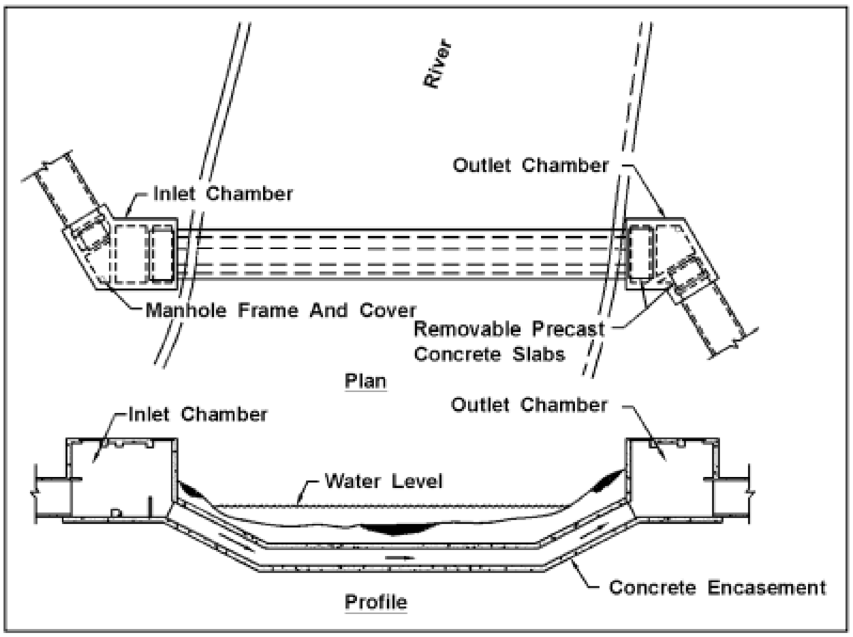
Siphon designs require review and concurrence by the State Hydraulics Office per Table 1-1. Also, the siphon design may need to be reviewed and approved by the owner of the features being crossed. A siphon carries the flow under an obstruction such as a depressed railroad, roadway, stream, sanitary sewer, water main, or any other structure or utility line that is in the path of the storm drain line. The storm drain invert is lowered at the obstacle and is raised again after the crossing. The siphon will remain full when there is no flow. AASHTO recommends a minimum of two barrels with 3 ft/s velocity. One of the barrels is designed to have a weir-type obstruction placed at the inlet and outlet structures to keep the normal flow in one barrel to provide the required minimum velocity for self-cleaning and servicing. The elevation of the weir crests is based on the depth of normal flows in the upstream storm drain. Maintenance access is to be provided at both the inlet and outlet chambers. Figure 3-11 illustrates a typical twin-barrel inverted siphon.

The following considerations from HEC-22, Chapter 6 (1) are important to the efficient design of siphons:

- Self-flushing velocities shall be provided under a wide range of flows
- Hydraulic losses shall be minimized
- Provisions for cleaning shall be provided
- Sharp bends shall be avoided
- The rising portion of the siphon shall not be so steep as to make it difficult to flush deposits (some agencies limit the rising slope to 15 percent)
- There shall be no change in pipe diameter along the length of the siphon
- Provisions for drainage shall be considered

Additional information related to the design of siphons is provided in HEC-22 (1) and United States Bureau of Reclamation (USBR) Design of Small Canal Structures (6), which includes a design example.

**Figure 3-11** Typical Twin-Barrel Inverted Siphon



# Chapter 4 Channels and Floodplains

## 4-1 Introduction

Channels and floodplains are runoff systems that include streams, rivers, ditches, and swales. Built extensions or modifications to these systems are included in this chapter.

Proper design requires sufficient hydraulic capacity to convey the flow of the design storm. All flow assessments require a hydrologic analysis with procedures and methodologies presented in Chapter 2. In the case of earth-lined channels or river channels, bank protection may also be required if the shear stress is high enough to cause erosion or scouring.

This chapter provides guidance for determining design velocity (Section 4-2) and critical depth (Section 4-4) for designing roadside ditches (Section 5-5), stormwater systems, swales, and roadway gutters. All other transportation hydraulic features require the use of a 2D hydraulic model; FHWA has developed a reference document for 2D hydraulic models, titled *Two-Dimensional Hydraulic Modeling for Highways in the River Environment* (FHWA 2019).

SRH-2D hydraulic modeling training is required for all WSDOT projects or WSDOT-managed infrastructure that requires hydraulic modeling as part of the hydraulic design process. Hydraulic modelers are required to obtain a training certificate from NHI for attending Course 135095, *Two-Dimensional Hydraulic Modeling of Rivers at Highway Encroachments*. Other equivalent SRH-2D hydraulic modeling training requires approval by the State Hydraulics Office.

Countermeasures for stream instability (Section 4-6) may be necessary for highly erosive, high-energy stream and river channels, to help stabilize the banks and/or channel bottom. The success of stabilization measures is dependent on the ability of the methods and materials used to withstand the hydraulic forces. For example, it is important to properly size the rock materials used for armoring; the methodology for sizing rock materials used in river stabilization is described in HEC-23, Volume 1 and Volume 2.

## 4-2 Uniform Flow Calculations

The determination of the flow characteristics for uniform flow conditions can be calculated based on the continuity equation (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$^2$

While channel geometry can be estimated or surveyed, the flow velocity may not be as practical to manually or directly measure. When actual channel or flow velocity measurements are not available, the velocity can be calculated using the Manning's equation shown in Equation 4-2.

$$V = 1.486 \left(R^{2/3}\right)\left(S^{1/2}\right)/n \tag{4-2}$$

where:

V = mean velocity of flow in feet per second

R = hydraulic radius in feet (R = area (A) of flow section / wetted perimeter (P) of flow in channel)

S = slope of the energy grade line (EGL)

n = Manning's roughness coefficient of the channel refer to Table 4-1.

The flow area of a channel can be determined by previous investigations, surveys, or studies, or can be estimated through measurements of the channel and corresponding flow conditions. Determinations of slope (S) can be directly measured in the field for typical uniform and non-uniform flow conditions; refer to Section 4-3 below for more guidance on measuring in the field. If one or more variables are unknown, the flow area or flow depth must be calculated by trial and error, as presented in HDS-4, or by using a computer hydraulic program, such as the FHWA Hydraulic Toolbox or StormShed. The hydraulic designer is also referred to HDS-4 for further information on channel flow rates and velocities.

## 4-3 Field Slope Measurements

The slope is calculated by dividing the vertical drop in the river channel by the horizontal distance measured along the channel centerline or along the thalweg, whichever applies for uniform flow or natural (non-uniform flow) channels, of a specific channel reach. Where slope (S) is needed to support Manning's equation calculations, it can be measured in the field for typical channel conditions. Calculated channel slope is often referred to as the "rise over run," whereby the "rise" in a channel is represented by the vertical change in channel elevation, and the run in a channel is the change in horizontal length between representative elevation points.

Both rise and run are measured along the lowest point of the channel. For channels that have assumed uniform geometries (i.e., same cross section and profile), which is typical of constructed gravity stormwater systems, roadside ditches and swales, roadway gutters, and can also include streams and conveyance channels, the lowest elevation point is typically along the middle of the bed of the channel, as shown in Figure 4-1 and Figure 4-2.

**Figure 4-1** Field Slope Measurement of Uniform Flow Channels Plan View

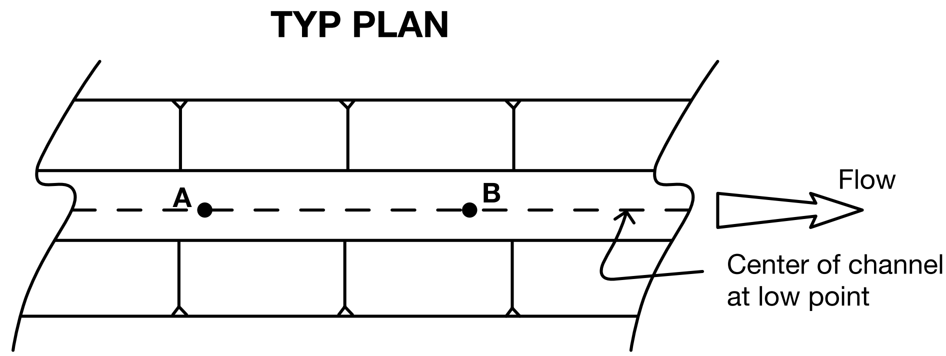
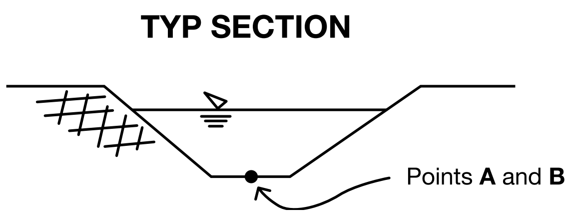



**Figure 4-2** Field Slope Measurement of Uniform Flow Channels Section View




Where the channel has non-uniform geometries (i.e., changes gradient or channel dimensions), which is more typical of natural stream and river channels that have geomorphically governed characteristics (e.g., pools and riffles) but can also be constructed channels, the slope shall be measured for each similar channel reach, and the results shall be incorporated into the analysis so as to accurately represent the overall channel hydraulics. A reach is defined as a segment of the channel with similar hydraulic and geomorphic characteristics. In particular for natural channels, the gradient is typically measured along the thalweg, as shown in Figure 4-3 and Figure 4-4. The thalweg is the lowest channel elevation point for any given flow, typically located along the outside of bends, and then moves more to the center of the channel in straight reaches. The thalweg can change during peak flows.

**Figure 4-3** Field Slope Measurement of Non-Uniform Flow Channels Plan View

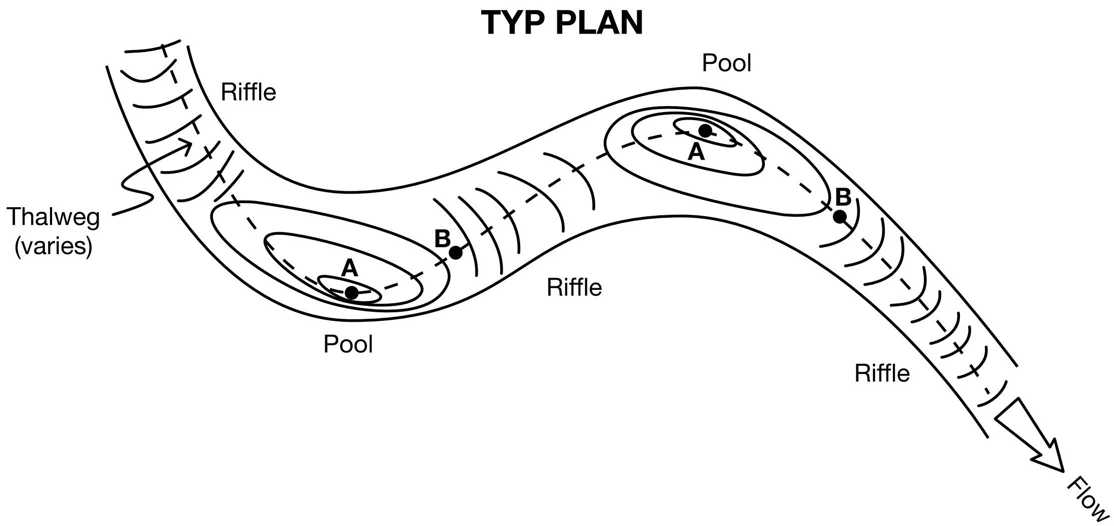
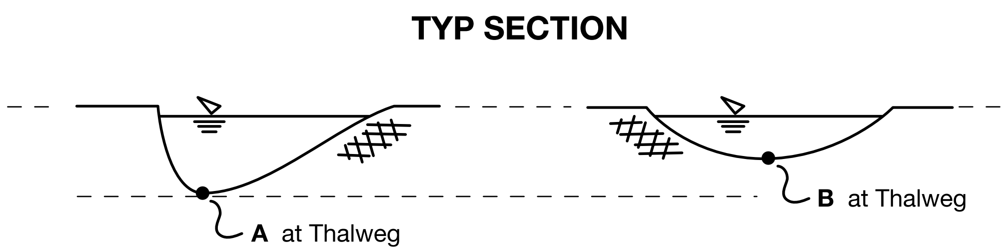



**Figure 4-4** Field Slope Measurement of Non-Uniform Flow Channels Section View



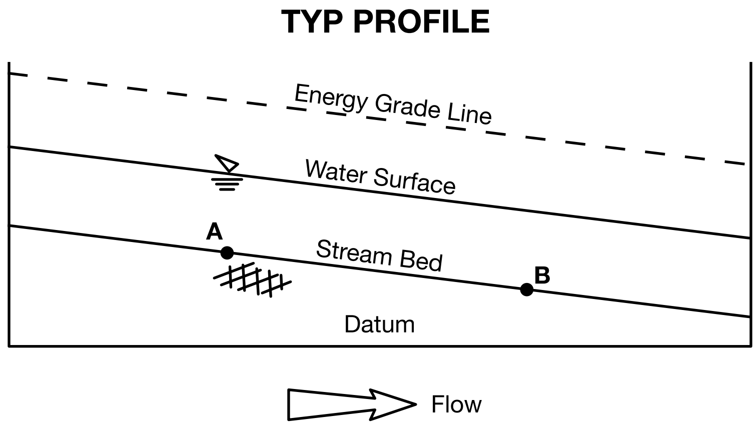

In both uniform and non-uniform channels, the engineer may need to apply discretion in how the gradient reaches are assessed and/or combined to best represent the channel hydraulic conditions, and where the thalweg is located.

## 4-3.1 *Uniform Flow Conditions: Gravity Stormwater Systems, Roadside Ditches and Swales, Roadway Gutters, Streams, and Conveyance Channels*

In constructed or natural channels with assumed uniform flow conditions (i.e., with corresponding uniform channel geometries and corresponding uniform flow depth, width, area, and velocity for the reach of interest) the channel bed gradient generally matches the top of flow gradient, as shown in Figure 4-5. Therefore, the vertical drop shall be measured at points along the bed elevation represented by points A and B in Figure 4-5. If the channel does not allow for practical or safe access to measure the channel bed (e.g., flows are too deep, or suspended sediment does not allow safe or practical visibility of bed conditions), then measure from the top of the water surface. The horizontal distance shall be measured between the two points where the bed or top of water points were located.

When discharge or flow is directed to cut slopes or fill slopes the designer shall include energy dissipaters along the drainage path to minimize erosion along the drainage path. The design shall follow Section 3-4.7.

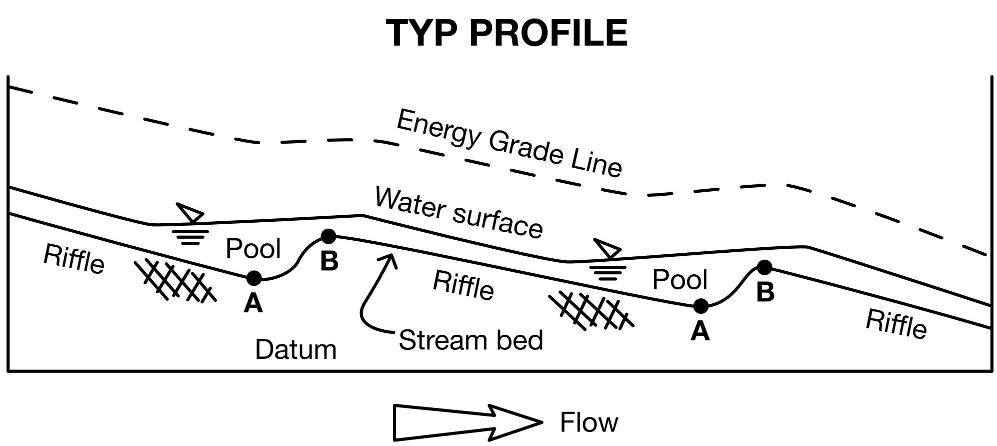
**Figure 4-5** Field Slope Measurement of Uniform Flow Channels Profile View




### 4-3.2 *Non-Uniform Flow Conditions: Streams and Rivers*

In natural channels with assumed non-uniform flow conditions (i.e., changes in channel depth, width, area, and/or velocity corresponding to variations in channel geometries at geomorphically governed pools or riffles along the channel reach of interest), the channel bed gradient may be different from the water surface gradient at various points along the channel, as shown in Figure 4-6. For example, the bed elevation may drop in pools along the channel, resulting in slower velocity and deeper flows, and then rise in riffles along the channel, resulting in shallower and faster velocity flows.

**Figure 4-6** Field Slope Measurement of Non-Uniform Flow Channels Profile View




In these situations, it is important to measure bed elevations at similar geomorphic locations; otherwise, the resulting channel gradient may represent only localized flow conditions and could be artificially high or low when considering the reach flow conditions. For example, measuring the channel gradient at a pool and the next downstream riffle (see Figure 4-6, points A and B) could result in a localized flatter gradient, and similarly measuring from a riffle to the following downstream pool could result in a locally steeper gradient; neither of these situations accurately represents the reach flow conditions. Measurements shall ideally be taken from “riffle-to-riffle,” shown in Figure 4-6 as point B at the upstream end of the riffle to point B at the following downstream riffle.

### 4-3.3 *Energy Grade Line*

Note that in both uniform and non-uniform channel flow conditions, the most accurate representation of gradient for input into calculations is represented by the energy grade line (EGL). The EGL is generally represented as the sum of the flow depth and the velocity head. The concept of the EGL is presented here to recognize the basis for the standard of practice, and be able to reference back to more complex analyses, where needed; in practical terms the channel bed and/or water level is commonly used as a means for characterizing slope in calculations.

In uniform flow conditions the flow depth is generally constant and the resulting water surface is generally parallel to the bed elevation; therefore, the EGL is also typically parallel to the water surface, as shown in Figure 4-5 above. Simplified calculations using measured rise over run to estimate slope of the channel are therefore applicable.

In non-uniform flow conditions, where the depth of flow and gradient can vary corresponding to changes in channel geometry along the channel, the corresponding channel slope is better represented by the EGL, as shown in Figure 4-6. Non-uniform flow conditions are more difficult to accurately characterize with manual channel bed measurements and calculations. If no other options are available, then incorporate the methods described above for measuring channel slope, and the results shall be qualified accordingly.

Because non-uniform flow conditions are more complex, and the measurement of channel geometries (i.e., elevations, sections, gradients, etc.) often requires special equipment and expertise to complete bathymetric surveys to capture that information, the methods of calculating corresponding hydraulic results incorporate the EGL and require using complex analyses and/or hydraulic modeling software tools. Contact the RHE or State Hydraulics Office for more information regarding more complex analyses.

## 4-4 Critical Depth

Before finalizing a channel design, the hydraulic designer must verify that the normal depth of a channel is either greater than or less than the critical depth. If this cannot be achieved contact the RHE for additional guidance. Critical depth is the depth of water at critical flow, an 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 hydraulic designer 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 shall refer to HDS-5 or HEC-14, for further information.

## 4-5 Manning's Roughness Coefficients (n)

Table 4-1 presents references for Manning's roughness coefficients.

**Table 4-1** References for Manning's Roughness Coefficients

| Category of Surface | Surfaces Included | Source |
|---|---|---|
| Open channel and pipe | Closed conduits<br>Pipes<br>Pavement<br>Gutter<br>Man-made channels | HEC-22 |
| River, stream, and culvert design for aquatic organism passage | Rigid channel<br>Minor streams<br>Floodplains<br>Major streams<br>Alluvial beds<br>Sand beds<br>Gravel beds<br>Cohesive soils<br>Composite roughness value | Aberle and Smart 2003<br>Barnes 1967<br>Bathurst 1985<br>Chow V.T. 1959<br>Griffiths 1981<br>Hey 1979<br>Jarrett 1984<br>Lee and Ferguson 2002<br>Limerinos 1970<br>Liu, X. et al. 2024<br>Rickenmann and Recking 2011<br>Yochum et al. 2012 |
| Channel lining | Rigid channel<br>Unlined channel<br>Grass<br>Gravel<br>Riprap<br>Gabion | HEC-15 |
| Storm sewer conduit[a] | Concrete pipe<br>Metal pipe<br>Polyethylene pipe<br>PVC pipe | HEC-22 |
| Street and gutter | Concrete gutter<br>Asphalt<br>Concrete pavement | HEC-22 |
| Maintained vegetation | Grass | HEC-15<br>Chow V.T. 1959 |

***Notes:***

a. For storm sewer pipes 24 inches or less in diameter, use n = 0.013.

# 4-6 Countermeasures for Stream Instability

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 protect transportation investments. New roadways and other infrastructure must be placed to minimize interaction with or effects on water bodies, avoiding them altogether if possible. This section discusses the options available for those cases where action must be taken and provides a subset of techniques and associated technical references to be used for those techniques. This is not a comprehensive guide, and as new techniques arise, all should be considered (in coordination with State Hydraulics Office for their cost-benefit in addressing interactions with water bodies. Countermeasures used for stream instability or bank protection have different design requirements from scour countermeasures used to protect a structure. Scour countermeasure design requirements for structures are provided in Section 7-4.3.

## 4-6.1 *Bank Protection*

Extensive guidance exists for numerous techniques for bank protection, from rock to revegetation. Many techniques recommended in Pacific Northwest rivers incorporate LWM; see Chapter 10 for guidance. Some of the most pertinent guidance documents are listed below:

- HEC-23, Volume 1 and Volume 2
- *Integrated Streambank Protection Guidelines* (ISPG) (WDFW 2002)
- *Bank Stabilization Design Guidelines* (Baird et al. 2015)
- WDFW's *Stream Habitat Restoration Guidelines* (Cramer 2012)

## 4-6.2 *Rock for Bank Protection*

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

There are disadvantages to using rock for bank protection. Replacing streambank vegetation with rock may create a relatively smooth surface, resulting in higher water velocities. This change may impact the channel downstream, and to some extent upstream, where the rock ends, creating a higher potential for erosion. Because of impacts to the adjacent channel, the hydraulic designer shall consider if using rock for bank protection would solve the problem or create a new problem. These aspects shall be considered when determining if rock is appropriate.

Rock bank protection is used primarily on the outside of curved channels or along straight channels when the streambank serves as the roadway embankment. 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.

#### 4-6.2.1 Rock Sizing for Bank Protection

For WSDOT projects, the rock material to be used will be quarry spalls or rock for erosion and scour protection (RESP) Class A, B, C, or D as defined in the *Standard Specifications*.

Once the hydraulic designer has completed a hydraulic analysis, the hydraulic designer shall consider the certainty of the velocity value used to size the rock along with the importance of the facility. For additional guidance and examples on rock sizing for bank protection design, see HEC-23, Volume 1 and Volume 2.

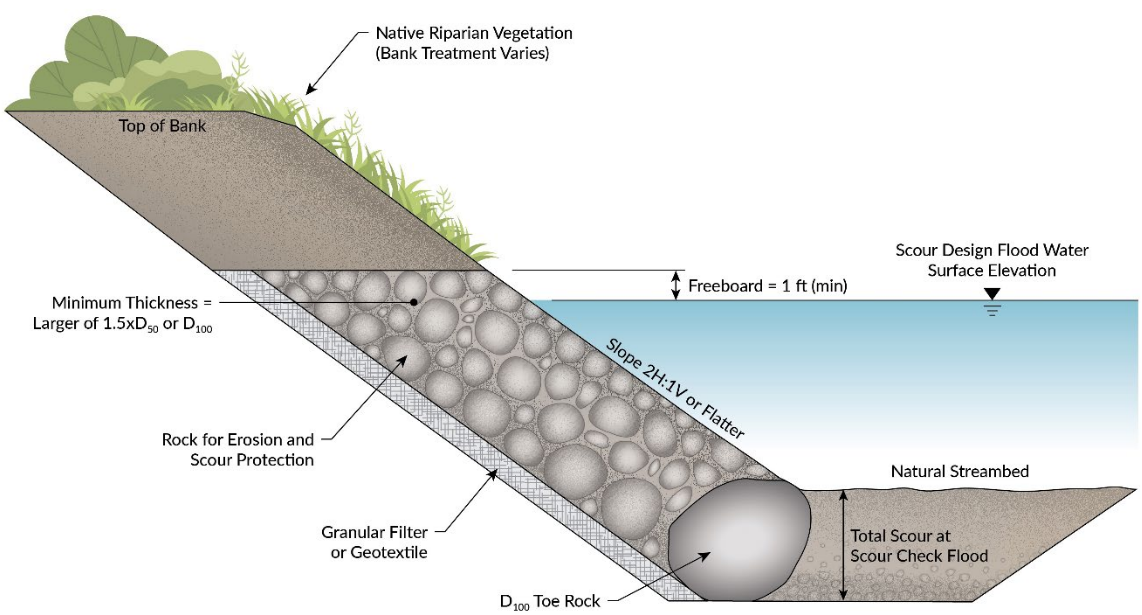
In some cases, on very high-velocity rivers or rivers that can transport large rocks downstream, even RESP Class D may not be adequate to control erosion and specially sized rock may need to be specified in the contract. The RHE, State Hydraulics Office, and HQ Materials Laboratory are available for assistance in writing a complete specification for special rock for erosion and scour protection.

#### 4-6.2.2 Placement of Rock Bank Protection

Once the type of rock has been selected, the next step is to determine the appropriate installation. Several factors affect the placement of rock including the type of filter material best suited for the project site, the thickness of rock placement, and the depth to key rock to prevent undermining.

Figure 4-7 illustrates a typical cross section of a rock bank protection installation.

**Figure 4-7** Typical Cross Section of Rock Bank Protection Installation



The filter material acts as a transition between the native soil and the rock, preventing the piping of fines through the voids of the rock structure while allowing relief of the hydrostatic pressure in the soil. Two types of filters are used: granular or geotextile. Filter materials are further described in the *Standard Specifications* and the *Geotechnical 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 type, the hydraulic designer shall first consult with the RME or geotechnical engineer and the RHE to determine if there is a preference. In areas of highly erodible soil (fine, clay-like soils), the State Hydraulics Office shall be consulted, and an additional layer of sand may be required. For additional guidance selecting the appropriate filter material, see HEC-23, Volume 1 and Volume 2. Use of the FHWA Hydraulic Toolbox is required for design of filters.

The thickness of rock placed (Figure 4-7) depends on which type of rock was selected: quarry spalls or RESP Class A, B, C, or D. Additional guidance for determining minimum rock thickness can be found in HEC-23, Volume 1 and Volume 2. Care should be taken during construction to ensure that the range of rock sizes, within each group, is evenly distributed to keep the rock stable. Rock is required to be extended to 1 foot above the scour design flood WSEL as shown in Figure 4-7. However, if severe wave action is anticipated, it shall extend farther up the bank.

In some circumstances, the rock bank protection slope face may be steeper than 2:1. The hydraulics designer shall coordinate with the RME or geotechnical engineer for feasibility prior to implementing into the design.

The hydraulic designer and construction inspectors must recognize the importance of a proper toe or key at the bottom of any rock bank protection. The toe of the rock is placed below the channel bed to a depth equaling total scour at the scour check flood (Figure 4-7). If the estimated scour is minimal, the toe is placed at a depth equivalent to the thickness of the rock to help prevent undermining. The toe of the revetment needs to be clearly detailed in the project plans to ensure that the revetment's foundation is solid. Without a toe, the rock has no foundation and the installation is certain to fail. 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*

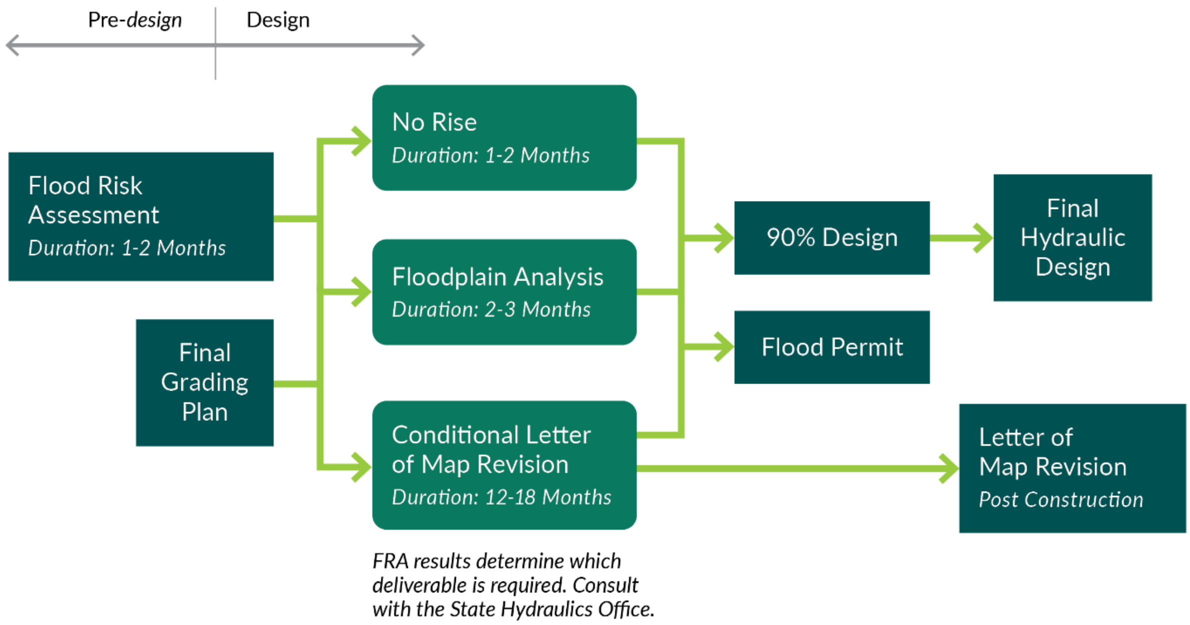
Channel stabilization, as opposed to bank stabilization, involves controlling and maintaining the channel cross section, alignment, and gradient, for some given length of the stream. There can be several reasons to stabilize a channel. At WSDOT, it is often to protect transportation infrastructure such as a culvert, bridge, or roadway embankment. These channel stabilization designs shall follow the guidance in HEC-23, Volume 1 and Volume 2. The major types of channel stabilization are concrete or rock linings, weirs, dams, and grade-control structures. Stabilization of roadside ditches and other constructed channels shall follow the guidance in HEC-15.

Notably, channel stabilization is a significant modification to natural processes, but is sometimes necessary for fish habitat or passage designs. It is not only technically challenging to design a maintenance-free, sustainable project of this nature, but it is also 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, and only in consultation with the State Hydraulics Office (see Chapter 7 and Chapter 10 for more details, as well as the ISPG (WDFW 2002).

## 4-7 Flood Risk Assessment

The Flood Risk Assessment (FRA) is a communication tool used to identify if there are potential risks of meeting FEMA, local jurisdiction, and public health and safety requirements in the preliminary stages of design. Specifically, the FRA identifies if there are potential risks (1) of meeting FEMA Code of Federal Regulations (CFR) requirements, (2) of meeting local jurisdiction code floodplain development requirements, and (3) to public health and safety in order for a project to be considered for permitting as a fish habitat enhancement project, as required per Revised Code of Washington (RCW) Section 77.55.181. The FRA also identifies subsequent deliverables (e.g., floodplain analysis, no-rise, Conditional Letter of Map Revision [CLOMR], etc.) that may be needed for the permitting process as shown in Figure 4-8. Each of these subsequent deliverables are covered in more detail in the following sections and are described on the FEMA website. This preliminary assessment should allow the PEO and other disciplines to know if the project may need a CLOMR, easement, ROW, temporary construction easement (TCE), etc. allowing the project schedule and budget to be modified, if needed, early in the project delivery process. These processes can be lengthy and add significant time to a project, so early coordination is critical. A Letter of Map Revision (LOMR) is completed after the project has been constructed. All stream projects, regardless whether they are in a FEMA special flood hazard area (SFHA), shall complete an FRA. The FRA template used by WSDOT and training can be found on WSDOT's Hydraulics website. For more information regarding the permitting process associated with floodplains, see the WSDOT *Environmental Manual*.

**Figure 4-8** Potential Deliverables for Permitting Process




### 4-7.1 *No-Rise Analysis*

A no-rise analysis is required when the project is located in a FEMA-designated floodway, or when local codes have requirements above the FEMA minimum standards. A no-rise analysis provides the required justification and technical data to support a no-rise certificate to obtain a flood hazard permit from a local jurisdiction. This permit is submitted and approved locally, and does not require further permitting by FEMA.

### 4-7.2 *Floodplain Analysis*

If a project is not located in a FEMA-designated floodway, a floodplain analysis shall be conducted. Contact the State Hydraulics Office for more information about the complexity of the floodplain analysis required.

### 4-7.3 *Conditional Letter of Map Revision*

FEMA requires a CLOMR when a no-rise cannot be met or when there is a realignment or change to a floodway. Local communities may require a CLOMR for other work done in the floodplain. Contact the State Hydraulics Office for information about when a CLOMR is needed and for assistance in requesting effective FEMA models.

### 4-7.4 *Letter of Map Revision*

Once a project is constructed an as-built survey is required to verify the results from the CLOMR (if required) and to submit a Letter of Map Revision (LOMR) request to FEMA. Contact the State Hydraulics Office for information about when a LOMR is needed and for assistance in requesting effective FEMA models.

# 4-8 Hydraulic Analysis for Riverine and Coastal Areas

WSDOT requires the use of SRH-2D with steady-state boundary conditions unless otherwise approved by the State Hydraulics Office for all riverine and coastal area projects. Determine modeling extents and terrain spatial resolution necessary to support the basis of design and coordinate early with survey crew to collect these data. For a FEMA no-rise assessment, CLOMR, or LOMR, the model required by the local floodplain manager is acceptable for the analysis; however, an SRH-2D model is still required for design. Any project that uses SRH-2D modeling will require a specialty report with model outputs as outlined in the WSDOT specialty report templates. All hydraulic modeling files need to be provided to HQ Hydraulics by uploading to the ProjectWise project folder: the files shall include all input and output files; remove extraneous or working files/simulations; coverages and simulations shall be clearly named. As a basis for 2D hydraulic modeling principles, FHWA has developed a reference document for 2D hydraulic models called *2D Hydraulic Modeling for Highways in the River Environment* (FHWA 2019). WSDOT has put together a 2D hydraulic modeling checklist that is used during model audits to ensure that stream designers are meeting the requirements of the *Hydraulics Manual* as well as the FHWA manual; this checklist can be found on the WSDOT Hydraulics Training web page.

SRH-2D hydraulic modeling training is required for all WSDOT projects or WSDOT-managed infrastructure that requires hydraulic modeling as part of the hydraulic design process. Hydraulic modelers are required to obtain a training certificate from NHI for attending Course 135095, *Two-Dimensional Hydraulic Modeling of Rivers at Highway Encroachments*. Other equivalent SRH-2D hydraulic modeling training requires approval by the State Hydraulics Office.

## 4-8.1 *Intermediate Conditions*

In a situation where an existing feature affects the hydraulics at the focused modeling location (e.g., upstream or downstream culvert, bridge, or weir) and the possibility exists that the structure could be removed or altered within the lifetime of the proposed construction, hydraulic modeling shall be completed for both the condition that the existing structure stays in place and having it removed. The proposed project shall meet design requirements for both current and future conditions.

## 4-8.2 *Tidal Crossings*

Tidally dominated crossings are crossings at locations where the flux varies with the tides and reverses direction during normal tidal events. These sites shall be modeled as unsteady-state simulations using the tidal hydrograph described in Section 7-5.3 as the downstream boundary condition. Tidally influenced crossings are affected by tides, and are further described in Section 7-3.5.4. These may be modeled as steady- or unsteady-state simulations. The decision to model as steady or unsteady state is site-dependent and modeling as steady state must be approved by the State Hydraulics Office. If the system is modeled as a steady-state simulation, each flood event must be modeled with both high and low tide WSELs as the downstream boundary condition.

# Chapter 5 Drainage of Highway Pavements

## 5-1 Introduction

Roadway and structure pavement drainage shall be considered early in a project design, while the roadway geometry is still being developed, because 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), roadside ditch design (Section 5-5), drainage structures and grate types and considerations (Section 5-6), and use of scupper barriers (Section 5-7). It concludes with a brief discussion of hydroplaning and hydrodynamic drag (Section 5-8).

The flatter the longitudinal profile is, 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 hydraulic designer must carefully examine the geometric profile of the gutter to eliminate standing water created by these transitions. These areas shall be identified and eliminated to the greatest extent feasible. This generally requires geometric changes stressing the need for early consideration of drainage; otherwise, additional drainage structures will be required.

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 shall not be allowed to discharge directly into waterways or wetlands without water quality treatment. Also, bridge drain downspouts have a history of plugging.

Inlets on bridges can usually be eliminated by considering drainage early in the design phase through geometric adjustments. Superelevation transitions, zero gradients, and sag vertical curves shall be avoided on bridges. Drainage design at bridge ends requires a great deal of coordination between the RHE, hydraulic designer, and State Hydraulics Office. All bridge drain designs shall be reviewed and approved by the State Hydraulics Office.

Multilane highways create unique drainage situations. The number of lanes draining in one direction shall be considered during the design phase. It may be necessary to complete a hydroplane analysis to assess risk. Coordinate with the RHE for additional requirements and guidance. “Part-time shoulder use” facilities shall be considered a lane. 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 5 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 or other minimizing erosion methods shall be included in projects as a means to protect the slopes from erosion until vegetation is established. Once sufficient vegetation is present to resist erosion and treat runoff, consideration shall be given to eliminating the curb in future overlay contracts as long as the runoff can be properly be dispersed with the use of an energy dissipater per Section 3-4.7, if needed.

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 Section 5-5, including energy dissipators as needed per Section 3-4.7.

## 5-3.1 *Bridge Deck and Downstream End Drainage*

The drainage design for bridge decks requires the coordination of the bridge designer, the State Hydraulics Office, and the hydraulic designer. The requirements of Table 5-1 for allowable spreads also apply to bridge decks and along the bridge barriers. The bridge drainage calculations must be included in the hydraulic design report. Chapter 2 of the *Bridge Design Manual* has additional information on bridge deck drainage.

The downstream ends of bridges need special attention. If a storm sewer inlet system is not provided, a channel shall be provided at the end of any significant barrier or curb to collect and convey concentrated stormwater away from the bridge.

Bridges with approach slabs generally have an extruded curb beginning at the bridge end and terminating past the approach slab. The concentrated flow shall be directed into a low-risk erosion area. The end of curb shall be located a minimum of 10 feet from an approach slab to avoid approach slab settlement due to the concentrated flow. Inlets also shall be located a minimum of 10 feet downstream from an approach slab to provide adequate construction clearance during installation or future drainage structure replacement.

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 and roadside slope erosion. If the design includes a new bridge or buried structure over a waterway, the hydraulic designer shall coordinate drainage outfalls with the Stream Team (defined in Chapter 7-1) to ensure that the outfalls do not cause erosion or interfere with any habitat or stream features.

### 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 Table 5-1.

For design-bid-build projects, the hydraulic designer shall perform a gutter flow analysis for each construction staging plan of the project using the same allowable spread design criteria in Table 5-1. Not meeting the criteria in Table 5-1 is not considered a *Hydraulics Manual* 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 Temporary Erosion and Sediment Control (TESC) Plan, Abbreviated TESC Plan, or region equivalent document and shall have concurrence from the RHE.

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 Table 5-1. Not meeting the criteria in Table 5-1 is not considered a *Hydraulics Manual* deviation. The purpose of the required analysis is to identify areas of ponding water for the design-builder to be aware of during 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 RHE.

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 shall use a uniform gutter section per HEC-22. The project shall only use uniform gutter sections as opposed to depressed gutter sections 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

The inlet spacing analysis shall take into account the effects of a shared-use path or bike lane that is already in existence or added as part of the project scope or as a requirement by Complete Streets.

For pedestrian safety considerations, the PEO shall assess the need to install an inlet near a marked pedestrian crossing even when the inlet spacing analysis or sag inlet analysis does not demonstrate the need for an inlet to satisfy flow spread requirements.

**Table 5-1** Design Frequency and Allowable Spread

| Road Classification | | Design Frequency (years) | Allowable Spread ($Z_d$) |
|---|---|---|---|
| Interstate | <45 mph | 10 | Shoulder + 2 feet |
| | ≥45 mph | 10 | Shoulder |
| | Underpasses and sag | 50 | Shoulder + 2 feet |
| Principal, minor arterial, or divided | <45 mph | 10 | Shoulder + 2 feet[a] |
| | ≥45 mph | 10 | Shoulder |
| | Sag | 50 | Shoulder + 2 feet[a] |
| Collector and local streets | <45 mph | 10 | Shoulder + one-half driving lane[b] |
| | ≥45 mph | 10 | Shoulder |
| | Sag | 50 | Shoulder + one-half driving lane[b] |
| Roundabouts (circulating roadway) | All design speeds | 10 | One-half driving lane[b] |
| Roundabouts entry lanes[c] | ≤45 mph | 10 | Shoulder + one-half driving lane[b] |
| | Sag | 50 | |
| Dedicated turn lanes | All design speeds | 10 | Shoulder + one-half driving lane[b] |
| | Sag | 50 | |
| Ferry terminals | <45 mph | 10 | Shoulder + 2 feet |
| | >45 mph | 10 | Shoulder |
| | Sag | 50 | Shoulder + 2 feet |
| Part-time shoulder use | All design speeds | 10 | Maintain at least 10 feet of driving width within the multi-use shoulder that is free of water |
| | Sag | 50 | |

***Notes:***
mph = miles per hour

a. 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.
b. For multi-lane roadways, only include the width of the driving lane adjacent to the shoulder or gutter.
c. Entry lanes include exit, bypass, and slip lanes.

## 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 shall 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 0.1 cfs for the 10-year MRI storm. The bypass flow rate of 0.1 cfs will not usually cause erosion or hydroplaning problems. The hydraulic designer 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 WSEL determined in the sag analysis. The spread width shall be compliant with Table 5-1. The spread width requirement also applies to the end of the curb or barrier even without an inlet.

A bypass flow more than 0.1 cfs at the curb or barrier end can be allowed with an approved deviation. To protect the roadside slope downstream of the bypass flow, employ erosion protection measures such as installation of rocks or filter blanket for energy dissipation. Coordinate with the RHE on the slope protection design.

In urban situations, with much lower speeds than noted in Table 5-1, it may not be feasible to use the allowable spread in the *Hydraulics Manual*. In this situation, the hydraulic designer shall first consider innovative solutions such as increasing the slope of the gutter (e.g., from 2 to 5 percent), depressing the inlet, or using a combination curb opening and grate inlet. If it is still not possible to meet the allowable spread in Table 5-1, the hydraulic designer shall 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 hydraulic designer shall 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 0.1 cfs. The hydraulic designer will find, by the time the roadway approaches the zero point, that 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 0.1 cfs.

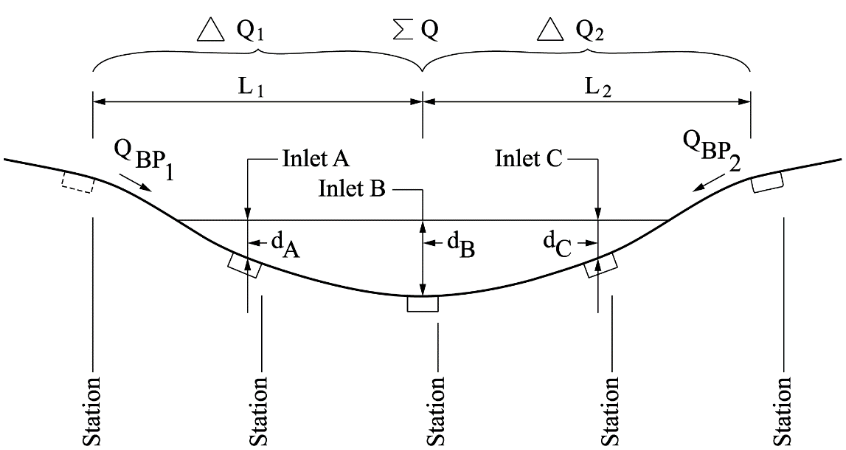
Roundabouts are typically designed to accommodate speed limits of 35 miles per hour (mph) or less; generally, the posted advisory speed limits are between 15 and 25 mph. Potentially, runoff from a roundabout is diverted to multiple different directions and, if it is possible, runoff from the upstream roadway shall be captured so that flow bypass shall 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. 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 because runoff at a roundabout could flow off in multiple directions. The hydraulic designer shall coordinate with the RHE and Maintenance to address all possible drainage issues expected with design and construction of the roundabout.

### 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 from 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) at 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 focuses on inlets operating as a weir with flow spilling in from the three sides of the inlet that are exposed to the ponding.

**Figure 5-1** 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 that the depth of water at the flanking inlets ponds to half the allowable depth at the sag (or $0.5d_{B\ allowable}$); see Figure 5-1 above. Flanking inlets are required only 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 because 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 Table 5-1 above, 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 hydraulic designer shall consider the potential damage of water going over the curb. The hydraulic designer shall use the guidelines provided in this section for locating flanking inlets. If the hydraulic designer suspects that flanking inlets are unnecessary, consult the RHE earlier in the design.

Any section of roadway located in a sag shall be designed according to the criteria described below and further detailed in the WSDOT Sag Worksheet located on the State Hydraulics Office web page.

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 5-1.

The effective perimeter of the flanking and sag inlets can be determined using the lengths and widths for various grates provided in Table 5-2. 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 5-1) would be assumed to be 50 percent plugged (except for the Combination Inlet, Standard Plan B-25.20-02, which shall be considered 0 percent plugged). This 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. Table 5-2 lists perimeters for various grates with reductions already made for bars.

$$P_n = L + 2\left({}^{W}\!/_{2}\right) \quad (5\text{-}2)$$

where:

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

$L$ = length of the inlet "n" from Figure 5-1

$W$ = width of the inlet "n" from Figure 5-1

When using a Combination Inlet, the width of the inlet, *W*, in Equation 5-2 shall not be divided by 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 shall be used:

$$Q_{allowable} = C_W \times P \times d_{B\,allowable}^{1.5} \tag{5-3}$$

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 that all grates are the same size and are oriented the same (all rotated or not rotated):

$$\Sigma Q = C_w \times P \times [2(0.5d_B)^{1.5} + (d_B)^{1.5}] \tag{5-4}$$

where:

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

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

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

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 0.5 $d_{B\,allowable}$, 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 Table 5-2

$d_B$ = actual depth of ponded water at the inlet in feet

After the analysis is completed, the PEO shall verify that 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 shall be placed close to the sag inlet to increase the flow interception and reduce the flow into the sag.

## 5-5 Roadside Ditch Design Criteria

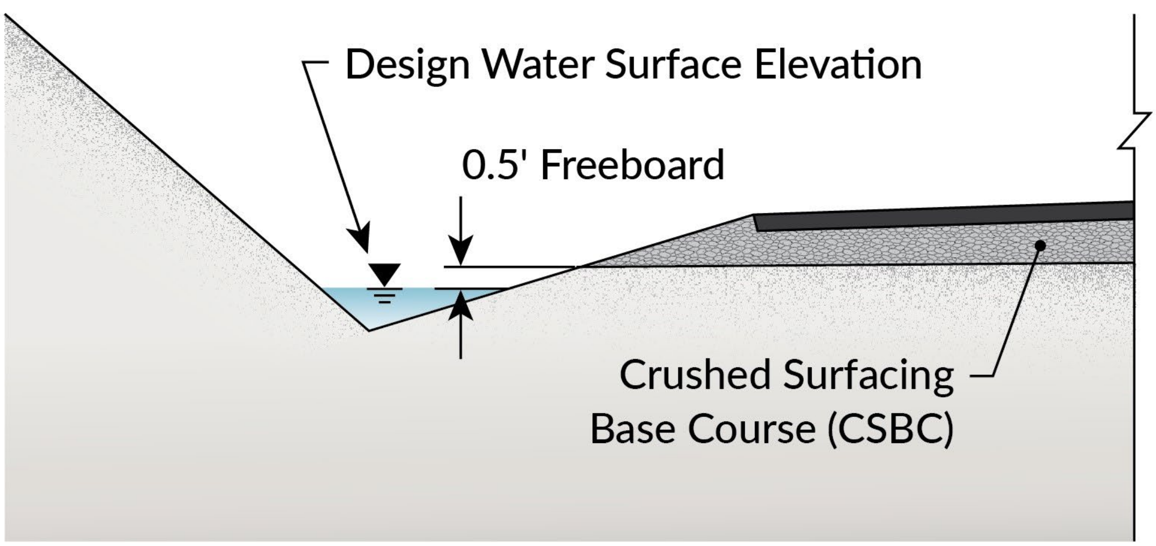
Roadside ditches are generally located alongside uncurbed roadways with the primary purpose of conveying runoff away from the roadway. Ditches shall be designed to convey the 10-year recurrence interval with 0.5 foot of freeboard (from the ditch design WSEL to the bottom of the pavement subgrade or ditch spill) and a maximum side slope of 2H:1V (Figure 5-2). Side slopes of 4H:1V or flatter are desirable; see WSDOT *Design Manual Exhibit 1239-4* for requirements for slopes steeper than 4H:1V.

The preferred cross section of a ditch is trapezoidal; however, a “V” ditch that meets the design requirements can also be used where ROW is limited. In those cases where the grade is flat, preventing adequate freeboard, the depth of channel shall still be sufficient to remove the water without saturating the subgrade shoulder.

If the freeboard is less than 0.5 foot, a deviation is required. Justification by the hydraulic designer including coordination with the RHE and Region Maintenance to allow the installation of an impermeable ditch liner or an underdrain system underneath the ditch to prevent saturation of the roadway subgrade.

To maintain the integrity of the channel, ditches are usually lined. See HDS-4 and HEC-15 for additional guidance.

**Figure 5-2** Drainage Ditch Detail



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

A drainage inlet can be placed at a low point or at the end of the ditch to convey the water to its intended discharge point. Ditch inlets operate as weirs under low water depth conditions or as orifices at greater depths. Orifice flow begins at depths dependent on the grate size. Flows in a transition stage could yield water depths fluctuating between weir and orifice control.

Ditch inlets are more susceptible to clogging from sediments and debris. Ensure that the grate is adequately sized to satisfy the ditch freeboard requirement or prevent water from spilling over onto the roadway. Contact the RHE for ditch inlet analysis.

## 5-6 Drainage Structures

Many variables are 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 shall not 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 the grate, not to the center of the structure, to ensure that 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.

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 is also known to cause debris problems. These areas may require additional maintenance.

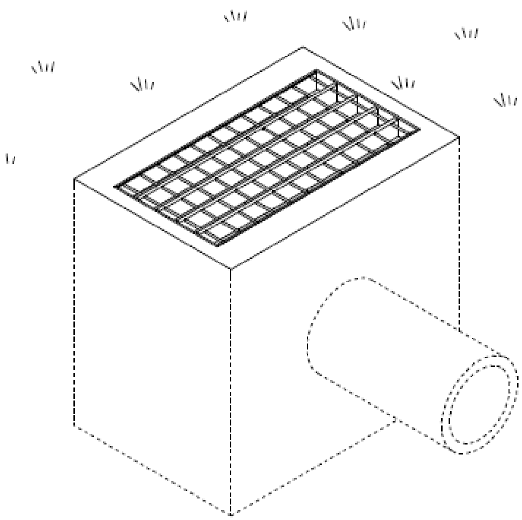
### 5-6.1 *Inlet Structure Types*

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 briefly describes each structure type.

#### 5-6.1.1 Grate Inlet Type 1 Structure: Standard Plan B-35.20-00

Grate inlet Type 1 structures are cast-in-place and use 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 hydraulic designer to tie into existing stormwater infrastructure without modifying the hydraulic gradient.

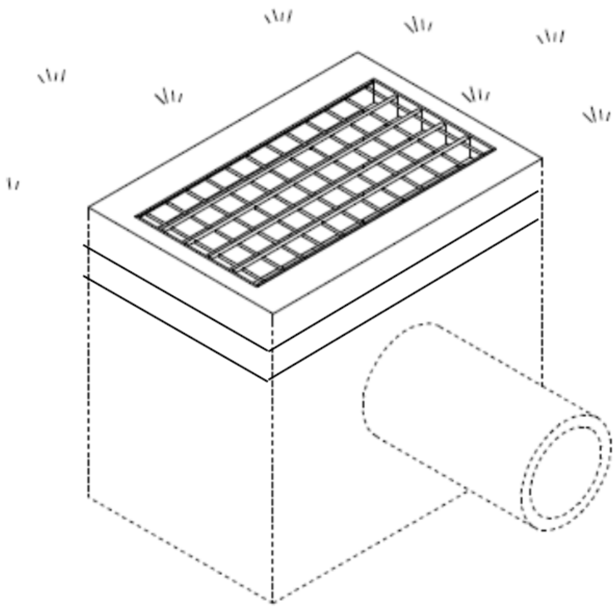
**Figure 5-3** Grate Inlet Type 1 Structure



#### 5-6.1.2 Grate Inlet Type 2 Inlet Structure: Standard Plan B-35.40-00

Grate inlet Type 2 structures are constructed using sections of precast reinforced concrete (Figure 5-4). These precast sections can be stacked to meet the required height, thus reducing construction time and cost. This inlet structure is similar to 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 shall be used in areas where existing infrastructure is easy to tie into.

**Figure 5-4** Grate Inlet Type 2 Structure



### 5-6.1.3 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 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-5.20-03 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-10.40-02 Catch Basin Type 2 with Flow Restrictor
- Standard Plan B-10.70-02 Catch Basin T-PVC

Within WSDOT ROW, a T-PVC catch basin can be used as an inlet or as a junction box in locations not subject to traffic loading such as ditches, landscaped or vegetated areas, and separated pedestrian paths. The use of a T-PVC catch basin requires the approval of the State Hydraulics Office through the RHE. The RHE shall not recommend approval without first getting concurrence from Region Maintenance. If approved for installation, T-PVC catch basin shall not be connected to a drainage system that is fully or partially installed within a roadway, sidewalk adjacent to the roadway, and the paved surface of a rest area.

#### 5-6.1.4 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

#### 5-6.1.5 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.

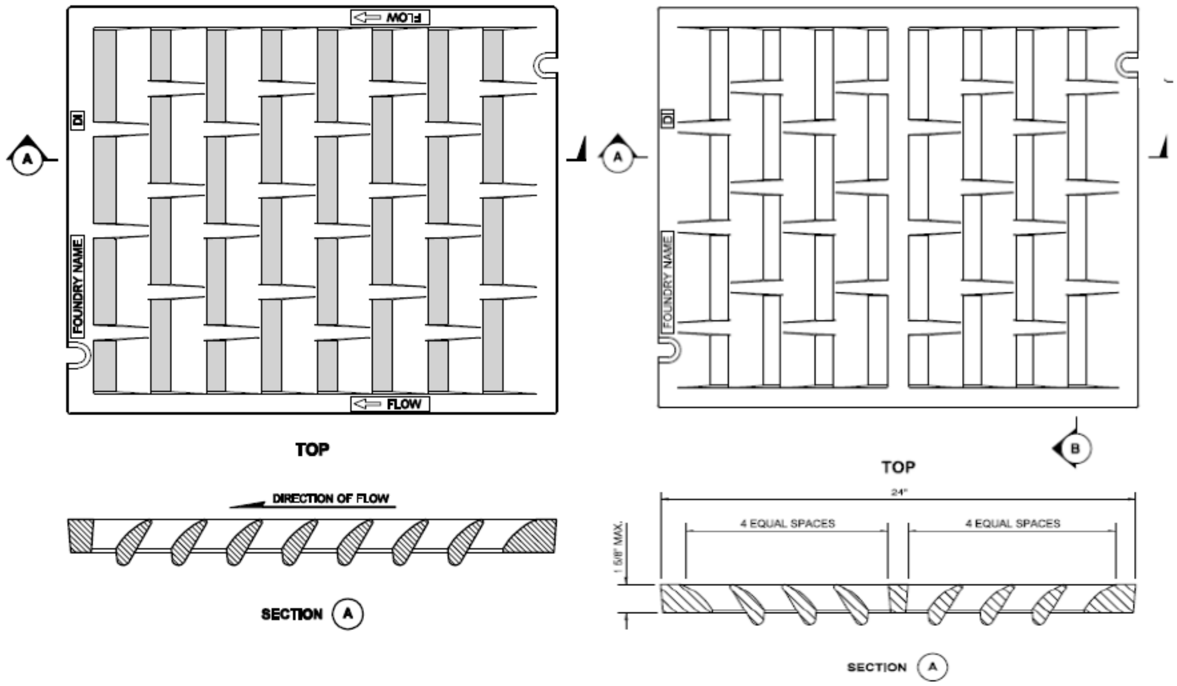
### 5-6.2 *Grate Types*

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.

#### 5-6.2.1 Rectangular Vaned Grate: Standard Plan B-30.30-03 and Rectangular Bi-Directional Vaned Grate: Standard Plan B-30.40-03

The vaned grate 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 backward the interception capacity is severely limited. The rectangular bi-directional vaned grate shall be used at all sump locations. Figure 5-5 depicts a rectangular vaned grate and a rectangular bi-directional vaned grate.

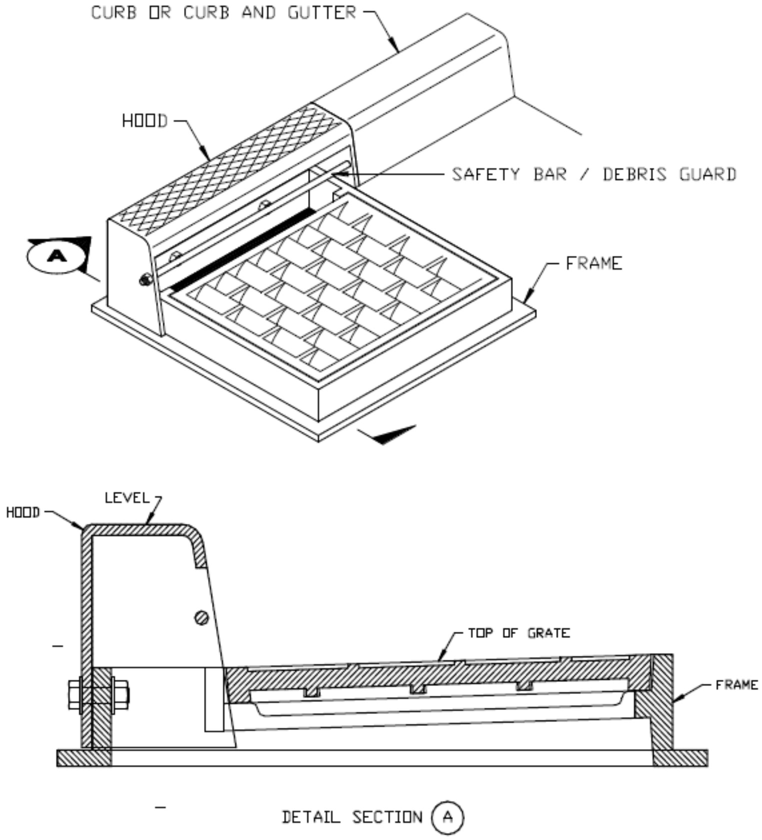
**Figure 5-5** Rectangular Vaned Grate and Rectangular Bi-Directional Vaned Grate




### 5-6.2.2 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 shall follow the same analysis as for a vaned grate alone (see Standard Plan B-30.30-03).

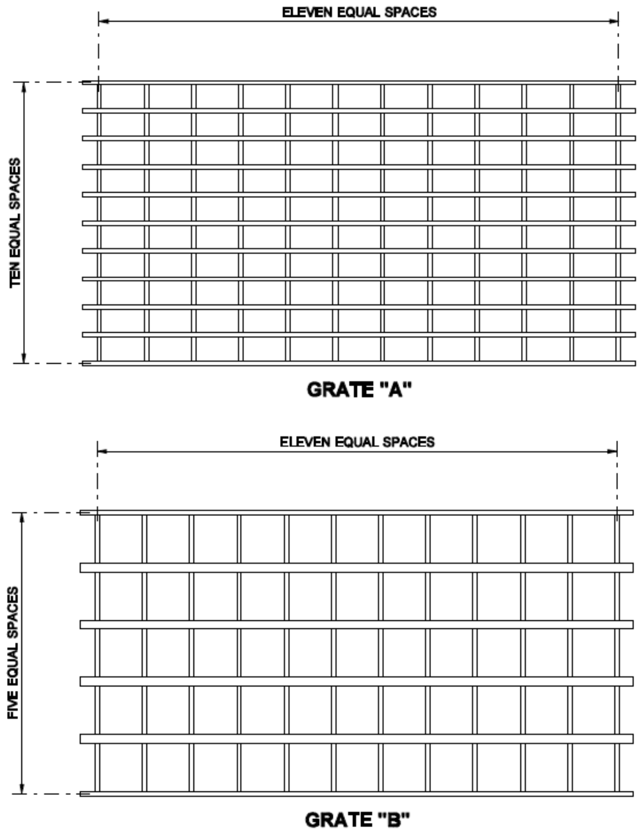
**Figure 5-6** Section and Isometric View Combination Inlet Frame, Hood, and Vaned Grate




### 5-6.2.3 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. Because of 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.

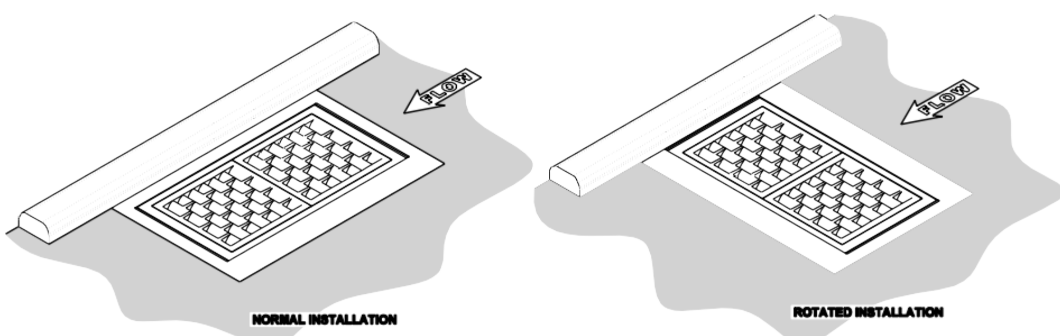
**Figure 5-7** Grates A and B




### 5-6.2.4 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 shall 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.

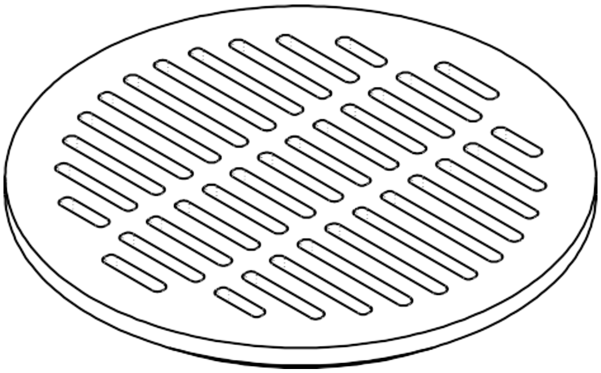
**Figure 5-8** Frame and Vaned Grates for Installation on Grate Inlet



### 5-6.2.5 Circular Grate or Standard Plan B-30.80-01

Circular grates are intended for use with dry wells, see *Standard Plans* 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.

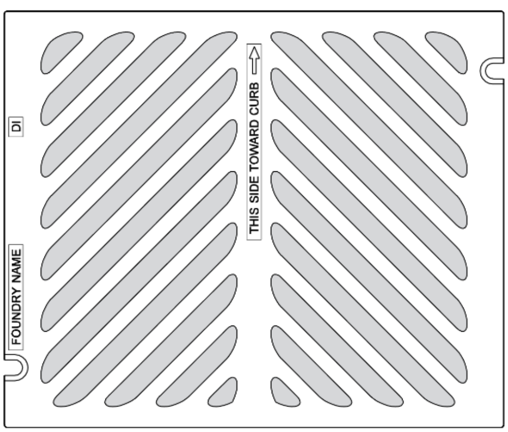
**Figure 5-9** Circular Grate



### 5-6.2.6 Rectangular Herringbone Grate: Standard Plan B-30.50-03

Herringbone grates (Figure 5-10) shall not be used on WSDOT projects. Replacement of existing herringbone gates shall be considered during preservation projects. Historically, use of the vaned grate was limited because of cost considerations. The cost difference now is minimal; the vaned grate is bicycle safe and is hydraulically superior under most conditions.

**Figure 5-10** Herringbone Pattern




Grate inlet properties are summarized in Table 5-2.

**Table 5-2** Properties of Grate Inlets

| Standard Plan | Description Properties of Grate Inlets | Continuous Grade[a] | | Sag Location[b] Perimeter Flows as Weir | |
|---|---|---|---|---|---|
| | | Grate Width (ft) | Grate Length (ft) | Width (ft) | Length (ft) |
| B-30.50-03[c] | Rectangular herringbone grate | 1.67 | 2.0 | 0.69 | 0.78 |
| B-30.30-03 or B-30.40-03[d] | Vaned grate for catch basin and inlet | 1.67 | 2.0 | 1.31 | 1.25 |
| B-25.20-02[b] | Combination inlet | 1.67 | 2.0 | 1.31 | 1.25 |
| B-40.20-00 | Grate inlet Type 1<br>(Grate A or B[e]) | 2.01<br>3.89[f] | 3.89<br>2.01[f] | 1.67<br>3.52 | 3.52<br>1.67 |
| B-30.80-01 | Circular grate | 1.52 | | 2.55[g] | |
| B-40.40-02 | Frame and dual vaned grates for grate inlet Type 1 or Type 2 | 1.75[h]<br>3.52[f] | 3.52[h]<br>1.75[f] | 1.29<br>2.58[f] | 2.58<br>1.29[f] |

***Notes:***

a. Inlet widths on a continuous grade are not reduced for bar area or for debris accumulation.
b. 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 5-6.2, except for the combination inlet, *Standard Plans*.
c. Shown for informational purposes only (see Section 5-6).
d. For sag conditions, inlets shall use a bidirectional vaned grate (as shown in *Standard Plans*).
e. Type B grate shall not to be used in areas of pedestrian or vehicular traffic (see Section 5-6 for further discussion).
f. Rotated installation (see *Standard Plans*).
g. Only the perimeter value has been provided for use with weir equations.
h. Normal installation (see *Standard Plans*).

## 5-7 Scupper Barrier

Scupper barrier designs are available for both Type F (Standard Plan C-60.15) and Single-Slope (Standard Plan C70.15) concrete barriers. See Design Manual 1610.06(1)(e) for more information.

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-95.20-02, allowing runoff to pass between the structures in a pipe.

In locations where a scupper barrier is used specifically to pass stormwater to flow across highway lanes on the other side of the median barrier, the scuppers shall be analyzed for potential plugging and consider site-specific details such as accumulation of debris or maintenance sand as well as impacts or risk associated with snow and ice obstructing the passage of stormwater. In sag profile locations, the project shall consider secondary means of removing stormwater, should scuppers be plugged, by installation of drainage structures. To analyze the hydraulic capacity of scuppers or curb-opening inlets, refer to Section 7-2.2 in FHWA's HEC-22 for guidance.

Contact the RHE to determine the appropriate level of consideration and analysis appropriate for a specific project or design.

## 5-8 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, Drain Pipe, Underdrain Pipe

## 6-1 Introduction

This chapter discusses the design criteria for storm sewers, drain pipe, and underdrain pipes. This chapter also briefly describes the potential design impacts on these types of pipes because of Complete Streets, and includes a discussion of drywells (Section 6-5).

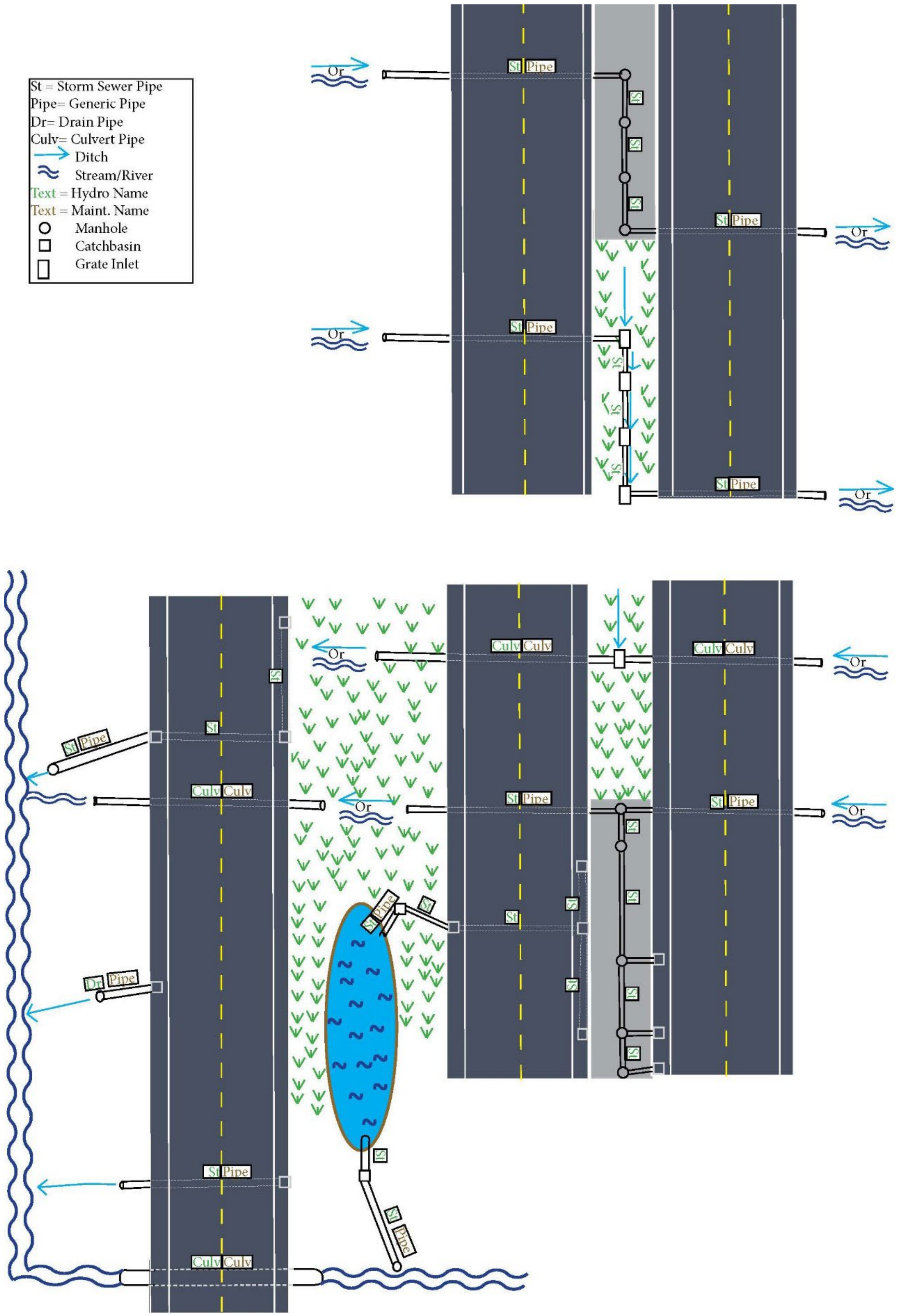
Implementing new Complete Streets and other active transportation design roadway features may require additional design considerations for storm sewers, drain pipes, and under drain pipes. A given project may need to move storm sewers to accommodate share use paths and bike lanes. Another scenario might require an existing run of storm sewer to be moved to the outside edge of pavement which would include the new shared use path or bike lane. The same types of adjustments may be needed for drain pipes and under drain pipe.

## 6-2 Storm Sewer

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 6-2.1); the data and process required to document the design (Section 6-2.2); and methods, tools, and concepts to help develop designs (Section 6-2.3 through Section 6-2.5).

Storm sewers are generally defined as closed-pipe networks connecting two or more inlets; see Figure 6-1. Typical storm sewer networks consist of laterals that discharge into a trunk line. The trunk line then receives the discharge and conveys it to an outlet location. For clarification on the difference between storm sewer and culvert configurations see Figure 6-1. See Section 8-2.4 for pipe testing requirements.

**Figure 6-1** 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 calculated using the Rational Method or the SBUH Method; see Chapter 1 and Chapter 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 6-2.3, or by computer program, as described in Section 6-5.

All storm sewer design shall consider climate resilience when determining required pipe sizes for flow conveyance; these factors include the following:

- Storm surges
- 24-hour peak precipitation (100-year event)
- Tidally influenced zones
- Sea level rise
- FEMA SFHAs
- Section 7-4.5.5 of WSDOT *Hydraulics Manual*
- Wildfires
- Landslides
- Sediment transportation
- Chronic events
- Population migration
- Future land use changes
- Heat waves

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

### 6-2.1 *Storm Sewer Design Criteria*

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

- **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 ensure stable soil conditions. Soil resistivity and pH must also be known so that the proper pipe material will be used. Section 8-5 contains further guidance.

- **Structure spacing and capacity:** Design guidelines for inlet spacing and capacity are detailed in Chapter 5. Structures (catch basins, grate inlets, and manholes) shall 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 shall consult the RHE for guidance.
- **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.
- **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.
- **Velocity:** The design velocity for storm sewers shall be between 3 and 10 ft/s. This velocity is calculated using Manning's equation, under full flow conditions even if the pipe is flowing only partially full with the design storm. The minimum slope required to achieve these velocities is summarized in Table 6-1.

  When flows drop below 3 ft/s, pipes can clog because of siltation. Flows can be designed to as low as 2.5 ft/s with justification in the hydraulic report. As the flow approaches (and exceeds) 10 ft/s, PEOs shall consult the RHE for abrasion design guidance.

**Table 6-1** Minimum Storm Sewer Slopes

| Pipe Diameter (in) | Minimum Slope (ft/ft) | |
|---|---|---|
| **N = 0.013** | **2.5 ft/s** | **3.0 ft/s** |
| 12 | 0.003 | 0.0044 |
| 15 | 0.0023 | 0.0032 |
| 18 | 0.0018 | 0.0025 |
| 24 | 0.0012 | 0.0017 |

- **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. Invert elevations of pipe draining a structure shall not be higher than any pipe discharging flow into the same structure unless a stilling structure is an intentional part of the storm sewer design.

- **Minimum pipe diameter:** The minimum pipe inside diameter for all storm sewer systems shall be 12 inches. If partially replacing or modifying an existing storm sewer system, the new or added storm sewer shall have at least the same diameter as the existing storm sewer even if the hydraulic analysis shows a smaller-diameter storm sewer would meet hydraulic design requirements in that location. If an existing culvert is replaced and converted to a configuration that would classify it as a storm sewer, coordinate with the RHE on the pipe sizing.
- **Structure constraints:** During the storm sewer layout design, PEOs shall 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 that 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 minimum clearance requirements must be met depending on the pipe diameter. PEOs can verify the minimum pipe angle with the Pipe Angle Calculation Worksheet.
- **Pipe material**: Storm sewers shall be designed to include all Schedule A pipe options, unless specific site constraints limit options (see Section 6-6 for further discussion).
- **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, because of maintenance concerns, WSDOT design practices do not allow pipe diameters to decrease in downstream runs. Consideration could be given to 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.
- **Discharge location:** A discharge location is where stormwater from WSDOT highways is conveyed off of the ROW by pipe, ditch, or other constructed conveyance. Additional considerations for discharge locations include energy dissipators and tidal gates. Energy dissipators prevent erosion at the discharge location. Based on the outlet velocity at the discharge location, the PEO shall install energy dissipation per Section 3-4.7. Installation of tide gates may be necessary when the discharge location is in a tidal area; consult the RHE for further guidance.
- **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. When a storm sewer is placed beyond the pavement edge, a

one-trunk system with connecting laterals shall be used instead of running two separate trunk lines down each side of the road.

- **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*.

### 6-2.2 *Storm Sewer 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 Chapter 1 for guidelines on what information shall be submitted and recommendations on how it shall be organized.

### 6-2.3 *Storm Sewer Design: Manual Calculations*

Manual calculations and spreadsheet calculations for storm sewer design are suitable only for pipe runs that do not include tailwater conditions or system losses that affect the capacity of the pipe. Project design teams shall consult the RHE prior to beginning design to determine if manual and spreadsheet calculations are acceptable for the project storm sewer design.

Storm sewer design is accomplished in two parts: (1) determine the pipe capacity and (2) evaluate the HGL. See the Storm Sewer Pipe Sizing Spreadsheet to determine the pipe capacity of the storm sewer system.

The Storm Sewer Pipe Sizing Spreadsheet does not currently calculate the HGL at each structure. The hydraulic designer must calculate them using hand calculations, per Section 6-2.5 and HEC-22, or use computer software per Section 6-2.4. The hydraulic designer shall consult with the RHE prior to design to determine if manual and spreadsheet HGL calculations are acceptable for the project storm sewer design.

### 6-2.4 *Storm Sewer Design: Computer Analysis*

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

### 6-2.5 *Storm Sewer Hydraulic Grade Line Analysis*

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 WSEL 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 WSEL in the catch basin/manhole needs to be calculated to verify that the WSEL is below the rim (top) elevation. When the WSEL 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). During the non-storm events (not raining), the HGL must be zero or at the same elevation as the pipe invert; no standing water inside the pipe would be allowed during non-storm events.

Regardless of the design conditions, the HGL shall 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 be calculated only 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 WSEL at the storm sewer outlet must be known. Refer to Chapter 3 for an explanation on calculating WSELs at the downstream end of a pipe (the tailwater is calculated the same for the 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 WSEL at the storm sewer outlet to obtain the WSEL at the first upstream structure (also the HGL at that structure, assuming that 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 WSEL of the first structure to obtain the WSEL 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:

$$\begin{aligned} \mathrm{WSEL_{J1}} &= \mathrm{WSEL_{OUTFALL}} + \mathrm{H_{f1}} + \mathrm{H_{e1}} + \mathrm{H_{ex1}} + \mathrm{H_{b1}} + \mathrm{H_{m1}} \\ \mathrm{WSEL_{J2}} &= \mathrm{WSEL_{J1}} + \mathrm{H_{f2}} + \mathrm{H_{e2}} + \mathrm{H_{ex2}} + \mathrm{H_{b2}} + \mathrm{H_{m2}} \\ \mathrm{WSEL_{Jn+1}} &= \mathrm{WSEL_{Jn}} + \mathrm{H_{fn+1}} + \mathrm{H_{en+1}} + \mathrm{H_{exn+1}} + \mathrm{H_{bn+1}} + \mathrm{H_{mn+1}} \end{aligned} \tag{6-1}$$

Where:

$\mathrm{WSEL}$ = Water surface elevation at structure noted

$\mathrm{H_f}$ = Friction loss in pipe noted

$\mathrm{H_e}$ = Entrance head loss at structure noted

$\mathrm{H_{ex}}$ = Exit head loss at structure noted

$\mathrm{H_b}$ = Bend head loss at structure noted

$\mathrm{H_m}$ = 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 because of 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.

## 6-3 Drain Pipe

In a highway setting, a drain pipe is defined as the single pipe that is connected to a single inlet but the pipe does not cross under the majority of the width of the highway or ramp. The pipe typically is in the roadway shoulder or edge of the traveled way if there is no roadway shoulder. If one pipe is connected to an inlet that is connected to another downstream pipe, then the pipes in this system would not be drain pipes. This configuration is either a storm sewer or culvert pipe. See Figure 6-1 for an illustration of a drain pipe. For other slope or groundwater applications for drain pipe, see Section 8-2.1. The design of a drain pipe follows the same methods for storm sewer design. The inlet associated with the drain pipe would also follow the inlet spacing design in Chapter 5. Drain pipes shall have outlet protection if they are discharging to a slope.

## 6-4 Underdrain Pipe

In a highway setting, an underdrain pipe can be used to drain groundwater or subsurface flow and turn it into surface runoff. Groundwater, as distinguished from capillary water, is free water occurring in a zone of saturation below the ground surface. If an underdrain pipe was installed in an area to drain groundwater, the discharge flow rate from the underdrain pipe depends on many variables that span both hydraulic and geotechnical disciplines. These variables may include the effective hydraulic head over the underdrain pipe, the permeability of the soil layer where the underdrain pipe is installed and any soil layer(s) above the underdrain pipe, the slope of the underdrain pipe, the gradient of the groundwater, and the area and volume of the groundwater layer being drained by the underdrain pipe. Sometimes the underdrain flow rate could be significant, especially when the roadway is located next to a big hillside that has visible seeps or springs. Any underdrain pipe flow rate must be thoroughly investigated and included in the project's drainage design. The PEO shall work directly with the RHE and RME to determine the necessary steps and actions needed to determine the discharge rate from an underdrain pipe installation. This may require significant engineering analysis and time.

The design of an underdrain pipe follows the same methods for storm sewer design. The only difference is that the flow rate used for the calculations is the predicted flow rate from groundwater into the underdrain pipe instead of flow entering the system from roadway drainage. When an underdrain pipe is connected to a storm sewer system, the invert of the underdrain pipe shall be placed at or above the top of pipe inside elevation in the storm sewer system. This is to prevent flooding of the underdrain pipe.

There are two distinct methods for estimating the amount of flow in an underdrain pipe. One method to get a site-specific predicted underdrain flow rate requires the PEO to work with the RHE and RME (and maybe HQ Geotechnical Office). This method may require extensive geotechnical investigations, computer modeling, and a stamped geotechnical report. The second method for estimating the amount of flow in an underdrain pipe is to assume full flow from the underdrain pipe based on the underdrain pipe diameter.

Underdrain pipes that convert groundwater or subsurface flows into surface flows need to be included in the project's drainage design. Increased surface flows from underdrain pipes to the stormwater drainage system need to be designed for and included in the conveyance calculations and possibly the stormwater BMP designs. The increased surface flow from the underdrain pipe shall be discussed in the project's downstream analysis. In some cases, the increased surface flows may need flow control stormwater mitigation. The PEO shall consult with the RHE when installing, removing, or modifying underdrain pipes within the project. Underdrain pipes shall have outlet protection if they are discharging to a slope. Underdrain pipes shall not drain water from natural wetlands, constructed stormwater treatment wetlands, or other treatment BMPs unless specified in the BMP design guidelines in the *Highway Runoff Manual*.

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

## 6-5 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 Washington Administrative Code (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*.

## 6-6 Pipe Materials for Storm Sewers, Drain Pipe, and Underdrain Pipe

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 Section 8-2.4.

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 calculations during the preliminary design and when the pipe materials have not been determined, the PEO shall use a roughness coefficient of 0.013 for schedule pipes

24 inches in diameter or smaller. 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 Table 4-1. In the event that 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 Table 4-1.

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

# Chapter 7 Water Crossings

## 7-1 Introduction

This chapter covers the design requirements for water crossings on state highways over fish-bearing waters, in addition to HEC-18, HEC-20, and HEC-23 Volume 1 and Volume 2. See Chapter 3 for the design of non-fish-bearing culverts, and HEC-18, HEC-20, and HEC-23 Volume 1 and Volume 2 for the design of bridges over non-fish-bearing waters, unless local requirements dictate otherwise. 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 crossings on WSDOT highways from current WAC Hydraulic Code Rules; the current USACE, Seattle District, Nationwide Permit Regional Conditions; and the 2013 Federal Court Injunction for Fish Passage (Injunction). 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 7-8 for more information.

All fish-bearing water crossings within Washington State must meet the requirements of WAC's Hydraulic Code Rules and the requirements of the *Hydraulics Manual*, unless a deviation is approved by the State Hydraulics Office. In Water Resource Inventory Areas (WRIAs) 1 through 23, the design must also meet the requirements of the Permanent Injunction Regarding Culvert Correction. This chapter uses WDFW's 2013 *Water Crossing Design Guidelines* (WCDG) as reference (WDFW 2013). Other published manuals and guidelines may be used with the approval of the State Hydraulics Office and permitting agencies.

New bridges and culverts in fish-bearing waters must be designed to meet current fish passage standards and WAC to ensure that 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 at which fish are expected to move. This is best accomplished by a multidisciplinary team, including engineers, biologists, and fluvial geomorphologists. Biologists are essential for understanding the habitat needs of the fish that use the site, whereas geomorphologists are essential for understanding the reach- and basin-scale stream processes that provide habitat and influence the crossing design.

WSDOT and WDFW have cooperated in a Fish Passage Barrier Removal Program since 1991. PEOs can check the WSDOT fish barrier database or contact the HQ Environmental Services Office Stream Restoration Program to determine whether the project has any fish barriers within its limits and whether the crossing will need to be included as part of the project. WDFW also maintains a database of fish barriers statewide. All water crossings over fish-bearing waters shall be designed by the State Hydraulics Office or by the Stream Team approved by the State Hydraulics Office (see Chapter 1).

Section 7-2 discusses requirements for assessing and documenting existing conditions to design a successful and fish-passable water crossing. Sections 7-3, 7-4 and 7-5 discuss the design process, considerations, criteria, and required scour analyses. Section 7-6 discusses the structure-free zone (SFZ). Section 7-7 provides guidance on temporary diversions, Section 7-8 describes the WSDOT monitoring process, Section 7-9 explains the performance management process, and Section 7-10 presents a discussion of additional resources. Section 7-11 provides the appendices.

This chapter uses the term "Stream Team" to denote work that either the State Hydraulics Office or the individual approved by the State Hydraulics Office performs and to separate that work from the work that the PEO would do in the rest of the *Hydraulics Manual*. At a minimum, the Stream Team consists of a stream design engineer, geomorphologist, and biologist who are leading or directly overseeing the work of other Stream Team staff. Minimum requirements for the stream design engineer include a Professional Engineering license in Washington and 2 years of design or construction experience in similar projects. The biologist shall be an aquatic or fisheries biologist with a minimum of 2 years of experience with similar projects, at least 1 year of which must be design experience and 1 year of construction experience. The geomorphologist must be a Licensed Geologist or Professional Engineer in Washington and have a minimum of 2 years of design and construction experience with similar projects. This chapter assumes that the stream designer or Stream Team 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 outlines the process that the State Hydraulics Office follows in designing a stream crossing, and what is expected on WSDOT projects. These designs require a specialty report. Additional requirements about specialty reports are provided in Chapter 1. The template used by WSDOT can be found on WSDOT's Hydraulics website along with training required to author a specialty report for a water crossing over fish-bearing waters. There is also a report checklist that outlines areas of focus during the specialty report review.

An FPSRD certificate number is required for all authors of any portion of a specialty report (including all members of the Stream Team). See Table 1-1 for a list of specialty reports and other requirements. An FPSRD certificate number is given to those who have viewed all of the training modules and successfully passed the comprehensive exam. Additional information, training resources, and the point of contact for this training can be found on the WSDOT Hydraulics Training web page. As WSDOT updates the FPSRD training modules a re-certification number is also required. Any updates to this training will be posted on the WSDOT Hydraulics Training web page.

A scour analysis is required for all WSDOT projects or WSDOT-managed infrastructure associated with scour or have a potential to be impacted by scour, such as water crossings, walls, roadway embankments, and other WSDOT infrastructure. A WSDOT *Scour Certification Record* number is required for all Stream Team members who are conducting scour calculations, lateral migration, scour analysis, and reviews as part of or supporting specialty reports. See Table 1-1 for a list of specialty reports and other requirements. A *Scour Certification Record* certificate number is given to those who have viewed all the WSDOT Scour Training Workshops and FHWA Bridge Scour Workshop Recordings; completed NHI Course 135046, *Stream Stability and Scour at Highway Bridges*, and NHI

Course 135048, *Countermeasures Design for Bridge Scour and Stream Instability*; and successfully passed the comprehensive exam. Additional information, training resources, and the point of contact for this training can be found on the WSDOT Hydraulics Training web page. As WSDOT updates the Scour Training modules a re-certification number is also required. Any updates to this training will be posted on the WSDOT Hydraulics Training web page.

The following training courses are required to obtain a scour certification:

- FHWA Bridge Scour Workshop Recordings
- NHI Course 135046, *Stream Stability and Scour at Highway Bridges*
- NHI Course 135048, *Countermeasures Design for Bridge Scour and Stream Instability*
- WSDOT 2023 Scour Training

Table 7-1 defines the design component of the stream channel that the individual members of the Stream Team, at a minimum, are responsible for in the design of fish-passable water crossings.

**Table 7-1** Stream Team Responsibilities

| Design Component | Stream Design Engineer | Geomorphologist | Biologist |
|---|---|---|---|
| Site assessment | ✓ | ✓ | ✓ |
| Watershed assessment | ✓ | ✓ | ✓ |
| Fish resources and habitat assessment | | | ✓ |
| Hydrology | ✓ | | |
| Hydraulic analysis | ✓ | | |
| Fish passage design | ✓ | | ✓ |
| Streambed design | ✓ | ✓ | ✓ |
| Habitat features | ✓ | ✓ | ✓ |
| Scour analysis | ✓ | ✓ | |

SRH-2D hydraulic modeling training is required for all WSDOT projects or WSDOT-managed infrastructure that requires hydraulic modeling as part of the hydraulic design process. Hydraulic modelers are required to obtain a training certificate from NHI for attending Course 135095, *Two-Dimensional Hydraulic Modeling of Rivers at Highway Encroachments*. Other equivalent SRH-2D hydraulic modeling training requires approval by the State Hydraulics Office.

## 7-2 Existing Conditions

The first step to designing a water crossing is understanding the behavior of the existing system and identifying a reference reach. There is no comprehensive set of biological and physical predictive equations for stream restoration design. Therefore, a reference reach approach is needed. This approach in channel design uses a reference reach, which exhibits channel and habitat properties that are not highly altered from natural, background conditions. By mimicking the reference reach, the design channel will approach (though not duplicate) natural, pre-crossing stream behavior and habitat. A thorough investigation of the site and adjacent stream reach, its history, and any known problems shall be performed prior to the field visit and confirmed during the field visit. Before or during the first field visit, the Stream Team shall complete the following:

- Determine whether the project is within a FEMA-mapped floodplain.
- Evaluate the watershed conditions/land cover (past, current, and future).
- Investigate the type of soils that are in the watershed and the underlying geology and consider how they might affect conditions and processes at the crossing.
- Look at historical aerial photographs and LiDAR for evidence of lateral migration of the channel, avulsion, debris flows, sediment pulses, LWM interactions, significant erosion, etc. Assess general watershed morphology and potential sediment sources using LiDAR, geologic maps, hazard maps, and other resources. Consider the location of the site within the context of watershed morphology and related processes.
- Discuss site history with the local agency and WSDOT area maintenance, specifically noting quantities of dredging, if available, scour repairs, and flooding.
- 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, transport investigations, etc.
- Review aerial photographs, topographic and survey maps, and previous watershed analyses for potential reference reach locations.
- Through site visits, the Stream Team will perform the following:
    - Determine the reference reach
    - Measure 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 are required, both before and after the survey has taken place, to ensure that all the necessary features are surveyed. The Stream Team 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 Team 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 *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. For example, watershed conditions have an impact on sediment yield to the site.

Historical and current aerial photographs shall be examined to determine what type of land cover the watershed has now and how that has changed over time. GIS layers are also available for displaying and approximating the areal extent of land cover types. Verifying whether the system is in an urban setting, within an urban growth area, or in an actively managed 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 how the watershed has changed over time will help the Stream Team create a successful crossing. Clearcut timber harvest, land conversion to agriculture, road building, bank hardening, log jam removal, stream relocation, and channel dredging are examples of watershed- and reach-level alterations that are likely to have occurred prior to the earliest available aerial photography. It is thus important for the Stream Team to find imagery dating as far back as they can find and to consider the impacts to the stream. Imagery dating back to the 1950s is often obtainable and shall be used when available.

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.

### 7-2.2 *Geology and Soils*

The soil types in the drainage basin not only assist the Stream Team 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 used to determine design flood events.

The surrounding geology will have an impact on susceptibility to mass wasting, and lateral migration and may influence where a new crossing is placed. It may also influence sediment load and size distribution in the channel, as well as long-term degradation (LTD). Generalized soil types may be found in soil surveys produced by NRCS. Surficial geology maps are also useful in determining soil information.

The Stream Team shall coordinate with the project geotechnical engineer while the specialty report is being authored and update the report as more geotechnical information becomes available. The WSDOT *Design Manual*, Chapter 800, provides additional information on coordination expectations.

### 7-2.3 *Fluvial Geomorphology*

Fluvial geomorphology is an integral part of determining where the crossing shall 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. Because the reach- and watershed-scale geomorphology is not the same for every site, failure to include an in-depth reach assessment of a stream or river may result in an inappropriate crossing design, requiring performance management.

The channel shall be examined to determine if there are signs of lateral and vertical stability or instability, the potential for changes in the base level, and how the stream may be impacted in the future. Delineation of channel migration zones (CMZs) shall be investigated (and may be required by local jurisdictions). The potential for channel avulsion shall also be assessed. Primary topics for analysis to determine the natural, geomorphic characteristics of a stream to appropriately design a water crossing include channel geometry, channel processes, lateral migration, and vertical stability. The analyses are informed by desktop review and site visits; the entirety of this process is referred to as a reach assessment and is further described in this section.

#### 7-2.3.1 Channel Geometry

Stream channel geometry is the combination of channel form in plan view, cross-section, and channel slope. Channel geometry is highly variable in undisturbed streams. In addition, streams have often been straightened or moved, simplifying channel geometry and 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 and may depict the stream alignment prior to roadway construction. Historical aerial photographs 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 helps identify grade breaks beyond the limits of the survey. LiDAR can also identify relic channel features, such as side channels, previous channel flow pathways, scroll bars, avulsions, and alluvial fans.

Many WSDOT roads were built along alluvial fans or at the edge of stream and river valleys. As a result, it is not uncommon for the roadway prism to have been built at a slope break or transition zone within the stream reach. This often leads to a historical slope that is 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 shall 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 shall 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.

### 7-2.3.2 Continuity of Channel Processes

WSDOT water crossings are designed using a reach-based approach to allow for continuity of channel processes such as the natural movement of water, sediment, wood, and aquatic organisms. This requires investigating the system as a whole, rather than focusing only on the channel corridor near the roadway. As part of the system evaluation, defining an appropriately sized channel corridor within a water crossing is essential for sustaining natural river function. A variety of techniques and tools are used to assess the continuity of natural channel processes. The Stream Team shall make sure to consider if the selected methodology fits or is appropriate and to make sure to include the surrounding constraints of the site. The Stream Team shall perform a meander belt assessment, and shall determine and document if a CMZ or other process is appropriate to include in the assessment. The combination of methods used for the final determination will be unique to each water crossing to account for site-specific variations and the data available. These assessments balance economic, social, and environmental values while also assisting WSDOT to understand future potential hazards posed by changes in a system due to natural channel processes, construction, or removal of infrastructure in the watershed and climate. Allowing continuity of channel processes also assists WSDOT with continuing to design sustainable, resilient, and reliable transportation networks for the traveling public.

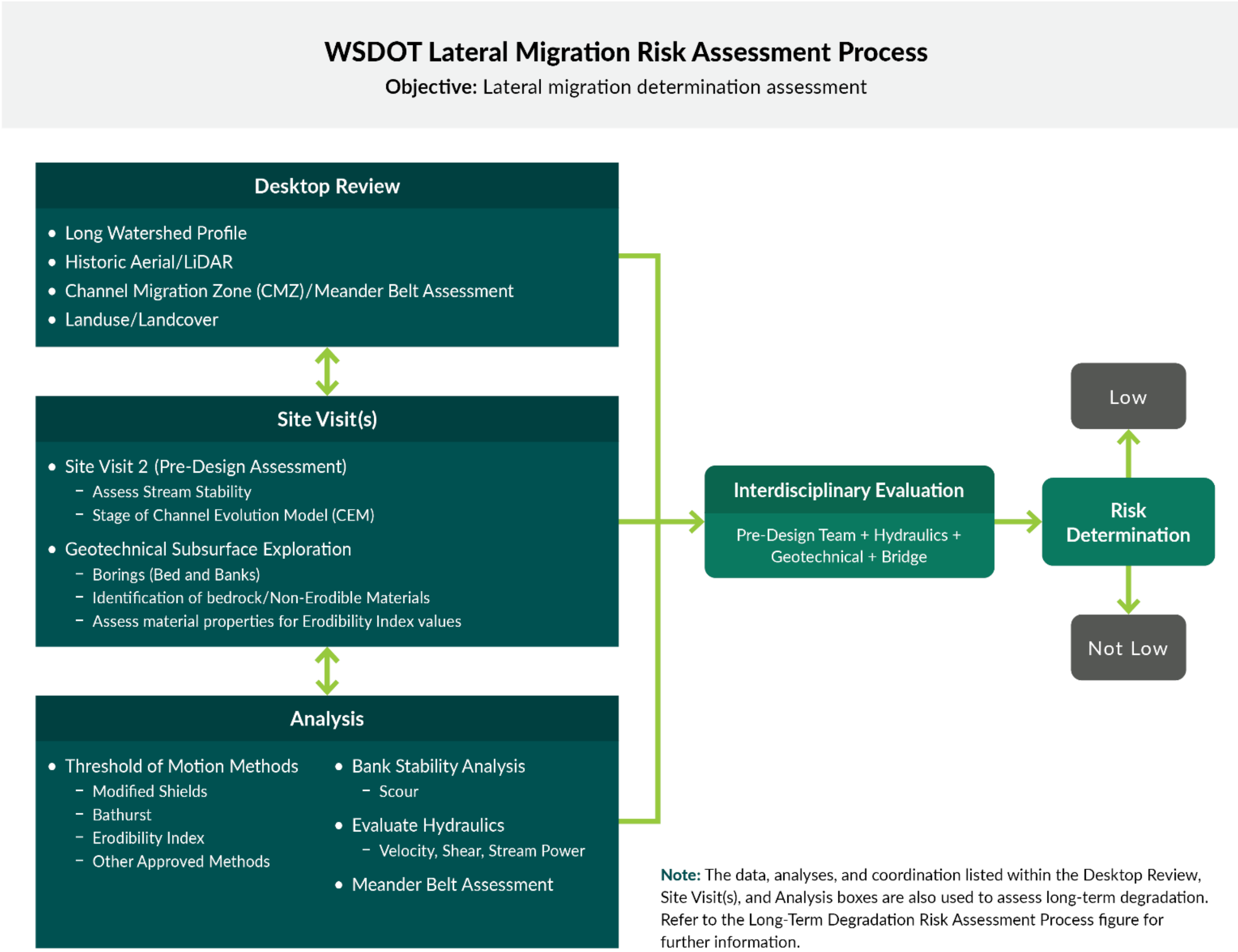
The following information is provided to assist project teams in considering continuity of channel processes in the design of water crossings. Future updates of this *Hydraulics Manual* will cover these topics in greater depth. Please check with the State Hydraulics Office for additional guidance.

1) As stated in Section 7-1, the Stream Team shall include an interdisciplinary team of hydrologists including a stream design engineer, geomorphologist, and biologist; the Stream Team shall also coordinate with the project geotechnical engineer. A desktop exercise shall be completed prior to a site reconnaissance (step 2) to determine available data, including existing reports, current and historical aerial imagery, LiDAR, existing topographic data, existing geologic information, and existing geotechnical investigations.
2) The Stream Team conducts a site reconnaissance to investigate the project reach, including documenting site-specific controls, constraints, and other information required in the specialty report.
3) The Stream Team selects the most appropriate methodologies to evaluate the continuity of natural channel processes of the stream system. Results of analyses/evaluation are documented in detail including assumptions and recommendations.
4) Meet with the State Hydraulics Office to discuss how various channel corridor widths based on the results of the analysis/evaluation may affect water crossing SFZ and general potential project impacts, and determine how to proceed. WSDOT applies professional judgment at step 4 with the information provided by the Stream Team in step 3.
5) Document the decisions that were made in step 4 in the specialty report.

### 7-2.3.3 Lateral Migration

The Stream Team shall assess lateral migration in the initial stages of design. All structure foundations shall be designed to account for the lateral migration expected to occur over the life of the structure. This does not require the full span of CMZs, but requires all structural elements to be designed considering the appropriate risk to lateral migration and for the structure to allow natural channel processes to the extent practicable. Lateral migration risk to water-crossing structures are classified as “low” or “not low.” Lateral migration risks shall be considered “not low” for all water crossings unless a detailed lateral migration risk assessment process is conducted and results in a determination that the risk for lateral migration to the structure is low and the determination is approved by the State Hydraulics Office. The process of determining lateral migration risk at water-crossing structures is illustrated below in Figure 7-1, including the necessary data, analysis, and coordination required. The determination is ultimately informed by data collection, site observations, and analysis, but most importantly by an interdisciplinary evaluation among the design, hydraulic, geotechnical, and bridge teams. The risk analysis shall consider risk during the expected project design life, which typically is 75 years. The flow chart is not meant to be exhaustive in analytical methods, data sources, or coordination across disciplines. Refer to the WSDOT *Design Manual*, Chapter 800, for additional information regarding interdisciplinary coordination.

**Figure 7-1** WSDOT Lateral Migration Risk Assessment Process




Note: For water crossing design projects, the Hydraulics team is the Stream Team.

### *7-2.3.3.1* Desktop Review

Prior to the site visit, a desktop review of available information shall be conducted for the purpose of conducting a qualitative geomorphic assessment of channel stability. The desktop analysis is intended to review factors that influence channel stability and identify additional data that shall be collected during the ensuing site visit. Desktop review includes review of historical imagery and elevation data, a meander belt assessment, or CMZ delineation and review of land use/land cover in the watershed, each of which is described in the following paragraphs. A longitudinal profile shall also be developed to assist with overall analysis of channel stability; the profile can be used to help assess lateral migration in some cases, but pertains more to vertical stability analysis. Refer to Section 7-2.3.4.1 to read a description of longitudinal profile development.

#### 7-2.3.3.1.1 Historical Aerial Photos and Elevation Data

Review of historical aerial photos and elevation data is the foundation of the desktop analysis and is used to quantify change over time to channel planform, profile, and watershed characteristics. The objective of reviewing the historical maps, elevation data, and aerial photographs is to understand channel migration within the current climatic regime. Reconstructing historical channel processes informs trends in future channel movement that may not be reflected in the historical record. Common sources for topographic elevation data and aerial photos include:

- Historical maps:
    - USGS Historical Topographic Maps (historical quad maps)
    - University of Washington River History (T sheets and survey plats)
    - BLM GLO Maps (survey plat maps, note these vary in quality)
    - As-builts or ROW maps
    - Others
- Elevation data:
    - Washington State Department of Natural Resources LiDAR Portal
    - Puget Sound LiDAR Consortium (PSLC)
    - U.S. Interagency Elevation Inventory
    - As-built data or survey from original construction
    - Others
- Aerial photos:
    - University of Washington River History (1930s-era aerial photos)
    - USGS Earth Explorer
    - USDA National Agriculture Imagery Program (NAIP)
    - Department of Ecology Coastal Atlas (obliques for shorelines)
    - Others

Review of aerial and elevation data for small streams with dense canopy cover can be challenging as the stream alignment is not readily identified from aerial photos. In this instance, information regarding lateral migration potential will be ascertained primarily from a detailed site visit, which is described in the following section.

#### 7-2.3.3.1.2 Channel Migration Zone/Meander Belt

A meander belt and/or CMZ delineation shall be conducted to characterize how the channel planform has changed over time—specifically, identification of channel meanders and how they have spatially varied over time in the vicinity of the project (both upstream and downstream). This analysis typically involves review of historical maps, aerial photos, and elevation data and digitizing bank location and channel centerlines at multiple dates to identify change over time. Where a smaller stream drains into a larger river, the river may require a CMZ delineation because it acts as the local base-level control for the small stream. CMZ delineations shall be conducted using historical maps, elevation data, and aerial photographs that go as far back as possible, i.e., at least over the last 100 years depending on data availability. Detailed methodology is not described in this document. Additional information, can be found in, but is not limited to, the following publications:

- HEC-20 Chapter 6.3
- Washington State Department of Ecology: Channel Migration Toolbox (Ecology 2014)
- Washington State Department of Ecology Screening Tools for Identifying Migrating Stream Channels in Western Washington: Geospatial Data Layers and Visual Assessments (Ecology 2015)
- Washington State Department of Ecology: A Framework for Delineating Channel Migration Zones (Ecology 2003)
- NCHRP Report 533: Handbook for Predicting Stream Meander Migration (NCHRP 2004)
- HEC-16

#### 7-2.3.3.1.3 Land Use/Land Cover

Aerial imagery shall also be reviewed to understand how the land use/land cover within the upstream watershed has changed or is expected to change. Land use/land cover is directly correlated to runoff rates as well as sediment supply, and large-scale changes can significantly impact both, ultimately impacting stream stability. For example, forest fires and silviculture can lead to increased peak flows and sediment supply as a direct result of loss of vegetation. Another common trend is associated with increased development/urbanization in a watershed, which will lead to increased peak flows and a decrease in sediment supply. Most streams and rivers in Washington that have experienced change because of anthropogenic influences likely started adjusting many decades ago with the arrival of the first European settlers. Therefore, it is important that the Stream Team understands that the record of available imagery may not reflect a stream or river’s extent of adjustments, and the Stream Team shall strive to find aerial imagery dating back as far as possible (e.g., 1950s) and understand that those images may not represent a “natural” condition. In addition to review of aerial photos, land use/land cover information can be determined from the National Land Cover Database (NLCD), which provides digital land cover data beginning in 2001. The NLCD data sets include land cover and impervious surface as well as tools for conducting comparisons between data sets. See Section 7-2.1 for additional discussion.

### *7-2.3.3.2* Site Visits

After the desktop review has been conducted, on-site investigations shall be conducted by both the Stream Team and geotechnical team. These on-site investigations are used to confirm, validate, or correct the assumptions established from the desktop review such as locations of control structures, any headcuts or knickpoints, etc. These visits may or may not be conducted at the same time. Early coordination among the teams is recommended if possible. The following paragraphs describe the data and observations that shall be collected in the field.

#### 7-2.3.3.2.1 Stream Site Visit

A site visit by the Stream Team is necessary to identify fluvial and geomorphic factors that influence stream stability as well as information to support the design of the proposed structure, which includes BFW measurements and pebble counts to characterize the streambed material gradation. See Chapter 2.3 of HEC-20 for an additional summary of the geomorphic factors related to stream stability. The site visit shall be conducted both upstream and downstream of the crossing. This site visit is conducted during the PHD phase. During the site visit, the Stream Team shall make observations regarding bank stability, lateral stability, and vertical stability. Observations related to bank and lateral stability are the most applicable to determine the lateral migration risk; however, vertical stability shall not be discounted and also needs to be considered during design. Observations shall be recorded with site notes, sketches and photographs, and locations captured on a field map or with a Global Positioning System (GPS) unit. HEC-20 provides more specific data regarding collection and example field forms are included in Appendices B, C, and D.

A Channel Evolution Model (CEM) is a qualitative method that can be used to predict how alluvial channels respond to changes involving lowering base level, incision, and alterations to hydrology and sediment supply. Field observations can be used to determine the current stage of channel evolution and stability. Once the current channel evolution stage is identified, the CEM can be used to identify expected responses of the channel as it progresses toward a stable configuration through predictable stages. Channel responses may include incision, channel widening, and bank erosion before arriving at a stable configuration. An example of a CEM is the model developed by Cluer and Thorne (2013). Please also see Castro and Thorne (2019) and Powers et al. (2019) for additional CEMs. It shall be noted that CEMs are not appropriate for bedrock channels or recently engineered reaches.

#### 7-2.3.3.2.2 Geotechnical Subsurface Exploration Site Visits

Geologic site reconnaissance shall be conducted by the geotechnical team to observe site conditions, including the extent and character of exposed soil units, and the condition of the roadway, bridge, channel banks, and embankment slopes. The exploration typically includes test borings conducted from the roadway and laboratory testing of selected samples retained from the test boring. Borings also identify if bedrock is present at the site and at what depths.

This information is typically summarized in a geotechnical scoping memorandum. The scoping memorandum also includes a summary of published geologic and soil data and a summary of historical borings in the project vicinity. Recommendations for hydraulic considerations, specifically regarding LTD, contraction scour, and local scour, are also included in the memorandum. It is critical that coordination between the geotechnical engineer and the stream designer or Stream Team is conducted early and ongoing through the design. The WSDOT *Design Manual*, Chapter 800, describes this coordination process. Pertinent parameters provided include a summary of HEC-18 Soil Type (Cohesive or Cohesionless), HEC-18 Erodibility Index (Low, Medium, High), and a median particle size ($D_{50}$) for the various stratigraphic units identified during the reconnaissance.

### *7-2.3.3.3* Analysis

Once the desktop review and site visits have been completed, detailed analysis can be performed using the collected information coupled with the results of hydraulic modeling. Analyses include the following:

- Threshold of motion
- Bank stability analysis
- Hydraulic analysis (modeling)
- CMZ/meander belt assessment

#### 7-2.3.3.3.1 Threshold-of-Motion Analysis

A threshold-of-motion (incipient motion) analysis is used to determine if a sediment particle of interest will mobilize under specific hydraulic conditions. For example, this analysis could determine if a particle of interest is mobilized during a specific flood event. Alternatively, it could be used to determine what hydraulic forces would be required to mobilize a particle of interest. Common methods used include the unit discharge method (Bathurst 1987), which identifies a stable $D_{84}$ particle size given a flood event of interest. This method is typically used for channels with gradients over 4 percent. For shallower slopes, the modified Shields approach (USDA 2008) is used to determine sediment mobility. WSDOT is currently working to incorporate another method of assessing the threshold of sediment transport and scour (the erodibility index) based on the work presented in HEC-18 and Annandale (2006). This work will be included in the next *Hydraulics Manual* update.

#### 7-2.3.3.3.2 Bank Stability Assessment

A Bank Stability Assessment considers if the toe of the bank is susceptible to scour given the hydraulic conditions and geotechnical properties of the streambank material. Bank failure occurs when the bank height exceeds the critical bank height for geotechnical slope stability. This assessment is meant to be qualitative in nature, using the site observations, CEM stage, bank material properties, and local hydraulics present at the bank to make an informed judgment about bank stability. More detailed methods exist for quantifying bank stability, such as the Bank Stability and Toe Erosion Model (BSTEM) (Simon et al. 2009), or sediment transport modeling, but these would require approval from the State Hydraulics Office before being used for assessment of bank stability.

#### 7-2.3.3.3.3 Evaluate Hydraulics

Pre- and post-project hydraulics shall be assessed and compared with the use of an SRH-2D hydraulic model. See Section 7-1 for further detail regarding WSDOT's hydraulic modeling requirements. Other modeling platforms or 1D modeling may be appropriate; however, they would require the approval of the State Hydraulics Office prior to being used. 2D modeling is required, as it provides more refined hydraulic results at locations of interest including flow and velocity distribution, WSELs and depths, shear stress, velocity magnitude, and direction.

Post-project hydraulics shall be reviewed for areas of high shear, stream power, and velocity, as these areas often are prone to erosion and scour. These hydraulic conditions are commonly located at the outside of bends. Often when a proposed project is replacing an undersized structure with a larger opening, the backwater upstream is eliminated, resulting in increases to shear and velocity upstream, and may mobilize material that had aggraded upstream because of the backwater.

An advantage of the 2D hydraulic model is the ability to predict flow patterns and velocity direction. Velocity vectors shall be reviewed at the proposed crossing and can be used to identify areas of contraction/expansion as well as determine the angle of attack on proposed structures. Velocity vectors entering channel meanders can be reviewed to provide an estimate of direction of potential lateral and down-channel migration paths.

#### 7-2.3.3.3.4 Meander Belt

See 7-2.3.3.1 for discussion on meander belt assessment. Results of the hydraulic analysis can be used to confirm assumptions used in the amplitude assessment.

### *7-2.3.3.4* Interdisciplinary Evaluation

Once the desktop review, fieldwork, and analysis have been completed, an interdisciplinary evaluation shall be conducted that includes members of the predesign, geotechnical, hydraulic (or Stream Team), and bridge teams to present the results of the site visits and analysis and ultimately determine the lateral risk on a project basis per the guidelines in the WSDOT *Design Manual*, Chapter 800.

### 7-2.3.4 Vertical Stability

Vertical stability must be assessed in the initial stages of design, specifically a longitudinal profile analysis (Section 7-2.3.4.1) prior to the initial site visit. It is important to understand the history and processes affecting the stream's longitudinal profile (Section 7-2.3.4.1). Events such as forest clearing, loss of instream wood, dams, beaver removal, urbanization, changes in peak flows, and uplift, along with other factors can have and have had a major impact on the overall stability of streams in the Pacific Northwest. Processes taking place at different time scales (geologic versus human) and spatial scales (watershed versus reach versus site) could affect the project's success. Identifying and understanding causal factors and related stream adjustments are necessary when designing robust and resilient instream projects, and shall be part of any engineering design analysis (Skidmore et al. 2011).

The “goal” of a river is to move sediment, debris, and water at a minimal expense of energy. To this end, the stream will smooth the longitudinal (or simply “long”) profile as much as possible. The long profile shape (usually convex downward) reflects the adjustment of the river to (1) the climate of the watershed (current and past), which controls the amount of runoff; (2) the tectonic setting of the watershed, which controls its overall relief as well as changes in base level; and (3) the geology of the watershed, which controls sediment supply and the bedrock’s resistance to erosion.

Tectonic activity and climate are not static phenomena, and bedrock is spatially variable. In addition, it takes time for a river to complete the job of adjusting its profile to these independent variables. Because of this, longitudinal profiles are in constant readjustment or dynamic equilibrium, never quite catching up to the changes that affect them (Mount 1995). Under natural, background conditions, the longitudinal profile of a river is in slow, constant adjustment to watershed conditions. Profiles are convex downward in shape with a steep gradient at the head and a low gradient at the mouth. Variations in the shape of profiles reflect the response of the river to the overall tectonic, climatic, geologic, and base level conditions. Changes in these conditions can produce regional shifts in profiles involving widespread river aggradation or incision to reestablish the ideal shape.

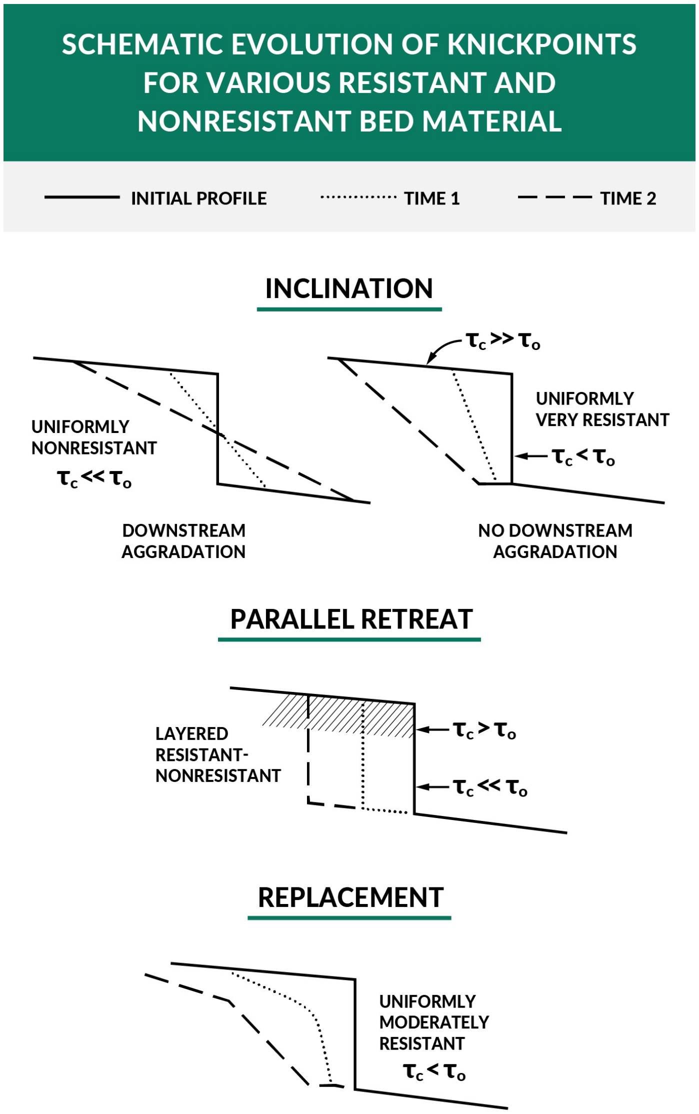
Rivers are constantly adjusting to local perturbations in their profile. Knickpoints are abrupt changes in stream gradient, and are often nearly vertical. However, they can also be less abrupt, and are sometimes call “knick zones.” In either case, the abrupt change is the stream’s response to a drop in a base level. The base level is a control on stream incision, and can be standing water—a wetland, lake, reservoir, or ocean—or it can be a resistant substrate. Downstream barriers or infrastructure shall not be considered a base-level control for the duration of the design life of a structure. In the case of the latter, bedrock is the ultimate base level control on the human time scale. On a larger time scale, bedrock is eroding, and depending on the strength of bedrock, incision can be relatively fast. Other types of substrate-related base level controls include log jams and boulder clusters. These types of base level controls are considered transitory, and can change during the human lifespan time scale.

Exactly how and how fast a knickpoint retreats in the upstream direction is highly specific to stream substrate and channel geometry (Gardner 1983). There are several styles of knickpoint retreat; these are illustrated in Figure 7-2. Parallel retreat can occur when a relatively resistant layer at the streambed surface is underlain by a weaker layer. The upper layer in this case gets undermined by the erosion of the weak layer, and collapses, allowing the process to begin all over again at a point upstream of the prior knickpoint location. Alternatively, if the substrate has a uniformly nonresistant material, the knickpoint can rapidly adjust profile by a combination of erosion upstream and deposition downstream. If material is uniformly resistant, the knickpoint is more persistent, with its slope decreasing gradually over time and almost no downstream aggradation. Slope replacement is another type of knickpoint evolution, in which the initial knickpoint changes by lowering in elevation but taking on a lower slope on the downstream side, and a steeper slope on the upstream side.

When assessing a stream for a new crossing, it is important to anticipate knickpoint migration and its implications for the new stream crossing. This may entail reconnaissance far downstream from the roadway. If necessary, survey may be needed to tie in a knickpoint that was observed. To understand the risk of knickpoints to a new crossing, the substrate must be examined and a knickpoint evolution model must be chosen based on professional judgment. If the knickpoint is relatively distant from the crossing, it may not pose a threat during the project design life. However, if there is evidence of rapid retreat of a knickpoint, even a distant knickpoint may pose a risk, particularly if the style of retreat is parallel.

Culverts that are replaced to provide fish passage often have served as grade control for 50 to 100 years. Removal and/or replacement of these grade control structures can set off a cascade of effects that negatively impact the habitat and passage that a project seeks to improve if the design does not account for the stability of the system. This instability can cause floodplain disconnection, loss of backwater and side channel habitat, increased levels of turbidity, and channel (and thus habitat) simplification. Evaluation of both the stage of stream evolution and a longitudinal profile analysis can help determine if morphologic grade control (Castro and Beavers 2016) is warranted, and if so, what type of structure is most geomorphically appropriate. Potential structures include placement of large wood and roughness elements, constructed riffles, step-pools, and cascades.

**Figure 7-2** Styles of Knickpoint Evolution

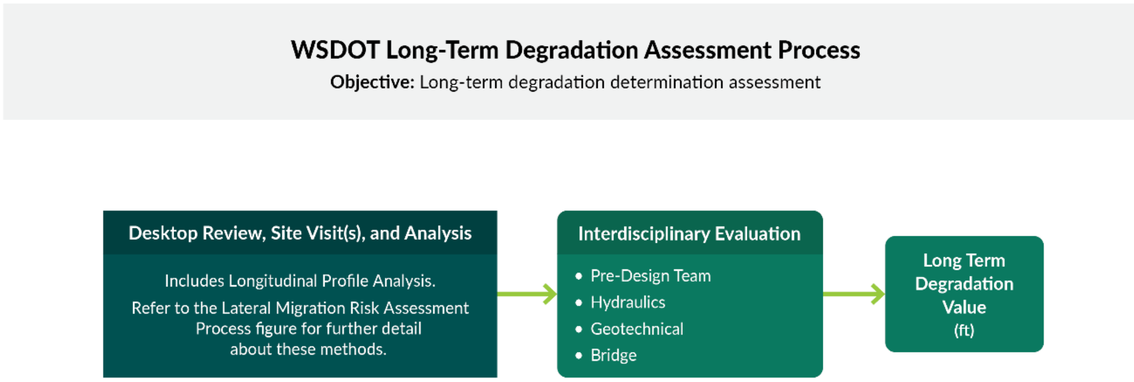



Adapted from Gardner 1983, where $\tau 0$ = bottom shear stress and $\tau c$ = critical shear stress needed to initiate motion.

Vertical stream stability shall be evaluated and documented in the specialty report for all WSDOT road/stream crossings to determine if morphologic grade control is necessary, if additional freeboard due to aggradation risk is required, and to estimate the LTD component of total scour. Similar analyses performed to assess lateral migration are also used to assess vertical stability; refer to Figure 7-3 for the long-term degradation assessment process, and to Sections 7-2.3.3.1 through 7-2.3.3.4 for a discussion of the applicable assessments and interdisciplinary coordination among the design, hydraulic (Stream Team), geotechnical, and bridge teams. Refer to the WSDOT *Design Manual*, Chapter 800, for additional information regarding interdisciplinary coordination.

A longitudinal profile is the primary tool used to assess vertical stream stability.

**Figure 7-3** WSDOT Long-Term Degradation Assessment Process




#### *7-2.3.4.1* Longitudinal Profile Analysis

A longitudinal profile is the elevation profile of a stream drawn along the length of the thalweg. A profile is plotted with elevation on the vertical axis and stationing along the horizontal axis. Typically, horizontal stationing is relative to a known point, for example, the distance from the mouth of the stream or confluence. Elevation data for the profile can be obtained from detailed topographic survey or LiDAR data, or they can be collected during a site visit. If multiple elevation data sets are available, consider displaying all data on the profile. Knickpoints identified through either fieldwork or topographic analysis must be included in the longitudinal profile analysis. Downstream infrastructure, as well as downstream knickpoints that can affect the proposed crossing during the design life of the proposed crossing, are required to be assessed in the initial stages of design. Similarly, upstream infrastructure that could be affected by the replacement of the proposed crossing during the design life of the proposed crossing are also required to be assessed in the initial stages of design. Once created, the vertical profile shall be reviewed for identification of slope breaks and discontinuities, existing grade control structures, and any headcuts or knickpoints. It is also helpful to include and label any other structures in the profile (e.g., culverts, bridges, dams, weirs, or bedrock features). If data are available they are required to include subsurface information provided by the geotechnical engineer. See Section 7-2.3.3.2.2 for additional information. It is not uncommon for other existing crossings downstream of a project to act as grade control. The longitudinal profile is a tool used to assess overall channel stability, and in some cases is also used in desktop review to determine lateral migration potential; see Figure 7-1.

Additional guidance on procedure and considerations for vertical stability will be provided in later iterations of this *Hydraulics Manual*. The Stream Team shall contact the State Hydraulics Office at the beginning of a project to determine if supplemental guidance is available for vertical stability.

### 7-2.3.5 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 provide a stream buffer that could recruit LWM. See Chapter 10 for additional information on how to document LWM in a reach assessment.

#### 7-2.3.6 Sediment

Sediment size in the reference reach is determined through Wolman pebble counts 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 is provided in scientific literature (e.g., Bunte and Abt 2001).

The sediment sampled shall be within the reference reach and a minimum of three samples is required. Note any large, naturally occurring material that is on site; it may not be appropriate to include the larger material in the gradation, but the material shall be noted within the design documentation. Depending on the stream regime, it may be appropriate to quantify all the larger material found on site. 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.

Where there is a natural streambed armor layer on the surface of the streambed, in addition to pebble counts, a sub-layer sample shall be used to capture the sediment size below the armored layer (see Section 7-3.8.3). For WSDOT projects, sampling below the ordinary high water level (OHWL) is allowed under General Hydraulic Project Approval. Work within the wetted perimeter may occur only during the periods authorized in the APP ID 21036 titled "Allowable Freshwater Work Times, May 2018." Work outside of the wetted perimeter may occur year round. For more information see the APPS website.

Samples collected below the OHWL must be documented in the current Hydraulics Field Report.

### 7-2.4 *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 is provided 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 buildout conditions may be available for use.

## 7-2.5 *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 State Hydraulics Office for additional guidance. Figure 7-4 depicts a flow chart that describes the steps below that shall be completed by an interdisciplinary team consisting of a hydraulics engineer, geomorphologist, and biologist.

### 7-2.5.1 Step A: Examine Adjacent Reaches

Examine the reaches with project resource co-managers and 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. Is there variability of geology, e.g., knickpoints, hard pan, or bank failure?
4. Are there anthropogenic features or other water crossings that impact the crossing within the project reach?
5. Are there any sudden changes in sediment size distribution?

In evaluating the project reach for the above points, the Stream Team 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 are an indication that sediment supply may be a concern, or that 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. Constructed 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, the channel shape shall be mimicked; in these instances, contact the State Hydraulics Office for additional guidance.

If the answer to any of the above questions is yes, proceed to Section 7-2.5.2. If the answers to all of the above questions are no, proceed to Section 7-2.5.3.

#### 7-2.5.2 Step B: Similar Reference Reach

If the adjacent reach is not representative, an appropriate watershed reference reach will need to be located. Locate the watershed 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 (plus or minus 20 percent) to the contributing area above the project reach?

If the reach meets criteria in item 2 above, go to Section 7-2.5.3. If it does not, look to adjacent watersheds with similar aspect, elevation, levels of development, and geology and follow the procedures in Step A for the location identified.

Prior to starting the stream design, the Stream Team must receive approval of the reference reach selection from the State Hydraulics Office.

#### 7-2.5.3 Step C: Reference Reach Data Collection

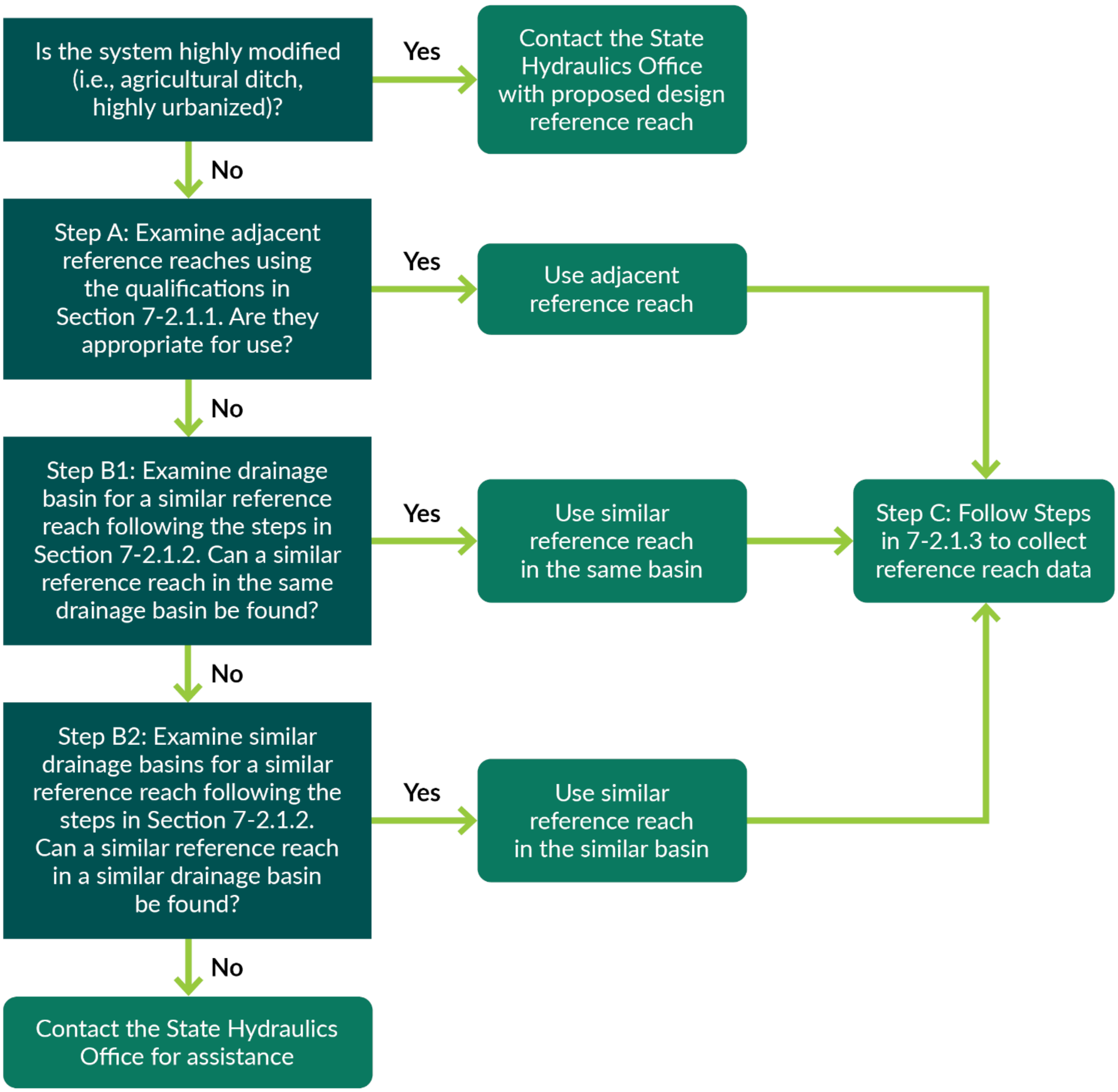
After locating an appropriate reference reach, collect data for the specialty report. At a minimum, collect the following information:

- Stage of channel evolution at the project reach (Cluer and Thorne [2013] evolution progression recommended)
- 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)
- Variability of sediment size throughout reach, i.e., armor layer, identification of largest size clasts
- Bank characteristics (i.e., height of banks, composition, cohesion, etc.)
- 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

#### 7-2.5.4 Project Constraints

Constraints in the project reach such as adjacent properties or railroads may limit the channel geometry, particularly the slope. In this case what would otherwise be the logical reference reach may not be suitable. In these cases, the Stream Team looks for a design reference reach that has the approximate slope of the project reach dictated by constraints. The process for design reference reach determination is similar to the reference reach process, but filtered by the parameter that is constrained (most likely channel slope). This process is outlined in Appendix 7A. If it is determined that a constraint is present requiring a design reference reach, contact the State Hydraulics Office for concurrence requirements for the use of a design reference reach.

**Figure 7-4** Reference Reach Determination



### 7-2.6 *Bankfull Width*

BFW is the most effective channel-forming flood event. Bankfull discharge is the flow at which the stream reaches BFW. Bankfull discharge occurs at a 1.2-year recurrence interval in western Washington and at a 1.5-year recurrence interval in eastern Washington (Castro and Jackson 2001). The bankfull discharge may be greater than the 2-year flood event for incised channels. Bankfull discharge may be exceeded multiple times within a given year. This may occur in a single flood event, or it might occur in different isolated flood events (Anderson et al. 2016).

An accurate BFW is critical. 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 (WAC 220-660-190). The Bankfull Width module of the FPSRD training provides guidance for measuring BFW for WSDOT water-crossing structures.

If there are significant differences between the measured and hydraulically modeled approximate BFW, further evaluation or justification will be required. The Stream Team shall verify that the channel hydrology is correct to the best of its knowledge, verify that the Manning's n values are appropriate for the crossing, and use engineering judgment as appropriate to ensure that the hydraulic model is accurate, and any differences are explained. Sites that are not typical shall be discussed with the tribe(s) and WDFW to come to an early understanding of the channel behavior.

In cases where BFW cannot be measured, regression equations provided in Castro 2001 shall be used to determine bankfull discharge that shall then be modeled to determine an estimate for BFW to be used for structure sizing in confined systems. Proposed channel width in these cases shall follow the process described in Section 7-3.4.

WDFW and Castro 2001 have developed a regression equation used for estimating BFW, which shall be used only as a check to determine what a reasonable measurement is on streams within the limitations of that equation.

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 flood 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 shall be checked to confirm that 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.

## 7-3 Design

This section covers the Bridge Design and Stream Simulation Design methodologies (Section 7-3.1). Other methods may be appropriate but must be approved by the State Hydraulics Office prior to use (Section 7-5).

The design flood event for WSDOT projects are listed in Table 7-2 below.

**Table 7-2** Flood Event for Hydraulic Design Elements

| Design Element | Flood Event |
|---|---|
| Structure freeboard | Scour design flood [a,b] |
| Structure foundation [c] | Scour design flood and scour check flood [b,d,e,f] |
| Scour countermeasure depth [g] | Scour check flood [b,d,f] |
| Scour countermeasure stability [c] | Scour check flood [b,d,h] |
| Scour countermeasure freeboard | Scour design flood [b,d,i] |
| LWM stability | 1% AEP (100-year) flood |
| Complex wood structures, flow deflectors, wood within a rock, and wood bank protection design | 2080 100-year projected flood [b] |
| Velocity ratio | 1% AEP (100-year) flood or the 2080 100-year projected flood [a,b] |
| Temporary bridges (freeboard and scour) [e, j] | 4% AEP (25-year) flood [e] |

**Notes:**

a. Discuss the impacts of structure size/impacts under climate predictions with State Hydraulics Office to determine how to proceed. PEO may need to be brought into discussion in case of low cover scenario. For tidally influenced areas, sea level rise shall also be taken into consideration. See Sections 7-3.5.4 and 7-3.5.5.

b. The 2080 100-year projected flood event shall be used for the design, unless the State Hydraulics Office has determined that the 2080 projected flood event is not practicable.

c. See the WSDOT *Bridge Design Manual* for more information on scour and how it pertains to structure foundations.

d. Collaborative discussion between Bridge and Structures Office, Geotechnical Office, State Hydraulics Office, and PEO to occur to determine risks and impacts and what is practicable.

e. For temporary bridges that will be in water for more than one season, use permanent structure design criteria.

f. Total scour shall be assessed for all flows up to the scour design flood and scour check flood events that results in worst-case total scour for each flood event.

g. Refers to location for toe of scour countermeasure.

h. Scour countermeasure stability shall be assessed for all flows up to the scour check flood that creates the greatest stresses on the countermeasure.

i. Scour countermeasures shall have 1 foot (minimum) of freeboard above the scour design flood. Scour countermeasures shall have 2 feet (minimum) of freeboard above the scour design flood when deep foundations have been designed to rely on the scour countermeasure.

j. For temporary bridges used only as work platforms or for construction equipment contact the State Hydraulics Office for additional guidance.

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

### 7-3.1 *Determining Crossing Design Methodology for Documentation*

The three most used design methodologies by WSDOT from WDFW's 2013 WCDG are the Unconfined Bridge, Confined Bridge, and Stream Simulation methodologies. 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 State Hydraulics Office. For all structures under 20 feet in width 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 State Hydraulics Office, the design process shall be documented as the process that was approved. See Section 7-5 for some other available methods and Appendix 7B for a summary of the necessary stream crossing elements and associated guidelines for the methodologies.

## 7-3.2 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 constructed or natural and, when encountered, shall be discussed with resource agencies, tribes, and 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 be resolvable within the scope of a project. There may be design constraints other than those covered in this section.

### 7-3.2.1 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.

Existing stormwater infrastructure is a key component to consider when determining stream gradient and grading impacts. Coordinate with the stormwater design engineer to verify that any changes in stream grade will not impact existing storm connections or ditches draining to the stream system. All stormwater discharges shall be placed above the 100-year WSEL.

### 7-3.2.2 Environmental Impacts

Environmental impacts shall 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 shall occur to determine the best design to move forward and whether mitigation (formal or informal) may be used in lieu of meeting requirements/recommendations. If impacts are temporary they may be more acceptable.

### 7-3.2.3 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 reaches. The vertical stability and historical profile can often be assessed through use of a longitudinal profile; see Sections 7-2.3.4 and 7-2.3.4.1. 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, or to produce a steeper reach with an overcoarsened channel. As much information as possible shall be obtained about historical conditions and the cause of the grade break and discussions with WDFW and the tribes shall occur to determine the best solution for the project.

### 7-3.2.4 Cultural Resources

Impacts to cultural resources shall 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 shall be consulted to determine the best way to proceed.

### 7-3.3 *Channel Alignment*

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 a covered stream shall be considered and the maximum angle of a bridge structure to the centerline of a roadway per the *Bridge Design Manual*, if a bridge structure is used. While the State Hydraulics Office does not typically recommend a structure type or layout, it is important for the Stream Team to know what this constraint is and keep it in mind while designing the layout to make an efficient crossing.

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 determined 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 State Hydraulics Office. If it is not practicable to meet the slope constraint, approval by the State Hydraulics Office is required.

If a channel is being realigned and the existing crossing is not abandoned or removed and is to remain in place and open, the Stream Team and PEO shall coordinate with the HQ ESO Stream Restoration Program to make sure that the crossing is not considered a fish barrier after the project is completed.

If allowing for natural regrade is determined to be desirable, the Stream Team must evaluate the LTD, 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 shall 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.

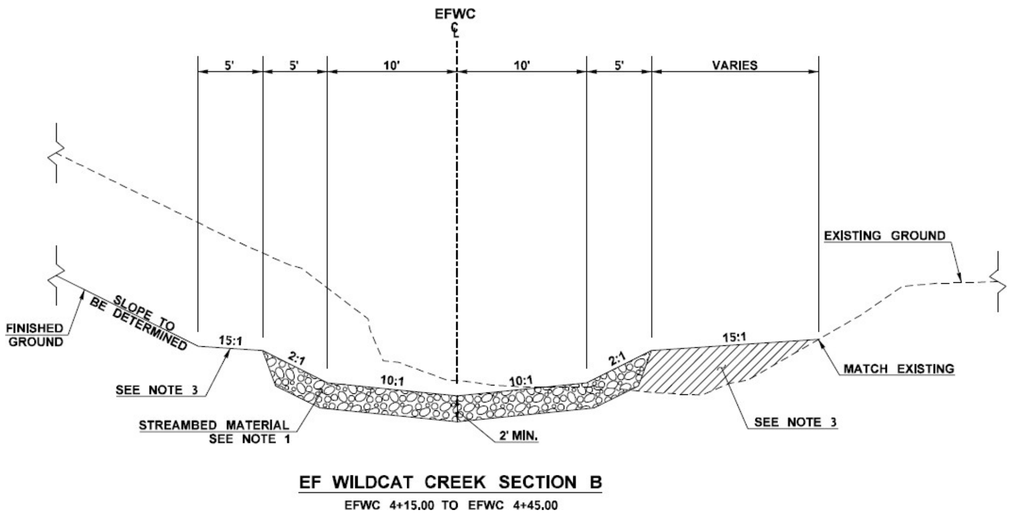
If regrade is determined not to be desirable, the reach must be designed to be stable. This may cause the project to be permitted as a fish passage improvement structure (see Section 7-5.2 and require long-term maintenance and monitoring. Additionally, extra consideration shall be given to bank integrity for these systems to help the water body dissipate energy. The Streambed Material Decision Tree found in Appendix 7A may help the Stream Team determine whether to allow for channel regrade.

### 7-3.4 *Channel Cross Section*

The channel cross section shall 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 State Hydraulics Office for assistance.

Cross-section lengths shall be rounded to the nearest 0.1 foot. Slope shall be rounded to the nearest 0.5:1. Example plans and plan requirements are provided in WSDOT's *Plans Preparation Manual*. An example cross section is illustrated in Figure 7-5. Natural channel cross sections are usually asymmetrical. However, these can be problematic to construct. Therefore, a symmetrical cross section like the one shown in Figure 7-5 is acceptable, knowing that the stream will self-adjust. A low-flow channel that connects habitat features is typically added during construction that will further help adjust the channel shape to something that is more natural and help encourage fish passage immediately after construction prior to the larger flows that shape the channel. In larger systems the main channel can migrate within its floodplain and, therefore, the floodplain width can vary. It may be desirable to describe that with different design cross sections.

**Figure 7-5** 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 flood event, the channel shall behave the same through the proposed graded reach.

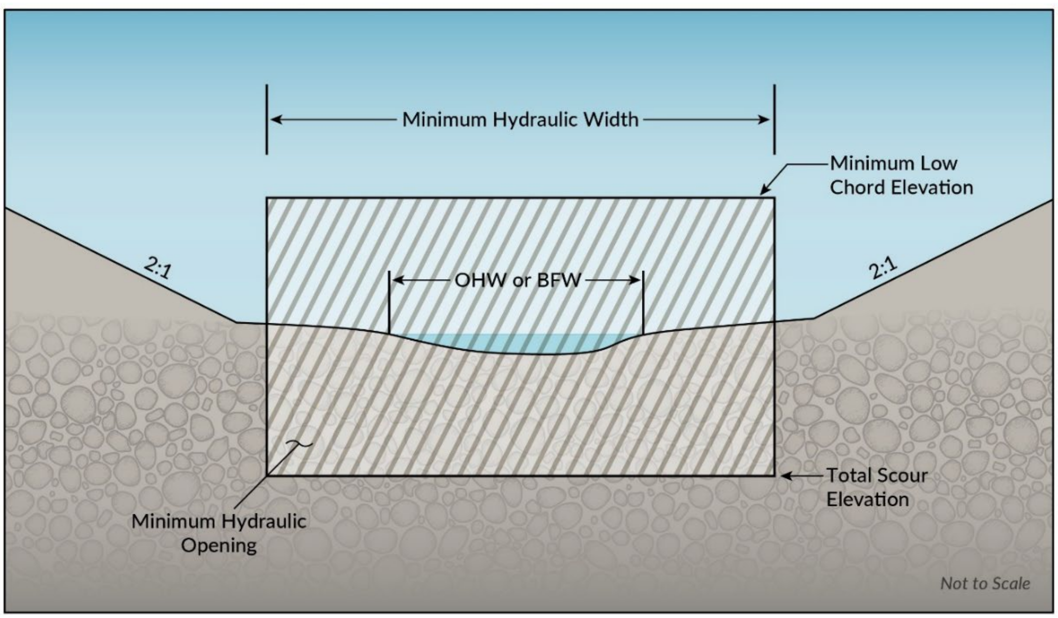
In crossings that serve a dual purpose for wildlife connectivity, consideration shall be given to whether the wildlife connectivity bench is to persist through the design life of the structure or a certain design event. If the wildlife connectivity bench is to remain stable, larger material or other means of bank stabilization may be necessary through the structure. The Stream Team shall coordinate with HQ ESO and the region to ensure that the proposed material will work with the wildlife for which that additional connectivity is provided.

### 7-3.5 *Hydraulic Opening*

For the purposes of this chapter, the minimum hydraulic width required by the specialty report and the hydraulic height defined by minimum low chord elevation and total scour elevation is defined as the minimum hydraulic opening (MHO). This section covers the hydraulic width portion of the definition. Freeboard and the maintenance clearance portion of the hydraulic height is covered in Section 7-3.6 and scour is covered in Section 7-4.1. The final SFZ determination made by the region in conjunction with the Bridge and Structures Office shall be, at minimum, the established MHO, but may be larger to include contextual needs (see Section 7-6). Any required scour countermeasure (Section 7-4.3) shall not encroach within the minimum hydraulic width and depth of scour. The depth of scour is determined as LTD + contraction scour at the scour check flood (minimum) or a minimum of 3 feet, whichever is greater, unless otherwise approved by the State Hydraulics Office and shall be set back horizontally far enough to establish planting as determined by the landscape architect. Coordination with a landscape architect is necessary to determine how far the countermeasure needs to be set back and maintain plant survivability. See the Plan Sheet Library for an illustration of the minimum structure width required by horizontal and vertical factors.

For preliminary plans, prior to the structure type being known, 2:1 cut slopes with a note that "grading limits to be based on final structure size, type and location" shall be 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 shall clearly depict where the minimum hydraulic width and MHO is, as shown in Figure 7-6.

**Figure 7-6** Minimum Hydraulic Width and MHO




There are three methods for determining the minimum hydraulic width: (1) stream simulation, (2) confined bridge, and (3) unconfined bridge. However, the process used for confined bridge is the same as that used for stream simulation with the exception that the confined-bridge method includes an additional factor of safety (FOS). All methods are dependent on the floodplain utilization ratio (FUR), which determines how confined a stream is. A meander belt assessment shall be conducted for all crossings. This information shall be used by the State Hydraulics Office to determine if there needs to be an increase in the hydraulic width based on the channel's ability to naturally meander through the crossing.

The hydraulic width shall not be less than Equation 7-1 (2013 WCDG, Equation 3.2) or Equation 7-2, unless otherwise approved by the State Hydraulics Office.

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

$$W_{HYO} = 1.3 * W_{bf} \tag{7-2}$$

where
$W_{HYO}$= width of hydraulic opening
$W_{bf}$= BFW

The minimum hydraulic width 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, as depicted in the SFZ Plans (see *Plan Sheet Library*).

### 7-3.5.1 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 flood-prone width (FPW) to the BFW. The FPW at a given location shall be divided by the BFW at the same location. The FPW and BFW must be measured in the same location along the stream alignment. If no measured FPW and BFW are available, then divide the modeled 100-year flood event width by the modeled 2-year flood event width at multiple representative locations. To determine what the FUR is through the upstream reach, the existing structure and roadway prism shall be removed from the model to remove any backwater from impacting FUR calculations.

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 is provided in the 2013 WCDG. For areas that are tidally influenced, see Sections 7-3.5.4 and 7-5.3.

### 7-3.5.2 Unconfined Systems

An unconfined system has a FUR of greater than 3.0. In these situations, the velocity ratio, as defined by the WCDG, must be computed and shall be close to 1, which means that the ratio when rounded to the nearest tenth shall be 1.1 or less. 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 State Hydraulics Office must approve in these instances. Design teams shall contact the Hydraulics Section in unconfined systems to determine the best path forward for modeling the proposed and natural conditions to determine the velocity ratio.

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 shall not have evidence of significant erosion, scour, or other performance issues. The State Hydraulics Office must approve in these instances.

When evaluating a crossing using the velocity ratio in the main channel, the floodplains shall also be considered. Floodplain velocity ratios do not need to be 1.1; rather, the velocities in the floodplains shall be similar to what is expected in the geomorphic context of the reach. Floodplain velocities shall not be accelerated to decrease main channel velocities. In some instances it is recognized that it may not be possible to mimic floodplain velocities through a structure because of a decrease in roughness (Manning's n) through the structure as compared to the adjacent floodplain; this shall be documented in the specialty report.

For preliminary design, the Stream Team is to assume vertical walls for the edge of structure while determining the MHO 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 or to accommodate the meander belt; see Sections 7-4.2.

#### 7-3.5.3 Confined Systems

For confined systems, the BFW plus an FOS shall be used. In the case of WSDOT crossings, minimum structure width shall not be less than the greater of Equation 7-1 or Equation 7-2 unless otherwise approved by the State Hydraulics Office. In many cases, this width is appropriate. In some cases, a wider structure may be more appropriate. The effects of LTD and aggradation shall be considered with regard to structure width.

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 undisturbed reach.[1]
- Lateral migration of the channel is expected throughout the system.
- The stream has a natural sinuosity that can be replicated and justified (see Section 7-2.3.2).
- The structure is considered a long crossing (see Section 7-2.3.2).
- The Stream Team has reason to believe that additional width is needed. This shall be justified in the specialty report.

#### 7-3.5.4 Tidally Influenced Systems

For tidally influenced systems follow at a minimum Appendix D from the 2013 WCDG and the guidance of this section. Tidally dominated crossings are crossings at locations where the flux varies with the tides and reverses direction during normal tidal events. Tidal datums

[1] In the case of a difference in velocities, if the structure size is not the cause of the velocity discrepancy, the cause shall be documented and efforts shall be made to reduce the difference if possible. An increase in structure size is not necessary if the difference in velocities is not tied to structure width unless other elements of the channel design leads to a change in structure width.

(except mean water level) are not computed beyond the head of tide (NOS CO-OPS 1 2000). The distance that the head of tide is located in a watercourse upstream from the coastline is dependent on the slope of the channel and the flow. Although the definition of the head of tide describes a point, it is really the zone of transition where the morphology of a watercourse changes from a fluvial to a tidal flow regime.

To design a fish passage structure on a watercourse that is tributary to the Salish Sea or the Pacific Ocean it is necessary to establish where the project is located with respect to sea level and the geomorphic processes that define the site. The structure must be appropriately sized and the channel through or under the structure must be appropriately shaped to facilitate passage. Because the "head of tide" may be miles upstream of the coastline, indicators can be used to locate the project on the continuum between the fluvial and tidal flow regimes.

#### *7-3.5.4.1* Elevation

Determine mean higher high water (MHHW) using local tidal datums or using the NOAA VDatum tool. If the invert or any portion of any structure involved in the project is at a lower elevation than MHHW, then the project is located in the tidal zone. Washington Sea Grant, a collaborative organization of NOAA and the University of Washington, has developed extreme tide frequencies for Puget Sound and coastal Washington (unpublished data).

#### *7-3.5.4.2* Indicators

The following field indicators that can be observed can then be used to help describe the project site:

- **Mud line:** A mud line demarks the elevation of transition between the frequently flooded zone and the uplands. In a tidal system the demarcation is normally bare soil or mud because of the twice daily inundation. This is different from an incised channel in a fluvial system, where the ordinary high water mark is characterized by reduced leaf litter and lack of woody vegetation. If a mud line is present, the location is likely in the zone below the "head of tide" and estuarine processes shall be considered in the crossing design.
- **Gravel bars:** Clean gravel bars are usually an indicator of fluvial processes. Gravels coated in fine sediments may be found in estuaries, especially in Puget Sound, where gravel beaches are common. Clean gravel bars would be found at the upstream limits of the "head of tide" zone. Projects in this area may be suitable for a stream simulation design.
- **Salt-intolerant vegetation:** Salt-intolerant vegetation would be found at the upstream limits of the "head of tide" zone. Hutchinson provides a comprehensive listing of the salt tolerance of vegetation associated with estuarine wetlands (Hutchinson 1988). Western hemlock, tall Oregon grape, yellow skunk cabbage, or pale yellow iris are common riparian species that are very sensitive to salt. If these species are observed at the project site, the site is probably fluvial. Projects in this

area may be suitable for a stream simulation design.

- **Reverse flow:** Flow upstream through the existing culvert would indicate that the site is located below the "head of tide." If possible, plan to visit the site during the flood tide during the daily higher high tide when the stream is at base flow. High stream flows following storm events may mask tidal flow. If reverse flow is observed, an estuarine solution shall be considered for the crossing design.
- **Salinity:** The salinity of the water can be measured with an electronic meter. The salinity of water in the ocean averages about 35 parts per thousand (ppt). The mixture of seawater and fresh water in estuaries is called brackish water and its salinity can range from 0.5 to 35 ppt. Fresh water has salinity of less than 0.5 ppt. The salinity of estuarine water can change from one day to the next depending on the tides, weather, or freshwater inflow. If the salinity is greater than 0.5 ppt, an estuarine solution shall be considered for the crossing design.

#### 7-3.5.5 Climate Resilience

WSDOT uses climate science and tools to evaluate the influence that 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 the State Hydraulics Office for guidance on incorporating climate resilience on projects.

The procedure as of the publication of this *Hydraulics Manual* is as follows:

1. Using the Climate-Adapted Culvert Design tool from WDFW, determine the percentage change in 100-year flood event. This tool can be accessed on WDFW's Designing climate-change-resilient culverts and bridges website.
2. The Stream Team uses the current 100-year design flow established from the hydrology evaluation process and applies the projected increase in 2080 to get the 2080 projected 100-year flow.
3. The Stream Team models the 2080 projected 100-year flow and evaluates whether the proposed hydraulic opening will see significant velocity increases through the crossing as compared to the adjacent reach. If the velocities are much higher, the Stream Team evaluates what size MHO is necessary to achieve similar velocities and discusses the results with the State Hydraulics Office to determine whether it is practicable to increase the structure size.
4. The Stream Team evaluates the 2080 projected 100-year WSEL and follows the guidelines outlined in Table 7-2. In situations where the system is tidally influenced, 2 additional feet shall be analyzed to account for sea level rise. Additional clearance shall be considered to account for sea level rise if applicable; refer to Projected Sea Level Rise for Washington State (Miller et al. 2018).
5. The Stream Team evaluates the 2080 projected 100-year scour elevation and follows the guidelines outlined in Table 7-2.

In steps 3, 4, and 5, the State Hydraulics Office may need to coordinate with the WSDOT Bridges and Structures Office, WSDOT Geotechnical Office, and PEO to determine what the effects of including climate change may be on the project, to ensure that all project impacts are quantified. See Table 7-1 above for more information.

Changes to this guidance will be provided in future revisions to the *Hydraulics Manual*. The Stream Team shall check with the State Hydraulics Office before beginning a WSDOT project to determine whether the process has changed. The process used for the project shall be included as an appendix in the specialty report.

Climate resilience shall 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.

## 7-3.6 *Vertical Clearance*

The vertical clearance under a structure is made up of two components: the freeboard and the maintenance clearance. Vertical clearance is one component to the hydraulic height aspect of the MHO.

### 7-3.6.1 Freeboard

The design freeboard is the minimum dimension from the 100-year or 2080 100-year projected flood event (Table 7-2) WSEL to the minimum low chord that is necessary to pass all expected debris, water, and sediment expected over the life of a structure. The figures in the *Standard Plans* and *Plan Sheet Library* further illustrate the terms used here.

A minimum of 3 feet of freeboard above the 100-year or 2080 100-year projected flood event (Table 7-2) WSEL is required on all structures greater than 20 feet in span measured along the centerline of the roadway and on all bridge structures unless otherwise approved by the State Hydraulics Office. The Stream Team shall also confirm that local ordinance requirements are met and any necessary permit conditions are satisfied.

The 100-year or 2080 100-year projected flood event design freeboard required on all buried structures unless otherwise approved by the State Hydraulics Office are listed in Table 7-3.

**Table 7-3** 100-Year Design Freeboard Requirements on Buried Structures

| Structure Bankfull Width | Required Freeboard |
|---|---|
| Less than 8-foot BFW | 1 foot above 100-year or 2080 100-year projected flood event [a] |
| 8- to 15-foot BFW | 2 feet above 100-year or 2080 100-year projected flood event [a] |
| Greater than 15-foot BFW | 3 feet above 100-year or 2080 100-year projected flood event [a] |

a. The 2080 100-year projected flood event shall be used for the design, unless the State Hydraulics Office has determined that the 2080 100-year projected flood event is not practicable.

In areas that are tidally influenced, the impacts of 2 feet of sea level rise shall be evaluated for the project to determine if it shall be included in the freeboard requirements. For all projects, the Stream Team shall consider providing the clearances in Table 7-3 above the 100-year projected 2080 WSEL.

The required minimum design freeboard shall be maintained across the entire hydraulic width, as shown in the SFZ figures in the *Plan Sheet Library*. If aggradation is expected to occur, additional freeboard shall be given above the design freeboard equal to the anticipated aggradation.

Allowable exceptions are as follows. Fillets or arches may be inside the SFZ provided that all three of the following are true:

- The sum of all fillet areas (or arch encroachment areas) in a given cross section is less than 2 percent of the area calculated as the SFZ width multiplied by the SFZ height
- All fillet and arch encroachments are entirely above the elevation of the hydraulic design flood event plus the hydraulic design flood event freeboard within the limits of the hydraulic width

Four-sided buried structure allowable exceptions in addition to the above are as follows:

- The bottom fillets are allowed within the area that is 2 feet below total scour
- If total scour is calculated to be less than 1 foot, the bottom fillets shall be allowed to encroach only within the last 1 foot below total scour

If the design requirements listed above cannot be met, a hydraulic deviation approved by the State Hydraulic Engineer will be required. At a minimum, the Stream Team shall demonstrate the following:

- The proposed freeboard will pass all expected debris, water, and sediment through the system
- There is no history of repetitive maintenance at the existing crossing location
- Providing the required freeboard would cause adverse environmental impacts, impacts from changes to roadway geometry, or other unacceptable impacts
- Efforts have been made to maximize the freeboard to the extent practicable, including evaluating different structure types
- Documented acceptance of the proposed freeboard from WDFW and the Tribes

#### 7-3.6.2 Maintenance Clearance

Maintenance clearance is the vertical dimension added to the height to allow for inspection, monitoring, and maintenance, and is measured from the highest ground elevation point on the floodplain bench within the hydraulic width. All structures are recommended to incorporate 6 feet of maintenance clearance.

Maintenance clearance is required for complexity features withing a water crossing as specified in Table 7-4.

Coordination with the PEO shall occur prior to proposing any habitat features that require additional maintenance clearance to determine if roadway geometrics would prohibit the incorporation of additional maintenance clearance. The roadway geometric impact may be unavoidable, depending on what is required for stream function. After the structure type, size, and location are determined and maintenance clearance is known, the Stream Team shall revisit the habitat elements listed in Table 7-4 to determine if any are appropriate given the updated geometric design.

Variance from the maintenance clearance requirements will require a Hydraulic Deviation approved by the State Hydraulics Office prior to implementation. More guidance on maintenance clearance can be found in the WSDOT *Design Manual*.

**Table 7-4** Maintenance Clearance for Complexity Features

| Item | Required Minimum Maintenance Clearance |
|---|---|
| Slash | Required design freeboard (see Section 7-3.6.1) |
| Small woody material (SWM) | 6 feet |
| Mobile woody material (MWM)[a] | 10 feet |
| Type one boulders | Discuss with State Hydraulics Office |
| Type two boulders | Discuss with State Hydraulics Office |
| Type three boulders | 10 feet |
| Stable wood[b] | 10 feet |
| Step pools | 10 feet |

a. Mobile wood may require scour countermeasures and may require an additional risk assessment; coordinate with State Hydraulics Office.
b. Stable wood will require scour countermeasures.

## 7-3.7 *Buried Structures*

Buried structures for WSDOT projects can follow either the bridge design or stream simulation design criteria. When a buried structure is used as the crossing structure, wing walls shall be used to minimize the overall length of the buried structure. Wing walls can also increase the efficiency of the crossing structure. Wing walls shall be designed in accordance with Section 8 of the *Bridge Design Manual*. Additional criteria are discussed below.

As discussed in Sections 7-2.3.2 and 7-2.3.3, a meander belt assessment shall be conducted for all crossings. If a structure length is more than 10 times its width, then the hydraulic width shall be increased to whichever is greater, a 30 percent increase, or incorporate the width necessary for the natural meander as determined through the meander belt assessment. A meander belt assessment and increased hydraulic width may also be warranted in crossings that are greater than 200 feet in length, for multiple crossings in a short length (interchange, divided highway, etc.), or in other situations for stream restoration as described in Section 7-2.3.2.

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 total scour. Scour analyses are considered acceptable engineering justification.

Four-sided buried structures shall be countersunk a minimum of 2 feet below total scour as defined in Section 7-4.1, regardless of span width. Round buried structures shall be countersunk a minimum of 2 feet below total scour at the scour design flood event throughout the horizontal limits of the minimum hydraulic width. If this requirement cannot be met, approval from the State Hydraulics Office is required. It is understood that four-sided structures are created in whole-foot increments because of construction practices, so if the countersink is slightly below 2 feet, contact the State Hydraulics Office to verify if additional depth is required.

The footings of three-sided buried structures shall be countersunk at minimum as described in Section 7-4.1.

In some cases, constructability is more straightforward if the structure is placed flat, but the Stream Team may recommend that the structure be placed at a different slope from that of the streambed. Buried structures may be placed at a different slope from 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 State Hydraulics Office. 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.

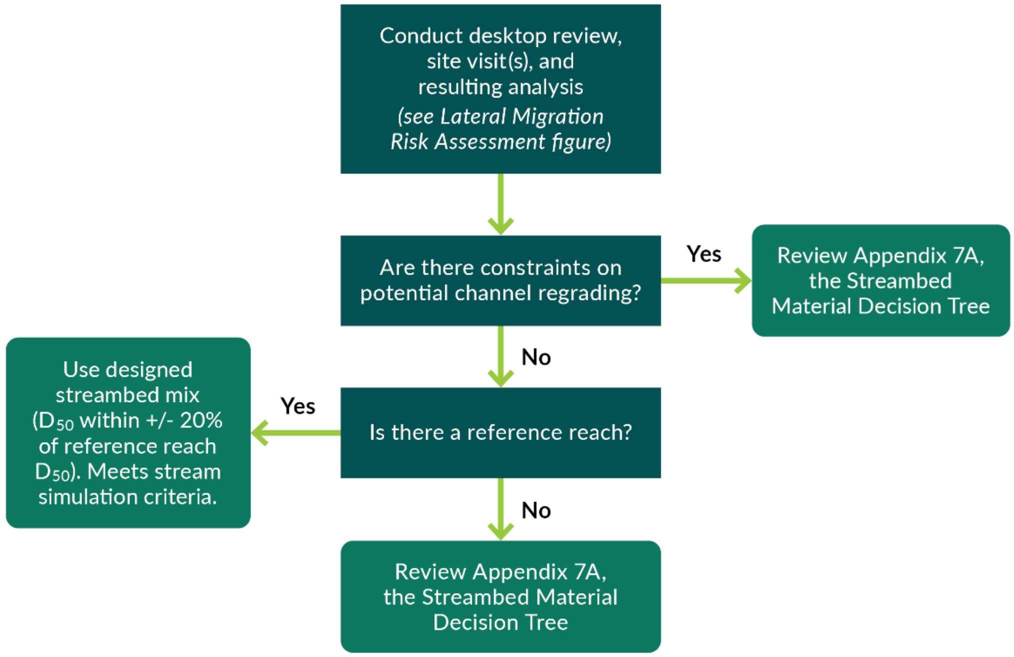
### 7-3.8 *Sediment Design*

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.

After reviewing existing conditions as discussed in Section 7-2, use the flow chart found in Figure 7-7 to determine the appropriate streambed material design methodology depending on site-specific conditions. Apply the stream simulation requirement of a $D_{50}$ that is within 20 percent of the reference reach unless constraints prevent this, or unless no reference reach is available. For these special cases, a Streambed Material Decision Tree to further assist the Stream Team in determining which methodology to use for streambed sediment sizing in these special cases is shown in Appendix 7A.

It may be appropriate to determine if other channel designs are applicable in certain situations; stream channels fall under the alluvial, threshold, or transition channel categories depending on their bed movement during a site-specific design flow event (NRCS 2007). After reviewing all streambed design methodologies within Appendix 7A, discuss with the State Hydraulics Office if an alluvial or threshold channel design could be appropriate.

**Figure 7-7** Streambed Material Design Methodology




For assessing sediment mobility, WSDOT requires the Modified Critical Shear Stress Approach, as described in Appendix E from the 2008 United States 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 flood event. If using WSDOT standard materials, it shall be noted that a minimum of 30 percent streambed sediment (9-03.11(1) is required to fill the voids in the various streambed cobbles mixes (9-03.11(4). Additional fines, typically using streambed sand (9-03.11(2) or native material), may be required to fully seal the bed.

### 7-3.8.1 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. 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 specialty report.

### 7-3.8.2 Constraints

If constraints in the systems, as described in Section 7-3.2, 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 first need to be evaluated for stability, using the appropriate methodology from Section 7-3.8. If the $D_{84}$ is not stable at the design flood event, then a risk assessment will need to be conducted to determine the next steps. The State Hydraulics Office and RHE shall be a part of the risk assessment process.

#### *7-3.8.2.1* 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 shall 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 flood event. 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 alternatives analysis needs to be conducted. The Stream Team 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 Team 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 event.

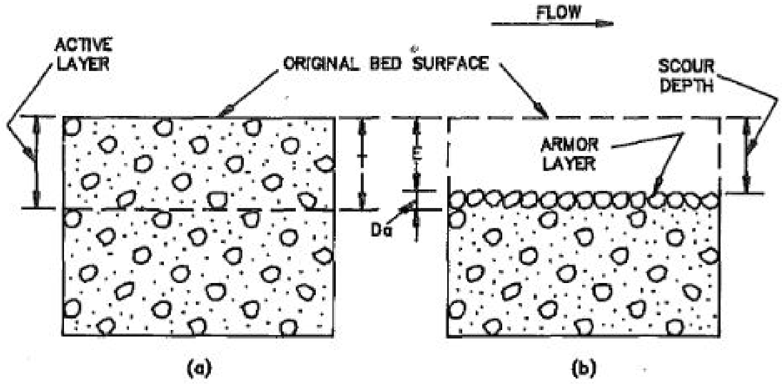
If a project is determined to be low risk, then the bed material shall match the pebble count in the reference reach and the process described in Section 7-3.8.1 applies.

### 7-3.8.3 Natural Streambed Armor Layer Design

The streambed material mix attempts to mimic the site-specific gradation of stream particles (sediment), normally prescribed via pebble count data, but also contains a large volume of fine-grained and highly mobile material with a desired outcome of bed sealing and relative bed stability. Streambed sediment can have as much as 20 percent by weight passing the No. 40 sieve, which is medium sand. In a gravel bed stream much of this finer material may be transported away from the active sediment layer during bed-forming discharges. This will be variable depending on sediment transported from upstream reaches. The bed will ultimately end at a state of dynamic equilibrium—a natural bed armor layer. The natural armor layer protects the integrity of the bed, adds stability, and renders the finer particles below it relatively immobile. However, a large volume of fine, highly mobile sediment must be “worked” by the stream to achieve this more stable state. The result is material transported downstream and likely lost within the reach. Figure 7-8 depicts formation of an armor layer.

**Figure 7-8** Formation of an Armor Layer

(a) Well-Mixed Original Bed Material (b) Armor Layer with Underlying Bed Material




Source: Borah 1989.

To prevent this loss, an active layer that matches the reference reach pebble count, but with no fines below a calculated surface layer particle size, could be designed. If the Stream Team is in a system in which this may be appropriate and wants to pursue this design, approval from the State Hydraulics Office is required.

### 7-3.8.4 Construction Requirements

The final stream grading limits horizontally and vertically shall be discussed with the Geotechnical Office to identify the composition and suitability of the surrounding native material. If the underlying material is evaluated as scour-resistant, it may not need to be replaced. Additionally, if the surrounding material meets the project requirements for the designed streambed material gradation, the depth and extents of the excavation may be adjusted as directed by the engineer in the field.

The final streambed material shall be placed in lifts no thicker than 12 inches. Streambed material shall be placed to ensure that stream low flow rate is conveyed above each channel layer. The contractor shall apply water and 0.5 to 1.0 inch of streambed sand to each layer to facilitate filling the interstitial voids of the streambed materials. The voids are satisfactorily filled when water equivalent to the low flow rate of the stream does not go subsurface and there is no perceivable difference in the low flow rate from upstream of the project limits to the downstream of the project limits. Refer to the *Standard Specifications*, Section 8-30 Water Crossings, for additional information.

### 7-3.8.5 Step-Pool Design

Step-pool systems occur naturally, between 3 and 8 percent slopes, and occur through natural material sorting or are forced through LWM. Many Washington streams are within this gradient range and special consideration is required for their design.

If the system's reference reach is step-pool in nature or the Stream Team has other reason to believe that a step-pool system is most appropriate for the site, the Stream Team must coordinate with the State Hydraulics Office regarding the proposed design and for any additional guidance that has been developed. The design of a step-pool system may require stability features that are larger than typical habitat structures or sediment size, channel-spanning wood, higher than normally recommended drop heights, etc. Working closely with the State Hydraulics Office will also help expedite any deviations from this *Hydraulics Manual* that are necessary to ensure a successful step-pool design.

### 7-3.9 Channel Complexity

Channel complexities are obstructions within the stream channel that support channel shape, diverse habitat for fish, and streambed stability. These features are discussed within the context of the constructed environment, though they are based on natural features as much as possible. Channel complexity features include both wood and non-wood structures. See Chapter 10 for additional guidance on channel complexity using woody material (WM).

Channel complexities are used to simulate natural characteristics in a stream. They are more important through water-crossing structures where vegetation and bank stability are absent or reduced. Simulating bank strength and naturally occurring channel complexity inside of a structure is difficult without soil cohesion and root strength.

It is important to consider the longevity of the channel complexity design: how it may change over time, its sustainability, and fish passability throughout the life of the crossing. The placement of complexity features can create a situation where the channel shape deteriorates over time, causing unintended aggradation or scour. When designing channel complexity features, the Stream Team shall protect the opposite bank from expected erosion using bioengineering and landscaping techniques unless bank protection is necessary for structural and roadway protection, in which case HEC-23, Volume 1 and Volume 2, measures would apply.

The following questions shall be considerations when designing channel complexity features:

- What is the design life of the structure?
- How could it change over time?
- Is it sustainable?
- Will it continue to serve its design functions after failure begins?
- Will it remain fish-passable throughout the design life of the crossing?
- How to incorporate slash? (see Chapter 10)

Channel complexities can be made up of coarser aggregate (cobbles and boulders) that is sized to be stable at the design flood events. Small woody material (SWM) (including slash) can be used in conjunction with coarse aggregate. Subsurface flow through channel complexities is a concern as voids in the coarser mixes allow low flows to penetrate below the stream profile. Layering the coarse aggregate and streambed fine sediment during placement and saturating the sediment between layers helps to seal the streambed. Streambed fine sediment bands have been installed upstream of complexity features to help seal the complexity features in situations where subsurface flow was a problem, post-construction.

WSDOT has used many types of channel complexity features, including single boulders, coarse bands, meander bars, and boulder clusters. To improve the success of complexity features, WSDOT has conducted research on meander bars to improve bank stability through water crossings. As additional research is conducted on other complexity features, further guidance will be provided in future revisions to the *Hydraulics Manual*. Confirm with the State Hydraulics office whether any new guidance has been released regarding complexity features since the last Hydraulics Manual revision.

#### 7-3.9.1 Boulder Features

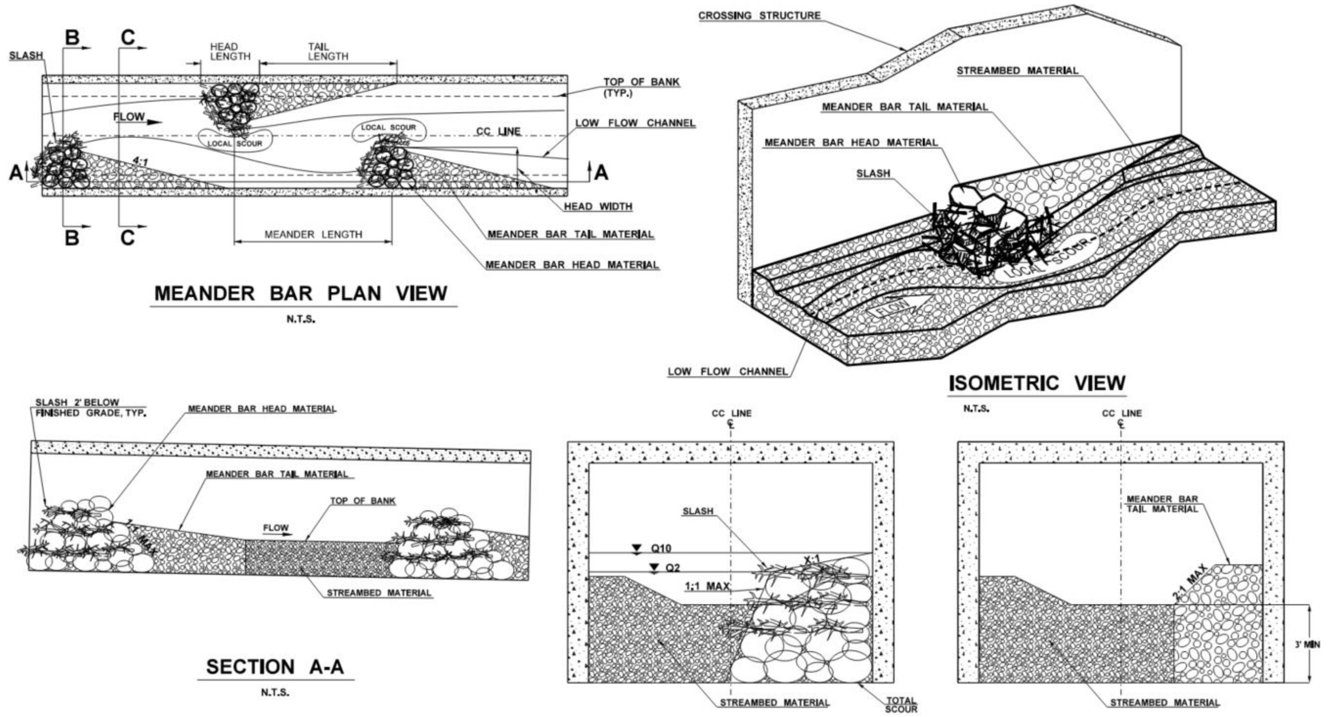
It may be necessary to have boulder features within water crossings to support channel complexity. In these cases, the Stream Team shall use engineering judgment to determine what this will look like and how it will tie in with other complexity features and the upstream and downstream planform.

If used, boulder features shall be spaced to simulate the expected sinuosity, and sized large enough to remain stable, be placed in a way that they promote localized scour/pool development, maintain high and low flow through the channel, do not create a low-flow barrier risk, and engage in the active channel. In addition to being stable during design flood events, consideration shall 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. Consider upsizing boulders when human contact is unavoidable; coordinate with the State Hydraulics Office and PEO to determine when upsizing may be appropriate. Boulder features are considered a channel complexity feature but with a hydraulic intention to direct flows away from a bank or structure where bank stability is critical.

#### 7-3.9.2 Meander Bars

Meander bars were conceived of and designed to replicate the natural forcing elements of a stream channel (e.g., banks) that create sinuosity in western Washington streams within a water-crossing structure. Typically, meander bars shall not be used upstream or downstream of the water-crossing structure. Meander bars are forcing elements that drive scour during higher discharge events and are not intended to be mobile. Their primary purpose is to reduce structure wall entrainment, to provide thalweg maintenance, and to prevent a plane bed from forming. The U.S. Fish and Wildlife Service recommends similar features to maintain streambanks within structures (Hanson 2022). Proper design and installation of meander bars provides additional benefits such as reach-scale hydraulic diversity/complexity, pool scour, sediment sorting (important for spawning salmonids), high flow refugia for migrating aquatic organisms (e.g., fish), and channel roughness. WSDOT published research and a case study indicating that meander bars also function to rack and attenuate organic debris (e.g., small wood), further providing significant habitat benefits. Figure 7-9 presents an example of meander bar detail. See Section 7-8 for additional information regarding monitoring; updated monitoring protocol will be determined in the future to evaluate and adjust design criteria for future updates to the WSDOT *Hydraulics Manual*.

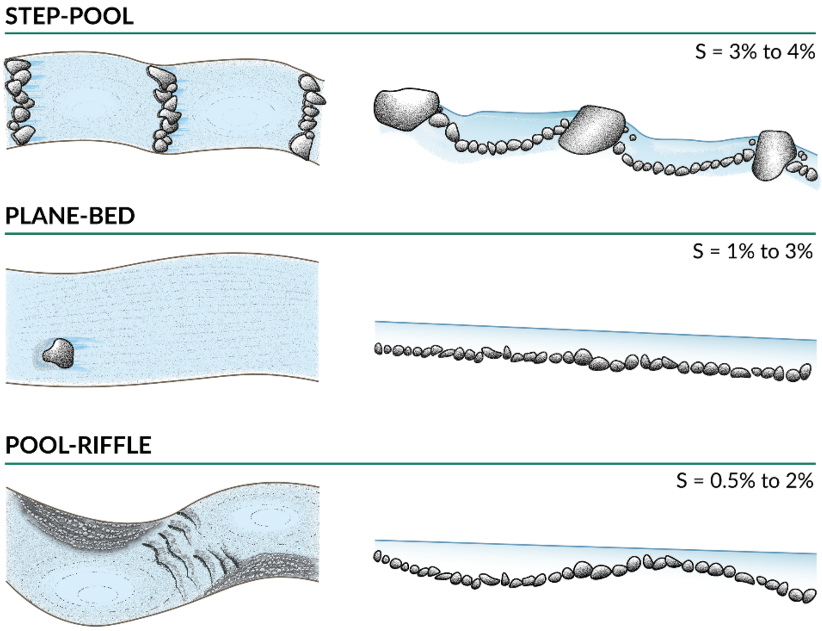
**Figure 7-9** Meander Bar Detail




### *7-3.9.2.1* Design Considerations: Slope—1–3 Percent

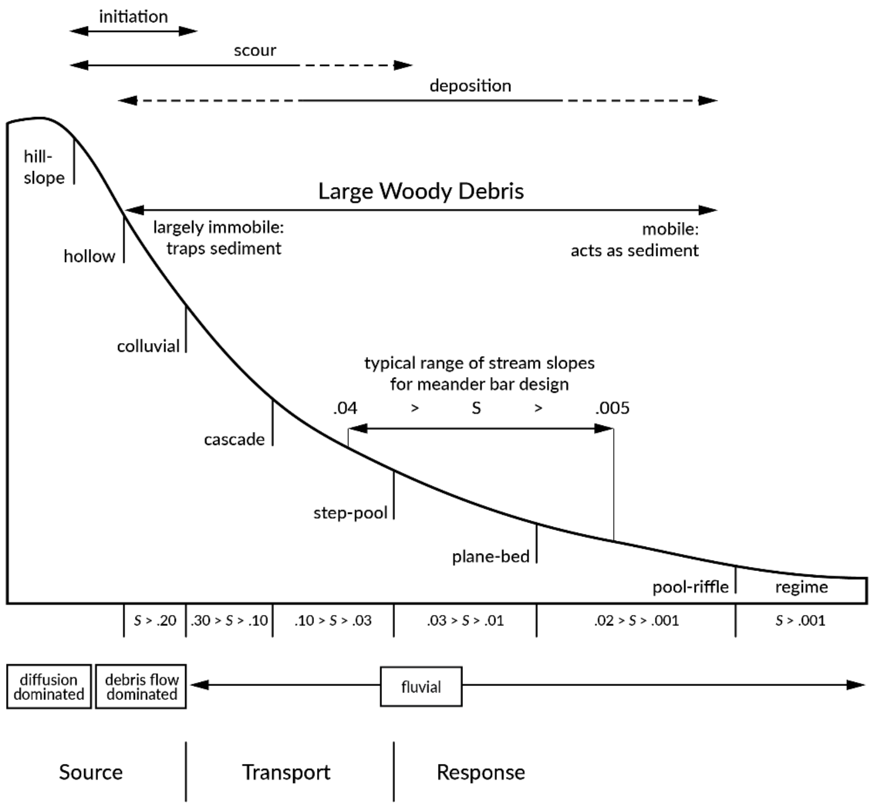
Meander bars shall be installed to simulate forcing elements typically found in riffle-pool systems or to re-form plane-bed streams into more productive, forced riffle-pool sequences (Figure 7-10). Montgomery-Buffington stream classification identifies a stream with a 1 to 3 percent gradient as a plane-bed response reach, unless there are forcing elements to create a riffle-pool system. Gradients less than 0.5 percent and between 3 and 4 percent could be acceptable depending on the stream characteristics (Figure 7-11). Meander bars shall not be used at gradients greater than 4 percent.

**Figure 7-10** Typical Stream Morphologies Suitable for Meander Bar Application




Typical stream morphologies with slopes suitable for meander bar placement. Note: meander bars are typically placed in plane-bed and pool-riffle channels (adapted from Montgomery and Buffington 1997).

**Figure 7-11** Range of Slopes Suitable for Meander Bar Application




Range of slopes suitable for meander bar placement (adapted from Montgomery and Buffington 1997).

### 7-3.9.2.2 Spacing

Meander bars shall be installed in an alternating pattern on the left and right banks of a channel and spaced to mimic natural sinuosity as seen in a reference reach at a similar gradient. If a natural sinuosity cannot be identified, hydraulic modeling may help inform appropriate spacing.

Lower-gradient streams require larger spacing between meander bars and additional consideration of complexity elements along the banks between the bars, while higher-gradient streams require closer spacing to generate natural sinuosity and mimic the observed pattern. Consideration of the banks between the meander bars shall be included. Variable spacing of meander bars may be appropriate and shall be considered.

#### 7-3.9.2.2.1 Guidelines/Recommendations

The following are guidelines and recommendations for spacing of meander bars:

- Meander bars shall be installed on both sides of a structure, unless approved by the State Hydraulics Office.
- Meander bars are intended for application in crossings of sufficient length to contain

one, or more, river-meander wavelengths.

- Crossings shorter than one wavelength shall limit extending the meander bar design upstream and downstream of the crossing and the design shall consider using other complexity applications outside of the crossing structure such as wood features, when possible.
- Ideally, two or more bars will be placed within the structure for structures longer than 50 feet.
- The application of meander bars in crossings shorter than one wavelength requires approval of the State Hydraulics Office.

#### 7-3.9.2.2.2 High Sediment Load Spacing

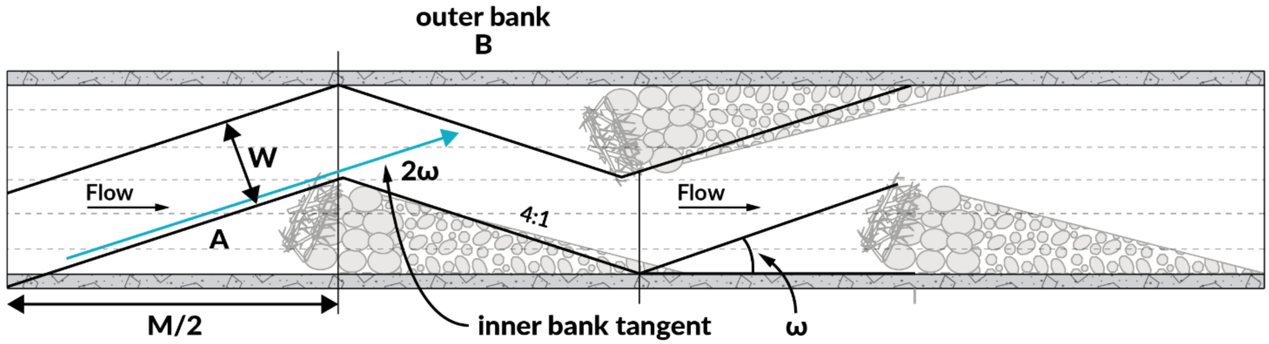
In the absence of natural meander forcing features, and if significant bedload sediment transport (sediment input is greater than 110 percent of sediment output) is anticipated through the crossing, the meander bars shall be designed to generate sediment deposition in consistent locations. The deposition of sediment in a consistently located gravel bar because of local hydraulic conditions is termed a forced bar. In the absence of local hydraulic controls on bar location, gravel bars can migrate downstream, a process termed free bars. Forced bars are recommended for crossings with high bedload transport rates to provide greater predictability of planform location and a lower rate of morphologic change (Figure 7-12 and Figure 7-13). Forced bars can be created by designing the meander bars to simulate a sufficiently high sinuosity.

Whiting and Dietrich (1993) define the threshold between forced bars and free bars. The authors place this threshold in a phase space with the ratio of the channel wavelength (M) to channel width (W) on the x-axis and the angle of the inner bank tangent (ω) on the y-axis (Figure 7-12 and Figure 7-13). The threshold of bar migration within this phase space is defined by Equation 7-3:

$$\frac{M}{W} = \frac{1}{\sin\omega\cos\omega} + 2 \qquad (7\text{-}3)$$

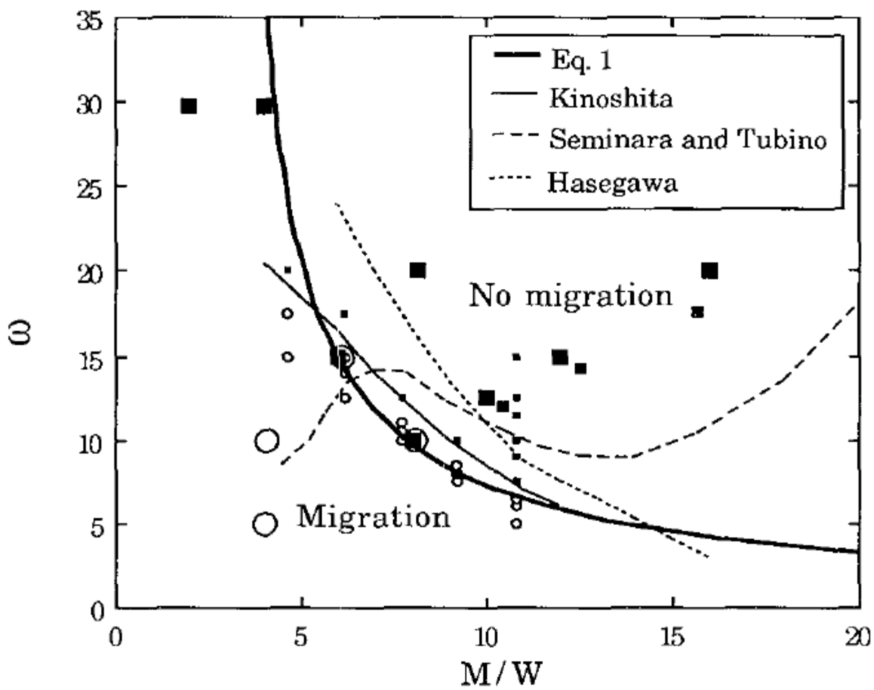
Note: In high sediment load conditions, the material behind the bar head may not be needed and requires coordination with the State Hydraulics Office.

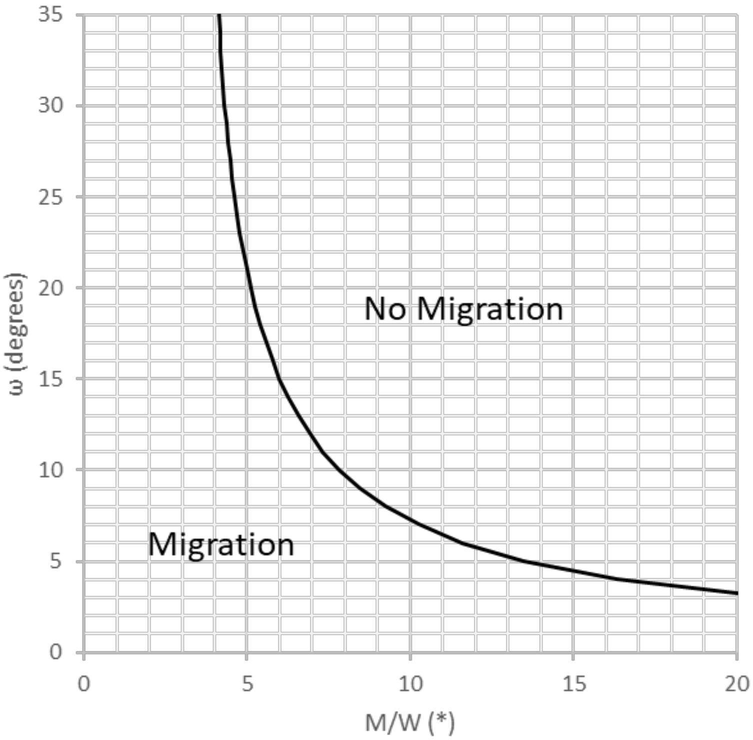
**Figure 7-12** Meander Bar Spacing Detail




Source: Whiting and Dietrich (1993).

**Figure 7-13** Forced Bar vs. Free Bar Threshold






Source: Whiting and Dietrich (1993).

### *7-3.9.2.3* Bar Height

Meander bars shall be designed to the full depth of the streambed and shall extend to the lesser of total scour or total excavation elevation if competent material exists.

The bar head shall be composed of stable large rock and be designed so that the top of the head is approximately at the 10-year flood event elevation measured at the structure wall and at the 2-year flood event elevation measured at the nose of the bar head, closest to the thalweg.

The bar tail shall be composed of a streambed cobble mix including boulders as necessary and be designed so that the top of the tail is approximately at the 10-year flood event elevation measured at the structure wall and tapers to the elevation of the streambed at the downstream end of the structure tail. Stable elements shall extend to a minimum of 3 feet or full design sediment thickness.

### *7-3.9.2.4* Additional Considerations

The following are additional considerations related meander bar design:

- Add a single boulder at the nose of the bar head, closest to the thalweg.
- Create a saddle between the meander bar and an additional boulder resulting in split flow at 2- to 5-year recurrence intervals. Coordinate with the State Hydraulics

Office for design considerations.

- Bar angle is an important component of design. Bars angled downstream will increase velocity and scour along the face. Bars angled upstream or perpendicular will create a pocket refugia upstream, keeping the thalweg more central, and will encourage deposition upstream of the bar head.
- Incorporation of SWM, slash, and/or boulders, if clearance allows, in the opposite bank of the meander bar head to reduce bank erosion and entrainment.

### *7-3.9.2.5* Channel Constriction

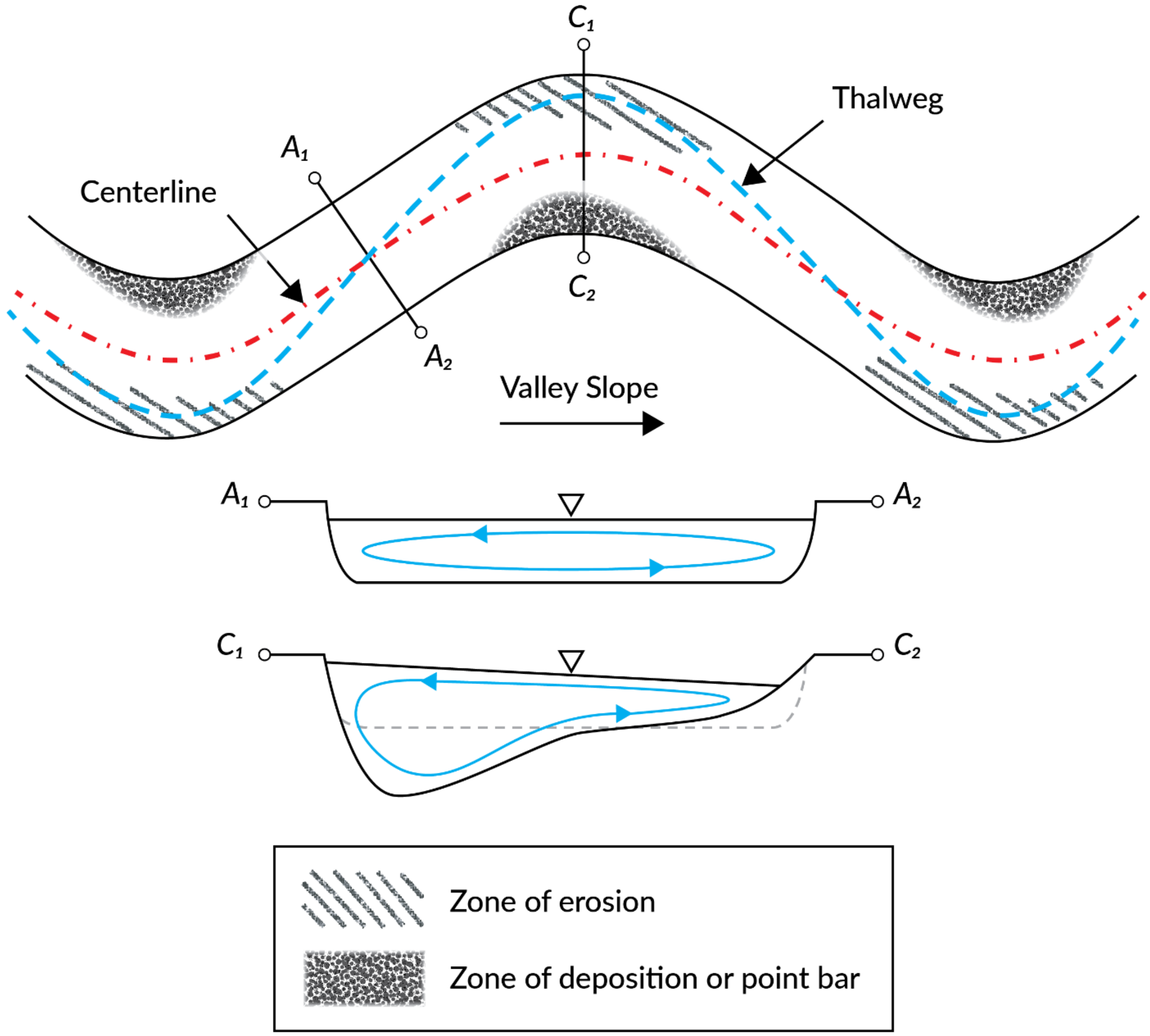
Meander bars shall occupy a minimum of 30 percent of the cross-sectional area of the channel to drive contraction scour, provide thalweg maintenance, and match the natural sinuosity of a reference reach. The meander bar shall constrict the channel width down to the minimum measured BFW. Larger structure widths require more obstruction width to perform the function needed and may require either the meander bar to extend farther into the channel and/or the use of slash and/or boulders in the opposite bank of the meander bar head to help maintain channel shape. Contraction scour shall be evaluated based on the width that is capable of moving sediment and documented in the specialty report.

### *7-3.9.2.6* Bar Shape

The following are bar shapes:

- **Teardrop or modified crescent:** Meander bars are intended to provide some of the functions similar to point bars, which are found in natural, undisturbed systems (Figure 7-14). Meander bars are three-dimensional features with a crown (high point), deflecting head (upstream proximal end), and tapering tail (downstream distal end). Meander bars differ in function from point bars in that they drive scour along the margin of the proximal end, which reduces structure wall entrainment and provides thalweg maintenance. They also help with sediment sorting as energy dissipates toward the distal tail.
- **Half-dome without tail:** If it is determined that a tail is not required because of high sediment load within the system, the meander bar shall be designed with a half-dome shape consisting completely of head material with slash or SWM.

**Figure 7-14** Typical Point Bar Formation in Meandering Streams




Source: Dey (2014). Meander bars are designed to imitate the functions of natural point bars.

### *7-3.9.2.7* Materials: Cobbles and Boulders Sized for Stability and Resilience

This section presents a discussion on bar materials, including bar head, bar tail, and other design.

#### 7-3.9.2.7.1 Bar Head

Materials used in the design and construction of the meander bar head shall consist of large rounded rock designed to be 100 percent stable at the 100-year flood event. Although the smallest stable material shall be used, the size might need to be increased for meander bars to be stable for the long term. The material shall be sized to allow for minimal maintenance, which can be difficult within structures and provides resilient complexity. The stability analysis shall consider flow overtopping the rock (see 2012 WDFW *Stream Habitat Restoration Guidelines* pages T6-20 and T6-21 for an example) (Cramer 2012). The length of the head shall be a minimum of twice the $D_{100}$ of the head material size at the top and will taper out at a 1:1 slope maximum. The head material shall be placed in lifts with well-graded stream material and fines to seal the bar head to prevent porosity. To prevent saltation of the head material and relocation of material by humans a minimum Type 2 Boulder is recommended. Consider upsizing boulders when human contact is unavoidable; coordinate with the State Hydraulics Office and PEO to determine when upsizing may be appropriate.

### *7-3.9.2.8* Bar Tail

If it is determined that a stable tail is needed for the meander bars, the $D_{30}$ of the material in the tail of the structure shall be larger than the $D_{84}$ of the observed streambed material and be stable at the 25-year flow event to dissipate overtopping energy. If this is larger than the material sized for the head, evaluate if the site is correct for meander bar installation. Fines shall also be incorporated into the bar tail to seal the bar tail to prevent porosity. In construction, the meander bars and tails shall be tested for subsurface flow similar to the streambed.

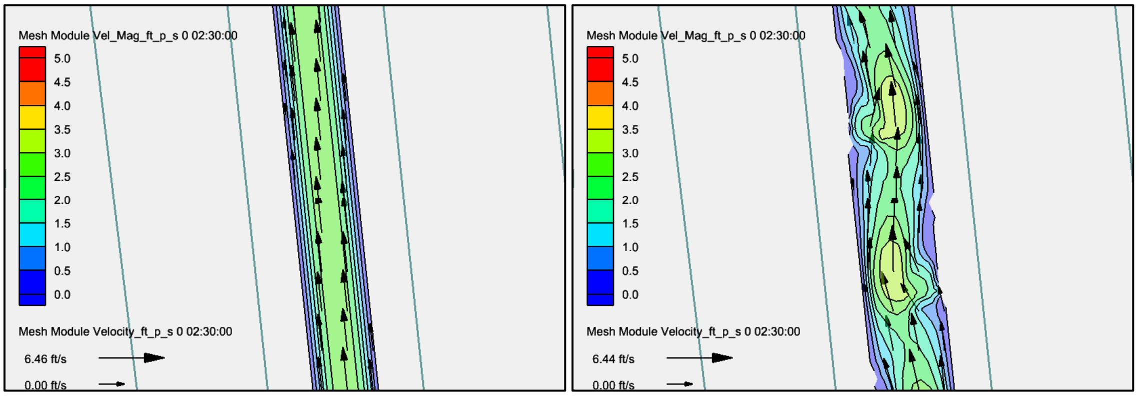
#### 7-3.9.2.8.1 Slash and Small Woody Material

SWM (if clearance allows) or slash shall be placed in the head of bars to encourage racking increase stability and add habitat complexity to the stream. Between 30 and 50 percent by volume of SWM or slash shall be interwoven between the boulders forming the meander bar head and shall also wrap around the stream side to the beginning of the tail to engage with all flow conditions and encourage a scour pool. See Figure 7-9 for an example of meander bar slash implementation.

### *7-3.9.2.9* Hydraulic Modeling of Meander Bar Features

Meander bars can be modeled with composite roughness values during the conceptual phase of a stream design. However, there are times when it is necessary to include meander bars as part of the surface during preliminary phases of a design and documented accordingly. Meander bars shall be included as part of the streambed surface in the hydraulic model prior to the FHD. Figure 7-15 shows an example of a hydraulic model where the proposed surface was modified to include the meander bars. Contact the State Hydraulics Office for additional information on scour associated with complexity features.

**Figure 7-15** Example Velocity Maps




Example modeled velocity maps for the McCormick crossing (left figure with composite roughness values in the model and right figure with meander bars included in the surface). This models the hydraulic diversity introduced by the meander bars.

#### 7-3.9.3 Construction Requirements

Most channels take a few large flows before natural 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 rootwads of LWM and other complexity elements at locations where a pool would naturally form is recommended as directed by the engineer in the field. A low-flow pilot channel is also required to be installed as directed by the engineer in the field, that connects the habitat complexity elements immediately after construction, unless otherwise approved by State Hydraulics Office. An example of a constructed meander bar is shown in Figure 7-16.

**Figure 7-16** Example of a Constructed Meander Bar with Slash

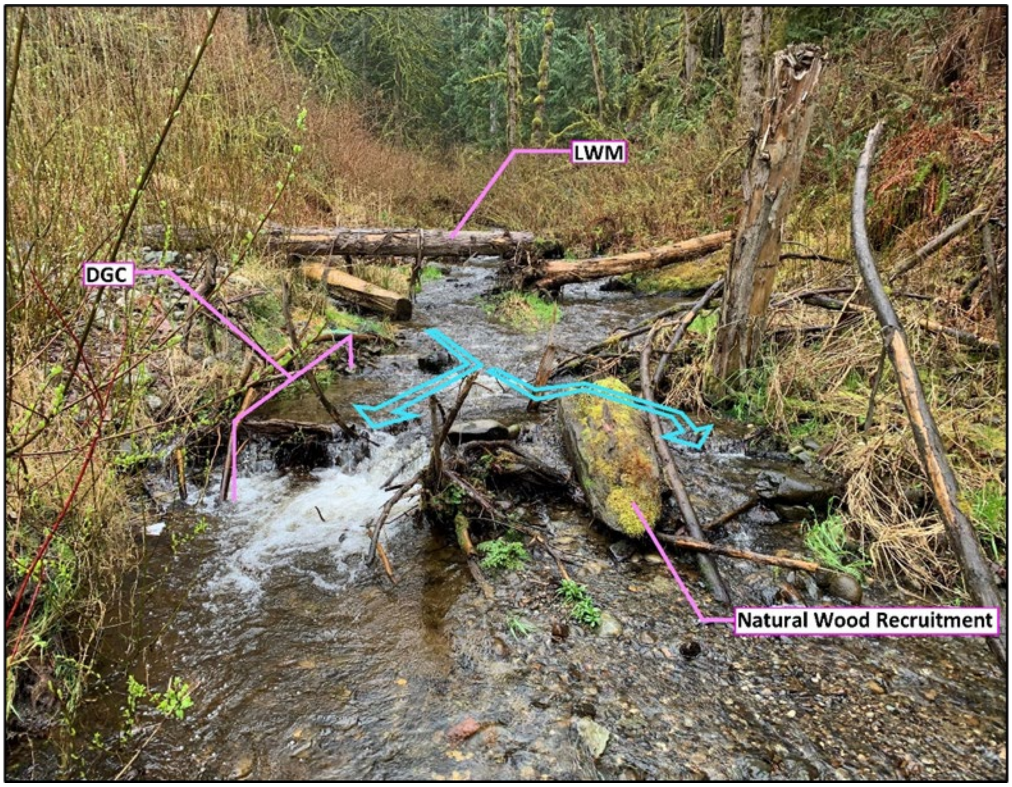
#### 7-3.9.4 Deformable Grade Control

Complexity features of creeks—specifically lower-order, tributary systems—are often a product of messy, interlocking matrices of roots, branches, rocks, and sediment. A creek over time accrues diverse wood debris and incorporates it within its streambed, while the riparian vegetation grows and intwines itself into this mix (Bilby 1980; Bretschko 1990; Dolloff 2000). In addition to the application of LWM as forcing structures, an accumulation of smaller materials introduces local complexity and in series these structures support reach-scale processes (Shahverdian et al. 2019). The intent of deformable grade control (DGC) is to replicate a natural, cohesive matrix of debris and sediment, to construct features resilient and adaptive in nature that decrease the rate of stream degradation and restore stream processes and complexity.

The empirical hydraulic functions of DGCs change as the feature weathers. The following design considerations describe DGCs' adaptive hydraulic functions and how the deformability of a DGC is a product of design choices. Locally, a DGC promotes upstream floodplain connectivity and thus sediment retention, and as they degrade (Figure 7-17), they promote downstream pool formation and sediment sorting as seen in post-assisted log structures (PALSs) and beaver dam analogs (BDAs) (Shahverdian et al. 2019). On a reach scale, a series of DGC features provides vertical stability, ultimately counteracting headcuts, and regulating channel degradation (Fouty 2023; Shahverdian et al. 2019).

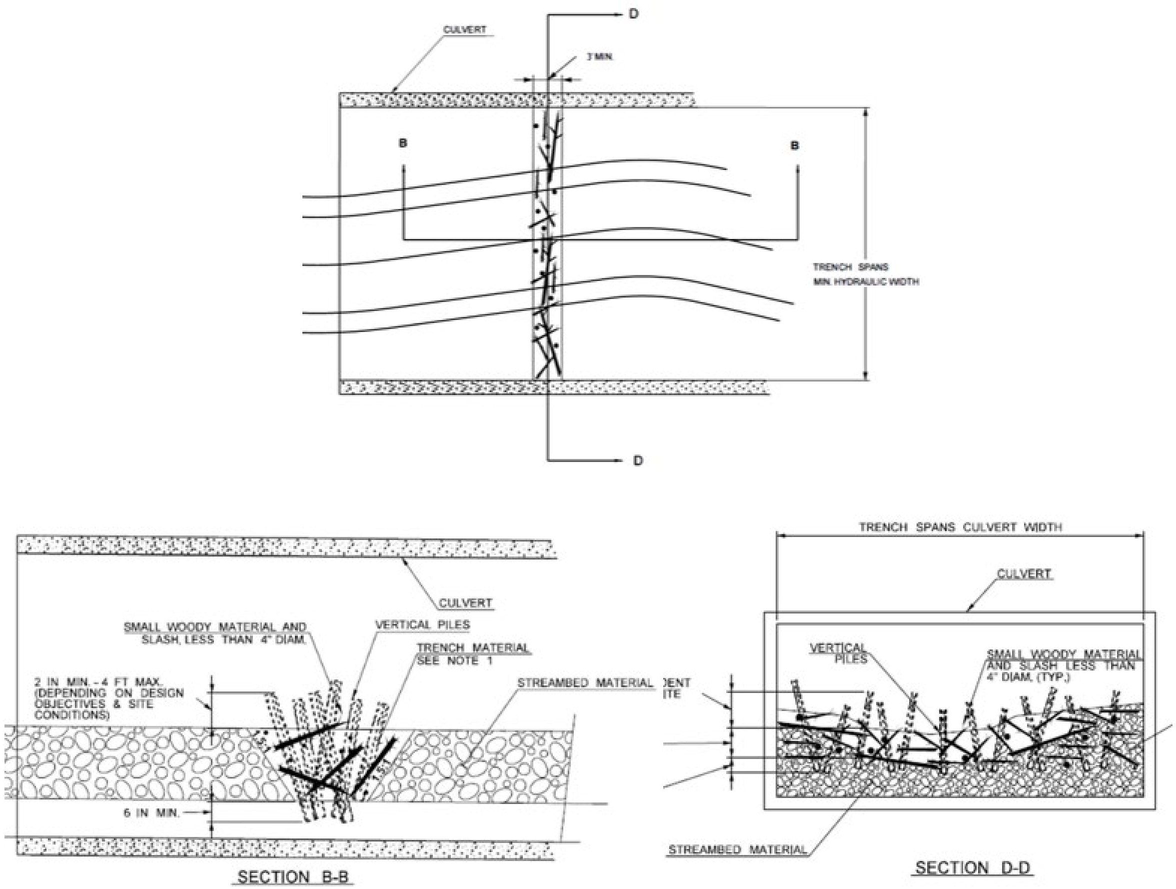
DGC, developed in partnership between WSDOT and the Tulalip Tribes, counteracts historical stream design methods and their legacy. In developed environments, streams were routed through undersized structures that fragmented riparian habitat, limited wood/sediment transport, were maintained to remove woody debris accumulation, and yet provided grade control. See Figure 7-18 for example details of the DGC. The exact sediment-to-wood ratio varies based on the design objectives and site conditions of the project, and the DGC shall be used in combination with other stream complexity features.

**Figure 7-17** Example of Constructed DGC Feature, 2 Years after Construction




A headcut propagated through a downstream structure and was arrested at the photographed DGC. Consequently, the DGC deformed and created a forced step pool (stream's right) and simultaneously maintained a side flow path with no water surface drop (stream's left). This deformation demonstrates the ecological function of DGC and how weathered DGC provides fish passage for multiple species and life stages.

**Figure 7-18** DGC Details




### *7-3.9.4.1* Design Considerations

DGC functions are fluid: initially, the channel roughness slows and deepens flow, accumulating more sediment and debris. The feature deforms as more overbank events occur and erosion and deposition patterns emerge. The weathered DGC may form a forced step pool, a slope transition, or alternate flow paths, promoting hydraulic diversity and heterogeneity in the gradient (Figure 7-17; Shahverdian et al. 2019). As noted, the deformability is a function of the site conditions (i.e., slope and reach characteristics) as well as the design choices (i.e., wood-to-sediment ratio; sediment and wood sizing; density, embedded depth, and exposure height of the vertical piles; and type of WM).

#### 7-3.9.4.1.1 Slope

DGC shall be installed to mimic an accrued matrix of material found in response and transport reaches. Fouty (2023) tested DGC performance at 2 to 4 percent slopes and found that in systems over 2 percent DGC will deform, and that as the slope increases the rate and magnitude of feature deformation will consequently increase.

Montgomery-Buffington (1997) classifies a stream with a 2 to 4 percent gradient as a plane-bed, pool-riffle, or step-pool system. Gradients outside this range are acceptable depending on the design elements and design intent. Slopes greater than 4 percent will require additional consideration and the combined use of LWM, rock, and DGC. Contact the State Hydraulics Office for current guidance for high gradient systems.

#### 7-3.9.4.1.2 Density and Location

Fouty (2023) tested the performance of an isolated DGC feature and concluded that a single DGC reduces upstream erosion and improves channel shape stability. A single DGC may be placed to provide localized function; i.e., channel roughness, targeted headcut arrest, or channel complexity with low impact and within tight construction limits.

If the intent of DGC is to promote reach-scale processes, DGCs shall be placed in series, and the recommended number and spacing is dependent on channel characteristics. Increasing the number of DGCs may result in extending the impacted reach length, increasing the stability of the grade controls, and intensifying the sediment retention. DGCs shall not be used in locations where these functions intensify risk to infrastructure or property. Consider the implications of site conditions (e.g., aggradation issues, beaver presence, limited structure clearance, and flooding) to evaluate if DGCs are acceptable.

#### 7-3.9.4.1.3 Trench Dimensions

The following are design considerations for trench dimensions:

- **Within structure:** The trench dimensions are dependent on design objectives. Typically, the DGC shall span the minimum hydraulic width and reach the estimated depth of long-term degradation to a maximum of 6 feet. At a minimum, the trench shall have a depth of 3 feet, a 3-foot base, and a 1.5:1 side slope (see Figure 7-18).
- **Outside of structure:** Typically, the DGC shall span the estimated lateral migration of the channel and reach the estimated depth of long-term degradation to a maximum of 6 feet. At a minimum the trench shall have a depth of 3 feet, a 3-foot base, and a 1.5:1 side slope.

#### 7-3.9.4.1.4 Trench Fill

The following are design considerations for trench fill:

- **Sediment sizing:** Fouty (2023) and WSDOT's constructed case studies recommend using the streambed material mix that is proposed for the project reach. Use of mixes that deviate from the general proposed gradation may be acceptable depending on the site conditions and design objective. Consider that the native and proposed sediment size and type does impact the stability of the DGC and will drive other design choices.
- **Wood-to-sediment ratio:** Fouty (2023) results suggest that at 2 to 4 percent slopes a trench fill consisting of 50 to 75 percent wood reduces sediment transport and optimizes channel shape stability. Consider how DGC influences sediment transport

and if there are implications to short- and long- term scour. If long-term scour is anticipated, consider how this shall be reflected in total scour.

- **Wood sizing:** According to Fouty (2023), a mixture of diameters provides reliable sediment retention and channel shape stability. The key finding from Fouty (2023) and supported by Shahveridan et al. (2019) is that a diverse distribution of wood diameter and length increases channel shape stability. The design shall allow these features to be complex and incorporate material available on site. The design may call out diameter and length ranges but does not need to specify precise mixes and rather could point to WSDOT's slash and SWM specifications. The vertical piles shall be 4 inches in diameter or smaller (i.e., SWM). Additional requirements for maintenance clearance may apply; see Table 7-4.

#### 7-3.9.4.1.5 Wood Orientation/Positioning

The following are design considerations for wood orientation/positioning:

- **Vertical pile density:** Incorporating a higher density of vertical piles increases the stability of the DGC.
- **Vertical pile embedded depth:** Consider the site conditions and design objectives to determine appropriate embedded depth. The minimum pile depth is 6 inches below the trench excavation depth; driving the piles deeper will increase the stability of the DGC. The recommended minimum buried length is 2/3 of the pile length.
- **Woody material exposure:** Consider the site conditions and design objectives to determine appropriate exposure height. The vertical pile exposure height can vary from 2 inches to up to 4 feet; positioning the wood with higher exposure to flow provides higher channel roughness and racking potential while increasing the deformability of the feature.

#### 7-3.9.4.1.6 Wood Type

The SWM and slash shall consist of a random assortment of branches, trees, brush, and treetops of the following native species: western red cedar (*Thuja plicata*), Douglas fir (*Pseudotsuga menziesii*), western hemlock (*Tsuga heterophylla*) coniferous trees, or various hardwood trees. No more than 50 percent of hardwood species shall be used. The needles shall be left intact to the extent possible given the mechanics of handling slash. Slash shall not contain any material that causes turbidity. DGC features shall incorporate slash and SWM available on site.

### *7-3.9.4.2* Hydraulic Modeling of DGC Features

DGCs can be modeled with discrete sections with higher roughness to match the proposed DGC locations. Guidance on modeling DGCs is still in development; coordinate with the State Hydraulics Office for other current modeling methods.

##### *7-3.9.4.3* Construction

Two common construction methods are outlined below. Other methods are acceptable if approved by the State Hydraulics Office:

- **Install DGC after streambed lifts are installed**: Construct streambed to final grade. Excavate trench per plan. Mix SWM, slash, and streambed material in specified ratios. Drive vertical SWM components into the trench. Care shall be taken to wash in streambed material including sand to seal as placement occurs.
- **Install DGC along with streambed lifts from subgrade**: Place SWM and slash as constructing streambed. Mix in streambed material at the ratio specified. This can be challenging to maintain vertical piles while constructing streambed lifts. Incorporate any streambed sand and water as required for adjacent streambed material installation sequencing.

### 7-3.10 *Landscaping/Planting*

The landscape architect will follow guidance for planting near streams located in WSDOT's *Roadside Manual* Chapter 830 for all projects located near streams. The Stream Team shall collaborate with the landscape architect to develop a restoration plan that includes the areas of bank stabilization countermeasures, habitat complexity, riparian restoration, and any planting that could be implemented prior to the first storm event post-construction to minimize erosion. The Stream Team will coordinate with the landscape architect regarding any plantings recommended for bank stability, and as discussed in Section 7-3.2.1, shall coordinate with the hydraulic designer doing the stormwater/drainage design for the project to ensure that the drainage works with the stream design and any outfalls properly drain to the stream without creating erosion and do not interfere with habitat features. 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*). 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 Scour

This section covers scour analysis requirements for all WSDOT water crossings structures (bridges and culverts). Scour is evaluated throughout the project delivery process through early and often coordination with various specialty groups. Refer to the WSDOT *Design Manual*, Chapter 800, for additional information regarding interdisciplinary coordination.

### 7-4.1 *Total Scour*

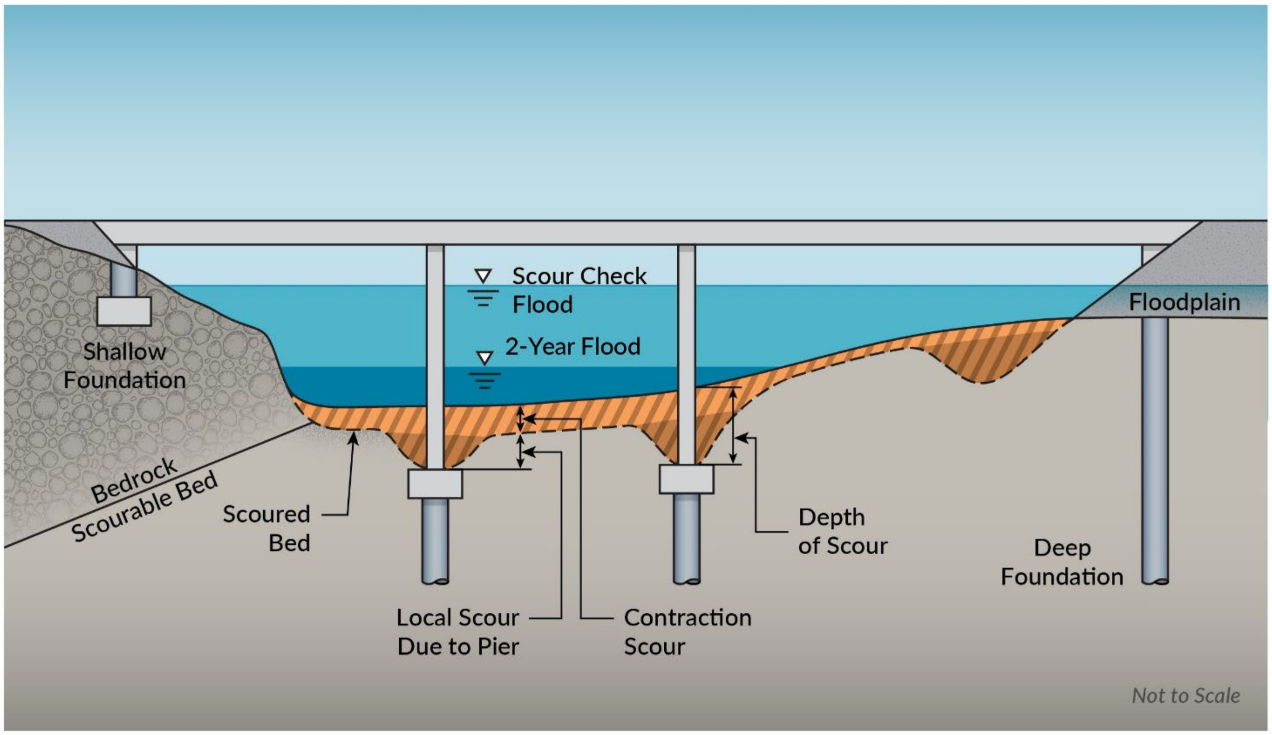
All water crossing structures (bridges and culverts) shall be designed for total scour, not just bridges. Total scour shall be assessed for all scenarios and flows up to the scour design flood and scour check flood events that results in worst-case total scour for each event. The hydraulic team or Stream Team shall follow appropriate method(s) depending on structure type, size, and location. A minimum of 3 feet of total scour is required to be assumed for all bridges and three-sided structures. Walls for all bridges and three-sided structures shall be designed for total scour and the length shall be based on the potential impacts of lateral migration as assessed by the hydraulic team or Stream Team. As defined by HEC-18, total scour is determined by the sum of various scour components—specifically, LTD, contraction scour, and local scour. Total scour must be computed using the $D_{50}$ for both the proposed design mix and subsurface material provided by the Geotechnical Engineer when total scour is anticipated to be deeper than the depth of placed streambed material. Determination of whether the contracted section is in a clear-water or live-bed condition must use a representative grain size at the approach section for the material that would be transported from upstream into the water crossing. ***Coordinate with HQ Hydraulics, HQ Geotechnical, and HQ Bridge to ensure the provided depths of total scour are being correctly applied to determine the total scour elevations at each infrastructure component and are commensurate with the level of risk warranted for the crossing location.*** Methodologies and equations used for determining total scour shall follow HEC-18. Refer to the FHWA *Pier Scour Estimation for Tsunami at Bridges* (FHWA 2021) to design for the effect of tsunami events on pier scour for those bridges in locations within identified tsunami design zones (*Tsunami Design Zone Maps*).

In addition to the three scour components mentioned above, the potential for lateral migration (Section 7-4.2) must be assessed to evaluate total scour at water-crossing structures. WSDOT has also developed a scour review checklist to identify a list of elements examined during scour review; this checklist can be found on the WSDOT Hydraulics Training web page. Wall scour analysis is not appropriate for every water-crossing project, and shall be included only on a case-by-case basis depending on the characteristics of the stream and structure type. Coordinate with the State Hydraulics Office if it is determined that wall scour may be required at the crossing and consider applying principles from HEC-23 Volume 1.

### 7-4.2 *Lateral Migration for Water-Crossing Structures*

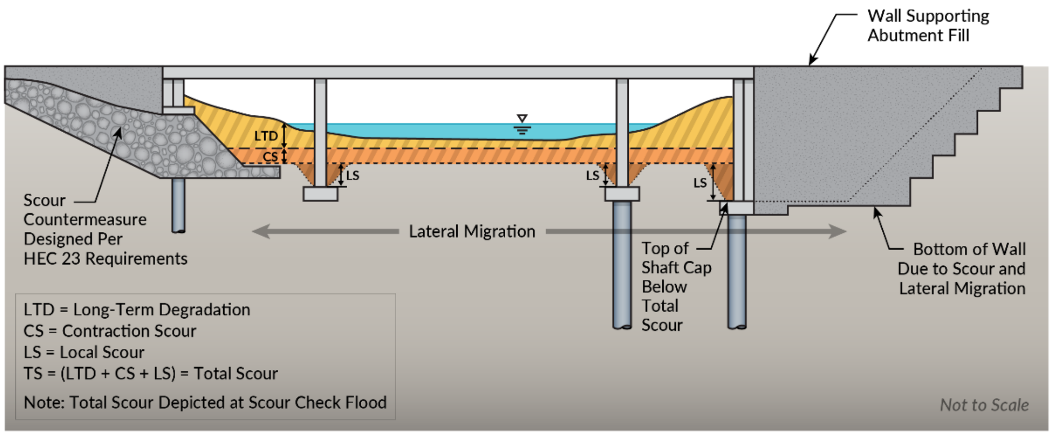
All structures shall be designed to account for the lateral channel migration expected to occur over the design life of the structure. See HEC-20 and Sections 7-2.3.2 and 7-2.3.3 for additional guidance on maintaining continuity of channel processes and assessing lateral migration and. If non-erodible soils are present such that no lateral migration is expected to occur over the design life of the structure, then LTD and contraction scour is a uniform offset from the existing channel section. Figure 7-19 illustrates various scour components for a channel that has been determined to be vertically and laterally stable. On the left side of Figure 7-19, based on geotechnical data, the channel bank and ground supporting the bridge foundation have been determined to be bedrock with low potential for erosion over the design life of the bridge. For these reasons, a shallow bridge foundation is acceptable because no scour is anticipated. Conversely, on the right side of Figure 7-19, a deep foundation is required because no bedrock or other non-erodible materials are present. The two intermediate piers are also deep foundations with shaft caps below anticipated total scour to minimize potential obstruction to the flow. The abutment scour occurring at the toe of the abutment on the right side of Figure 7-19 is above the channel thalweg because it is outside the main channel and there is no potential for lateral migration. For these reasons, the deep foundation needs to be designed only for abutment scour. Prior to using various scour equations, the hydraulic team or Stream Team needs to confirm what reference elevation a given scour equation uses. For example, some scour equations estimate scour as depth of flow after the scoured condition (e.g., measured from water surface to scoured bed), while others estimate scour as the vertical distance from the pre-scoured bed to scoured bed.

**Figure 7-19** Total Scour Components without Potential of Lateral Migration




If lateral migration can occur over the design life of the structure, the hydraulic team or Stream Team shall document in the specialty report the risk of lateral migration at each pier and/or abutment and whether any scour countermeasures and potentially an increase in structure size (or SFZ) are recommended. The thalweg is the starting elevation for determining total scour for all infrastructure components that are within the extents of potential lateral migration. Figure 7-20 provides an example for a water crossing with deep foundations and abutments with potential of lateral migration. On the left side of Figure 7-20 a scour countermeasure designed meeting requirements, specifically the use of an apron below LTD and contraction scour at the scour check flood, is used to mitigate abutment scour. On the right side of Figure 7-20, no scour countermeasures are used, resulting in a greater depth of scour because of the requirement to account for abutment scour at the structure and wall foundations.

**Figure 7-20** Total Scour Components with Potential of Lateral Migration




## 7-4.3 *Scour Countermeasures*

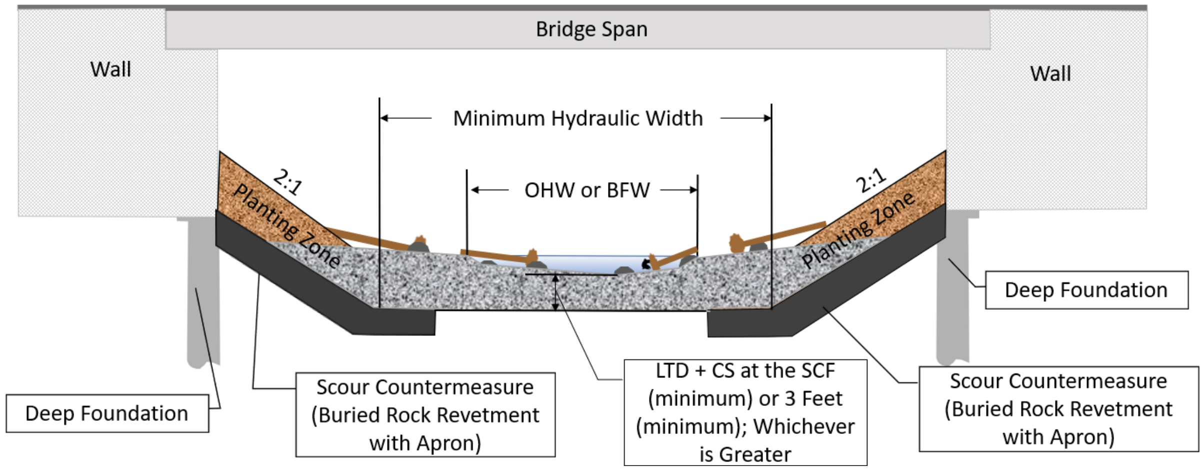
Scour countermeasures are used to protect the structure itself or to protect other elements of the roadway adjacent to a water body and have different design requirements from countermeasures used for stream instability or bank protection. Countermeasure design requirements for stream instability and bank protection are provided in Chapter 4. Scour countermeasures are required when stable wood is proposed and may be required when mobile wood or other large complexity features are proposed; refer to Section 7-3.6 and coordinate with the State Hydraulics Office. When a scour countermeasure is necessary, the specialty report shall document the risk to the infrastructure asset and rationale for the protection, any current evidence of erosion, and the countermeasure design standard. See HEC-23 Volume 1 and Volume 2 for additional guidance on the implementation of scour countermeasures.

For new structures, scour countermeasures shall not encroach within the minimum hydraulic width and depth of scour. The depth of scour is determined as LTD + contraction scour at the scour check flood (minimum) or a minimum of 3 feet, whichever is greater, unless approved by the State Hydraulics Office. The design of scour countermeasures first relies on an understanding and agreement of the asset they intend to protect and the required design standard for the asset. Elements of a water crossing that may need a scour countermeasure include but are not limited to the abutments, roadway approach walls, and the roadway embankment. Each of these elements can have varying levels of acceptable risk and thus different design standards. Scour countermeasure may be used to prevent scour at deep foundation abutments when recommended by the hydraulic team or Stream Team and the project shall require maintenance access per the Roadside Manual 830. When used with deep foundation, scour countermeasure rock class shall exceed the required design by one rock class. Figure 7-21 and Figure 7-22 provide conceptual sketches for where a scour countermeasure can be placed in relation to the minimum hydraulic width and depth of scour for a water crossing in a fish-bearing stream with and without abutment scour, respectively. The limits of scour countermeasure shall be determined based on the lateral migration determination process; see Sections 7-2.3.3 and 7-4.2. In the examples shown in

Figure 7-19 and Figure 7-20, the bridge is founded on deep foundations, which are designed to meet HEC-18 requirements and do not rely on the integrity of the scour countermeasure. The Stream Team shall also consider the effect of any placed habitat features and ensure that the opposite banks are properly stabilized and that the revetment will not become exposed because the stream migrates around, and interacts with, the habitat features.

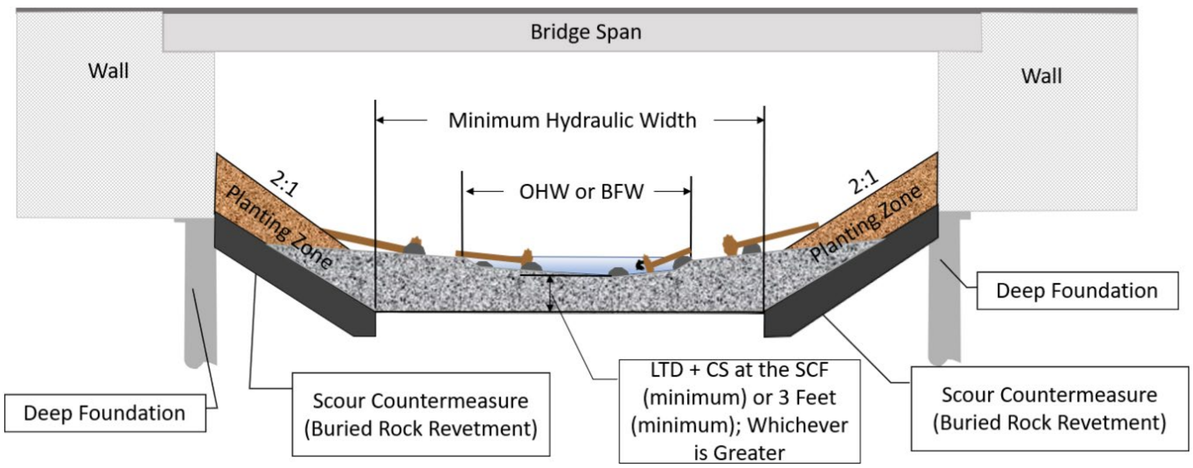
Also depicted in Figure 7-21 is a very important but often overlooked scour countermeasure feature for water crossings with abutment scour, the apron. Guidance for design of the apron can be found in HEC-23, Volume 1 and Volume 2 and the FHWA *TechBrief: Hydraulic Considerations for Shallow Abutment Foundations* (FHWA 2020). The example figures also contain curtain walls, which assist to retain the roadway embankment fill and were decided by the PEO, for this specific crossing, to rely on the integrity of the scour countermeasure for their design. Because of the site-specific nature of water crossings, the State Hydraulics Office shall be contacted to assist in coordinating with the appropriate subject matter experts to determine the design standards for the scour countermeasure and the level of protection they can assume to provide for a given asset. If scour countermeasures are included in the design, a maintenance access shall be included as part of the project to access the stream for future repairs as needed. The Stream Team shall coordinate with Maintenance to determine what is required for access.

**Figure 7-21** Scour Countermeasure Design with Deep Foundation and Calculated Abutment Scour Greater than Zero




Coordinate with the project landscape architect regarding planting details.

**Figure 7-22** Scour Countermeasure Design with Deep Foundation and Calculated Abutment Scour of Zero




Coordinate with the project landscape architect regarding planting details.

## 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 7-3, other available design methodologies can be accepted on a case-by-case basis with the approval of the State Hydraulics Office. This section briefly describes some of the other methodologies available.

Some of these design methodologies may need to include project objectives with performance measures, inspection schedules, maintenance triggers, and a contingency plan shall 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 WAC. The no-slope designs are 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 typically used on WSDOT water crossings and requires approval from the State Hydraulics Office.

## 7-5.2 *Fish Passage Improvement Structures*

Fish passage 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 passage improvement structures. Fish passage improvement structures require approval from the State Hydraulics Office. Additional information about roughened channels, roughened rock ramps, and structural retrofits is included below. Other fish passage improvement structures exist but are not covered here.

A fish passage 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 passage improvement structures must meet WAC 220-660-200.

### 7-5.2.1 Roughened Channel Design Methodology

A roughened channel is a constructed channel with streambed material and configuration designed to be non-deformable up to the design flood event. 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 retrofit will be allowed within WRIAs 1 through 23 because of the Injunction.

## 7-5.3 *Tidal Crossing Structures*

Tidal crossings are those water crossings on state highways in which the hydraulics are either influenced or dominated by tidal cycles that must be considered in the crossing design. Flow through structures at tidal sites are bi-directional and typically subject to a mixed semi-diurnal tidal cycle, unlike the one-way flow of riverine systems. Mixed semi-diurnal tides have two unequal high and low tides each tidal day (24 hours and 50 minutes).

At tidally influenced crossings it is necessary to assess the hydraulics through the tidal cycle as well as during events such as the tidal flood event and in conjunction with the design riverine flood event. Site assessments using topographic data compared with local tidal datums (refer to Section 7-3.5.4) can be used to evaluate the thalweg elevation relative to the local tidal datums. Sites with thalweg elevations at or below mean sea level are likely to be tidally influenced or dominated, depending upon the tidal prism. The tidal prism is the volume of water that is exchanged during a typical tidal cycle, excluding freshwater flow; the greater the tidal prism that is exchanged, the higher the design velocity. 2D modeling may be used to evaluate tidal hydraulics for tidally influence and tidally dominated crossings.

Crossings of embayments and lagoons with substantial tidal prisms would typically be tidally dominated for freeboard, scour, and stability. The location of a crossing at an embayment or lagoon must consider the effects of local waves and nearshore sediment transport on channel stability and meandering. Embayment and lagoon crossings may experience muted tide ranges because of local bathymetry of the typical shallow bays and estuaries where these crossings are located. Depending on the tidal prism, natural embayments and lagoons may have velocities that regularly exceed desirable fish passage velocities during peak ebb and flood tides. 2D modeling shall be applied to evaluate the typical range of velocities during typical spring and neap tides, in addition to flood event scenarios.

Crossings of coastal creeks are not typically associated with substantial tidal prisms and therefore are not typically tidally dominated. However, design freeboard, scour, and stability may be governed by either tidal or riverine processes depending upon local conditions. 2D modeling shall be applied to evaluate the typical range of water levels and velocities during typical spring and neap tides, in addition to flood event scenarios that combine both riverine and tidal events, to determine the governing processes for hydraulic design. Where tidal creek crossings occur at or near the shoreline, structure design shall incorporate study of coastal geomorphology on past, present, and future conditions.

River deltas are typically broad-low gradient areas that require long crossings to minimize impact to wetlands, essential fish habitat, flooding, and nearshore processes. Depending on river basin size, the sites may fluctuate between river and tidal dominance. 2D modeling shall be applied to evaluate the typical range of water levels and velocities during typical spring and neap tides in addition to flood event scenarios that combine both riverine and tidal events to determine the governing processes for hydraulic design.

Relative sea level rise (RSLR) data shall be acquired from NOAA or another appropriate source and validated using on-site observations. RSLR refers to sea level rise adjusted for changes in local land elevation due to either subsidence or glacial rebound. WSDOT recognizes that coastal terrain can be highly variable and that there may be no nearby tidal gage. In such instances, it is acceptable to use data from the nearest gage and adjust the data as necessary to obtain a tidal hydrograph that corresponds with field observations. Structure design must consider the RSLR in addition to the predicted 2080 100-year increase in riverine flow unless otherwise justified. A king tide event shall also be used in the hydraulic analysis unless otherwise justified.

It is not necessary to design a crossing that spans the full extent of the Tidal Design Event provided that there is a point of diminishing returns in terms of hydraulics in relation to structure size. 2D modeling shall be used to determine the point of diminishing returns.

Scour must be evaluated at tidal crossings; refer to HEC-25 for guidance on estimating scour at tidal structures.

Modeling guidance is provided in Section 4-8.

# 7-6 Structure-Free Zone

The SFZ is an imaginary prism of infinite length both upstream and downstream that is horizontally centered on the stream and represents the minimum boundary within which no part of the fish passage structure (footings, chamfers, etc.) shall be allowed (Plan Sheet Library).

The components of the SFZ that determine the boundaries are width, height, and length. The specialty report documents the MHO (width and height including freeboard, scour, and bed thickness), and length of the structure. However, there may be other reasons to increase the SFZ that are not hydraulic related, such as constructibility, maintenance access, wildlife connectivity, or cost, and the specialty report does not document justification for additional width or height outside of what is necessary to allow for stream processes.

## 7-6.1 *Complete Streets and Effect on Structure-Free Zone*

The inclusion of active transportation design elements or application of the Complete Streets program (see Section 1-5) could have an impact on the SFZ. It may be recommended to increase the hydraulic length of a water-crossing structure to accommodate pedestrians, bicyclists, or any other type of network connectivity supported by the program; discuss with the PEO whether an increase in hydraulic length is appropriate. If deemed appropriate, discuss with the PEO if a resulting increase in hydraulic width could be warranted.

# 7-7 Temporary Stream Diversions

Temporary stream diversions shall be designed following the methodology described in Chapter 3. Under most circumstances, determination of the design and configuration of temporary diversions for streams is left to the contractor. This allows the contractor to create the most efficient and innovative work plan. If the PEO wishes to design the temporary diversions, coordination with the State Hydraulics Office is required.

# 7-8 Monitoring

In September 2015, as part of the Culvert Injunction, state agencies and tribal nations agreed upon and finalized a set of Injunction 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.

The Fish Passage Monitoring Plan provides a protocol that can be broadly applied to ensure a consistent and efficient post-project monitoring process for all WSDOT fish passage projects. WSDOT's Fish Passage Monitoring Plan and the Injunction Implementation Guidelines are available by request from the State Hydraulics Office. Fish passage monitoring results are available for barriers corrected since 2013, and are available publicly online through WSDOT's interactive Fish Passage Webmap; click on a corrected barrier and select "more info" under the site attributes (reports available for barriers corrected since 2013).

There are four basic types of monitoring inspections:

- **Post-construction compliance inspection:** WSDOT evaluates all fish passage projects to ensure that 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 are evaluated 5 and 10 years after construction to determine whether the project still provides fish passage and stream function. Monitoring protocols described for the Over-Winter inspection will be repeated to determine if the project still meets design expectations.
- **Additional monitoring:** Ad hoc evaluations can take place anytime between regular monitoring intervals at the discretion of the WSDOT monitoring biologist to reevaluate project performance based on responses recorded during a previous assessment.

The results of the monitoring efforts are summarized each year in the Fish Passage Annual Report, which can be found on the WSDOT Fish Passage Program website. WSDOT uses the information from the monitoring efforts to work alongside 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.

### 7-8.1 *Streambed Camera Monitoring*

Since July 2021, WSDOT has included monitoring with cameras for selected fish passage sites. The purpose of monitoring with cameras is to collect live data during storm events to observe complexity features and evaluate how the streams are reacting/adjusting during various flow conditions, including winter storm events and during summer low flow periods. The data are used to validate the design technique and inform design changes to improve the overall function of stream features.

Pre-project streambed camera monitoring data that are available will be shared with the Stream Team. Contact the State Hydraulics Office for additional information on available data. The time-lapse photos/videos may inform design features including:

- Sediment observations (mobility, supply, erosion/scour, degradation/aggradation)
- LWM (transport, presence, racking)
- High flow events with associated high water marks (validate hydrology)
- Beaver activity
- Wildlife observations
- Low flow events/dry channel (in summer or not)
- Mobility of habitat features (wood, steps)
- Seasonal channel variation with roughness

Post-construction data, trends, and observations will be reviewed, distributed, and communicated to the State Hydraulics Office. Observations that could inform the design may include meander bars, step pools, and LWM. Any items of concern will be communicated and may trigger additional monitoring and potential adjustment to design criteria.

## 7-9 Performance Management

WSDOT is committed to managing fish passage sites to ensure continued fish passage and stream function. WSDOT's goal for performance management is to continuously improve policies, practices, and design guidance by learning from outcomes of post-project monitoring.

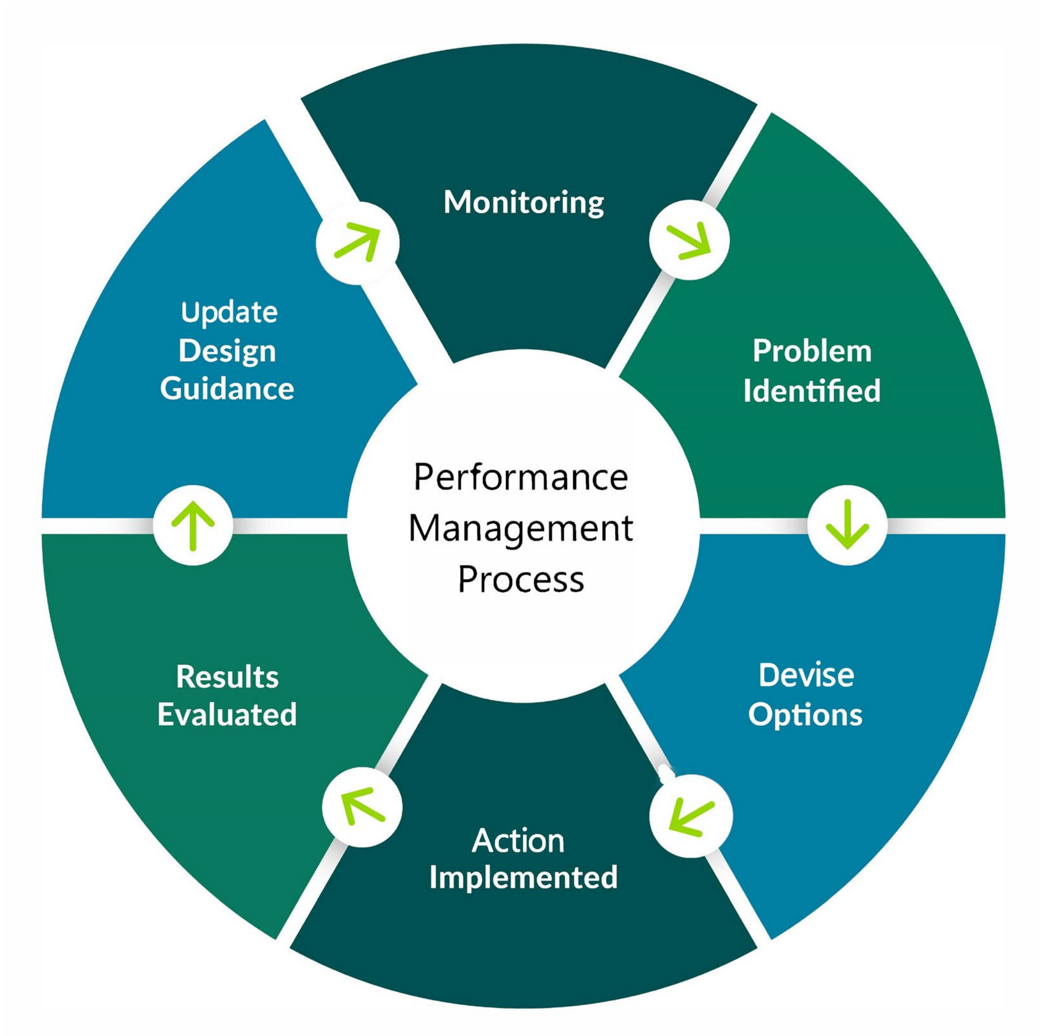
Monitoring is conducted by HQ Stream Restoration Program staff and reviewed by the Fish Passage Monitoring and Performance Coordinator. Any project trending toward becoming a barrier to fish passage or losing stream function receives an increase in the frequency of monitoring for a period to determine if an action is needed to correct the deficiency. If an observed deficiency is noted during the monitoring process as described above that hinders fish passage or stream function, the WSDOT performance management process is initiated (see Figure 7-23). WSDOT's performance management process is for repairs or modifications that are deemed necessary to maintain fish passage or stream function.

Once an action is proposed, the Fish Passage Monitoring and Performance Coordinator notifies the State Hydraulics Office and Regional Project Office of the status and refers it for further hydraulic evaluation. The State Hydraulics Office will either refer it back to Stream Restoration for continued monitoring or assign a status of action needed; if action is needed, the State Hydraulics Office will draft a technical memorandum documenting the design conditions, the existing conditions, and a concept for repair (not yet a barrier condition) or modification (barrier condition).

The State Hydraulics Office determines the appropriate repair or modification options and refines the technical memorandum into a Fish Passage Performance Management Recommendation document. The document is provided to the region for implementation.

Once a correction is designed, permitted, and implemented, the modification or repair is monitored for success and the design guidance is reviewed for potential updating. Contact the State Hydraulics Office for more information.

**Figure 7-23** WSDOT's Performance Management Process




## 7-10 Additional Resources

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

- HEC-16: Highways in the River Environment: Roads, Rivers, and Floodplains (FHWA 2023b)
- 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, Volume 1 and Volume 2

- HEC-25: Highways in the Coastal Environment
- *TechBrief: Hydraulic Considerations for Abutments on Deep Foundations and Bridge Embankment Protection* (FHWA 2023a)
- *TechBrief: Hydraulic Considerations for Shallow Abutment Foundations* (FHWA 2020)
- 2013 WDFW WCDG
- 2008 USFS Manual: Stream Simulation: An Ecological Approach to Providing Passage for Aquatic Organisms at Road-Stream Crossings
- WDFW ISPG
- WDFW *Stream Habitat Restoration Guidelines* (Cramer 2012)

## 7-11 Appendices

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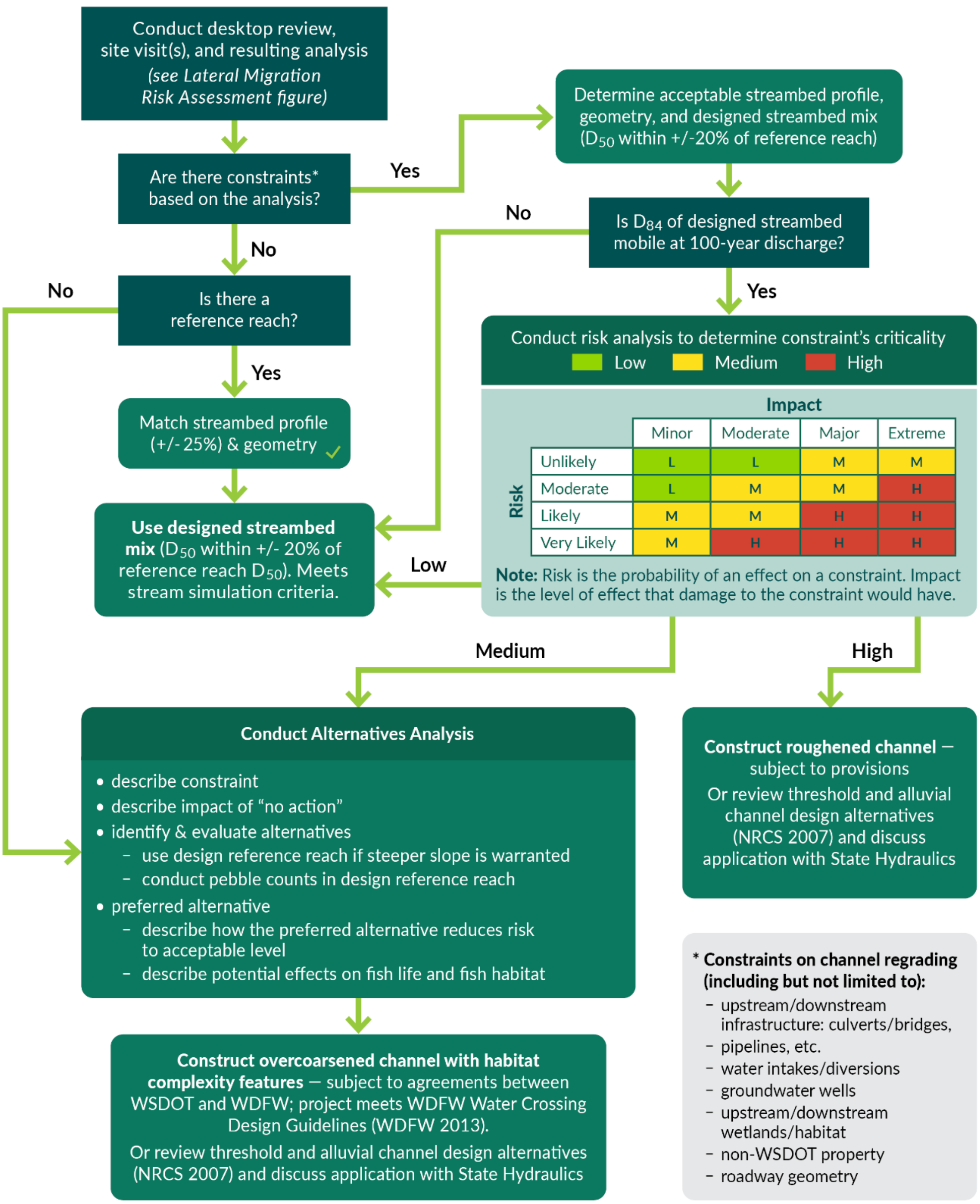
|---|---|
| Appendix 7A | Streambed Material Decision Tree |
| Appendix 7B | Stream Simulation and Bridge Design Methodology Requirements |

# *Appendix 7A 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.*

# Appendix 7B Design Methodology Requirements for Bridges and Stream Simulation Culverts

| Stream crossing element | Goals | Stream Simulation Methodology | Bridge Design Methodology |
|---|---|---|---|
| **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. | WAC: 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.<br><br>WDFW: Appendix C provides recommended methods to determine bankfull width.<br><br>WSDOT: Bankfull in highly modified (urban/agricultural) determined by hydraulic modeling, reference reach or comparative analysis. See WSDOT Hydraulics Manual, Chapter 7, Water Crossings. | |
| **Channel slope/gradient** | The slope of the bed inside the culvert is within 25% of the slope of the upstream channel. | WAC: 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.<br>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.<br><br>WDFW: The slope of the bed inside a stream-simulation culvert must not exceed the slope of the upstream channel by more than 25%. ($S_{culvert}/S_{upstream}$ ch) < 1.25<br>Slope ratios greater than 1.25 require a bridge or the application of the Hydraulic Design Option, specifically, the roughened channel option.<br><br>WSDOT: Slope ratio greater than 1.25 or more than 1' of uncontrolled regrade needs justification. In low-gradient systems, provide explanation if designed gradient is outside slope ratio. See WSDOT Hydraulics Manual, Chapter 7, Water Crossings.<br>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 long as the necessary embedment depth and freeboard are met and the engineering justification is provided. | |
| **Countersink/scour** | Bridge foundation / culvert bottom does not become exposed for life of structure and substrate size is similar to adjacent channel. | WAC: 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.<br><br>WDFW: 30%–50%, not less than 2 feet unless justified by analysis.<br><br>WSDOT: WSDOT designs all water crossing structure foundations (bridges and culverts) to account for total scour at the scour design flood and scour check flood. A minimum of 3 feet of total scour is required to be assumed for all bridges and three-sided buried structures. See WSDOT Hydraulics Manual, Chapter 7, Water Crossings. | WAC: 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.<br><br>WDFW: Follow AASHTO and FHWA guidelines. Prevent or limit local scour and coarsening of the stream substrate.<br><br>WSDOT: WSDOT designs all water crossing structure foundations (bridges and culverts) to account for total scour at the scour design flood and scour check flood. A minimum of 3 feet of total scour is required to be assumed for all bridges and three-sided buried structures. See WSDOT Hydraulics Manual, Chapter 7, Water Crossings. |
| **Scour countermeasures** | Minimize risk to the structure or elements of the roadways from scour by using scour countermeasures. | WSDOT: Stable wood within the structure requires scour countermeasures; mobile wood may require scour countermeasures. See WSDOT Hydraulics Manual, Chapter 7, Water Crossings. | WAC: The bridge design must minimize the need for scour protection. Where midchannel 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.<br><br>WDFW: Encroachments of abutments or embankment end slopes into the bankfull channel is unacceptable.<br>Riprap placed above Q100 elevation does not require mitigation for instream functions unless the bridge span is inadequate to allow meander migration or the rock significantly affects riparian vegetation.<br><br>WSDOT: Stable wood within the structure requires scour countermeasures; mobile wood may require scour countermeasures. See WSDOT Hydraulics Manual, Chapter 7, Water Crossings. |
| **Channel geometry / cross section** | Continuity of channel shape maintained throughout reach [channel complexity]. | WAC: All water crossings must retain upstream and downstream connection in order to maintain expected channel processes.<br>If the channel is heavily degraded, the cross section must match expected stream measurements in order to limit main crossing channel velocity and scour to prevailing conditions.<br><br>WDFW: 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.<br>Bed cross section should be similar to the adjacent stream cross section.<br><br>WSDOT: See WSDOT Hydraulics Manual, Chapter 7, Water Crossings. | WAC: 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.<br><br>WDFW: The stream channel created or restored near the bridge should have a gradient and cross section similar to the existing morphology of the upstream and downstream adjacent channel.<br><br>WSDOT: See WSDOT Hydraulics Manual, Chapter 7, Water Crossings. |

| Stream crossing element | Goals | Stream Simulation Methodology | Bridge Design Methodology |
|---|---|---|---|
| **Floodplain continuity** | Constructed channel mimics adjacent floodplain habitat conditions and allows for floodplain connectivity. | WAC: 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.<br><br>WSDOT: See WSDOT Hydraulics Manual, Chapter 7, Water Crossings. | WAC: 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 encroachment 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 event or the design flood event 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.<br><br>WDFW: Allow continued down-valley flow of water on the floodplain. The bridge/culvert design must comply with legislation governing development within floodplains.<br><br>WSDOT: If the V2/V1 is less than 1.1, no additional justification needed. If V2/V1 is greater than 1.1, State Hydraulics Office approval is needed. See WSDOT Hydraulics Manual, Chapter 7, Water Crossings. |
| **Freeboard** | Crossing provides unimpeded passage of fish, 100-year flood event, LWM, and sediment. | **WDFW**: Culverts shall be installed to an approved design to maintain structural integrity to the 100-year flood event with consideration of the debris loading likely to be encountered. A list of suggested clearances is provided, though the values are not based on hydraulic modeling or empirical studies and therefore should be used with caution.<br>-Small streams less than 8 ft BFW: clearance of 1 foot above the 100-year water surface<br>-Medium streams from 8-15 ft BFW: clearance of 2 feet above the 100-year water surface<br>-Large streams over 15 ft BFW: clearance of 3 feet above the 100-year water surface<br><br>**WSDOT**: Same as listed above substituting the 100-year event with the 2080 100-year projected flood event. See WSDOT Hydraulics Manual, Chapter 7, Water Crossings. | **WAC**: 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 flood event unless engineering justification shows a lower clearance will allow the free passage of anticipated debris.<br><br>**WDFW**: Culverts shall be installed to an approved design to maintain structural integrity to the 100-year flood event with consideration of the debris loading likely to be encountered. A list of suggested clearances is provided, though the values are not based on hydraulic modeling or empirical studies and therefore should be used with caution.<br>-Small streams less than 8 ft BFW: clearance of 1 foot above the 100-year water surface<br>-Medium streams from 8-15 ft BFW: clearance of 2 feet above the 100-year water surface<br>-Large streams over 15 ft BFW: clearance of 3 feet above the 100-year water surface<br><br>**WSDOT**: A minimum of 3 feet of freeboard above the 100-year or 2080 100-year projected flood event is required on all structures greater than 20 feet in span measured along the centerline of the roadway and on all bridge structures. See WSDOT Hydraulics Manual, Chapter 7, Water Crossings. Additional justification possible when recommended freeboard is not achievable. |
| **Substrate** | Channel substrate mimics reference reach. | **WAC**: 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.<br><br>**WDFW**: 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.<br><br>**WSDOT**: Streambed Material Decision Tree and WSDOT Hydraulics Manual, Chapter 7, Water Crossings. | **WAC**: 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.<br><br>**WDFW**: 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.<br><br>**WSDOT**: Streambed Material Decision Tree and WSDOT Hydraulics Manual, Chapter 7, Water Crossings. |

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| **Structure span** | Crossing width (span) allows for geomorphic processes to occur including 100-year flood event; minimize the need for scour protection: maintain structural integrity for the duration of the design life; maintain water and sediment transport continuity. | **WAC:** 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.<br><br>**WDFW:** 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.<br><br>**WSDOT:** Starting point for sizing is 1.2*BFW+2 or 1.3*BFW (the larger of the two). A meander belt assessment shall be conducted for all crossings to determine if there are any changes to the minimum hydraulic width. If a structure length is more than 10 times its width, then the hydraulic width shall be increased to whichever is greater, a 30% increase or incorporate the width necessary for the natural meander as determined through the meander belt assessment. See WSDOT Hydraulics Manual, Chapter 7, Water Crossings. | **WAC:** 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 event or the design flood event 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.<br><br>**WDFW:** 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 floodplain 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.<br><br>**WSDOT:** Starting point for sizing is 1.2*BFW+2 or 1.3*BFW (the larger of the two). The confined bridge methodology may include an additional factor of safety. The unconfined bridge methodology requires the hydraulic opening to provide a velocity ratio of less than 1.1 (see "floodplain continuity" row). A meander belt assessment shall be conducted for all crossings to determine if there are any changes to the minimum hydraulic width. See WSDOT Hydraulics Manual, Chapter 7, Water Crossings. |
| **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 bridge/culvert inlet and from the bridge/culvert outlet. | **WDFW**: Culverts with a length-to-span ratio of greater than 10 are considered long and special consideration should be given to their design. Three alternatives for long culverts are proposed; the first two suggest increasing width and the third a change of crossing type.<br><br>**WSDOT**: If a structure length is more than 10 times its width, then the hydraulic width shall be increased to whichever is greater, a 30% increase or incorporate the width necessary for the natural meander as determined through the meander belt assessment (see "culvert size" row). | **WSDOT**: See WSDOT Hydraulics Manual, Chapter 7, Water Crossings. |
| **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. Determine if unconfined bridge design criteria are adequate for the bridge or buried structure. | **WDFW**: FUR < 3 indicates a confined channel where a culvert is better suited. FUR is defined as the flood-prone width (FPW) divided by the bankfull width (BFW).<br><br>**WSDOT**: When FUR > 3, use unconfined bridge method for minimum channel span.<br>Measure FUR outside the influence of any crossing structures. See WSDOT Hydraulics Manual, Chapter 7, Water Crossings. | **WSDOT**: Measure FUR outside the influence of any crossing structures. See WSDOT Hydraulics Manual, Chapter 7, Water Crossings. |
| **Streambank protection / stabilization** | Minimize armoring (use of riprap or concrete) and use bio-engineering techniques where appropriate. | WAC: Any proposed bank hardening must include:<br>(i) An analysis performed by a qualified professional assessing the level of risk to existing buildings, roads, or services being threatened by the erosion;<br>(ii) Technical rationale specific to the project design, such as a reach and site assessment;<br>(iii) Evidence of erosion and/or slope instability to warrant the work.<br>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:<br>(i) No action-Natural channel processes to occur;<br>(ii) Biotechnical techniques;<br>(iii) Combination of biotechnical and structural techniques; and<br>(iv) Structural techniques<br>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 flood event.<br>Use natural materials whenever feasible, including large wood and vegetation; protect existing spawning and rearing habitat.<br><br>WDFW: See Integrated Streambank Protection Guidelines (WDFW 2002)<br><br>WSDOT: See WSDOT Hydraulics Manual Chapter 4, Open-Channel Flow. | |

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| **Hydrology / design flood events** | Correlate to watershed conditions and land use, while avoiding over-engineered channels and banks. Develop design flood events that accurately reflect watershed conditions, including future conditions. | **WDFW**: See (Appendix G) Design Flows for Fish Passage<br><br>**WSDOT**: Address potential effects of extreme events (e.g., 500-year); climate resilience should also be considered as current science suggests that both the magnitude and frequency of peak flows are expected to increase (WDFW 2016a).<br><br>See WSDOT Hydraulics Manual, Chapter 2, Hydrology and Chapter 7, Water Crossings for design flood events and guidelines. |

**NOTES:**
This table provides a brief summary of design criteria. It is recommended to read the full design criteria in each of the references to fully understand water crossing methodology and how the design criteria may apply to each water crossing site. In this table, the references denoted by bold and underlined characters are listed below.
**WAC** refers to the Washington Administrative Code 220-660-190 Water Crossing Structures or 220-660-130 Stream Bank Protection and Lake Shoreline Stabilization, published in 2015.
**WDFW** refers to the Water Crossing Design Guidelines, published in 2013.
**WSDOT** refers to the current WSDOT Hydraulics Manual.

# Chapter 8 Pipe Classifications and Materials

## 8-1 Introduction

WSDOT uses several 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
- Sanitary sewer pipe

Each category is intended to serve specific purposes and is described further in Section 8-2.

Within each pipe classification 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 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 [AWWA]) 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 used 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.

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 using 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. Underdrain applications use 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* list 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 *Plan Sheet 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 State Hydraulics Office 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 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 alternatives available for a given culvert diameter and fill height. Additionally, Figure 8-8, Figure 8-10, and Figure 8-12 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.

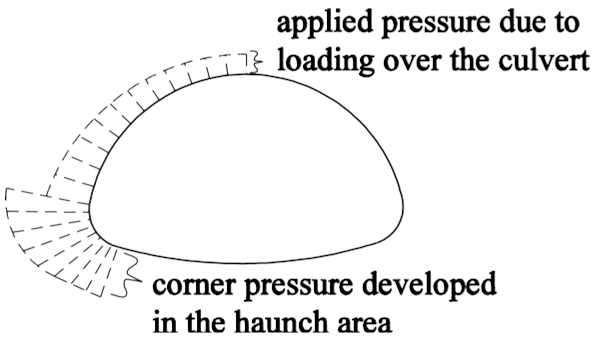
The type of material for circular culvert pipe measuring 54 to 120 inches in diameter shall be designated on the plan sheets. The structure notes sheet shall include any acceptable alternative material for that particular installation. A schedule table for these large sizes has not been developed because of 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 most 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. Because of 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

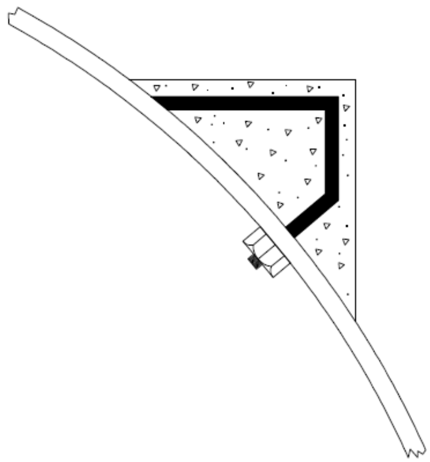
Structural plate culverts are steel or aluminum structures delivered to the project site as unassembled plates of material and bolted together. Structural plate culverts are large diameter—from 10 to 40 feet or more—and are available in several different shapes including circular, pipe arch, elliptical, 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 (oz/ft$^2$) of galvanized coating on each plate surface (galvanized culvert pipe is manufactured with 1 oz/ft$^2$ of galvanized coating on each pipe surface). The design 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 gage 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 State Hydraulics Office shall 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.

For the purpose of this chapter the terms “access,” “approach,” and “driveway” are referred as “driveway” to remain consistent with the WSDOT *Design Manual*.

#### *8-2.3.4.1* Applicable Criteria

The requirements in this section apply to a drainage pipe constructed within an existing WSDOT drainage ditch to accommodate and maintain stormwater drainage underneath a driveway. Driveway culverts are off the main line of the highway, so minimal hazard is presented to the traveling public if a failure occurs. The requirements for culverts placed under driveways are less stringent than the requirements for culverts placed under roadways except those identified as fish barriers by WDFW. Fish barrier private road approach and driveway culverts need to follow WDFW water crossing design guidelines. Culverts that cross bioswales are treated in a different manner. See Section 8-2.3.4.9.

#### *8-2.3.4.2* Culvert Replacement

At a minimum, the replacement culvert shall have the same size, slope, and material type as the existing culvert. If the culvert is replaced because of the failure of the existing culvert, an appropriate hydraulic evaluation shall be done to prevent future problems.

#### 8-2.3.4.3 Construction Material

Within the WSDOT ROW, driveway culverts shall be constructed from material selection guidance as described in Section 8-3.

#### 8-2.3.4.4 Minimum Size

Private road approach and driveway culverts shall be sized to pass the 10-year ditch flow capacity without overtopping the driveway. The minimum size for driveway culverts shall be 12 inches in diameter for round pipe or an equivalent cross-sectional area for arch or elliptical shapes.

#### 8-2.3.4.5 Maximum Length

The length of a culvert will vary depending on the connection width, side slopes, and ditch depth. Use the minimum length of pipe necessary to span a driveway plus allow for appropriate end walls because a longer pipe may get clogged more easily, which frequently creates maintenance problems.

#### 8-2.3.4.6 Minimum Cover

Driveway culverts shall be provided with the minimum cover recommended by the pipe structural design requirements, or 1 foot, whichever is greater. It is difficult to provide a minimum 2-foot cover over the top of these culverts. Therefore, private road approach 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).

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 Section 8-12 shall be specified.

#### 8-2.3.4.7 Culvert End Treatments

All driveway culverts shall be provided with end treatments on the upstream and downstream ends of the culvert to protect and help maintain the integrity of the culvert opening. Headwalls and/or wingwalls and flared end sections are acceptable end treatments.

#### 8-2.3.4.8 Minimum Slope

A minimum slope shall be provided to achieve the minimum velocities outlined in Section 3-3.5.

#### 8-2.3.4.9 Design Documentation of Driveway Culverts

Additional information must be included in the drainage report and on the construction drawings for new developments, where the use of roadside ditches and driveway culverts is proposed. Driveway culverts shall be designed and documented in the development's drainage report, based on the tributary area at the downstream lot line. The construction drawings shall include information regarding sizes, materials, locations, lengths, grades, and end treatments for all driveway culverts. Typical driveway crossing/culvert details shall be included in the construction drawings. The construction drawings must address the roadside ditch section in detail to ensure that adequate depth is provided to accommodate the driveway culverts, including the minimum cover, and considering overtopping of the driveway when the culvert capacity is exceeded.

If driveways or approach roads cross a bioswale, the culvert shall be checked to establish that the backwater elevation would not exceed the banks of the swale. See Section 3-4.7 for energy dissipation requirements.

#### *8-2.3.4.10* Culvert Extension

Culvert extension shall be as per guidance outlined in Section 3-3.1.6.

### 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 State Hydraulics Office.

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 less than 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 shall 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 structures shall meet the design criteria as specified in the *Bridge Design Manual* and the *Standard Specifications*. 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. A scour analysis is required for designs of all three-sided structures.

## 8-2.4 *Storm Sewer Pipe*

A storm sewer is defined as one 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 alternatives available for a given culvert diameter and fill height. Additionally, Figure 8-8, Figure 8-10, and Figure 8-12 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 must be pressure tested. Pressure testing indicates the presence of leaking seams or joints or other structural deficiencies that may have occurred during the manufacturing or installation of the pipe. The *Standard Specifications* describe the types of pressure tests that are available.

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. 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 alternatives that will function properly at a reasonable cost.

If one or more of the schedule pipe alternatives 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 limitations of pH for steel and aluminum pipe shall be justified in the hydraulic report (Chapter 1) 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 alternatives; however, the city or county must submit a letter stating its justification. Existing culverts shall be extended with the same pipe material and no alternatives are required.

## 8-3.1 *Concrete Pipe*

This section presents design criteria for concrete pipe, including drain pipe; underdrain pipe; and culvert, storm, and sanitary sewer 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 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. Because of 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. 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 8-15, 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 use 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 shall not be used 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 (Table 4-1). 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. Table 8-3 lists the class of pipe that shall 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 provided in Section 8-10.3. During the installation process, pipe shall 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.

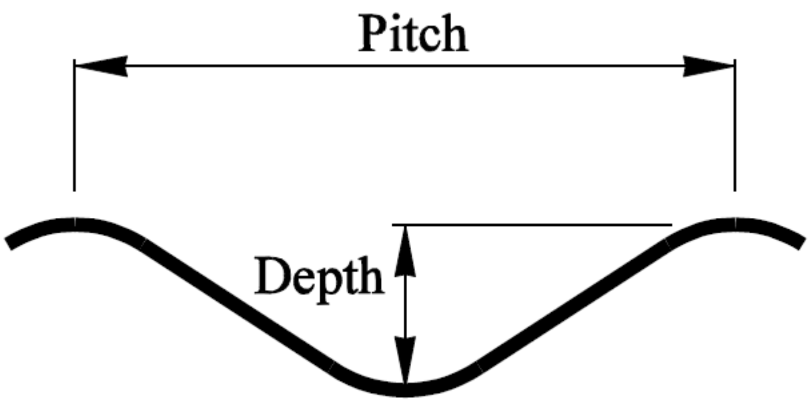
Metal pipe is available in a wide range of sizes and shapes and, depending on the type of material corrugation configuration, can be used with fill heights up to 100 feet or more. Metal pipe is susceptible to both corrosion and abrasion; methods for limiting these issues are covered in Sections 8-5.3 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 gage thicknesses, depending on the pipe diameter and fill height. Larger corrugation sizes are used 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 Table 4-1 to determine if a larger-diameter metal pipe alternative 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 Table 4-1. The fill heights shown in Section 8-12 apply to both helical and annular corrugated metal pipe.

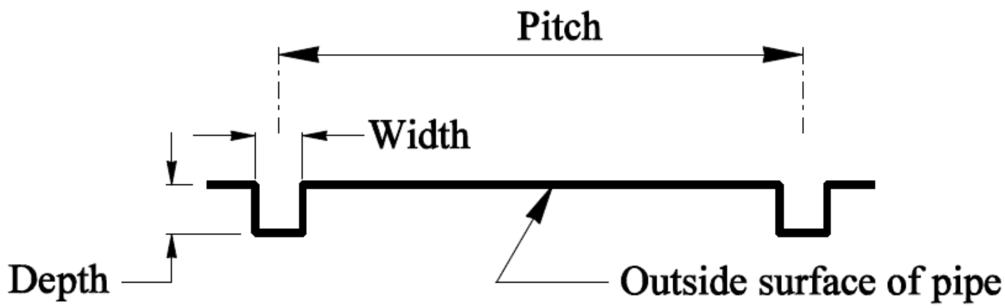
The typical corrugation section shown in Figure 8-3 is the same for annular corrugations, except that annular corrugations are available only in 2⅔-inch by ½-inch and 3-inch by 1-inch sizes.

#### 8-3.2.3 Spiral Rib

Spiral rib pipe uses the same manufacturing process as helically wound pipe but, instead of using a standard corrugation pitch and depth, spiral rib pipe comprises 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 Table 4-1.

**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 5.0 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 Zones I and II without the use of protective coatings. Aluminized steel is not permitted when the pH is less than 5.0 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 Zones 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 shall not be used 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, aluminum pipe generally shall not 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 designed primarily 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 foot of cover in most installations. Deep fill heights are available from manufacturers and concurrence with the State Hydraulics Office. Joint 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 several types of pipes including corrugated polyethylene, solid-wall 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 shall not be used for severely abrasive conditions as described in Table 8-1.

Thermoplastic pipe is lightweight 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 are 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 mitered to match the surrounding embankment or ditch slope. The ends shall be mitered 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 mitered pipe shall be at least 6 times the diameter of the pipe, measured from the toe of the miter to the first joint under the fill slope. 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-75.20-03 headwall shall be used in conjunction with a mitered end.

#### 8-3.3.1 Corrugated Polyethylene for Drains and Underdrains

Corrugated polyethylene 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 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 Polyethylene Culvert and Storm Sewer Pipe

Corrugated polyethylene 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 polyethylene used for culvert applications and polyethylene 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, the joint used for storm sewer applications shall 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 polyethylene 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, polyethylene shall not be allowed as a pipe alternative.

#### 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 8-5. 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 8-5** Typical Profile Wall PVC Cross Sections


#### 8-3.3.6 Polypropylene Culvert and Storm Sewer Pipe

PP pipe is similar in style to corrugated polyethylene 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-tight and watertight.

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 polyethylene pipe.

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

Steel rib reinforced polyethylene pipe has a fairly thin wall profile; the inner wall is smooth, and the outer wall has ribs that are steel encased in polyethylene. 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, water line 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 foot. 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 leakproof connection that is as strong as the pipe itself. There are a wide variety of grades and cell classifications for this pipe; contact the State Hydraulics Office for specific pipe information.

## 8-4 Pipe Corrosion Zones and Pipe Alternative Selection

Once a PEO has determined the pipe classification needed for an application, the next step is to ensure that 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 8-6. A flow chart and corresponding acceptable pipe alternative list have been developed for each of the corrosion zones and are shown in Figure 8-7 through Figure 8-12. The flow charts and pipe alternative lists can be used to develop acceptable pipe alternatives for a given location.

The flow charts and pipe alternative 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 Table 8-1 shall be followed.

When selecting a pipe alternative, the PEO shall 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 used 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 shall 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 Figure 8-7 and Figure 8-8 for a complete listing of acceptable pipe alternatives for culvert and storm sewer applications.)

The following 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. Untreated aluminized steel, aluminum alloy, thermoplastic, and concrete pipe may be used in Corrosion Zone II. (See Figure 8-9 and Figure 8-10 for a complete listing of acceptable pipe alternatives 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 shall 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 shall 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 Figure 8-11 and Figure 8-12 for a complete listing of all acceptable pipe alternatives for culvert and storm sewer applications.)

**Figure 8-6** Washington State Corrosion Zones

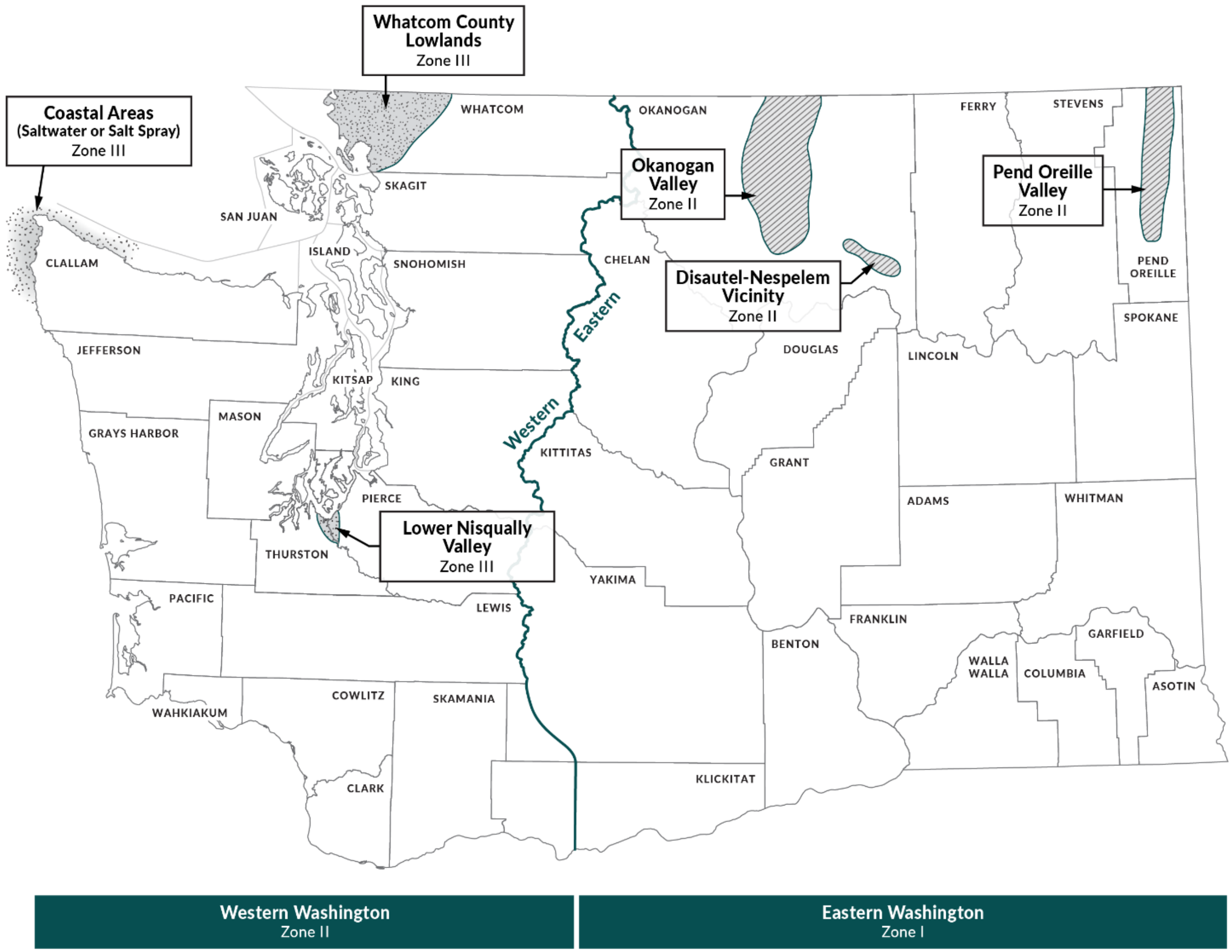


**Figure 8-7** Corrosion Zone I: Flow Chart of Acceptable Pipe Alternatives and Protective Treatments

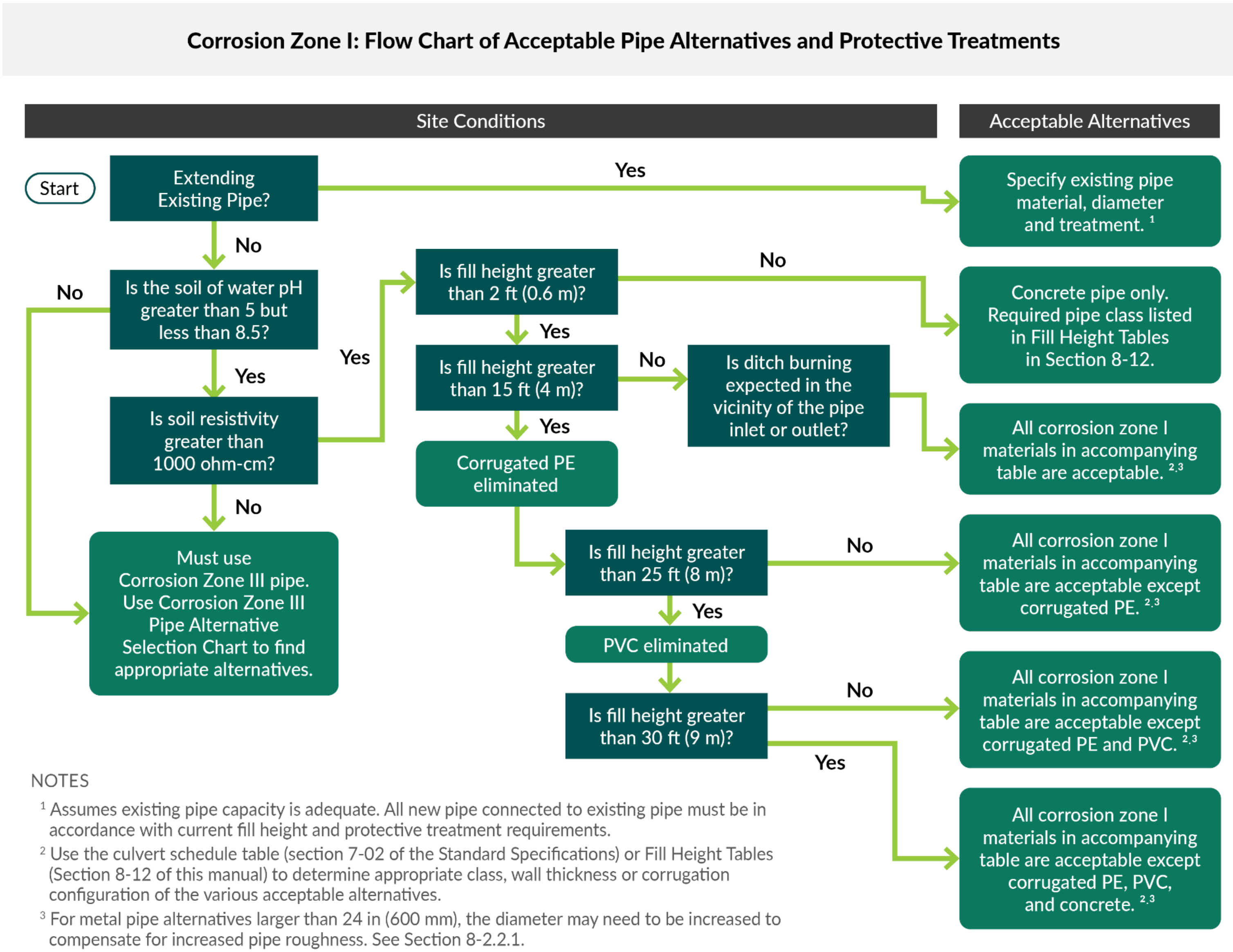
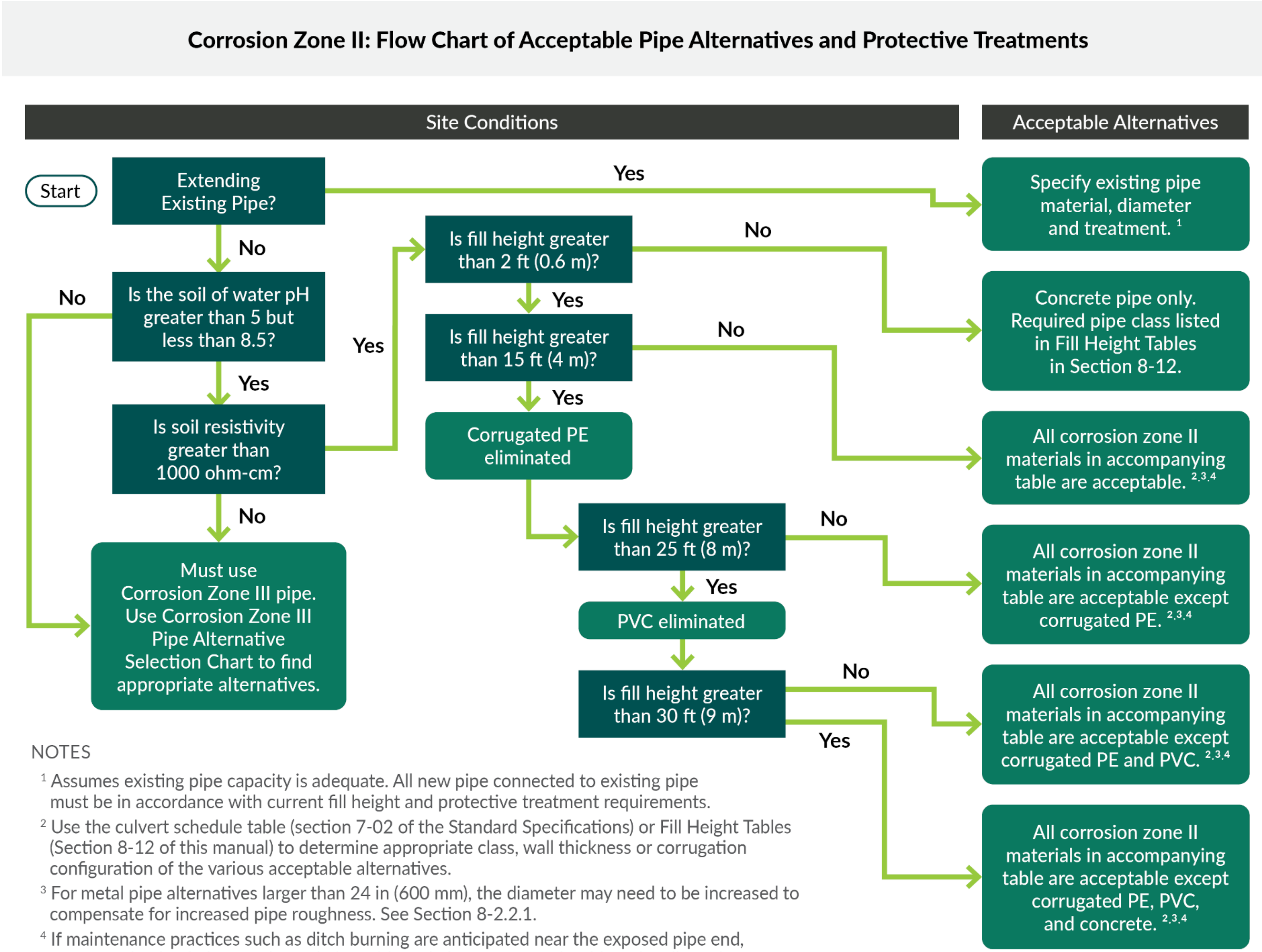


**Figure 8-8** Corrosion Zone I: Acceptable Pipe Alternatives and Protective Treatments

| Culverts | Storm sewers |
|---|---|
| **Schedule pipe:**<br>Schedule_____culvert pipe<br><br>If Schedule pipe not selected, then:<br><br>**Concrete:**<br>• Plain concrete culvert pipe<br>• Cl__reinforced concrete culvert pipe<br>**PVC:**<br>• Solid wall PVC culvert pipe<br>• Profile wall PVC culvert pipe<br>**Polyethylene**<br>• Corrugated polyethylene culvert pipe<br>• Solid-wall HDPE pipe<br>**Polypropylene culvert pipe**<br>**Steel**<br>• Plain galvanized steel culvert pipe<br>• Plain aluminized steel culvert pipe<br>**Aluminum:**<br>• Plain aluminum culvert pipe | **Concrete:**<br>• Plain concrete storm sewer pipe<br>• Cl.___Reinforced concrete storm sewer pipe<br>**PVC:**<br>• Solid-wall PVC storm sewer pipe<br>• Profile-wall PVC storm sewer pipe<br>**Polyethylene:**<br>• Corrugated polyethylene storm sewer pipe<br>• Solid-wall HDPE pipe<br>**Polypropylene storm sewer pipe**<br>**Steel:**<br>• Plain galvanized steel storm sewer pipe with gasketed or welded and remetallized seams<br>• Plain aluminized steel storm sewer pipe with gasketed or welded and remetallized seams<br>**Steel spiral rib:**<br>• Plain galvanized steel spiral rib storm sewer pipe with gasketed or welded and remetallized seams<br>**Aluminum spiral rib:**<br>• Plain aluminum spiral rib storm sewer pipe with gasketed seams |

**Figure 8-9** Corrosion Zone II: Flow Chart of Acceptable Pipe Alternatives and Protective Treatments




NOTES

[1] Assumes existing pipe capacity is adequate. All new pipe connected to existing pipe must be in accordance with current fill height and protective treatment requirements.

[2] Use the culvert schedule table (section 7-02 of the Standard Specifications) or Fill Height Tables (Section 8-12 of this manual) to determine appropriate class, wall thickness or corrugation configuration of the various acceptable alternatives.

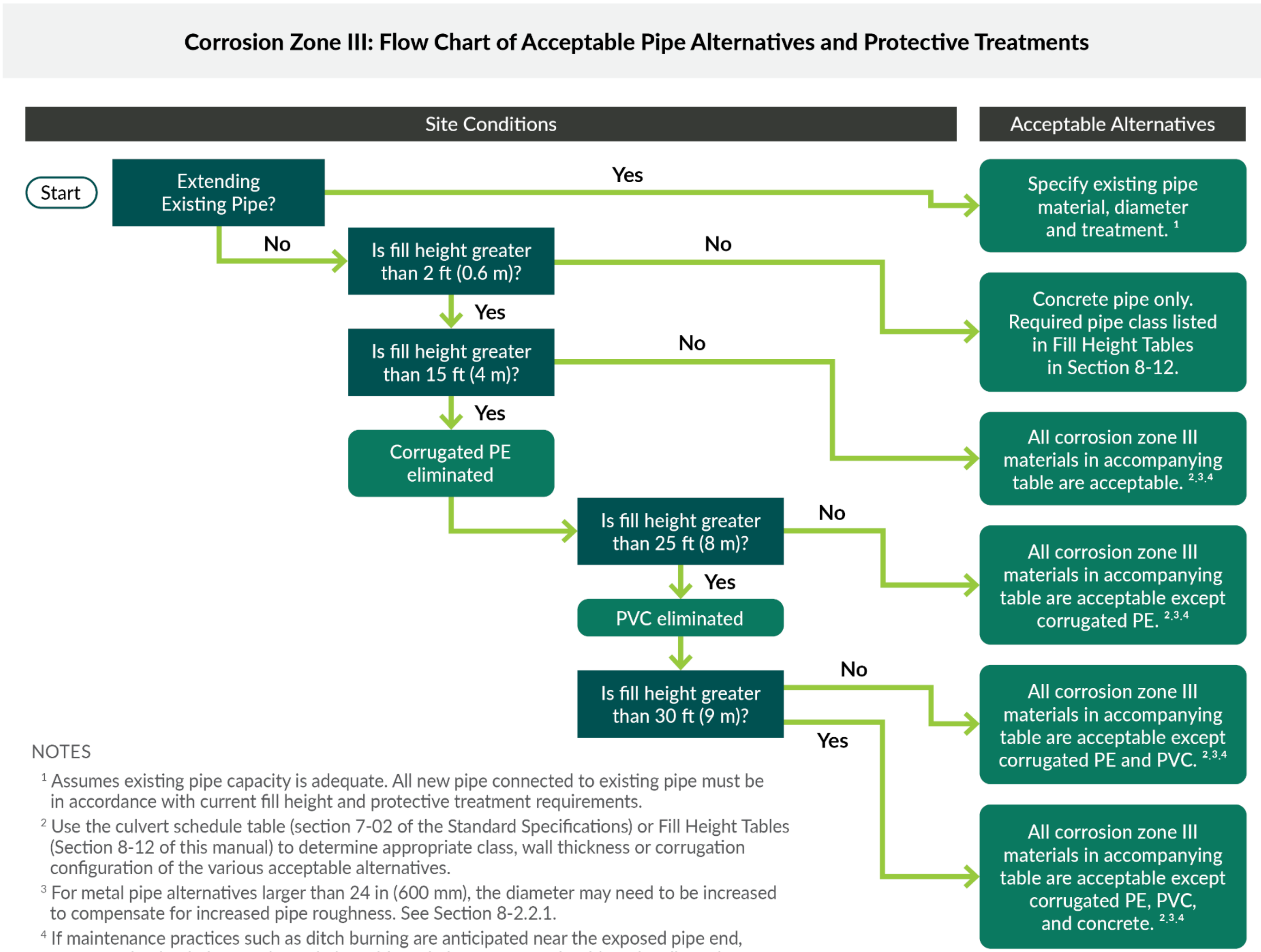
[3] For metal pipe alternatives larger than 24 in (600 mm), the diameter may need to be increased to compensate for increased pipe roughness. See Section 8-2.2.1.

[4] If maintenance practices such as ditch burning are anticipated near the exposed pipe end, corrugated polyethylene and metal pipe with asphalt treatments should not be allowed.

**Figure 8-10** Corrosion Zone II: Acceptable Pipe Alternatives and Protective Treatments

| Culverts | Storm Sewers |
|---|---|
| **Schedule pipe:**<br>Schedule_____culvert pipe<br>If Schedule pipe not selected, then:<br>**Concrete:**<br>• Plain concrete culvert pipe<br>• Cl___reinforced concrete culvert pipe<br>**PVC:**<br>• Solid wall PVC culvert pipe<br>• Profile wall PVC culvert pipe<br>**Polyethylene:**<br>• Corrugated polyethylene culvert pipe<br>• Solid-wall HDPE pipe<br>**Polypropylene culvert pipe**<br>**Steel**<br>• Plain aluminized steel culvert pipe<br>**Aluminum:**<br>• Plain aluminum culvert pipe | **Concrete:**<br>• Plain concrete storm sewer pipe<br>• Cl.___Reinforced concrete storm sewer pipe<br>**PVC:**<br>• Solid-wall PVC storm sewer pipe<br>• Profile-wall PVC storm sewer pipe<br>**Polyethylene:**<br>• Corrugated polyethylene storm sewer pipe<br>• Solid-wall HDPE pipe<br>**Polypropylene storm sewer pipe**<br>**Steel:**<br>• Plain aluminized steel spiral rib storm sewer pipe with gasketed or welded and remetallized seams<br>**Steel spiral rib:**<br>• Plain aluminized steel spiral rib storm sewer with gasketed or welded or welded and remetallized seams<br>**Aluminum:**<br>• Plain aluminum storm sewer pipe with gasketed seams<br>**Aluminum spiral rib:**<br>• Plain aluminum spiral rib storm sewer pipe with gasketed seams |

**Figure 8-11** Corrosion Zone III: Flow Chart of Acceptable Pipe Alternatives and Protective Treatments




NOTES

[1] Assumes existing pipe capacity is adequate. All new pipe connected to existing pipe must be in accordance with current fill height and protective treatment requirements.

[2] Use the culvert schedule table (section 7-02 of the Standard Specifications) or Fill Height Tables (Section 8-12 of this manual) to determine appropriate class, wall thickness or corrugation configuration of the various acceptable alternatives.

[3] For metal pipe alternatives larger than 24 in (600 mm), the diameter may need to be increased to compensate for increased pipe roughness. See Section 8-2.2.1.

[4] If maintenance practices such as ditch burning are anticipated near the exposed pipe end, corrugated polyethylene and metal pipe with asphalt treatments should not be allowed.

**Figure 8-12** Corrosion Zone III: Acceptable Pipe Alternatives and Protective Treatments

| Culverts | Storm Sewers |
|---|---|
| **Schedule pipe:**<br>Schedule____culvert pipe____in. diam.<br>If schedule pipe not selected, then: **Concrete:**<br>• Plain concrete culvert pipe<br>• Cl___reinforced concrete culvert pipe<br>**PVC:**<br>• Solid wall PVC culvert pipe<br>• Profile wall PVC culvert pipe<br>**Polyethylene**<br>• Corrugated polyethylene culvert pipe<br>• Solid-wall HDPE pipe<br>**Polypropylene culvert pipe**<br>**Aluminum:**<br>• Plain aluminum culvert pipe | **Concrete:**<br>• Plain concrete storm sewer pipe<br>• Cl.___Reinforced concrete storm sewer pipe<br>**PVC:**<br>• Solid-wall PVC storm sewer pipe<br>• Profile-wall PVC storm sewer pipe<br>**Polyethylene:**<br>• Corrugated polyethylene storm sewer pipe<br>• Solid-wall HDPE pipe<br>**Polypropylene storm sewer pipe**<br>**Aluminum:**<br>• Plain aluminum storm sewer pipe with gasketed seams<br>**Aluminum spiral rib:**<br>• 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 5.0 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 serve as minimum guidelines for determining the appropriate number of tests for a project:

1. **Size and importance of the drainage structure:** A project comprising large culverts or storm sewers under an interstate or other major arterial warrant testing at each culvert or storm sewer location, while a project comprising small culverts under a secondary highway may need only 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 soil types and geologic conditions, increasing the likelihood of variable pH readings. Tests shall 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 shall 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 5.0 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 is, 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 alternatives. Resistivity tests 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*

This section presents corrosion control methods, including protective treatments and increased gage thickness.

### 8-5.3.1 Protective Treatments

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 polyethylene and acrylic acid copolymer.

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 unusually abrasive or corrosive conditions are anticipated, and it is difficult to determine which treatment would be adequate, either the HQ Materials Laboratory or State Hydraulics Office shall be consulted.

### 8-5.3.2 Increased Gage 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. The California Transportation Department (Caltrans) has developed a methodology to estimate the expected service life of untreated corrugated steel pipes. The method uses 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 State Hydraulics Office.

## 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 Table 8-1.

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.

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.

**Table 8-1 Pipe Abrasion Levels**

| Abrasion Level | General Site Characteristics | Recommended Invert Protection |
|---|---|---|
| Non-abrasive | • Little or no bed load<br>• Slope less than 1%<br>• Velocities less than 3 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<br>• Slopes 1%–2%<br>• Velocities less than 6 ft/s | For metal pipes, an additional gage 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. |
| Moderately abrasive | • Moderate bed loads of sands and gravels, with stone sizes up to about 3 inches<br>• Slopes 2%–4%<br>• Velocities from 6 to 15 ft/s | Metal pipe thickness shall be increased one or two standard gages. The PEO may want to consider a concrete-lined alternative.<br>Concrete pipe and box culverts shall be specified with an increased wall thickness or an increased concrete compressive strength.<br>Thermoplastic pipe may be used without additional treatments. |
| Severely abrasive | • Heavy bed loads of sands, gravel, and rocks, with stone sizes up to 12 inches or larger<br>• Slopes steeper than 4%<br>• Velocities greater than 15 ft/s | Metal pipe thickness shall be increased at least two standard gages, or the pipe invert shall be lined with concrete.<br>Box culverts shall be specified with an increased wall thickness or an increased concrete compressive strength.<br>Sacrificial metal pipe exhibits better abrasion characteristics than metal or concrete. However, it generally cannot be reinforced to provide additional invert protection and shall not be used 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* shall 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 above ground 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 polyethylene tubing may be used without the need for anchors because there are no joints in the pipe. On larger pipes, solid-wall HDPE pipe with fused joints may be used without the use of pipe anchors. For further design guidance, contact the State Hydraulics Office.

### 8-8.1 *Thrust Blocks*

Thrust blocks shall 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 1,000 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-90.40-01, Standard Plan for Concrete Thrust Block, for details on placement and sizing of a thrust block for various fittings.)

## 8-9 Pipe Rehabilitation: Trenchless Technology

Deteriorated pipes can affect the pipes' structural integrity and lead to roadway failures and development of sinkholes. Pipe deterioration could include longitudinal or circular cracks, joint separations, root intrusions, deformation, erosion, voids outside the pipes, and bedding erosion. Depending on the type of deterioration, failure to repair deteriorated pipes within certain time frames , which can lead to roadway failures, embankment failures, or sinkhole development.

The most common option for a deteriorated pipe is to remove the existing culvert and replace it with a new one.

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 methods are referred to as trenchless technology because minimal trenching is needed.

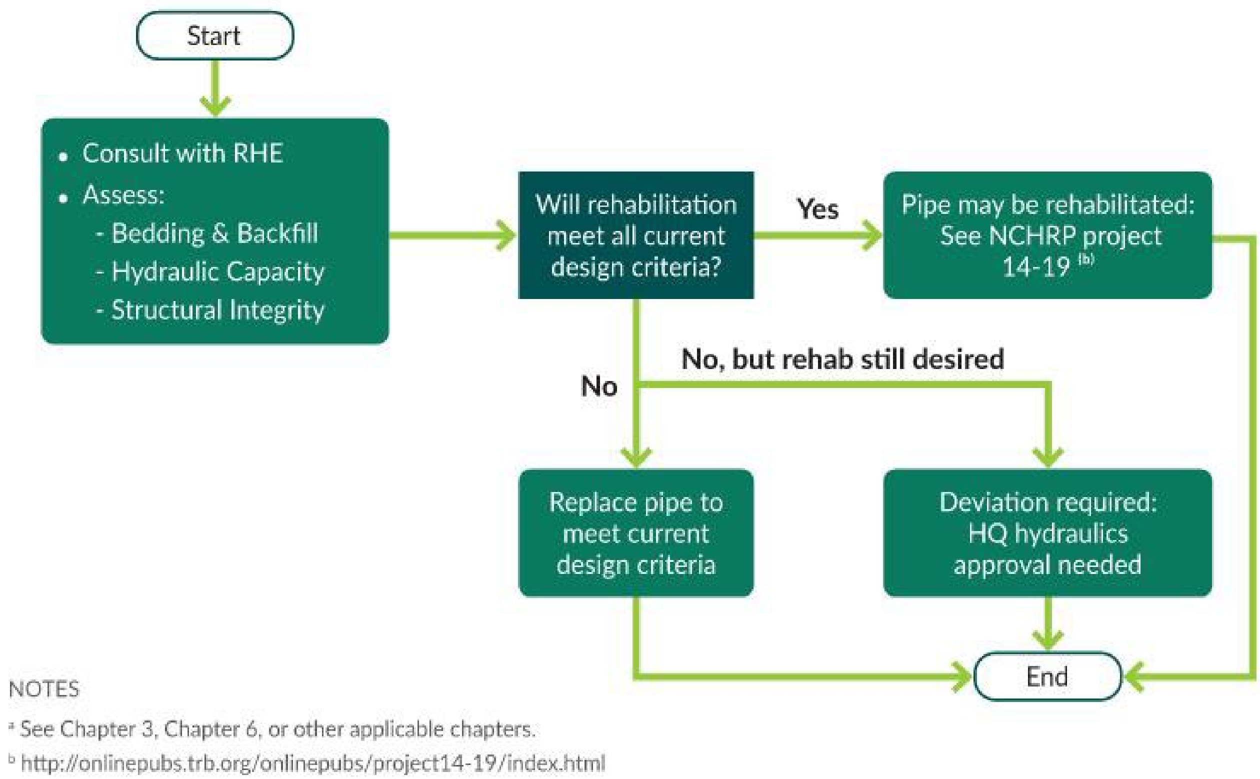
Prior to selecting a trenchless technology method, the PEO shall investigate the feasibility of a pipe being rehabilitated to provide a long-term fix. The investigation shall include, at a minimum:

- **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. For crossing culverts, if the pipe diameter is reduced, it must be analyzed as a culvert. Evaluate the inlet or outlet control and upstream and downstream impacts, and maintain the minimum pipe diameter requirement. HDPE and PVC liners are typically strong enough to withstand the loads, and they can last more than 50 years. However, these liners would reduce the inside diameter of the pipes, and this could be an issue for crossing culverts. Minimum pipe diameters must be maintained. The Manning's n values of HDPE and PVC liners are typically smaller than those of corrugated metal pipes and cement concrete pipes; therefore, flow capacity might not be an issue. However, flow capacity analysis is still required.
- **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 State Hydraulics Office for guidance on inspecting the pipe and developing a risk assessment. The vendors providing the trenchless technology shall also be consulted for determining the minimum structural requirements of the pipe. When evaluating the structural integrity of the pipes, the host pipes are excluded in the calculation. The liners must be able to withstand the dead loads and live loads. All pipes under rails must be sleeved. Cured-in-place pipe (CIPP) liners are typically very thin, and they may not be able to withstand the loads as required. If selected, certification from the manufacturer is required to testify that the liner is capable of withstanding the loads.

- **Evaluation of cost and age of the pipe:** The rehabilitative cost shall be determined as well as the replacement the replacement cost. Determine the age of the pipe as well as its original design life when installed.
- **Evaluation of design life:** All liners must have a lifespan of 50 years or longer. Certification from the manufacturer is required.
- **Evaluation of environmental impacts:** All liners must not have negative impacts on the environment. Consult with HQ ESO and Hydraulics for review and approval.

If this analysis indicates that rehabilitating the pipe using trenchless methods will meet all current design criteria, then the pipe may be rehabilitated. 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 State Hydraulics Office. See Figure 8-13.

**Figure 8-13** Replace or Rehabilitate Decision Tree




## 8-9.1 *Trenchless Techniques for Pipe Rehabilitation*

Several 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.

- **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 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: (1) drilling a pilot hole, (2) pilot hole enlargement, and (3) 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
- **CIPP lining** is a trenchless method of storm sewer pipe rehabilitation. It requires little or no digging and significantly less time to complete than other sewer repair methods. CIPP involves inserting a resin-impregnated glass-reinforced thermosetting plastic (GRP) liner or flexible liner inside the existing pipe, inflating the liner, and exposing it to heat or ultraviolet (UV) light to dry and harden the liner inside the pipe. The liner essentially forms a smooth surface inside the existing pipe, restoring it to near-new condition. The host pipe is assumed to be fully deteriorated in the structural integrity calculations to determine the required thickness of the liner itself may not be able to withstand the design live and dead loads. CIPP liners are relatively less expensive than other materials, and they are easier to install. However, certain installation protocols must be followed; otherwise, temporary impacts on the environment could occur. GRP lining using UV cure shall be the preferred method. See additional guidelines in General Special Provision 7-SA1.FR7 (currently under final subject-matter expert review) for specifics. Consult with HQ ESO or the State Hydraulics Office for more guidance.

GRP liner or flexible felt tube liner are placed inside an existing host pipe by one of the following methods:

- Inverting in place using compressed air
- Pulling in place with a winch

The lining does not come in standard sizes but is designed specifically for the individual pipeline to be rehabilitated, with variable diameters/shapes (i.e., round, elliptical, oval, etc.) and wall thickness. When necessary, a minimum thickness of the liner can be specified to provide additional service life for abrasive conditions. See *Hydraulics Manual* Section 8-6 and Table 8-1 for guidelines regarding abrasion. Grouting may be required if the host pipe has minor corrosion or minor cracks. There shall be no annular space between the host pipe and liner.

A GRP liner, or felt tube liner, saturated with a thermosetting resin is either pulled into the existing pipe or inverted through as air pressure pushes the tube tightly against the pipe wall. The UV light source is then inserted in the tube and heated to the curing temperature of 160 to 180 degrees Fahrenheit. The plastic resin on the tube cures to solid pipe inside the existing pipe, creating a new lining. Installation goes quickly, leaving no annular space to be sealed. Odd cross sections, bends, and minor deformations can be accommodated. This method is particularly useful when flow capacity must be maintained or slightly increased by lowering the Manning's n value.

Concrete culverts subject to sulfate attack are especially good candidates for this repair method or metal pipes where the reduction in diameter using other lining methods is not acceptable.

For the pulled-in-place installation method, a winched cable is placed inside the existing pipe. The resin-impregnated liner is connected to the free end of the cable and then pulled into place between drainage structures or culvert ends. The cable is disconnected, the two ends are plugged, and the liner is inflated (approximately 8 pounds per square inch [psi]) before curing by use of UV light. For resin control, the General Special Provisions (GSP 7-SA1.FR7) require chemically resistant UV-cured isophthalic polyester resin or vinyl ester resin. The contractor shall also send resin samples per GSP 7-SA1.FR7 to an independent third-party laboratory certified by the American Association for Laboratory Accreditation (A2LA) for quality assurance infrared fingerprinting. The tube shall include an impermeable inner and outer foil layer (liner) to contain potential resin immigration and contamination, and the inner foil layer should easily remove from the inside tube wall or remain if fabricated as a permanent part of the cured fabric tube (refer to GSP 7-SA1.FR7).

When curing using UV light a fiberglass and resin tube is used and no refrigeration is necessary; no heated water/steam is used. Cure times are quicker than the other methods; however, there is a thickness limitation of 1 inch because the maximum thickness for light curing is limited to 0.5 inch per run.

Site setup is a high proportion of costs on small projects. Prior to UV cure, the host pipe shall be cleaned to remove debris, sediment, and any other accumulated material. Removed sediment-laden washout and debris shall be disposed of per WSDOT *Standard Specifications*. After pipe cleaning, the host pipe shall also be inspected per GSP "Video Pipe Inspection" (currently under final subject-matter expert review) to check for unanticipated obstructions, reduction in cross-sectional areas, sags, and structural defects to determine all the point repairs prior to lining the pipe to be rehabilitated.

After completion of UV cure, core restrained samples shall be obtained to be sent to an independent third-party laboratory certified by A2LA (refer to GSP 7-SA1.FR7) for physical properties tests such as flexural strength and flexural modulus of elasticity. Post-installation inspection shall be conducted per GSP "Video Pipe Inspection" to check if there are any imperfections such as wrinkles, fins, tears, holes, blisters, and delamination. Failed installations shall not be accepted by WSDOT. For full details of failed installations and required remedies, refer to GSP 7-SA1.FR7.

In general, the following steps are sequentially performed:

1. Install pipe plugs upstream and downstream of the storm sewer pipes and install diversion (if needed)
2. Clean, inspect (pre-install), and prepare host pipe for cleaning (voids in backfill may need grouting; remove protrusions greater than 0.5 inch, record exact locations of lateral pipes)
3. Prepare liner: GRP or felt tube liner is vacuum impregnated with resin
4. Install liner
5. Cure (UV) liner
6. Take test samples
7. Conduct final (post-install) inspection
8. Repair as needed
9. Remove pipe plugs and diversion (if needed)

**Spray lining** could be an option if the host pipes are big enough. The materials could be cement or polymer, and the liners could be installed with or without the wire mesh or reinforced bars. Without the host pipe, the liners could provide very little strength to significant strength to withstand the loads. Similarly, the liner lifespan depends on the material and construction method.

## 8-10 Pipe Design

This section presents pipe design alternatives.

### 8-10.1 *Categories of Structural Materials: Rigid or Flexible*

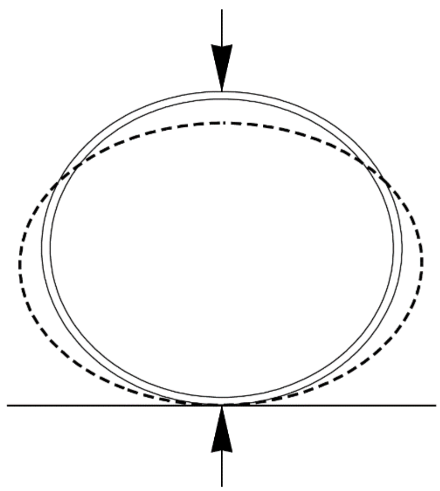
Based upon material type, pipes can be divided into two broad structural categories: flexible and rigid. Flexible pipes have little structural bending strength. The material they are made of, such as corrugated metal or thermoplastic, can be flexed or distorted significantly without cracking. Flexible 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 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 8-14. 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 8-14** Deflection of Flexible Pipes

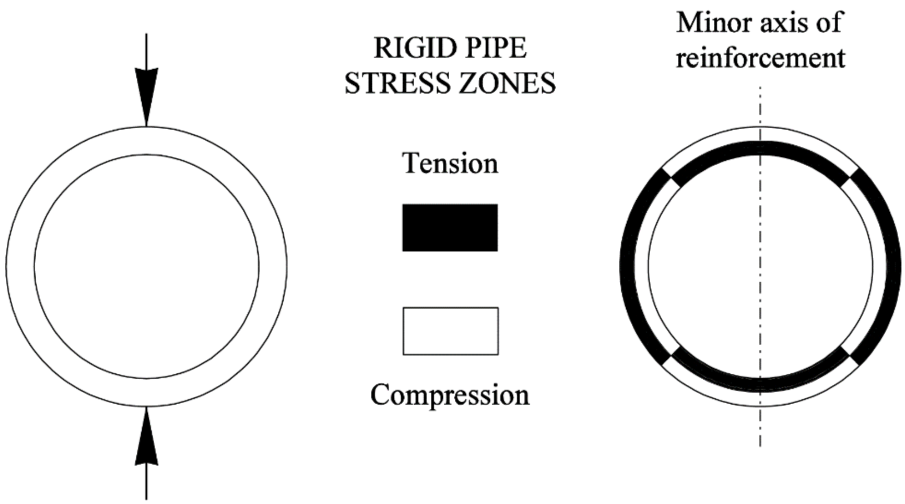


The flexibility of a pipe also allows for some bend in the horizontal when designing the pipe layout. The PEO shall limit the bend to a maximum of 1.5 degrees. This same allowable bend does not apply to pipe profiles, which shall be designed to be straight. When bends occur in the profile, “bellies” 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 8-15. 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 8-15** 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-55.20-03 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 State Hydraulics Office.

The bedding equal to one-third of the pipe outside diameter shall be loosely placed directly under the pipe, while the remainder shall be compacted to a minimum 90 percent of maximum density per AASHTO guidelines. 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, plugged per *Standard Specification 7-08.3(4),* or filled with controlled-density fill (CDF) per *Standard Specification 2-09.3(1)E.* If it is not practical to remove the pipe, the pipe can be abandoned in place and the pipe ends can be plugged as specified in the *Standard Specifications.* All pipes shall be evaluated prior to abandonment by the project PEO, RHE, or State Hydraulics Office 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 CDF that meets the requirements per the *Standard Specifications.* See the decision tree for pipe abandonment in Figure 8-16 and pipe abandonment determination schematic in Figure 8-17.

**Figure 8-16** Decision Tree for Pipe(s) to be Abandoned

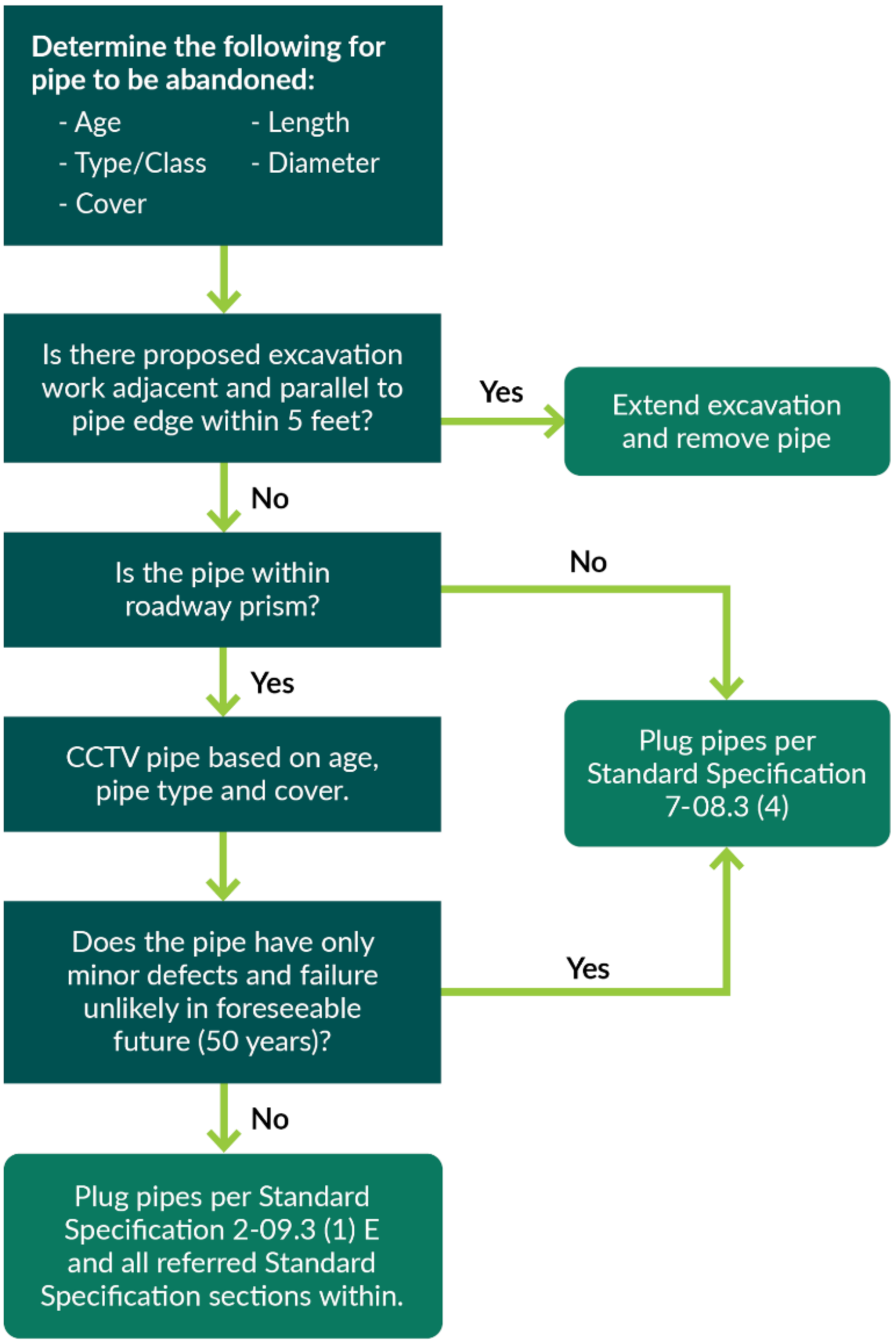
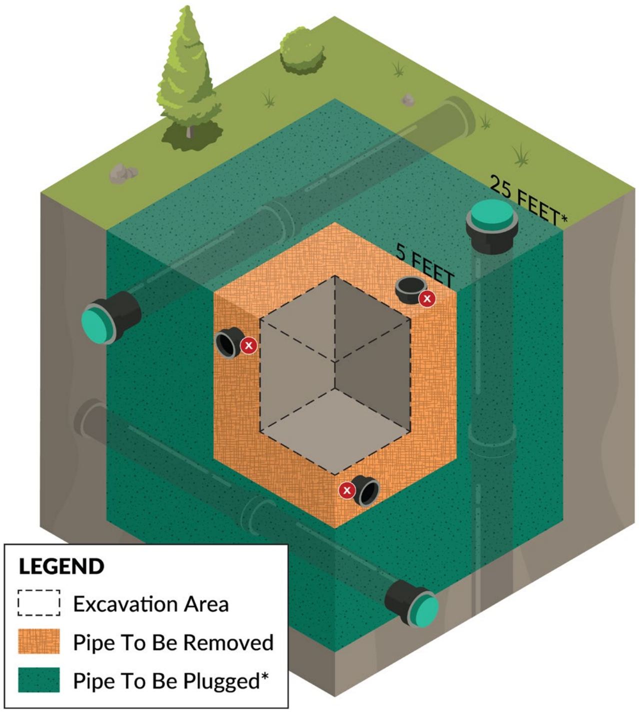


**Figure 8-17** Pipe abandonment determination schematic




Note: if the distance between the edge of the excavation area and the edge of the pipe is greater than 5 feet horizontally or vertically, plug and abandon pipe. Refer to Section 8-11 and pipe abandonment tree chart above.

## 8-12 Structural Analysis and Fill Height Tables

The State Hydraulics Office, 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 (Table 8-2 through Table 8-19). 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 specification 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 shall 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, coordination with the HQ Geotechnical, Bridge and Structures, and Hydraulics Offices is required for review and approval.

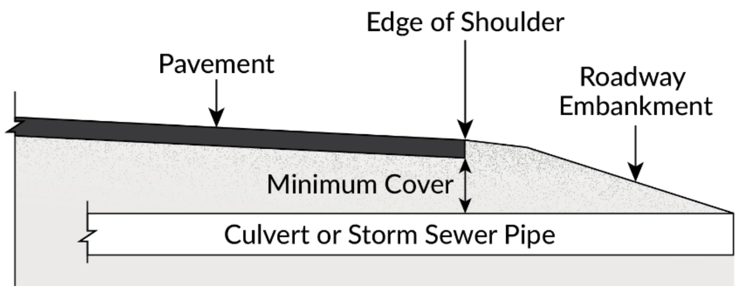
When a pipe is rehabilitated with a liner, the liner must be able to withstand the loads without the host pipe included in the calculations.

## 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 (see Figure 8-18). This measurement does not include any asphalt or concrete paving above the top course. 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 State Hydraulics Office through a deviation.

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.

**Figure 8-18** Pipe Minimum Cover




NOTES:
(1) Minimum thickness of cover is measured at edge of shoulder.
(2) Minimum cover is measured from outside diameter of pipe to bottom of pavement.
(3) All pipes not listed in Table 8-19 of Hydraulic Manual shall have minimum cover of 2.0 feet.
(4) Provide supporting calculations and references for the proposed pipes if minimum cover is less than 2.0 feet.
(5) Consult RHO or State Hydraulics Office if minimum cover is less than 2.0 feet.

### 8-12.1.1 Pipe Sleeve

The pipe shall be sleeved when it is located underneath railroad guideways. The sleeves must be able to withstand the dead and live loads. The sleeve must be extended 10 feet out from the edge of the guideway.

## 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 Table 8-19 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.

In addition to circular pipe, concrete box culverts and concrete arches are 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 shall consult with either the RHO/contact or the State Hydraulics Office for additional guidance on the use of these structures in this application.

### 8-12.3 *Fill Height Tables*

Table 8-2 through Table 8-19 are fill height tables.

**Table 8-2** 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 | 12 | 17 | 38 | 42 |
| 18 | 18 | 13 | 17 | 40 | 42 |
| 24 | 16 | 13 | 17 | 40 | 42 |
| 30 | -- | 13 | 17 | 40 | 42 |
| 36 | -- | 12 | 17 | 40 | 42 |
| 48 | -- | 12 | 17 | 40 | 42 |
| 60 | -- | 12 | 17 | 40 | 42 |
| 72 | -- | 12 | 17 | 39 | 42 |
| 84 | -- | 12 | 16 | 39 | 42 |

***Notes:***
-- = not applicable
Minimum cover is 2 feet.
In. = inch

**Table 8-3** Concrete Pipe for Shallow Cover Installations

| Pipe Diameter (in.) | Pipe Wall Thickness (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 | 1.0 | 1.0 | 0.5 |
| 48 | 5 | -- | 1.0 | 1.0 | 0.5 |
| 60 | 6 | -- | 1.0 | 1.0 | 0.5 |
| 72 | 7 | -- | 1.0 | 1.0 | 0.5 |
| 84 | 8 | -- | 1.0 | 1.0 | 0.5 |

***Notes:***
-- = not applicable
in. = inch

**Table 8-4** 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 |
| 36a | 65 | 81 | 100 | 100 | 100 |
| 42[a] | 56 | 70 | 98 | 100 | 100 |
| 48[a] | 49 | 61 | 86 | 100 | 100 |
| 54[a] | -- | 54 | 76 | 98 | 100 |
| 60[a] | -- | -- | 68 | 88 | 100 |
| 66[a] | -- | -- | -- | 80 | 98 |
| 72[a] | -- | -- | -- | 73 | 90 |
| 78[a] | -- | -- | -- | -- | 80 |
| 84[a] | -- | -- | -- | -- | 69 |

***Notes:***
-- = not applicable
ga = gage
in. = inch
Minimum cover is 2 feet.
a. The PEO shall consider the most efficient corrugation for the pipe diameter.

**Table 8-5** 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 |

| 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 |
| 108 | -- | -- | 44 | 56 | 69 |
| 114 | -- | -- | 41 | 53 | 65 |
| 120 | -- | -- | 39 | 50 | 62 |

***Notes:***
-- = not applicable
ga = gage
in. = inch
Minimum cover is 2 feet.

**Table 8-6** 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 = gage
in. = inch
Minimum cover is 2 feet.

**Table 8-7** 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 |

| 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 |
| 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 = gage
in. = inch
6 in. × 2 in. corrugations require field assembly for multiplate; diameter is too large to ship in full section.

**Table 8-8** Corrugated Steel Pipe Arch: 2⅔ in. × ½ in. Corrugations—AASHTO M 36

| Span × Rise (in. × in.) | Min Corner Radius (in.) | Thickness | | Minimum Cover (ft) | Maximum Cover in Feet for Soil-Bearing Capacity of: | |
|---|---|---|---|---|---|---|
| | | in. | Gage | | 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 = gage
in. = inch
NS = not suitable

**Table 8-9** Corrugated Steel Pipe Arch: 3 in. × 1 in. Corrugations—AASHTO M 36

| Span × Rise (in. × in.) | Corner Radius (in.) | Thickness | | Minimum Cover (ft) | Maximum Cover in Feet for Soil-Bearing Capacity of: | |
|---|---|---|---|---|---|---|
| | | in. | Gage | | 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 = gage

in. = inch

**Table 8-10** 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. | Gage | 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[a] |
| 14-2 × 9-10 | 31 | 0.111 | 12 ga | 2 | 12 | 2 | 16[a] |
| 15-4 × 10-4 | 31 | 0.140 | 10 ga | 2 | 11 | 2 | 15[a] |
| 16-3 × 10-10 | 31 | 0.140 | 10 ga | 2 | 11 | 2 | 14[a] |
| 17-2 × 11-4 | 31 | 0.140 | 10 ga | 2.5 | 10 | 2.5 | 13[a] |
| 18-1 × 11-10 | 31 | 0.168 | 8 ga | 2.5 | 10 | 2.5 | 12[a] |
| 19-3 × 12-4 | 31 | 0.168 | 8 ga | 2.5 | 9 | 2.5 | 13 |

***Notes:***

ft. = feet
ga = gage
in. = inch
TSF = tons per square foot

a. Fill limited by the seam strength of the bolts. Additional sizes are available. Contact the OSC Hydraulics Office for more information.

**Table 8-11** 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 = gage
Minimum cover is 2 feet.

**Table 8-12** 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 = gage
Minimum cover is 2 feet.

**Table 8-13** 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.

**Table 8-14** Aluminum Pipe Arch: 2⅔ in. × ½ in. Corrugations—AASHTO M 196

| Span × Rise (in. × in.) | Corner Radius (in.) | Thickness | | Minimum Cover (ft) | Maximum Cover in Feet for Soil-Bearing Capacity of: | |
|---|---|---|---|---|---|---|
| | | in. | Gage | | 2 tons/ft$^2$ | 3 tons/ft$^2$ |
| 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 = gage
in. = inch
NS = not suitable

**Table 8-15** Aluminum Pipe Arch: 3 in. × 1 in. Corrugations—AASHTO M 196

| Span × Rise (in. × in.) | Corner Radius (in.) | Thickness | | Minimum Cover (ft) | Maximum Cover in Feet for Soil-Bearing Capacity of: | |
|---|---|---|---|---|---|---|
| | | in. | Gage | | 2 tons/ft$^2$ | 3 tons/ft$^2$ |
| 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 = gage
in. = inch

**Table 8-16** 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. Gage Thickness (in.) | Min. Cover (ft) | Maximum Cover[a] in Feet for Soil-Bearing Capacity | |
|---|---|---|---|---|---|---|
| | | | | | 2 tons/ft$^2$ | 3 tons/ft$^2$ |
| a | 5-11 × 5-5 | 31.8 | 0.100 | 2 | 24[b] | 24[b] |
| b | 6-11 × 5-9 | 31.8 | 0.100 | 2 | 22[b] | 22[b] |
| c | 7-3 × 5-11 | 31.8 | 0.100 | 2 | 20[b] | 20[b] |
| d | 7-9 × 6-0 | 31.8 | 0.100 | 2 | 28[b] | 18[b] |
| e | 8-5 × 6-3 | 31.8 | 0.100 | 2 | 17[b] | 17[b] |
| f | 9-3 × 6-5 | 31.8 | 0.100 | 2 | 15[b] | 15[b] |
| g | 10-3 × 6-9 | 31.8 | 0.100 | 2 | 14[b] | 14[b] |
| h | 10-9 × 6-10 | 31.8 | 0.100 | 2 | 13[b] | 13[b] |
| i | 11-5 × 7-1 | 31.8 | 0.100 | 2 | 12[b] | 12[b] |
| j | 12-7 × 7-5 | 31.8 | 0.125 | 2 | 14 | 16[b] |
| k | 12-11 × 7-6 | 31.8 | 0.150 | 2 | 13 | 14[b] |
| l | 13-1 × 8-2 | 31.8 | 0.150 | 2 | 13 | 18[b] |
| m | 13-11 × 8-5 | 31.8 | 0.150 | 2 | 12 | 17[b] |
| n | 14-8 × 9-8 | 31.8 | 0.175 | 2 | 12 | 18 |
| o | 15-4 × 10-0 | 31.8 | 0.175 | 2 | 11 | 17 |

| Span × Rise (ft-in. × ft-in.) | | Corner Radius (in.) | Min. Gage Thickness (in.) | Min. Cover (ft) | Maximum Cover[a] in Feet for Soil-Bearing Capacity | |
|---|---|---|---|---|---|---|
| | | | | | 2 tons/ft$^2$ | 3 tons/ft$^2$ |
| 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

a. Additional sizes and varying cover heights are available, depending on gage thickness and reinforcement spacing. Contact the State Hydraulics Office for more information.

b. Fill limited by the seam strength of the bolts.

**Table 8-17** 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 = gage

in. = inch

Minimum cover is 2 feet.

**Table 8-18** 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 = gage
in. = inch
Minimum cover is 2 feet.

**Table 8-19** Thermoplastic and Ductile-Iron Pipe

| Solid-Wall PVC | Profile-Wall PVC | Corrugated Polyethylene |
|---|---|---|
| ASTM D 3034 SDR 35<br>3 in. to 15 in. diameter<br><br>ASTM F 679 Type 1<br>18 in. to 48 in. diameter | AASHTO M 304<br>or<br>ASTM F 794 Series 46<br>4 in. to 48 in. diameter | AASHTO M 294 Type S<br>12 in. to 60 in. diameter |
| 40 ft max, 2 ft min.<br>All diameters | 40 ft max, 2 ft min.<br>All diameters | 18 ft max, 2 ft min.<br>All diameters |
| **Solid-Wall HDPE** | **Polypropylene** | **Ductile-Iron Pipe** |
| Std Spec 9-05.23 | Std Spec 9-05.24<br>12 in. to 60 in. diameter | Std Spec 9-05.13<br>12 in. to 48 in. diameter |
| 18 ft max, 0.5 ft min.<br>All diameters | 21 ft max, 1 ft min.<br>All diameters | 25 ft max, 0.5 ft min.<br>All diameters |

***Notes:***
in. = inch
For cover, refer to Section 8-12.3.

# Chapter 9 Highway Rest Areas

Contact the State Hydraulics Office for design guidance

# Chapter 10 Woody Material

## 10-1 Introduction

WM plays a critical role in many Washington streams through its influence on stream geomorphic processes and aquatic habitat formation. This chapter determines when LWM is appropriate, and how to design WM features that meet habitat and stability objectives. The best approach for habitat enhancement and restoration is to mimic or replicate natural conditions to which salmon and other aquatic species have adapted. Site natural wood loading conditions provide a reference to guide quantities, sizes, and placement of WM as a component of habitat enhancement and restoration.

Installation of instream wood has become a common stream enhancement and restoration practice in Washington State. 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 WM, particularly large wood in many of these stream systems, have been altered through decades of timber harvesting, channel clearing, snag removal, and human alterations to stream channels and riparian zones, resulting in changes to stream channel form, function, and degradation of aquatic habitat. Placement of WM can achieve a variety of physical and biological benefits to stream morphology and aquatic habitat. WM can be used to directly provide habitat cover, complexity, and natural levels of streambank stability, or may provide indirect benefits through its influence on pool development, sediment trapping, hydraulic roughness, lateral channel dynamics, and maintenance of channel bedform.

This chapter provides policy on the use of WM in all water bodies—streams, rivers, lakes, and marine shorelines. WSDOT WM is divided into three categories: LWM, SWM, and slash. LWM can be designed to be stable or mobile. Mobile LWM is referred to as mobile woody material (MWM). See the Main Glossary of Terms for formal WSDOT definitions of the types of WM.

Section 10-1.1 gives an overview of the design process, while Section 10-2 describes reach assessments. Risk considerations are described in detail in Section 10-3, and detailed design is described in detail in Section 10-4. Design criteria, including using MWM, are discussed in Sections 10-4.1 and 10-4.2. Section 10-5 provides guidance on inspection and maintenance, and Section 10-6 provides the appendices.

Project designs that include WM require expertise in hydrology, hydraulics, and geomorphology and designs will need to be documented in a specialty report. Additional requirements about specialty reports are provided in Chapter 1. An FPSRD certificate number is required for all authors of any portion of a specialty report, if the project is related to fish passage barrier removal or scour. See Table 1-1 for a list of specialty reports and other requirements. An FPSRD certificate number is given to those who have viewed all of the training modules and successfully passed the comprehensive exam. Additional information, training resources, and the point of contact for this training can be found on the WSDOT Hydraulics Training web page. As WSDOT updates the FPSRD training modules a re-certification number is also required. Any updates to this training will be posted on the WSDOT Hydraulics Training web page.

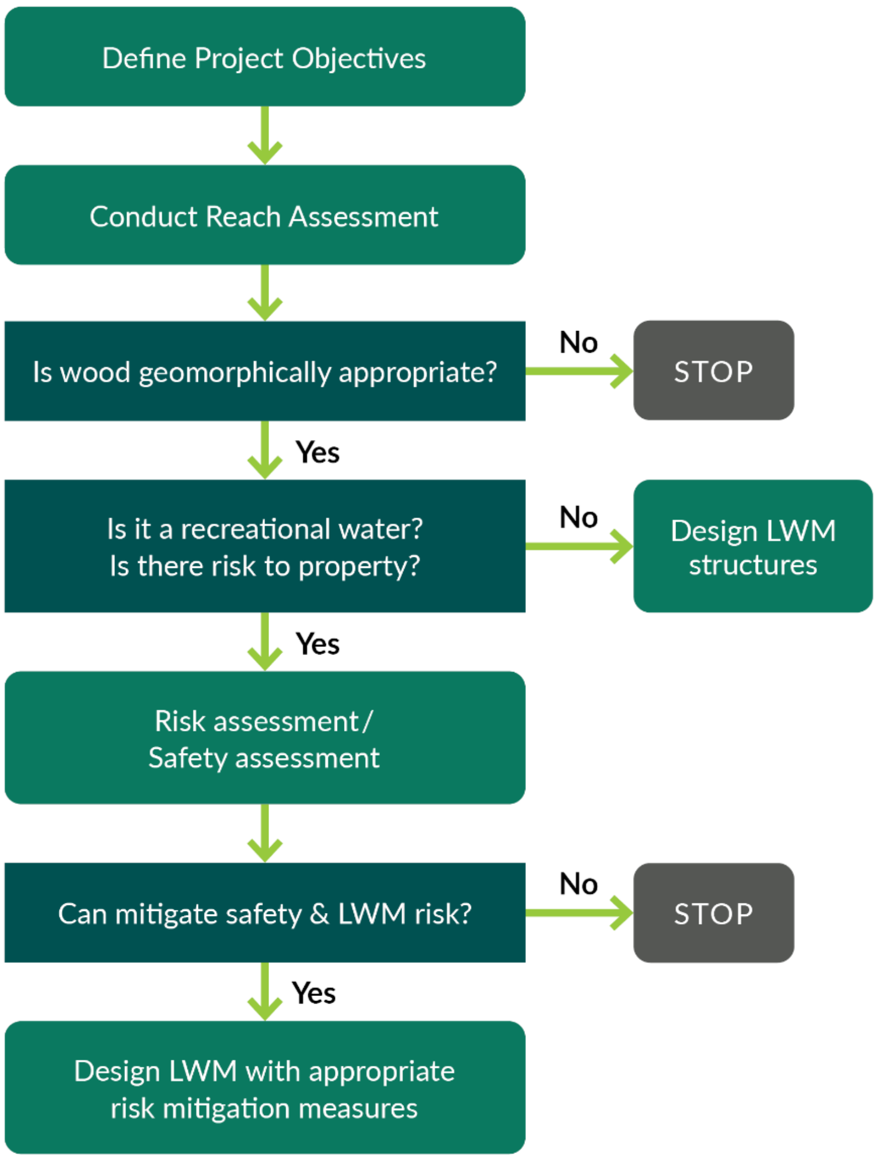
WSDOT is actively monitoring completed projects that include WM and will update this chapter as new information becomes available. Contact the State Hydraulics Office for additional or updated guidance.

### 10-1.1 *Design Process*

Design and placement of WM shall follow a geomorphic and ecological assessment of the watershed and a similar, more detailed assessment of the river reach and site to be treated, including an analysis of existing conditions and anticipated hydraulic and geomorphic responses. The following multi-step design process is shown in Figure 10-1:

1. The project objectives are identified.
2. A reach assessment describes the geomorphic and habitat conditions. It also informs habitat and bank stability objectives of the reach, the constraints, and the existing wood in the system and to determine if the use of wood is suitable for the site conditions (Section 10-2).
3. A risk assessment is completed to identify potential risks to infrastructure and the public, and to provide guidance to reduce potential risks (Section 10-3).
4. The design is created using general and project-specific design criteria.

**Figure 10-1** Wood Design Process




### 10-1.2 *Guidance for Emergency Large Woody Material Placement*

Generally, failure of a water crossing or streambank requires rapid response to stabilize and prevent additional damage to WSDOT infrastructure and to restore a safe travel corridor. In these cases, regional maintenance staff likely need to act quickly and engineering judgment calls are needed during such situations. Incorporation of LWM could be considered a mitigation element for aquatic habitat impacts as a result of the emergency action. LWM shall be placed during emergency repairs only in consultation with the State Hydraulics Office. The maintenance or project office in charge of emergency repairs must also consult with WDFW and the appropriate tribal contacts for the area.

## 10-2 Reach Assessment

The reach assessment discussed in Chapter 7 is essential for developing and justifying the wood layout design. The reach assessment serves as the basis for applying large wood to aid in restoring, partially restoring, or enhancing geomorphic and biological processes at the project site. The reach assessment should provide the following context for developing the wood layout design:

- Reconstruct the historical processes that delivered large wood to the site and/or reach prior to floodplain settlement in North America during the 19th and 20th centuries (e.g., local recruitment via bank erosion, windfall, exhumation; wood supply delivered from upstream via debris flows, mass wasting)
- Reconstruct the geomorphic and biological impacts of removing large wood from the channel (e.g., impacts of log jam removal on channel incision, channel simplification, loss of pools), the floodplain (e.g., depletion of wood supply via loss of riparian forest), and possibly the watershed (e.g., clearcut logging)
- Document current conditions for large wood density, recruitment processes, wood sourcing, and geomorphic and biological functions within the project reach (if applicable, answer the question: "Why is wood absent?")
- Assess risk of wood transport downstream to adjacent property owners and/or infrastructure

Effective design of the wood layout hinges on defining specific geomorphic functions to address:

- Is geomorphic grade control necessary to mitigate channel incision and knickpoint migration (e.g., channel-spanning buried large wood, channel-spanning log steps?)
- Is flow deflection and bank protection needed for protecting WSDOT infrastructure?
- Are engineered log jams (ELJs) recommended for pool formation, in-channel deposition, and gravel retention?
- Is surface and/or subsurface large wood needed to redistribute flow hydraulics (partition shear stress) and offer secondary stability to other design elements?

## 10-3 Risk Assessment

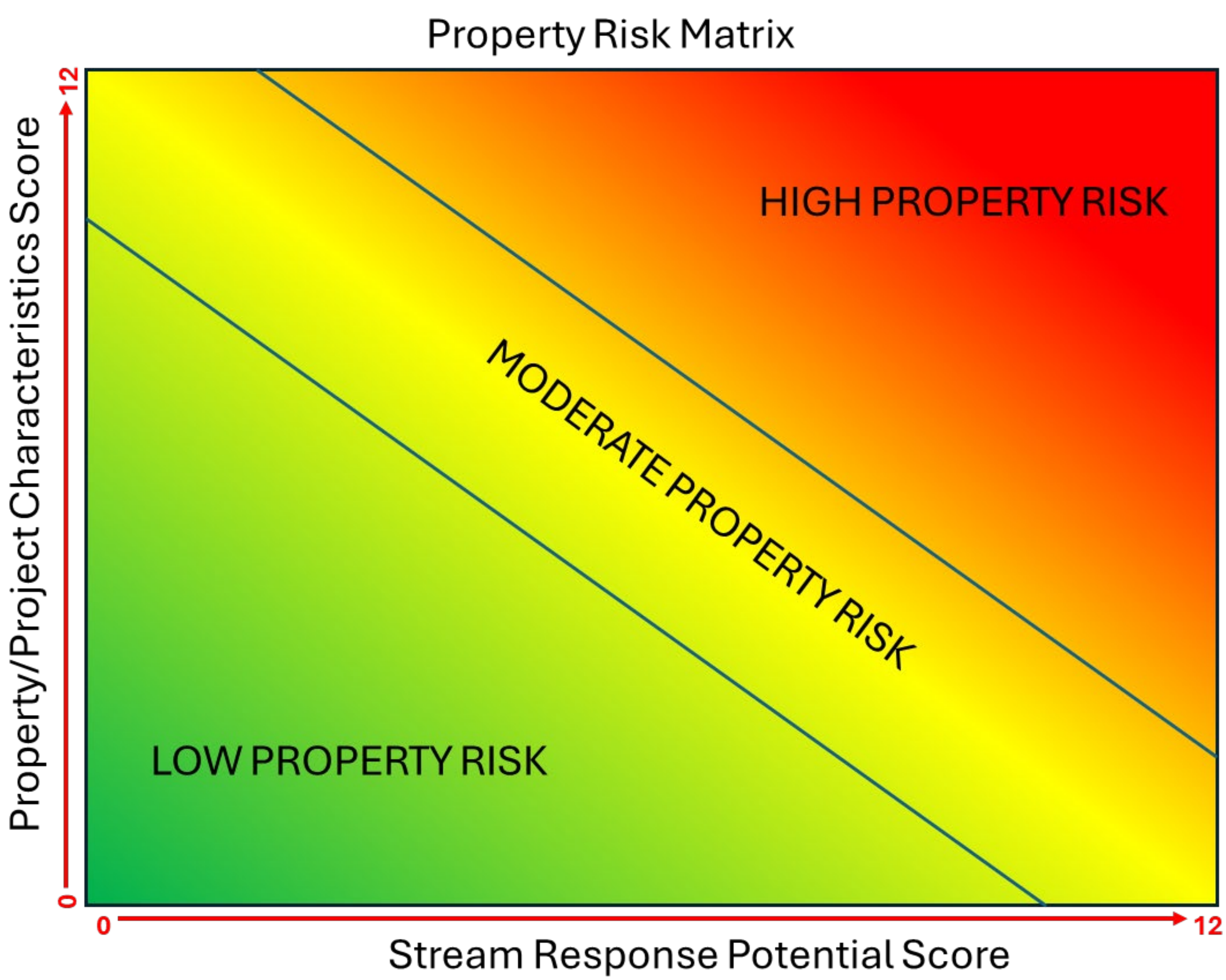
This section presents the risk assessment, including LWM and MWM, recreational water safety, and FEMA and local floodplain permit requirements.

### 10-3.1 *LWM and MWM Risk Assessment*

Risk shall be considered for all projects that propose WM and shall be incorporated into the PHD and FHD. There are two levels of risk evaluation—the first level is to assess whether adding large wood, in general, is appropriate for the project reach. This occurs during the site and reach assessment (Section 10-2). The second level is a more formal risk assessment, which shall address risks associated with infrastructure, MWM, long-term morphological changes, etc.

Some existing documentation providing guidance for evaluating risk includes the NOAA-produced guidance on conducting risk assessments for LWM placement (NOAA 2011). This document presents a risk matrix that is helpful in categorizing risk to infrastructure, even when risk cannot be quantified. This matrix is presented in Figure 10-2. NOAA 2011 discusses how to fill out the inputs on the X axis (stream response potential) and the inputs on the Y axis (property/project characteristics). In summary, the various factors affecting modification and movement of wood over time, along with the type and proximity of infrastructure downstream, are scored on the Y axis. The factors of stream response are scored on the X axis. The total score for each axis is plotted against each other, and the coordinates' location indicate the relative risk to infrastructure. The matrix has been modified somewhat from the original.

**Figure 10-2** Large Wood Property Damage Risk Matrix (modified from NOAA 2011)



Stream Response Scoring (X-axis):

| | | | | | Score (0-2) |
|---|---|---|---|---|---|
| **Scale of problem** | | | | | |
| Site | Reach | | Multiple Reaches | Watershed | ______ |
| **Landscape Sensitivity/Stream Type** | | | | | |
| Bedrock | Colluvial | Alluvial | Incised Channel | Alluvial Fan | ______ |
| **Riparian Corridor** | | | | | |
| Continuous/wide | | Discontinuous | | Urbanized/levees | ______ |
| **Bank Characteristics** | | | | | |
| Bedrock/till | | Erosion resistant | | Highly erodible | ______ |
| **Bed Mobility** | | | | | |
| Low (Coarse/clay) | | Medium (gravel) | | Fine (sand/silt) | ______ |
| **Dominant Hydrologic Regime** | | | | | |
| Spring-fed | Snowmelt | Rain | Rain-on-snow | Thunderstorms | ______ |
| | | | | **TOTAL SCORE** | ______ |

Project/Property Characteristics Scoring:

| | | | | | Score (0-3) |
|---|---|---|---|---|---|
| **Project Scale** | | | | | |
| Site Scale | | Reach Scale | | Multi-reach Scale | ______ |
| **Wood Length (multiple of channel width)** | | | | | |
| >2.5x with rootwad | 2x | 1.5x | | <1.0x, no rootwad | ______ |
| **Wood Properties** | | | | | |
| High Density, Slow Decay | | | | Low Density, Fast Decay | ______ |
| **Infrastructure/distance downstream from crossing** | | | | | |
| None/>1000' | Parallel Roads | Crossings <500' | Piers | Crossings <250' | ______ |
| | | | | **TOTAL SCORE** | ______ |

Additionally, NRCS's *National Engineering Handbook* (Technical Supplement 14J: Use of LWM for habitat and bank protection) provides discussion on the limitations of using LWM (NRCS 2010). The *National Large Wood Manual*, produced by USBR and the U.S. Army Engineer Research and Development Center (ERDC) (2016), provides additional discussion on projects involving WM.

MWM is used for habitat restoration and enhancement, recognizing that wood moves through aquatic systems across a variety of flow levels. However, MWM can pose risks to downstream infrastructure and properties. 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 and infrastructure are all factors. MWM shall not be placed when it could result in flood risk to infrastructure or properties, or damage to downstream crossings.

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 because of 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 (Diehl and Bryan 1993).

Studies of MWM blockages at culverts in small streams indicate that the plugging of culverts by MWM is initiated by one or more "initiator pieces" lodging across the culvert inlet during high flows (Furniss et al. 1998; Flanagan 2005). 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 rootwads attached that had lodged themselves on the barrel edges of the culverts. 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.

Based on the above research, MWM shall not be used when there is a potential to impact downstream infrastructure. SWM and slash by its nature does not pose a risk to infrastructure because of its mobility, size, and rate of decay relative to large wood pieces. However, the infrastructure present downstream of the project shall be considered, particularly if it is in close proximity to the crossing or reach in question. The quantity and placement of SWM used in the design may be constrained if there is risk to infrastructure. An example would be a tide gate flap or undersized culvert located within 100 feet of a project.

### 10-3.2 *Recreational Water Safety Risk Assessment*

WM may present risks to recreational users and these risks shall be considered in the planning and design phases of project development. The Recreational Water Safety Risk Assessment (RWSRA) shall identify the likely recreational activities that could occur at the site or in the project reach, and risks or hazards that WM may pose to recreational users. The assessment shall also determine if risk posed by WM can be reduced to an acceptable level. This type of assessment is often required by the Washington State Department of Natural Resources (DNR) for aquatic land use permits, if required, and shall 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 and 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.
- All rivers and streams designated as State-Owned Aquatic Land by DNR.

An RWSRA is required if any the stream or river in question meets any of the above criteria.

When an RWSRA is required, the following must be considered to mitigate the recreational risk:

- WM placement in confined channels shall be limited to grade control on the streambed and not structures obstructing flow.
- WM structures shall not be placed where there is poor visibility from upstream. A minimum visibility of 50 feet or three BFWs, whichever is greater, must be maintained.
- WM structures shall not be put in channels that do not allow for circumnavigation.
- Larger LWM structures shall not be constructed in close proximity upstream or downstream (within 100 feet or three BFWs, whichever is greater) of boat ramps.
- 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.
- Signage shall be addressed on a case-by-case basis, particularly where upstream visibility is limited because of meandering channels, etc.
- Multi-log LWM structures shall 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 is, the less this tends to occur. LWM structures shall be designed to limit flow-through characteristics by including an impermeable core to prevent "straining."

At sites with heavy recreational use, public notification and involvement may be desired to minimize the risks of LWM structures. Public notification shall 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 State Environmental Policy Act shall 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*, Chapter 400. Additional guidance for public involvement can be found in WSDOT's *Design Manual*.

Basic engineering standards require consideration of safety and risk and, ultimately, design decisions regarding the use of WM in recreational waters must be left to the State Hydraulics Office. The methods and assumptions used for the recreational water safety assessment analysis will be fully documented in the project's Hydraulic Design Report.

### 10-3.3 *FEMA and Local Floodplain Permit Requirements*

Introduction of WM into a stream will change the WSELs in the immediate vicinity. While this is often desirable for habitat and hydraulic objectives, it may have an undesirable effect on adjacent property or infrastructure. During project designs, every project that includes WM shall evaluate the effects the WM has on the WSELs. If the stream has a FEMA-designated SFHA, the local flood manager may also require that the project meet specific floodplain requirements. The designer shall determine the FEMA designations for the stream and floodplain and ensure compliance with local and federal floodplain regulations.

## 10-4 Design

The design of WM structures requires a comprehensive understanding of hydraulics, geomorphic, and ecological factors to achieve project objectives. A successful design ensures that WM placements are stable as intended, functional, and align with project goals. Key considerations include selecting appropriate materials; evaluating forces acting on the structure; and incorporating safety measures to mitigate risks to infrastructure, the environment, and public safety. The stream design engineer shall ensure that banks opposite any WM are appropriately stabilized against erosion. For WM intended to be used as grade control, the stream design engineer shall coordinate with the State Hydraulics Office for approval. This section outlines the design principles, criteria, and methodologies for designing WM structures.

### 10-4.1 *Bank Protection Design Criteria*

WM influences river systems by increasing flow resistance, reducing velocity, and decreasing sediment transport. Designers can recreate this natural function to protect streambanks by using wood-dominated features like ELJs or log crib walls. These features function by increasing hydraulic roughness along the streambank and thereby protecting the underlying material from erosion. When designed and constructed appropriately, they are effective at addressing lateral instability but are not suitable as a scour countermeasure for critical infrastructure like bridges or walls. WM shall be placed outside of any scour countermeasure footprint. WM shall be placed such that it does not conflict with the scour policies presented in the *Bridge Design Manual*, nor with Chapter 4 or Chapter 7 of this *Hydraulics Manual*.

Extensive guidance exists for numerous techniques for bank protection, from rock to revegetation. See Section 4-6 for guidance on using rock for bank protection. Some of the most pertinent guidance documents are listed below:

- HEC-23, Volume 1 and Volume 2
- ISPG (WDFW 2002)

- *Bank Stabilization Design Guidelines* (Baird et al. 2015)
- WDFW's *Stream Habitat Restoration Guidelines* (Cramer 2012)

#### 10-4.1.1 Wood Selection

Where WM is to be incorporated into bank protection design, the decay and degradation of the wood over time shall be considered. Coniferous species of wood are acceptable for bank stability design, aside from western hemlock. The density of the wood species used must be accounted for in the stability calculations. Per the WSDOT GSP for "Woody Material," western red cedar is disallowed. However, if the density is accounted for in the stability calculations, then it may be used. Deciduous trees, which are prone to decaying sooner, shall not be used for bank stability. Refer to Section 10-4.3 for additional information regarding WM stability analyses. See Table 10-1 below for the relevant properties of different species to use in stability analyses.

**Table 10-1** 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[a] | Modulus of Rupture $N/m^2$ | Modulus of Elasticity $N/m^2$ | Specific Gravity[a] | 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 |
| Bigleaf 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.

a. Specific gravity computed from oven-dry weight (0% moisture) and volume at 12% moisture content.

#### 10-4.1.2 Design Flows

LWM bank protection features are intended to function over a long project design life (50 years or longer), and therefore the design flood event shall be the 1 percent annual exceedance probability (AEP) (100-year) used for the stability analysis. For complex wood structures, such as ELJs, flow deflectors, or wood incorporated into a combined rock and wood bank protection, the design flood shall be the 2080 100-year projected flood. Anchoring techniques, which are described in Section 10-4.3.1.4, may be necessary to ensure that the WM does not mobilize during the design flood event. Refer to Section 10-4.3.2 for additional information regarding required Factors of Safety for design as part of the stability analysis.

#### 10-4.1.3 Placement Criteria

As noted previously, wood-dominated features can be effective at addressing lateral instability but are not suitable as a scour countermeasure for critical infrastructure like bridges or walls. WM shall be placed outside of any scour countermeasure footprint and such that it does not conflict with the scour policies presented in the *Bridge Design Manual*, nor with Chapter 4 or Chapter 7 of this *Hydraulics Manual*. The risks described previously in Section 10-3 shall also be considered when evaluating whether bank protection design incorporating WM is appropriate.

During design, the appropriate extents for the bank protection in plan view, as well as the top and bottom elevations necessary for design features to provide full bank protection, shall be evaluated. This evaluation shall be conducted by an interdisciplinary team and include hydraulic modeling, scour analysis, and floodplain analysis. A risk assessment shall also be conducted on the design features to evaluate longevity (for example, pile failure, erodible bank materials, and/or long-term WM integrity). The bottom elevation of the bank protection shall be designed to accommodate scour at the design flood. The top elevation of the bank protection shall extend a minimum of 1 foot above the scour design flood.

Several examples of bank protection designs including WM are included in the appendix.

### 10-4.2 *Habitat Enhancement Design Criteria*

WSDOT performs stream habitat restoration or enhancement to reconstruct stream corridors through new water crossings. Habitat restoration or enhancement 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 wood into these projects as sustainable habitat features. These features increase channel complexity and diversity of habitat necessary to support a healthy aquatic ecosystem. They must be designed based on the expertise and input from all members of a project's Stream Team (defined in Chapter 7-1), including a stream design engineer, geomorphologist, and biologist.

Conceptually, stream restoration refers to restoring or partially restoring geomorphic processes that were present at the site prior to Euro-American settlement. For example, WSDOT has several stream crossings that traverse alluvial fans. The streams are often confined between berms and levees upstream of the crossing. The disruption of alluvial fan processes frequently results in excessive, chronic sedimentation at the highway crossing. Repetitive dredging is usually required, often under emergency conditions. Berm or levee removal, partial or complete restoration of alluvial fan floodplain processes, and/or road relocation are examples of stream restoration by reestablishing alluvial fan processes to decrease sedimentation at the crossing.

The concept of stream enhancement refers to improving or enhancing geomorphic processes and biological conditions at a site that may not result in full restoration of a site. For example, a stream may have been relocated from its lowland, floodplain environment (pool riffle morphology) to flow over a steep glacial escarpment. If the highway was constructed through the floodplain (burying the original channel course), channel design of the affected reach will need to reflect the appropriate target morphology of the steeper gradient (e.g., step pool or cascade morphology). Because restoration or partial restoration of a pool riffle system is not possible, the channel design will need to enhance geomorphic and biological conditions appropriate to its current governing conditions (e.g., slope, confinement, and so forth).

All channel designs should go beyond consideration of flow conveyance to include continuity of sediment and wood transport processes. In moderately confined and unconfined alluvial systems, stream enhancement or restoration will incorporate floodplain and channel migration processes. For example, sediment yield and sediment transport are critical to consider for sizing a crossing span width and vertical clearance in a response reach affected by debris flows draining an upper watershed composed of weak bedrock.

Many streams have been severely impacted by land clearing, channelization, stream relocation, wood removal, and urban development. Channel incision is a common consideration in urbanizing systems. The impacts of changes to watershed hydrology, sediment transport regime, loss of streambank vegetation, and channel alterations are critical to understand for defining the objectives of a wood layout design. Stream enhancement or restoration upstream of crossings can help to reduce risks by capturing mobile wood that might otherwise cause blockages. Stream enhancement or restoration can also be instrumental in preventing channel incision and knickpoint propagation through a new crossing.

Stream enhancement and restoration activities include the following:

- Construct channels with the appropriate planform, grade, width, depth, and channel substrate, as discussed in Chapter 4 and Chapter 7
- Construct overbank and floodplain areas, where appropriate
- Stabilize the channel banks and disturbed floodplain and upland areas with revegetation and bioengineering

Wood provides habitat and geomorphic functions within a stream, including the following:

- Create stable obstructions that capture organic debris and form log jams
- Form pools
- Contribute to eddy creation and flow complexity
- Cause the deposition of finer sediments to create substrate diversity

- Enhance hyporheic flow by locally increasing hydraulic head
- Provide cover for aquatic organisms
- Provide woody substrate for invertebrates and other aquatic species
- Accumulate mobile wood and other organic debris
- Activate side channels with flood flows

Note that all vegetation to be cleared on a site shall be evaluated for use for habitat purposes and so used if determined to be acceptable quality.

#### 10-4.2.1 Wood Selection

The type of WM used for habitat enhancement is based on the size or mobility of the wood as defined below, as well as in the *Hydraulics Manual* Main Glossary of Terms and "Woody Material" GSP. Acceptable species for these types of WM are included below.

- **Large woody material (LWM):** LWM and MWM consist of trees and parts of trees including any variation of logs, rootwads, or stumps greater than 4 inches in diameter and larger than 6 feet in length. These shall be of a native coniferous tree species. Western red cedar cannot be used unless the density is accounted for in the stability calculations (see Table 10-1). Deciduous trees obtained from clearing or grubbing on site may be used for stable LWM or MWM if approved by the State Hydraulics Office.
- **Small woody material (SWM):** A random assortment of branches, trees, brush, and treetops of the following native species: western red cedar (*Thuja plicata*), Douglas fir (*Pseudotsuga menziesii*), western hemlock (*Tsuga heterophylla*) coniferous trees, or various hardwood trees. The maximum diameter of any piece of SWM shall be 4 inches. The maximum length of any piece of SWM shall be 6 feet. SWM shall not contain any material that causes turbidity.
- **Slash:** A random assortment of branches, trees, brush, and treetops of the following native species: western red cedar (*Thuja plicata*), Douglas fir (*Pseudotsuga menziesii*), western hemlock (*Tsuga heterophylla*), Sitka spruce (*Picea sitchensis*) coniferous trees, or various hardwood trees. The maximum diameter of any piece of slash shall be 2 inches. The maximum length of any piece of slash shall be 6 feet. Slash shall not contain any material that causes turbidity.

#### 10-4.2.2 Design Flows

LWM used for habitat enhancement or restoration shall be designed and placed with specific project objectives in mind. The appropriate design flood event must be determined based on habitat objectives, hydraulic opening width, and on-site constraints (see Section 10-4.2.3 for additional information related to placement considerations). Maintenance clearance requirements and the potential for scour countermeasures must also be considered. Stable LWM shall be designed based on the 1 percent AEP (100-year) flood event. For complex wood structures, the design flood shall be the 2080 100-year projected flood; contact the State Hydraulics Office for additional information. MWM shall be designed based on a target flood event and is in alignment with the results of a risk assessment and use of MWM shall be approved by the State Hydraulics Office prior to incorporating into the design. Refer to Section 10-4.3.2 for additional information regarding required FOSs for design as part of the stability analysis.

#### 10-4.2.3 Placement Criteria

Before laying out an aquatic habitat enhancement design, it is important to have some understanding of the species that use the stream and what habitat features the design will provide based on the reach assessment completed (see Section 10-2). The Stream Team needs to identify what kind of fish and habitat is needed and whether the channel has been impacted by the loss of functional wood. The reach assessment (see Section 10-2) shall assist with evaluating this. For example, many channels experience incision or downcutting after wood is removed, which can impact water crossings. To provide the best certainty for fish habitat, natural configurations and spatial organizations known to foster adaptations by salmonids shall be mimicked. For example, see Fox (2003) and Abbe and Montgomery (1996).

Knowing the species life history and habitat needs, as well as an understanding of the stream system, helps to identify an appropriate wood configuration. For example, wood 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, wood placements low in the channel to enhance low-flow habitat values may not provide high-flow refuge. The purpose of the overall design, including the intended function of proposed wood structures, shall be documented by the Stream Team in a hydraulic design report.

Habitat-limiting factors shall be considered for some types of projects, such as ones addressing certain chronic environmental deficiencies or restoration-based projects. Common limiting factors in Washington's waterways include water quality (temperature, sediment), stream flow, instream 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 WRIA can be found at Ecology's website.

Wood placement includes orientation, dip angle, and spacing. The configuration of wood will depend on the project objectives and specifically the intended objective for each log. Configuration of LWM for bank protection is different from that for aquatic or floodplain habitat enhancement. WSDOT expects a diversity of wood sizes, orientations, and elevations that are appropriate for the channel size. Wood can be placed in single logs or multiple-log groupings, depending on the intended purpose and both short- and long-term function. Complex placements with multiple logs with interlocking pieces of wood provide better habitat and mimic wood accumulation (log jams) over time. Channel-spanning WM may be included but requires approval by the State Hydraulics Office.

WM can pose a risk for critical infrastructure as noted in Section 10-3. Wood shall be located so that it does not create scour that could compromise bridge members (e.g., piers, abutments), road embankments, walls, or scour countermeasures. State Hydraulics Office approval is required for any projects with stable LWM proposed within a water crossing. If stable LWM is proposed within the channel under a permanent water crossing, appropriate scour countermeasures are required and must be designed to protect the structure's foundations in accordance with the *Bridge Design Manual* and Chapter 7 of this *Hydraulics Manual*. The inclusion of MWM in a design requires approval from the State Hydraulics Office. SWM and slash is generally acceptable without State Hydraulics Office approval.

Maintenance and freeboard requirements shall be taken into account by the Stream Team when proposing WM near or through a permanent water crossing. Refer to Table 7-3 and Table 7-4 in Sections 7-3.6.1 and 7-3.6.2, respectively, for additional information on these requirements. Localized aggradation occurs upstream of WM and shall be considered when determining minimum required freeboard.

As described in Section 10-2, WM can play a significant role in affecting reach-scale processes within a stream, including the channel's overall gradient. Depending on the arrangement and stability of wood pieces or jams, they may function as grade control for the system. The Stream Team must contact the State Hydraulics Office if using WM as a permanent grade control feature is being considered for a project. Less stable forms of grade control also occur naturally, consisting of matrices of smaller pieces of wood, sediment, and other debris. Section 7-3.9.4 includes guidance for designing deformable grade control features.

Constructing WM structures as designed can be challenging based on site-specific conditions. The State Hydraulics Office must be contacted if a Stream Team's designed layout is modified during construction. The modifications shall not substantially alter the intent of the design or redirect the expected flow path for the waterway in a manner that could put the structure or scour countermeasures at greater risk.

Several examples of habitat enhancement designs are included in the appendix for reference.

#### 10-4.2.4 LWM Targets

For WSDOT projects LWM targets apply as a starting point in stream restoration design. These targets are adopted from the recommendations in Fox and Bolton (2007). The targets need to be adjusted based on site-specific constraints and considerations and shall not create risks to infrastructure or fish passage. Target values need to be adjusted based on what is geomorphically appropriate for the project site. This could be an increase or decrease from the Fox and Bolton starting point. The hydraulic design report shall include documentation for the proposed targets used for the stream restoration design and discussed with co-managers.

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 four key metrics found by Fox and Bolton (2007) is the LWM target. This was identified by the authors of that study to compensate for cumulative deficits of wood loading due to development. The four metrics are:

- Key piece volume
- Key piece density
- Total number of LWM pieces (key and non-key)
- Total volume of LWM (key and non-key)

Table 10-2 shows the LWM targets for each of the four metrics, by BFW, and forest zone of the categories of streams. A "log metrics calculator," a spreadsheet tool supplied by the State Hydraulics Office, is available and shall be used to tabulate proposed LWM compared to these targets.

**Table 10-2** Large Wood Target Metrics

<table>
<tr><th colspan="2">KEY PIECE VOLUME</th><th colspan="3">KEY PIECE DENSITY</th><th colspan="3">TOTAL LWM VOLUME</th><th colspan="3">TOTAL PIECES OF LWM</th></tr>
<tr><th>BFW class (ft)</th><th>volume (yd3)</th><th>Forest zone</th><th>BFW class (feet)</th><th>75th percentile (per/ft stream)</th><th>Forest zone</th><th>BFW class (feet)</th><th>75th percentile (yd3/ft stream)</th><th>Forest zone</th><th>BFW class (feet)</th><th>75th percentile (yd3/ft stream)</th></tr>
<tr><td>0-16</td><td>1.31</td><td rowspan="3">Western WA</td><td>0-33</td><td>0.0335</td><td rowspan="3">Western WA</td><td>0-98</td><td>0.3948</td><td rowspan="3">Western WA</td><td>0-20</td><td>0.1159</td></tr>
<tr><td rowspan="2">17-33</td><td rowspan="2">3.28</td><td rowspan="2">34-328</td><td rowspan="2">0.0122</td><td rowspan="2">99-328</td><td rowspan="2">1.2641</td><td>21-98</td><td>0.1921</td></tr>
<tr><td>99-328</td><td>0.6341</td></tr>
<tr><td>34-49</td><td>7.86</td><td rowspan="3">Alpine</td><td>0-49</td><td>0.0122</td><td rowspan="3">Alpine</td><td>0-10</td><td>0.0399</td><td rowspan="3">Alpine</td><td>0-10</td><td>0.0854</td></tr>
<tr><td rowspan="2">50-66</td><td rowspan="2">11.79</td><td rowspan="2">50-164</td><td rowspan="2">0.0030</td><td rowspan="2">11-164</td><td rowspan="2">0.1196</td><td>11-98</td><td>0.1707</td></tr>
<tr><td>99-164</td><td>0.1921</td></tr>
<tr><td rowspan="2">67-98</td><td rowspan="2">12.77</td><td rowspan="2">Douglas Fir/Pond. Pine (much of eastern WA)</td><td rowspan="2">0-98</td><td rowspan="2">0.0061</td><td rowspan="2">Douglas Fir/Pond. Pine</td><td rowspan="2">0-98</td><td rowspan="2">0.0598</td><td rowspan="2">Douglas Fir/Pond. Pine</td><td>0-20</td><td>0.0884</td></tr>
<tr><td>21-98</td><td>0.1067</td></tr>
<tr><td>99-164</td><td>13.76</td><td colspan="9" rowspan="2"></td></tr>
<tr><td>165-328</td><td>14.08</td></tr>
</table>

To account for portions of the channel where infrastructure may limit LWM placement (e.g., under a buried structure), a higher density may be needed in some channel segments to achieve the target density for the entire restored segment if this is considered appropriate.

Density targets assume that the LWM will be engaged with instream flows so that it functions to create habitat such as pools, low-velocity refugia, cover, capture sediment, or sediment retention. To best achieve these functions, LWM shall be placed within the low-flow channel.

Using the BFW, the LWM designer first selects the corresponding 75th percentile key piece volume, then the 75th percentile key piece density, and 75th percentile total LWM volume. When using the log metrics calculator, when BFW, length of regrade, and forest zone are entered, the target metrics for the project reach are automatically calculated.

When the LWM targets are determined, the designer then enters log dimensions (midpoint diameter and length) and number for each log type to match the proposed design. The log metrics calculator helps the designer quickly determine target numbers and how the proposed design compares to the targets. Contact the State Hydraulics Office for additional or updated guidance.

## 10-4.3 *Stability*

Stability of WM in the aquatic environment refers to the ability to remain in place under hydraulic forces throughout its intended lifespan. Stability analysis evaluates the vertical, horizontal, and rotational forces acting on WM and their interactions with anchoring and resisting forces. Section 10-4.3.2 provides an overview of suitable FOSs and Section 10-4.3.1 provides an overview of performing stability analysis on WM.

### 10-4.3.1 Stability Analysis

A WM stability analysis consists of a static evaluation of the forces acting upon the WM using a free-body analysis. Vertical and horizontal forces are analyzed separately, with rotational forces considered for bank protection and stable LWM structures. The vertical and horizontal forces acting upon the WM are compared with their resisting forces, like anchoring and ballast, to determine an FOS for the vertical, horizontal, and, if applicable, rotational force components.

Numerous guidance documents deal with the stability analysis equations for estimating these forces. A description of applicable equations and their use can be found in *Large Woody Material – Risk Based Design Guidelines* (USBR 2014), NRCS (2007), and *Large Woody Debris Fish Habitat Structure Performance and Ballasting Requirements* (D'Aoust 1991). More recently, USFS has published the *Computational Design Tool for Evaluating the Stability of Large Wood Structures* (Rafferty 2016). The WSDOT-approved methodology for assessing WM stability is a modified version of the Rafferty (2016) spreadsheet. Contact the State Hydraulics Office to obtain the most up-to-date copy. Other methods may be acceptable upon review and approval by the State Hydraulics Office.

A discussion of vertical, horizonal, and rotational forces, as well as the design and selection of anchoring techniques, is provided in the sections below. Designers are responsible for selecting appropriate methods and documenting all assumptions and calculations, including determining the applicable horizontal and vertical forces acting upon the WM. The State Hydraulics Office may request that additional forces be considered in the WM stability analysis based upon project-specific considerations.

Bank protection and stable LWM stability analyses shall consider anticipated short- and long-term lateral and vertical channel changes. WM for habitat enhancement shall also consider these scour components. Assumptions for these channel changes and how they impact WM stability shall be documented in the hydraulic design report.

#### *10-4.3.1.1* Vertical Forces

Vertical forces on WM are driven primarily by buoyant force, which acts upward and is determined by the submerged volume of the wood and its unit weight. An additional upward force, lift, arises from flow velocity and the lift coefficient of the WM. Lift forces are typically a small component to the overall vertical force acting upon the WM, but it can still influence stability.

These upward forces are counteracted by resisting forces that act downward. Key resisting forces include the weight of the WM, vertical soil loading, and anchoring. In multi-log structures, interactive forces between individual logs may contribute to resistance or, in some cases, add to the upward forces.

Further discussion of anchoring techniques and interactive forces is included in Section 10-4.3.1.4.

#### *10-4.3.1.2* Horizontal Forces

Horizontal forces on WM are driven primarily by drag, which acts along the direction of flow and results from the interaction between the submerged portion of the WM and the water's velocity. The magnitude of the drag is influenced by the flow velocity, the cross-sectional area of the submerged wood, and its drag coefficient.

Additional driving horizontal forces that may arise in site-specific scenarios include impact from MWM striking the structure during high flow events, hydrostatic force caused by water surface differential across the structure, debris loading from accumulation of transported material against the structure, and ice loading.

Resisting horizontal forces counteract these driving forces and provide stability to the WM. Common resistance mechanisms include friction from the interaction between the channel bed and WM, passive forces from soil surrounding the WM, and lateral resistance provided by anchoring systems such as timber piles or boulders.

Interactive forces with other WM pieces can act as either driving or resisting forces. Further discussion of anchoring techniques and interactive forces is included in Section 10-4.3.1.4.

### *10-4.3.1.3* Rotational Forces

Rotational forces on WM occur when loading on the WM is asymmetrical, creating moments that may cause the structure to rotate. These forces are most relevant for WM placed along channel banks or in configurations where flow is unevenly distributed.

A rotational force evaluation assesses the driving and resisting moments acting on the WM. A rotational force analysis is required for all bank protection and stable LWM structures. For MWM structures, a rotational force analysis may be requested by the State Hydraulics Office based on project-specific considerations.

### *10-4.3.1.4* Anchoring and Interacting Forces

Anchoring techniques include a variety of design elements that help WM structures achieve the target FOS for vertical, horizontal, and rotational forces. WSDOT prioritizes the use of "self-ballasting" WM, which achieves the intended FOS at the design flow event without additional anchoring. However, in high-risk sites or when additional stability is required, anchoring or interactive forces with other stable logs may be employed to achieve the necessary FOS.

A variety of anchoring techniques may be employed depending on site-specific conditions, design requirements, and project constraints. It is the responsibility of the stream design engineer to select the most appropriate technique and document that basis for the selection and analysis. Factors influencing anchoring technique selection may include project permit conditions, constructability, geotechnical conditions, required FOS, and other project-specific factors. Commonly used anchoring techniques include soil ballast, boulder ballast, wood ballast, and boulder anchors. Additional anchor techniques that are not commonly used but may be considered based upon case-by-case approval by the State Hydraulics Office include dolosse-timber, earth anchors, and timber piles. For any anchoring technique that uses ferrous hardware or material, stainless-steel cable and components shall be required. Chain is not allowed within WSDOT projects or projects within WSDOT ROW. No galvanized hardware shall be used below the 100-year WSEL.

WM designs often include multiple logs, ranging from small-scale structures with a few logs to complex arrangements with hundreds of logs. In multi-log structures, interacting forces play a critical role by redistributing forces from more stable logs to less stable ones. This interaction can enhance the stability of both individual elements and the structure as a whole. For example, a log placed on top of a complex structure can transfer vertical forces downward to the logs beneath it, or timber piles placed directly behind a log can counterbalance the drag forces acting on the upstream side of the structure. For all interacting forces, the stream design engineer is responsible for determining appropriate assumptions, documenting these assumptions, and providing supporting calculations.

In simpler structures with relatively few individual logs, force interactions can be explicitly analyzed for each individual log using tools such as the *Computational Design Tool for Evaluating the Stability of Large Wood Structures* (Rafferty 2016). In larger structures, where it is impractical to account for individual forces on each log, designers may need to assume force distribution across the structure and treat it as a cohesive unit. Approval must be obtained from the State Hydraulics Office prior to adopting this approach.

### 10-4.3.2 Factor of Safety

Design criteria for WM are covered in Sections 10-4.1 and 10-4.2 with the following section providing an overview of selection of suitable FOS for WM design. FOS is defined as the ratio of the resisting forces divided by the driving forces and is evaluated for vertical, horizontal, and rotational forces separately. Selection of FOS for WM design is influenced by the site-specific purpose of the WM placement, risks to public safety and property damage, and the desired lifespan of the WM. Differing FOSs may be required for different WM placements within a single project based upon the risks to public safety and private property and design intent of the WM placement. Additional resources for evaluating risks to public safety and property damage are included in Section 10-3.

#### *10-4.3.2.1* Bank Protection

Design of WM for bank protection is covered in Section 10-4.1. The application and placement of bank protection structures are often included in a project design to protect existing or proposed infrastructure along a river or streambank in a manner that provides improvements to habitat conditions within the stream and increases overall wood loading in the project reach. As this type of design is typically in locations where risks to public safety and/or property damage are higher, a higher FOS is required for structure design. Bank protection structures shall be designed to a minimum FOS of 2 for the vertical and 1.75 for horizontal and moment FOS components. Additionally, bank protection stability analyses require stability analyses to account for impact to the structure from MWM. Refer to Section 10-4.3.1 for further details on WM stability analysis.

#### *10-4.3.2.2* Habitat Enhancement

Design of WM for habitat enhancement is covered in Section 10-4.2. Habitat enhancement WM structures are intended primarily to provide benefits to aquatic habitat rather than protection of banks or infrastructure. Habitat enhancement structures can be placed in conjunction with bank protection structures to provide a variety of habitat and infrastructure protection goals in a project design.

##### 10-4.3.2.2.1 Stable Large Woody Material

The primary purpose of stable LWM is to serve as a key structural element in habitat enhancement WM structures. Stable LWM can be placed as individual pieces or small assemblages to increase wood loading within a project reach, contributing to ecological and hydraulic benefits.

Stable LWM may be placed in locations with varying levels of risk and therefore must have a minimum FOS of 1.5 for the vertical, horizontal, and moment components. Higher FOS may be appropriate because of site-specific considerations. Additionally, stability analyses shall consider impact to the structure from MWM. Refer to Section 10-4.3.1 for further details on WM stability analysis.

#### 10-4.3.2.2.2 Mobile Woody Material

MWM is LWM that is designed to move at target design flood events. MWM placements are intended to be applied in low-risk settings where the movement of MWM pieces is anticipated to occur over the lifespan of the project. MWM shall be approved by the State Hydraulics Office. MWM shall not be placed where movement of individual or multiple pieces, including out of the project, would pose a risk to public safety or private property. FOS for MWM shall be set to 1 for both the vertical and horizontal FOS components at the target design flood event. Target design flood events shall be approved by the State Hydraulics Office. For stability analysis of MWM, moment and impact forces may be disregarded.

Designs shall not incorporate a large quantity of MWM. Designers shall provide a design where MWM mobilizes at a variety of flow events and consider rootwads on some pieces to prevent mass mobilization of all the placed MWM at the same time.

## 10-4.4 *Scour*

Scour is the principal failure mechanism of many instream structures, and it is also a primary threat to wood structures. Scour at wood placements creates important habitat features but can also cause undesirable movement or destabilization of logs and/or streambanks. Bank protection projects incorporating WM must be designed to accommodate anticipated scour conditions including, but not limited to, bendway scour, long-term degradation, and lateral migration. WM for habitat enhancement shall also consider these scour components when evaluating the FOS based on the required stability. Appropriate anchoring methods shall be used to minimize the risk for wood structures intended to be stable from mobilizing (see Section 10-4.3). Stability analyses using soil ballast as an anchoring technique shall evaluate and take into consideration the potential for the overburden/backfill material to erode. Bioengineering techniques shall also be considered whenever it is expected that the placed WM will direct flow toward the opposite bank.

Reliable methods for estimating local scour near WM have not yet been developed in either the engineering or scientific communities. In some cases, equations developed for bridge piers and abutments have been used to predict scour around wood structures, but these are overly conservative for gravel bed streams found in much of Washington and may not accurately represent the unique geometry of wood. 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 the project analysis shall be fully documented in the project's hydraulic design report.

Additional guidance may be found in Chapter 6 of the *National Large Wood Manual* (USBR 2016). This document also cites the following references as being useful for specific situations:

- **Empirical formulas for scour:** WDFW (2012), Arneson et al. (2012), Shields (2007)
- **Scour analysis applied to LWM:** Brooks et al. (2006), Abbe and Brooks (2011)
- Scour computations for ELJs: Papanicolaou et al. (2018)

## 10-5 Inspection and Maintenance

As wood members decay, they lose strength and may ultimately fail and then may be transported. LWM may also capture MWM transported from upstream in which the accumulation of wood becomes a hazard by either 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 that it provides the function intended and does not become mobilized or present a risk to infrastructure.

If a maintenance or repair action is identified, the RHE shall coordinate with the State Hydraulics Office to determine an appropriate course of action. Additional guidance will be provided in future revisions to the *Hydraulics Manual*.

## 10-6 Appendices

Appendix 10A Woody Material Structure Examples

# Appendix 10A Woody Material Structure Examples

## 10A-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.

**Figure 10A-1** Self-ballasting Large Wood Structure, Swauk Creek, Kittitas County

## 10A-2 Rootwad Habitat Structures

As the name implies, these structures consist of logs with rootwads or a series of logs with rootwads located to interact with the channel at low and high flows to provide habitat variability and structure in the stream corridor. These may or may not have anchors.

**Figure 10A-2** Rootwad Habitat Structures, Evans Creek, King County

## 10A-3 Log and Rock Revetments

These revetments consist of a rock revetment with one or two layers of logs with rootwads at the toe of the streambank. These structures provide roughness, energy diffusion, some habitat value, and minor flow deflection. They are relatively simple to install and often can be done with WSDOT Maintenance resources.

**Figure 10A-3** Log and Rock Revetments, Newaukum River, Lewis County

## 10A-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. Depending on the scour risk, the design may include wood or steel piles for added stability. Several examples of crib walls are shown below.

**Figure 10A-4** Crib Wall with Wood Piles, Beaver Creek, Okanogan County

**Figure 10A-5** Crib Wall with Steel Piles, Sauk River Side Channel, Skagit County

**Figure 10A-6** Crib Wall with Soil Lifts (No Piles), Sauk River, Skagit County

## 10A-5 Flow Deflection Jams

Flow deflection jams consist of a series of logs with attached rootwads (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 10A-7** Flow Deflection Jams, Hoh River, 2004, Clallam County

## 10A-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 10A-8** Apex Bar Jams, Hoh River, 2004, Clallam County

## 10A-7 Dolotimber

The use of dolotimber structures, or other ballasted prefabricated LWM structure matrices, 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 10A-9** Dolotimber Structures, Skagit River, Skagit County

## 10A-8 Log Jacks

Log jacks are discrete structural units that are composed of four to six logs that hold a central ballast rock. The logs are connected to each other with cable, threaded rods, or chains. The rock in turn is connected to the logs with a wire rope cradle, and secured with wire rope clips or brackets. They can be assembled in a nearby spot with ample work space and then moved into position on the water body. Each log jack is a component of a larger array of log jacks. The array is deformable, and can respond to scour.

A major advantage of log jacks is that they can be deployed without flow diversion. Being modular, log jack design can be easily adapted to various scenarios/terrains. A potential disadvantage is that portions of the log jacks that are subaerially exposed can degrade quickly over time, and may come apart. However, when used in a river with significant recruitable wood, log jacks can rack and trap wood, which can reinforce the array's stability.

**Figure 10A-10** Log Jacks, Wynoochee River, Grays Harbor County

# *Glossary and Sources*

## Abbreviations

| | |
|---|---|
| 1D | one-dimensional |
| 2D | two-dimensional |
| AASHTO | American Association of State Highway and Transportation Officials |
| ADA | Americans with Disabilities Act |
| AEP | annual exceedance probability |
| AMC | antecedent moisture condition |
| ASTM | American Society for Testing and Materials |
| AWWA | American Water Works Association |
| BFW | bankfull width |
| BMP | best management practice |
| BSTEM | Bank Stability and Toe Erosion Model |
| Caltrans | California Transportation Department |
| CCTV | closed-circuit television |
| CDF | controlled-density fill |
| CEM | Channel Evolution Model |
| CFR | Code of Federal Regulations |
| cfs | cubic foot/feet per second |
| CIPP | cured-in-place pipe |
| CLOMR | Conditional Letter of Map Revision |
| CMP | corrugated metal pipe |
| CMZ | channel migration zone |
| CN | curve number |
| D | diameter |
| DBH | diameter at breast height |
| DDP | Design Decision Package |
| DI | ductile iron *(pipe)* |
| DNR | (Washington State) Department of Natural Resources |
| ECM | Enterprise Content Management |

| | |
|---|---|
| Ecology | Washington State Department of Ecology |
| EGL | energy grade line |
| ELJ | engineered log jam |
| EOE | Office of Equal Opportunity |
| ERDC | (U.S. Army) Engineer Research and Development Center |
| FEMA | Federal Emergency Management Agency |
| FHD | final hydraulic design |
| FHWA | Federal Highway Administration |
| FOS | factor of safety |
| FPSRD | *Fish Passage and Stream Restoration Design* |
| FPW | flood-prone width |
| FRA | Flood Risk Assessment |
| ft | foot/feet |
| $ft^2$ | square foot/feet |
| ft/ft | foot/feet vertical per 1 foot horizontal |
| ft/s | foot/feet per second |
| FUR | floodplain utilization ratio |
| ga | gage |
| GIS | geographic information system |
| GPS | Global Positioning System |
| HATS | Highway Activities Tracking System |
| HDD | horizontal directional drilling |
| HDPE | high-density polyethylene |
| 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 |
| H:V | horizontal:vertical *(slope)* |
| HW | headwater |
| ID | identifier |

| | |
|---|---|
| IDF | intensity, duration, and frequency |
| in. | inch(es) |
| Injunction | 2013 Federal Court Injunction for Fish Passage |
| ISPG | *Integrated Streambank Protection Guidelines* |
| LiDAR | light detecting and ranging |
| LOMR | Letter of Map Revision |
| LTD | long-term degradation |
| 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) |
| m | meter(s) |
| $m^2$ | square meter(s) |
| MDL | master deliverable list |
| MHHW | mean higher high water |
| MHO | minimum hydraulic opening |
| mph | mile(s) per hour |
| MRI | mean recurrence interval |
| MW | mobile wood *(also known as MWM)* |
| MWM | mobile woody material *(also known as MW)* |
| N | newton(s) |
| NAIP | National Agriculture Imagery Program |
| NCHRP | National Cooperative Highway Research Program |
| NHI | National Highway Institute |
| NLCD | National Land Cover Database |
| NOAA | National Oceanic and Atmospheric Administration |
| NRCS | Natural Resources Conservation Service |
| OHWL | ordinary high water level |
| oz | ounce(s) |
| PDF | Portable Document Format |
| PE | Professional Engineer |
| PEO | Project Engineer's Office |
| PHD | preliminary hydraulic design |
| PP | polypropylene |

| | |
|---|---|
| ppt | part(s) per thousand |
| PS&E | plans, specifications, and estimates |
| psi | pound(s) per square inch |
| PSLC | Puget Sound LiDAR Consortium |
| PVC | polyvinyl chloride |
| RCP | reinforced concrete pipe |
| RCW | Revised Code of Washington |
| RESP | rock for erosion and scour protection |
| RHE | Region Hydraulics Engineer |
| ROW | right-of-way |
| RSLR | relative sea level rise |
| SBUH | Santa Barbara Urban Hydrograph |
| SCR | *Scour Certification Record* |
| SCS | Soil Conservation Service |
| SFHA | special flood hazard area |
| SFZ | structure-free zone |
| SR | State Route |
| SRH-2D | Sedimentation and River Hydraulics – 2D Model |
| Standard Specifications | *Standard Specifications for Road, Bridge, and Municipal Construction Specifications* |
| SWM | small woody material *(also known as slash)* |
| TBD | to be determined |
| $T_c$ | time of concentration |
| TCE | temporary construction easement |
| TDA | threshold discharge area |
| TESC | temporary erosion and sediment control |
| TSF | ton(s) per square foot |
| $T_t$ | travel time |
| USACE | United States Army Corps of Engineers |
| USBR | United States Bureau of Reclamation |
| USDA | United States Department of Agriculture |
| USFS | United States Forest Service |
| USGS | United States Geological Survey |

| | |
|---|---|
| UV | ultraviolet |
| WAC | Washington Administrative Code |
| WCDG | *Water Crossing Design Guidelines* |
| WDFW | Washington Department of Fish and Wildlife |
| WRIA | Water Resource Inventory Area |
| WSDOT | Washington State Department of Transportation |
| WSEL | water surface elevation |

## Main Glossary of Terms

### *A*

| | |
|---|---|
| **abrasion** | Wearing or grinding away of material by water laden with suspended material. |
| **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. |
| **aggradation** | Accumulation of sediment deposited by a river or stream. |
| **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. |

### *B*

| | |
|---|---|
| **backfill** | The soil material used refill the pipe trench after excavation and placement of pipe. |
| **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. |
| **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 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. |
| **Biologist** | One member of the Stream Team (see "Stream Team" in the Glossary). The Biologist shall meet all outlined requirements and certifications listed in Chapter 1 and Chapter 7 and is responsible for the design components of the stream channel listed in Chapter 7. |
| **bridge** | Any structure that is 20 feet or larger in span measured along the centerline of the roadway. |
| **buckling** | Failure by an inelastic change in barrel cross-section shape. |
| **bulging** | A condition where the pipe wall swells outward or protrudes from the nominal shape. |

**buried structures**
See definition in *Bridge Design Manual*, Chapter 8.

## *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).

**circumferential cracking**
A crack that occurs perpendicular to the pipe circumference.

**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 that a transportation facility will be impacted by the effects of climate change.

**coating** Any material used to protect the integrity of a structural element from the environment.

**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.

**contraction scour**
Contraction scour, in a natural channel or at a bridge crossing, involves the removal of material from the bed and banks across all or most of the channel width. This component of scour results from a contraction of the flow area at the bridge, which causes an increase in velocity and shear stress on the bed at the bridge.

**contractor** The individual or legal entity contracting with WSDOT for performance of work.

**corrosion** Deterioration or dissolution of a material by chemical or electrochemical reaction with its environment.

**countermeasure** An action or approach intended to monitor, prevent, delay, or mitigate the severity of hydraulic and/or erosion problems.

**crack** A fissure in finished materials.

**crimping** The buckling of the metallic shell of a pipe into many small waves along the perimeter of the pipe wall.

**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*

**$d_c$** Critical depth, ft

**deliverable** Any unique and verifiable product, result, or capability to perform a service that must be produced to complete a process, phase, or project.

**depth of scour** The vertical distance a streambed is lowered by scour below a reference elevation.

**design approval** Documented approval of the design at this early milestone locks in design policy for 3 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).

**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).

**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, local programs project design staff, developer project design staff, 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** The discharge that is selected as the basis for the design or evaluation of a hydraulic structure including a hydraulic design flood, scour design flood, and scour check flood.

**design methodology**
Design methodology has the meaning used in the Washington Department of Fish and Wildlife *Water Crossing Design Guidelines*.

**design reference reach**
A stable segment of stream with consistent geometry and planform, that has the slope desired for the designed project reach.

**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.

**energy grade line (EGL)**
The measure of the friction slope or rate of energy head loss due to friction losses from flows along a channel, typically represented at any given point by the sum of the potential energy (i.e., elevation head including bed elevation and flow depth) and the kinetic energy (i.e., velocity head).

## *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.

**five-hundred-year flood**
The flood due to storm and/or tide having a 0.2 percent chance of being equaled or exceeded in any given year. Commonly denoted as Q500.

**floodplain utilization ratio (FUR)**
The floodplain utilization ratio is the flood-prone width (FPW) (100-year top width) divided by the bankfull width.

**freeboard** The vertical distance above the water surface elevation (WSEL) that is allowed for waves, surges, drift, and other contingencies.

## *G*

**Geomorphologist**
One member of the Stream Team (see “Stream Team” in the Glossary). The Geomorphologist shall meet all outlined requirements and certifications listed in Chapter 1 and Chapter 7 and is responsible for the design components of the stream channel listed in Chapter 7.

**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*

**headwater (HW)** Depth from inlet invert to upstream total energy grade line, feet.

**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 design flood**
The discharge and associated probability of exceedance that reflects the desired level of service for a roadway/bridge crossing a watercourse and/or floodplain. This flood drives the capacity design (i.e., size and configuration) of the waterway opening. By definition, the approach roadway or bridge shall not be inundated by the water levels produced by this flood.

**hydraulic height**
The minimum height required for hydraulic-related purposes, including freeboard, scour, bed thickness, and appropriate maintenance clearance. Maintenance clearance shall be included in hydraulic height only if necessary to maintain habitat elements.

**hydraulic length**
The horizontal length along the stream of all components of a structure within 10 feet of the structure-free zone (SFZ) including bridges, culverts, walls, wing walls, and scour countermeasures.

**hydraulic opening**
Represents the hydraulic width and height necessary to convey the design flood and stream processes.

**hydraulic width** The minimum width perpendicular to the creek that is necessary to convey the design flood and stream processes.

***I***

**Injunction, the** United States of America et al., v. State of Washington et al. Permanent Injunction Regarding Culvert Correction, United States District Court, Western District of Washington at Seattle, No. C70-9213 Subproceeding No. 01-1 (Culverts), ordered March 29, 2013.

**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. Key pieces are independently stable.

***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 *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.

**large woody material (LWM)**
Trees and tree parts where the trunk is larger than 4 inches in diameter and larger than 6 feet in length.

**lateral (storm sewer)**
These are the first inlets that contribute 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.

**meander belt** Measurement of the width of a stream's natural meander and planform variability.

**median** The portion of a divided highway separating vehicular traffic traveling in opposite directions.

**minimum hydraulic opening (MHO)**
The minimum structure width required by the specialty report and the total height defined by minimum low chord elevation and total scour elevation.

**mobile woody material (MWM)**
Large woody material that is designed to move at target design flood events.

***N***

**non-erodible** Material that is erosion-resistant and not anticipated to degrade or erode significantly over the design life of the structure. Additional guidance and definitions will be provided in future iterations of this manual.

### *O*

**one-hundred-year flood**
The flood due to storm and/or tide having a 1 percent chance of being equaled or exceeded in any given year. Commonly denoted as Q100.

**over-coarsened channel**
A constructed channel with a median particle size that is greater than 20 percent larger than the median particle size of the 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.

**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.

### *Q*

**Q** Discharge, cfs.

**Qc** Culvert discharge, cfs.

**Qo** Overtopping discharge over total length of embankment, cfs.

**Qt** Total discharge, cfs.

### *R*

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

**regrade, channel regrade, natural channel regrade, natural regrade**
Each of these terms shall be understood to mean the natural process of a stream to establish an equilibrium slope by means of aggradation or degradation over time. Regrade is expected to effect changes to the stream, its bed and banks, and may include at a minimum, incision, deposition, debris loading, downstream flooding, lateral shifting, and

bank erosion. The regrade process will be set in motion by removal of the existing barrier to fish passage, and is intended to allow the stream to return to its natural channel, by processes that are unencumbered by the design and construction of a new fish-passable stream crossing. Furthermore, the regrade process may extend to areas outside of State right-of-way, although the degree, extent, and timing are unpredictable.

**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***

**scour** Erosion of streambed or bank material due to flowing water; can be localized around bridge piers and abutments (see long-term degradation as defined in HEC-18, local scour, contraction scour, and total scour).

**scour check flood**
The discharge associated with the 0.2 percent annual exceedance probability (e.g., 500-year) flood or the 2080 100-year projected flood (whichever is greater).

**scour design flood**
The discharge associated with the 1 percent annual exceedance probability (e.g., 100-year) flood or the 2080 100-year projected flood (whichever is greater).

**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.

**slash** Small trees and parts of trees where the trunk is less than 2 inches in diameter.

**small woody material (SWM)**
Small trees and parts of trees where the trunk is 4 inches in diameter or smaller.

**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 or degrading (Rosgen 1996).

**state highway system**
All roads, streets, and highways designated as state routes in compliance with RCW 47.17.

**Stream Design Engineer**
One member of the Stream Team (see "Stream Team" in the Glossary). The Stream Design Engineer shall meet all outlined requirements and certifications listed in Chapter 1 and Chapter 7 and is responsible for the design components of the stream channel listed in Chapter 7.

**stream simulation**
The design methodology outlined in the 2013 *Water Crossing Design Guidelines* defined as Stream Simulation.

**Stream Team**
This team is composed of a Stream Design Engineer, a Geomorphologist, and a Biologist that shall lead the day to day effort for designing the stream and its habitat in fish-passable water crossing projects. See definitions for "Stream Design Engineer", "Biologist", and "Geomorphologist" for more information. This term applies to

hydraulic design personnel and is used to distinguish the work that is performed using Chapter 7 and Chapter 10 from the rest of the *Hydraulics Manual*. Wherever "Stream Team" 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.

**streambed mix** Sediment size distribution that uses pebble counts from the reference reach for the $D_{50}$ and $D_{84}$ and an even, designed distribution of sizes for finer classes (USFS 2008).

**structure-free zone (SFZ)** The minimum boundary within which no part of the fish passage structure, including footings, shall be allowed. SFZ incorporates additional width and height beyond the minimum hydraulic opening, not hydraulic related, such as constructability, maintenance access, wildlife connectivity, or other project-specific needs.

**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***

**tailwater (TW)** Tailwater depth measured from culvert outlet invert, feet.

**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.

**total scour** The sum of long-term degradation, contraction scour, and local scour. Total scour shall be evaluated for all scenarios and flows up to and including the scour design flood and scour check flood that create worst-case total scour.

**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. This version of the document has been approved for use on WSDOT projects with exceptions as noted in Chapter 7 and Chapter 10. If a newer version of the document is published, the Hydraulics Section must approve of it prior to use.

## *Z*

**Zone A** FEMA Zone designation. Areas with a 1 percent annual chance of flooding and a 26 percent chance of flooding over the life of a 30-year mortgage. Because detailed analyses are not performed for such areas, no depths or flood elevations are shown within these zones.

**Zone AE** FEMA Zone designation. The base floodplain where base flood elevations are provided. AE Zones are on new format FIRMs instead of A1–A30 Zones.

**Zone A1-30** FEMA Zone designation. These are known as numbered A Zones (e.g., A7 or A14). This is the base floodplain where the FIRM shows a BFE (old format).

## Sources

AASHTO (American Association of State Highway and Transportation Officials). 2017. Standard Specification for Precast Reinforced Concrete Box Sections for Culverts, Storm Drains, and Sewers, January.

Abbe, T., B. Belby, and F.D. Shields. 2016. Geomorphology and hydrology considerations. Chapter 4 in Bureau of Reclamation and U.S. Army Engineer Research and Development Center. National Large Wood Manual: Assessment, Planning, Design, and Maintenance of Large Wood in Fluvial Ecosystems: Restoring Process, Function, and Structure. 628 p. + Appendix. http://www.usbr.gov/pn/ and http://naturaldes.com/resources/

Abbe, T., M. Ericsson and L. Embertson. 2015. Channel incision and floodplain abandonment due to historic wood removal in Washington State, USA. International Conference on Wood in World Rivers Conference. University of Padova, Padova, Italy.

Abbe, T. B., and A. P. Brooks. 2011. "Geomorphic, Engineering, and Ecological Considerations when Using Wood in River Restoration." 419–451 in A. Simon, S. J. Bennett, and J. M. Castro (eds.), *Stream Restoration in Dynamic Fluvial Systems: Scientific Approaches, Analyses, and Tools*. Geophysical Monograph Series 194. Washington, D.C.: American Geophysical Union.

Abbe, T.B., S. Dickerson-Lange, S., M. Kane, M., P. Cruickshank, P., M. Kaputa, M., and J. Soden, J. 2019. Can wood placement in degraded channel networks result in large-scale water retention? Proceedings of SEDHYD 2019: Conferences on Sedimentation and Hydrologic Modeling, 24-28 June 2019 in Reno, Nevada, USA. Volume 1, pp.408–427.

Abbe, T. B., and D. R. Montgomery. 1996. "Large Woody Debris Jams, Channel Hydraulics and Habitat Formation in Large Rivers." https://doi.org/10.1002/(SICI)1099-1646(199603)12:2/3%3C201::AID-RRR390%3E3.0.CO;2-A

Aberle, J. and G. Smart. 2003. The influence of roughness structure on flow resistance on steep slopes. *Journal of hydraulic research 41*(3), 259–269.

American Whitewater. 2019. www.americanwhitewater.org.

Anderson, Paul S., Susan Meyer, Dr. Patricia Olson, Erik Stockdale. 2016. Determining the Ordinary High Water Mark for Shoreline Management Act Compliance in Washington State. Washington State Department of Ecology, Olympia, Washington.

Annandale, G. W. 2006. "Review of scour technology."

ASTM (American Society for Testing and Materials). 2016. C1433-16b, Standard Specification for Precast Reinforced Concrete Monolithic Box Sections for

Culverts, Storm Drains, and Sewers, ASTM International, West Conshohocken, Pennsylvania. www.astm.org.

Baird, D. C., L. Fotherby, C. C. Klumpp, and S. M. Sculock. 2015. *Bank Stabilization Design Guidelines*. U.S. Department of the Interior, Bureau of Reclamation, Technical Service Center, Denver, Colorado.

Barnes, H. H. 1967. *Roughness characteristics of natural channels*. *US Geological Survey Water-Supply Paper 1849*. U.S. Government Printing Office.

Bartels, R., J. D. Dell, R. L. Knight, G. Schaefer. 1985. "Dead and Down Woody Material." *Management of Wildlife and Fish Habitats in Forests of Western Oregon and Washington*. USDA Forest Service. Portland, Oregon.

Chow, V. T. 1959. Open-channel hydraulics. McGraw-Hill civil engineering series.

Bathurst, J. C. 1985. Flow resistance estimation in mountain rivers. *Journal of Hydraulic Engineering* 111(4), 625–643.

Bathurst, J. C. 1987. "Critical conditions for bed material movement in steep, boulder-bed streams." International Association of Hydrological Sciences Publication 165: 309–318.

Bilby, Robert E., and Gene E. Likens. "Importance of organic debris dams in the structure and function of stream ecosystems." Ecology 61, no. 5 (1980): 1107-1113.

Booth, D. 1990. Stream-Channel Incision Following Drainage-Basin Urbanization. Water Resources Research. American Water Resources Association. 26(3) 407–417.

Borah D. K. 1989. Scour-depth prediction under armoring conditions. *Journal of Hydraulic Engineering*, ASCE Vol. 115, No. 10.

Braudrick, C. A., and G. E. Grant. 2000. "Why Do Logs Move In Rivers?" *Water Resources Research* 36-2, John Wiley & Sons, LTD.

Braudrick, C. A., G. E. Grant, Y. Ishiharu, and H. Ikeda. 1997. "Dynamics of Wood Transport in Streams: A Flume Experiment." *Earth Surface Processes and Landforms*. 22: 669–683.

Bretschko, Gernot. "The dynamic aspect of coarse particulate organic matter (CPOM) on the sediment surface of a second order stream free of debris dams (RITRODAT-LUNZ study area)." Hydrobiologia 203, no. 1-2 (1990): 15-28.

Brooks, A.P., Brierley, G.J. and Millar, R.G. 2003. The long-term control of vegetation and woody debris on channel and floodplain evolution: insights from a paired catchment study between a pristine and disturbed lowland alluvial river in southeastern Australia. Geomorphology 51, 7–29.

Brooks, A.P., T. Abbe, T. Cohen, N. Marsh, S. Mika, A. Boulton, T. Broderick, D. Borg, and I. Rutherford. 2006. Design guideline for the reintroduction of wood into Australian streams. Land & Water, Australia, Canberra.

Brummer, C.J., Abbe, T.B., Sampson, J.R. and Montgomery, D.R. 2006. Influence of vertical channel change associated with wood accumulations on delineating channel migration zones, Washington, USA. Geomorphology 80, 295–309.

Bunte, Kristin; Abt, Steven R. 2001. Sampling surface and subsurface particle-size distributions in wadable gravel- and cobble-bed streams for analyses in sediment transport, hydraulics, and streambed monitoring. Gen. Tech. Rep. RMRS-GTR-74. Fort Collins, Colorado: U.S. Department of Agriculture, Forest Service, Rocky Mountain Research Station. 428 p.

Castro, J. 2003. Geomorphologic Impacts of Culvert Replacement and Removal: Avoiding Channel Incision; Unpublished Report. U.S. Fish and Wildlife Service: Portland, Oregon.

Castro, J.M. and A. Beavers. 2016. Providing Aquatic Organism Passage in Vertically Unstable Streams. Water 8(133), doi:10.3390/w8040133.

Castro, Janine M., and Colin R. Thorne. 2019. "The stream evolution triangle: Integrating geology, hydrology, and biology." River Research and Applications 35, No. 4: 315–326.

Cluer, Brian, and Colin Thorne. 2013. “A stream evolution model integrating habitat and ecosystem benefits.” River Research and Applications 30, No. 2: 135–154.

Collins, B.D., D.R. Montgomery, and A.D. Haas, 2002. Historical Changes in the Distribution and Functions of Large Wood in Puget Lowland Rivers. Canadian Journal of Fisheries and Aquatic Sciences 59:66–76.

Cramer, M.L. (managing editor). 2012. *Stream Habitat Restoration Guidelines*. Co-published by the Washington Departments of Fish and Wildlife, Natural Resources, Transportation and Ecology, Washington State Recreation and Conservation Office, Puget Sound Partnership, and the U.S. Fish and Wildlife Service. Olympia, Washington.

D’Aoust, S. G. 1991. “Large Woody Debris Fish Habitat Structure Performance and Ballasting Requirements.” Master of Applied Science Thesis, University of Ottawa. Revised 1999.

D’Aoust, S. G., and R. G. Millar. 2000. “Stability of Ballasted Woody Debris Habitat Structures.” *Journal of Hydraulic Engineering*. November. www.wsdot.wa.gov/publications/manuals/fulltext/M23-03/M23-03.05Revision.pdf.

Dey, Subhasish. 2014. *Fluvial Hydrodynamics: Hydrodynamic and Sediment Transport Phenomena.* Fluvial Processes: Meandering and Braiding.

Diehl, T. H., and B. A. Bryan. 1993. Supply of Large Woody Debris in a Stream Channel. *Hydraulic Engineering*, 1055.

Dolloff, C. Andrew, and Jackson R. Webster. "Particulate organic contributions from forests and streams: debris isn't so bad." In: Verry, Elon S.; Hornbeck, James W.; Dolloff, C. Andrew, eds. Riparian management in forests of the continental Eastern United States. Boca Raton, FL: Lewis Publishers, CRC Press LLC: 125-138. (2000).

Downs, P.W. and Piégay, H. 2019. Invited review: Catchment-scale cumulative impact of human activities on river channels in the late Anthropocene: implications, limitations, prospect. Geomorphology. 338, 88–104.

Ecology (Washington State Department of Ecology). 2003. A Framework for Delineating Channel Migration Zones. Ecology Publication 03-06-027 (Final Draft). November. https://apps.ecology.wa.gov/publications/documents/0306027.pdf

———. 2014. Channel Migration Toolbox: ArcGIS® Tools for Measuring Stream Channel Migration. Publication 14-06-032. October. https://apps.ecology.wa.gov/publications/documents/1406032.pdf

———. 2015. Screening Tools for Identifying Migrating Stream Channels in Western Washington: Geospatial Data Layers and Visual Assessments. Publication 15-06-003. February. https://apps.ecology.wa.gov/publications/documents/1506003.pdf

———. 2019. *Stormwater Management Manual for Western Washington*. Publication 19-10-021. https://apps.ecology.wa.gov/publications/SummaryPages/1910021.html.

FHWA (Federal Highway Administration). 2005. “Debris Control Structures; Evaluation and Countermeasures, Third Edition.” *Hydraulic Engineering Circular No. 9*, Publication FHWA- IF-04-016. U.S. Department of Transportation, Washington, D.C.

———. 2006. “Hydraulic Design of Energy Dissipators for Culverts and Channels, Third Edition.” *Hydraulic Engineering Circular No. 14*, Publication FHWA-NHI-06-086. U.S. Department of Transportation, Washington, D.C.

———. 2009. “Bridge Scour and Stream Instability Countermeasures: Experience, Selection, and Design Guidance, Third Edition.” *Hydraulic Engineering Circular No. 23*, Publication FHWA-NHI-09-111. U.S. Department of Transportation, Washington, D.C.

———. 2009. “Urban Drainage Manual, Third Edition.” *Hydraulic Engineering Circular No. 22*, Publication FHWA-NHI-10-009. U.S. Department of Transportation, Washington, D.C.

———. 2010. "Culvert Design for Aquatic Organism Passage" *Hydraulic Engineering Circular No. 26*, Publication FHWA-HIF-11-008. U.S. Department of Transportation, Washington, D.C.

———. 2012. "Design of Roadside Channels with Flexible Linings, Third Edition." *Hydraulic Engineering Circular No. 15*, Publication FHWA-HIF-12-004. U.S. Department of Transportation, Washington, D.C.

———. 2012. "Evaluating Scour at Bridges, Fifth Edition." *Hydraulic Engineering Circular No. 18*, Publication FHWA-HIF-12-003. U.S. Department of Transportation, Washington, D.C.

———. 2012. "Stream Stability at Highway Structures, Fourth Edition." *Hydraulic Engineering Circular No. 20*, Publication FHWA-HIF-12-004. U.S. Department of Transportation, Washington, D.C.

———. 2016. "Highways in the River Environment: Floodplains, Extreme Events, Risk, and Resilience." *Hydraulic Engineering Circular No. 17*, Publication FHWA-HIF-16-018. U.S. Department of Transportation, Washington, D.C.

———. 2019. *Two-Dimensional Hydraulic Modeling for Highways in the River Environment*. Publication FHWA-HIF-19-061. October. https://www.fhwa.dot.gov/engineering/hydraulics/pubs/hif19061.pdf.

———. 2020. "Highways in the Coastal Environment: Third Edition." *Hydraulic Engineering Circular No. 25*, Publication FHWA-HIF-19-059. U.S. Department of Transportation, Washington, D.C.

———. 2020. *FHWA Tech Brief "Hydraulic Considerations for Shallow Abutment Foundations."* Office of Bridges & Structures. Publication FHWA-HIF- 19-007. December 15 (originally published December 21, 2018). https://www.fhwa.dot.gov/engineering/hydraulics/pubs/hif19007.pdf.

———.2021. *FHWA Tech Brief "Pier Scour Estimation for Tsunami at Bridges."* Office of Bridges & Structures. Publication FHWA-HRT-21-073. August. https://www.fhwa.dot.gov/publications/research/infrastructure/structures/bridge/21073.pdf.

———. 2023a. *FHWA Tech Brief "Hydraulic Considerations for Abutments on Deep Foundations and Bridge Embankment Protection."* Office of Bridges & Structures. Publication FHWA-HIF-23-048. September. https://www.fhwa.dot.gov/engineering/hydraulics/pubs/hif23048.pdf.

———. 2023b. *Highways in the River Environment: Roads, Rivers, and Floodplains, Second Edition.* Publication FHWA-HIF-23-004. Hydraulic Engineering Circular No. 16. January.

https://www.fhwa.dot.gov/engineering/hydraulics/library_arc.cfm?pub_number=2&id=185.

Flanagan, S. A. 2003. “How Culverts Fail.”

———. 2005. “Woody Debris Transport at Road-Stream Crossings.” *Stream Notes*. USDA, Forest Service. Stream Systems Technology Center - Rocky Mountain Research Station. Fort Collins, Colorado. https://www.fs.usda.gov/biology/nsaec/assets/sn_10_05.pdf October.

Fouty, Tyler. “Flume-Based Design Recommendations for Deformable Grade Control in Stream Simulation Culverts.” Microsoft Teams Presentation, March 10, 2023.

Fox, M. J. 2003. *Spatial Organization, Position, and Source Characteristics of Large Woody Debris in Natural Systems*. Doctoral dissertation, University of Washington, Seattle, WA. Copyright 2003 by ProQuest Information and Learning Company, Ann Arbor, Michigan, UMI No. 3102652.

Fox, M., and S. Bolton. 2007. A regional and geomorphic reference for quantities and volumes of instream wood in unmanaged forested basins of Washington State. North American Journal of Fisheries Management 27:342–359.

Furniss, M. J., T. S. Ledwith, M. A. Love, B. C. McFadin, and S. A. Flanagan. 1998. *Response of Road-Stream Crossings to Large Flood Events in Washington, Oregon, and Northern California*. USDA, Forest Service. San Dimas Technology and Development Center. San Dimas, California. https://www.fs.usda.gov/t-d/pubs/html/wr_p/98771807/98771807.htm

Gardner, James W. 1983. *Experimental study of knickpoint and longitudinal profile evolution in cohesive, homogeneous material.* Department of GeoSciences, The Pennsylvania State University.

Griffiths, G. A. 1981. Flow resistance in coarse gravel bed rivers. *Journal of the Hydraulics Division 107*(7), 899–918.

Hanson, Heather. 2022. *Culvert Design Guidelines for Ecological Function*. U.S. Fish and Wildlife Service. Updated March 14, 2022. https://www.fws.gov/alaska-culvert-design-guidelines

Hey, R. D. 1979. Flow resistance in gravel-bed rivers. *Journal of the Hydraulics Division 105*(4), 365–379.

Hutchinson, I. 1988. *Salinity tolerance of plants of estuarine wetlands and associated uplands*. Report to the Washington State Shorelands and Coastal Zone Management Program. Contract C0088137. Simon Fraser University, Burnaby, British Columbia.

Jarrett, R. D. 1984. Hydraulics of high-gradient streams. *Journal of Hydraulic Engineering 110*(11), 1519–1539.

Johnson, A. W., and J. M. Stypula (eds.). 1993. *Guidelines for Bank Stabilization Projects in the Riverine Environments of King County*. King County Department of Public Works, Surface Water Management Division. Seattle, Washington.

Johnson, P.A. 2006. Assessing Stream Channel Stability at Bridges in Physiographic Regions. Federal Highway Administration Office of Infrastructure Research and Development. Publication FHWA-HRT-05-072.

Kondolf, M. G, and T. E. Lisle. 2016. "Measuring bed sediment," chap. 13 in *Tools in Fluvial Geomorphology*. M. G. Kondolf and M. H. Piégay, eds., 278–305. John Wiley & Sons, Ltd.

Kramer, Natalie, and Ellen Wohl. 2016. "Rules of the Road: A Qualitative and Quantitative Synthesis of Large Wood Transport Through Drainage Networks." *Geomorphology*, 279. 10.1016/j.geomorph.2016.08.026.

Kuichling, E. 1889. "The Relation Between the Rainfall and the Discharge of Sewers in Populous Districts." *Transactions*, American Society of Civil Engineers 20, 1–56.

Lagasse, P.F., P. E. Clopper, L. W. Zevenbergen, and J. F. Ruff. 2006. *Riprap Design Criteria, Recommended Specifications, and Quality Control*. NCHRP Report 568, Transportation Research Board, Washington, D.C. http://onlinepubs.trb.org/onlinepubs/nchrp/nchrp_rpt_568.pdf.

Lagasse, P.F., P. E. Clopper, L. W. Zevenbergen, W. J. Spitz, and L. G. Girard. 2010. "Effects of Debris on Bridge Pier Scour." *NCHRP Report 653*, Transportation Research Board, Washington, D.C. doi: 10.17226/22955.

Lane, E.W. Design of stable alluvial channels. Trans. Am. Soc. Civ. Eng. 1955. 120, 1234–1260.

Lee, A. J. and R. I. Ferguson. 2002. Velocity and flow resistance in step-pool streams. *Geomorphology* 46(1-2), 59–71.

Limerinos, J. T. 1970. Determination of the manning coefficient from measured bed roughness in natural channels. *Geologic Survey Water-Supply Paper 1898-B.* U.S. Government Printing Office.

Liu, X., Mahdavi, A., Zevenbergen, L. and Kramer, C. 2024. *Selection and Application of Mannings's Roughness Values in Two-Dimensional Hydraulic Models*. NCHRP Report 1077, Transportation Research Board, Washington, D.C. https://www.trb.org/Publications/Blurbs/183186.aspx

Mackin, J.H., 1948. Concept of a graded river. Geological Society of America Bulletin 59, 463–511.

Marden, M., Arnold, G., Gomez, B. and Rowan, D. 2005. Pre- and post-reforestation gully development in the Mangatu Forest, East Coast, North Island, New Zealand. River Research and Applications 21, 757–771.

Mastin, M. C., C. P. Konrad, A. G. Veilleux, and A. E. Tecca. 2016. *Magnitude, Frequency, and Trends of Floods at Gaged and Ungaged Sites in Washington, Based on Data through Water Year 2014* (ver 1.2, November 2017): USGS Scientific Investigations Report 2016–5118, 70 p., http://dx.doi.org/10.3133/sir20165118.

MGS Software. 2018. MGSFlood. http://www.mgsengr.com/mgsfloodhome.html.

Miller, J. F., R. H. Frederick, and R. J. Tracey. 1973. *Precipitation—Frequency Atlas of the Western United States*. National Oceanic and Atmospheric Administration, U.S. Department of Commerce, Washington, D.C.

Miller, I.M., H. Morgan, G. Mauger, T. Newton, R. Weldon, D. Schmidt, M. Welch, and E. Grossman. 2018. *Projected Sea Level Rise for Washington State – A 2018 Assessment*. A collaboration of Washington Sea Grant, University of Washington Climate Impacts Group, University of Oregon, University of Washington, and U.S. Geological Survey. Prepared for the Washington Coastal Resilience Project. Updated July 2019.

Montgomery, D. R., and J. M. Buffington. 1993. *Channel Classification, Prediction of Channel Response, and Assessment of Channel Conditions*. Timber, Fish & Wild-Life Publication TFW-SH10-93-002.

Montgomery, D. R., Buffington, J. M. 1997. Channel-reach morphology in mountain drainage basin. GSA Bulletin; May; v. 109; no. 5; p. 596–611.

Montgomery, D.R. and Abbe, T.B. 2006. Influence of logjam-formed hard points on the formation of valley-bottom landforms in an old-growth forest valley, Queets River, Washington, USA. Quaternary Research 65, 147–155.

Mount, J.F. 1995. California Rivers and Streams. University of California Press. Berkeley, California.

NCHRP (National Cooperative Highway Research Program). 2004. Report 533: Handbook for Predicting Stream Meander Migration. Transportation Board of the National Academies. https://www.trb.org/publications/nchrp/nchrp_rpt_533.pdf

NRCS (National Resources Conservation Service). 2007. *Stream Restoration Design National Engineering Handbook 654*. U.S. Department of Agriculture, Washington, D.C.

———. 2010. *National Engineering Handbook Part 630 Hydrology*. U.S. Department of Agriculture, Washington, D.C.

———. n.d. NRCS Snow Map, https://wrcc.dri.edu/summary/climsmwa.html.

Phelps, J.D., 2011. The Geomorphic Legacy of Splash Dams in the Southern Oregon Coast Range. University of Oregon.

Pollock, M.M., Beechie, T.J. and Jordan, C.E. 2007. Geomorphic changes upstream of beaver dams in Bridge Creek, an incised stream channel in the interior Columbia River basin, eastern Oregon. Earth Surface Processes and Landforms 32, 1174–1185.

Pollock, M.M., T.J. Beechie, J.M. Wheaton, C.E. Jordan, N. Bouwes, N. Weber, and C. Volk, 2014. Using Beaver Dams to Restore Incised Stream Ecosystems. BioScience 64:279–290.

Pollock, M.M., J.M. Wheaton, N. Bouwes, C. Volk, N. Weber, and C.E. Jordan, 2012. Working with Beaver to Restore Salmon Habitat in the Bridge Creek Intensively Monitored Watershed Design Rationale and Hypotheses. Seattle, Washington.

Powers, Paul D., Matt Helstab, and Sue L. Niezgoda. 2019. “A process‐based approach to restoring depositional river valleys to Stage 0, an anastomosing channel network.” River Research and Applications 35, no. 1: 3–13.

Prosser, I.P. and Soufi, M. 1998. Controls on gully formation following forest clearing in a humid temperate environment. Water Resources Research 34 (12), 3661–3671.

Rafferty, M. 2016. *Computational Design Tool for Evaluating the Stability of Large Wood Structures*. Technical Note TN-103.2. Fort Collins, Colorado: U.S. Department of Agriculture, Forest Service, National Stream and Aquatic Ecology Center. 27 p.

Rickenmann, D. and A. Recking. 2011. Evaluation of flow resistance in gravel-bed rivers through a large field data set. *Water Resources Research 47*(7).

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.

Ruiz-Villanueva, V., H. Piégay, A. M. Gurnell, R. A. Marston, and M. Stoffel. 2016. “Recent Advances Quantifying the Large Wood Dynamics in River Basins: New Methods and Remaining Challenges.” *Reviews of Geophysics*. 54,611–652. https://agupubs.onlinelibrary.wiley.com/doi/full/10.1002/2015RG000514.

Schanz, A.A, Montgomery, D.R., Collins, B. 2019. Anthropogenic strath terrace formation caused by reduced sediment retention. PNAS, 116(18) 8734–8739.

Schumm, S.A.; Harvey, M.; Watson, C. 1984. Incised Channels: Morphology, Dynamics, and Control; Water Resources Publications: Littleton, Colorado.

Sear, D.A., Millington, C.E., Kitts, D.R. and Jeffries, R. 2010. Logjam controls on channel: floodplain interactions in wooded catchments and their role in the formation of multi-channel patterns. Geomorphology 116, 305–319.

Shahverdian, Scott & Wheaton, Joseph & Bennett, Stephen & Bouwes, Nick & Camp, Reid & Jordan, Chris & Portugal, Elijah & Weber, Nick. (2019). Chapter 4 – Mimicking and Promoting Wood Accumulation and Beaver Dam Activity with Post-Assisted Log Structures and Beaver Dam Analogues. 10.13140/RG.2.2.22526.64324.

Simon, A.; Rinaldi, M. 2000. Channel instability in the loess area of the Midwestern United States. J. Am. Water Resour. Assoc., 36, 133–150.

Simon, A.; Rinaldi, M. Disturbance, stream incision, and channel evolution: The roles of excess transport capacity and boundary materials in controlling channel response. Geomorphology. 79, 361–383.

Simon, A., R. Thomas, A. Curini, and N. Bankhead. 2009. "Bank stability and toe erosion model (BSTEM) Static version 5.2."

Skidmore, P.B., C.R. Thorne, B.L. Cluer, G.R. Pess, J.M. Castro, T.J. Beechie, and C.C. Shea. 2011. Science base and tools for evaluating stream engineering, management, and restoration proposals. U.S. Dept. Commerce, NOAA Tech. Memo. NMFS-NWFSC-112, 255 p.

Stock, J.D., Montgomery, D.R., Collins, B.D., Dietrich, W.E. and Sklar, L. 2005. Field measurements of incision rates following bedrock exposure: implications for process controls on the long profiles of valleys cut by rivers and debris flows. Geological Society of America Bulletin 117 (11/12), 174–194.

Stubchaer, J. M. 1975. "The Santa Barbara Urban Hydrograph Method," in Proceedings of National Symposium on Urban Hydrology and Sediment Control. University of Kentucky, Lexington, Kentucky, 131–141.

USBR (United States Bureau of Reclamation). 2014. *Large Woody Material—Risk-Based Design Guidelines*. Knutson, P.E., and Fealko, P.E., co-authors, USBR Pacific Northwest Region Boise, Idaho. 115 p.

USBR and ERDC (United States Bureau of Reclamation and U.S. Army Engineer Research and Development Center). 2016. *National Large Wood Manual: Assessment, Planning, Design, and Maintenance of Large Wood in Fluvial Ecosystems: Restoring Process, Function, and Structure*. https://www.usbr.gov/research/projects/download_product.cfm?id=2219 (click on "River Restoration," then "Techniques").

USDA (United States Department of Agriculture). 2008. Stream Simulation: An Ecological Approach to Providing Passage for Aquatic Organisms at Road-Stream Crossings, Appendix E.

USDOT (United States Department of Transportation). 2008. "Introduction to Highway Hydraulics." Hydraulic Design Series 4, Publication FHWA-NHI-08-090. U.S. Department of Transportation, Washington, D.C.

———. 2012a. "Hydraulic Design of Highway Culverts, Third Edition." *Hydraulic Design Series 5*, Publication FHWA-HIF-12-026m. U.S. Department of Transportation, Washington, D.C.

———. 2012b. "Hydraulic Design of Safe Bridges." *Hydraulic Design Series 7*, Publication FHWA-HIF-12-018. U.S. Department of Transportation, Washington, D.C.

———. 2016. HY-8 Culvert Hydraulic Analysis Program software, Version 7.50. U.S. Department of Transportation, Washington, D.C. www.fhwa.dot.gov/engineering/hydraulics/software/hy8/. Build date July 28.

USFS (United States Forest Service). 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.

USGS (United States Geological Survey). n.d. PeakFQ website: https://water.usgs.gov/software/PeakFQ.

———. n.d. StreamStats website: https://streamstats.usgs.gov/ss.

———. n.d. Surface-Water Data for Washington Website: https://streamstats.usgs.gov/ss. Washington Climate Summaries. n.d., Western Regional Climate Center Webpage, https://wrcc.dri.edu/summary/climsmwa.html.

WDFW (Washington Department of Fish and Wildlife). 2002. *Integrated Streambank Protection Guidelines*. Washington State Aquatic Habitat Guidelines Program, Olympia, Washington.

———. 2013. *Water Crossing Design Guidelines*. Olympia, Washington, 300 p.

———. 2016a. *Incorporating Climate Change into the Design of Water Crossing Structures; Final Project Report*. Habitat Program—Science Division, Olympia, Washington, 300 p.

Whiting, Peter J., and William E. Dietrich. 1993. "Experimental constraints on bar migration through bends: Implications for meander wavelength selection." *Water Resources Research* 29, no. 4: 1091–1102.

Wolman, M. G. 1954. "A Method of Sampling Coarse River-Bed Material." *Transactions—American Geophysical Union*, 35, 951–956.

WSDOT (Washington State Department of Transportation). 2011. *Climate Impacts Vulnerability Assessment*. November.

———. 2019a. Utilities Manual M 22-87.10. Engineering and Regional Operations, Development Division, Design Office. February 28.

———. 2019b. *Highway Runoff Manual* M 31-16.05. Engineering and Regional Operations, Development Division, Design Office. Supplement May 22.

———. 2019c. *Development Services Manual* M 3007-01, Engineering and Regional Operations, Development Division, Design Office. October 28.

———. 2020. *Design Manual* M 22-01.19. Engineering and Regional Operations, Development Division, Design Office. September.

———. 2021a. *Local Agency Guidelines* M 36-63.36. Local Programs, Engineering Services. June 22.

———. 2021b. *Design-Build Manual* M 3126.02. Construction Division. Design-Build Office. August 9.

———. 2021c. *Standard Specifications for Road, Bridge, and Municipal Construction* M 41-10. August 22.

———. 2021d. *Standard Plans* M 21-01. Engineering and Regional Operations, Development Division, Design Office. September 21.

Yochum, S. E., B. P. Bledsoe, G. C. David, and E. Wohl. 2012. Velocity prediction in high-gradient channels. *Journal of Hydrology 424*, 84–98..