Chapter 4 Open-Channel Flow

4-1 Introduction

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

- Riverbank stabilization (Section 4-6)
- Roadside ditches (Section 4-3)

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

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

River stabilization (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.

The flow capacity of a culvert is often dependent on the channel upstream and downstream from that culvert. For example, the tailwater level is often controlled by the hydraulic capacity of the channel downstream of the culvert. Knowing the flow capacity of the downstream channel, open-channel flow equations can be applied to a channel cross section to adequately determine the depth of flow in the downstream channel. This depth can then be used in the analysis of the culvert hydraulic capacity. See Chapter 3.

Biofiltration swales are shallow, grass-lined, open channels that treat stormwater runoff before it reaches a receiving body. See the *Highway Runoff Manual* for a full list of approved stormwater BMPs.

A downstream analysis identifies and evaluates the impacts that a project will have on the hydraulic conveyance system downstream of the project site. For example, the downstream analysis looks for possible interaction and/or impacts to the channel with other infrastructure. See Section 1-3.3.

Measurement and/or estimates of flow in channels can be difficult because of non-uniform channel conditions, governed by varying channel dimensions (i.e., changes in width or depth) and have corresponding variations in flow characteristics such as flow area, depth, and velocities along the channel. Weirs, for example, allow water to be routed through a structure of known dimension, permitting flow rates to be directly measured. Other methods can be used for indirectly estimating channel flows, which require calculating flows using channel geometry and characteristics; for example, channel dimensions, velocities, assumed roughness coefficient, and flow gradients can be used to calculate flow through the channel.

4-2 Determining Channel Flow Rates

In open-channel flow, the characteristics of flow such as the discharge rate of flow, cross-sectional flow area, and the velocity at which flow travels are useful in designing the channel. As mentioned above, the discharge rate can be measured directly by collecting manual measurements of velocity relative to channel shape; or using a weir, which would require installing a temporary or permanent structure in the channel. Either method may pose practicality, safety, or feasibility challenges.

For the purposes of the *Hydraulics Manual*, the determination of the flow characteristics can be calculated based on the continuity of flow equation, or Equation 4-1. This equation states that the discharge (Q) is equivalent to the product of the channel velocity (V) and the area of flow (A).

$$Q = V A \quad (4\text{-}1)$$

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.

$$V = 1.486(R^{2/3})(S^{1/2})/n \quad (4\text{-}2)$$

where:

V = mean velocity of flow in feet per second

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

S = slope of the energy grade line (EGL) or, for assumed uniform flow, the slope of the channel in feet (vertical) per foot (horizontal), to calculate see points shown in Figure 4-1, Figure 4-2, and Figure 4-3 in following sections, where $S_{UF} = (A_{us} - B_{ds})/HMD_{AB}$), and S_{UF} is the slope for uniform flow conditions, A_{us} is the elevation at the upstream located Point A, B_{ds} is the elevation at the downstream located Point B, and HMD_{AB} is the horizonal measured distance between Points A and B; for assumed non-uniform flow, see points shown in Figure 4-4, Figure 4-5, and Figure 4-6 in following sections, where S_{NUF} = (Bus - Bds)/HMD_{BB}), and S_{NUF} is the slope of non-uniform flows, Bus is the elevation at the upstream located Point B, Bds is the elevation at the downstream located Point B, and HMD_{BB} is the horizonal measured distance between Points B and B.

n = Manning's roughness coefficient or friction factor of the channel lining, and 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 4-2.1 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 computer hydraulic simulation modeling programs. The hydraulic designer is also referred to HDS-4 for further information on channel flow rates and velocities.

4-2.1 *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

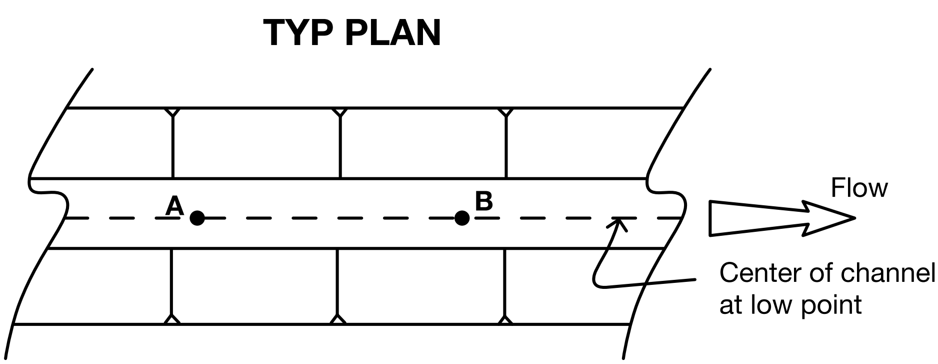
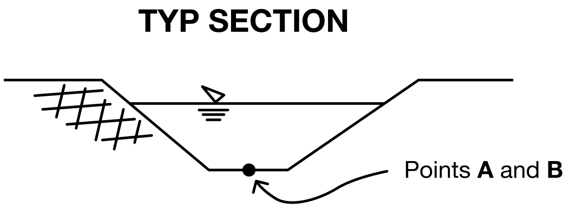


Figure 4-2 Field Slope Measurement of Uniform Flow Channels Section View



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 should be measured for each similar channel reach, and the results should 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

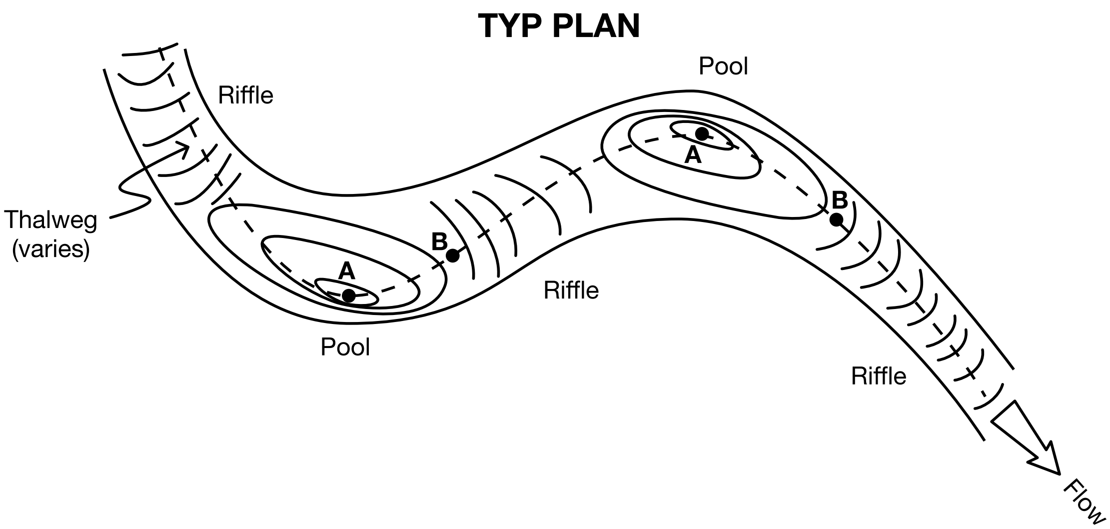
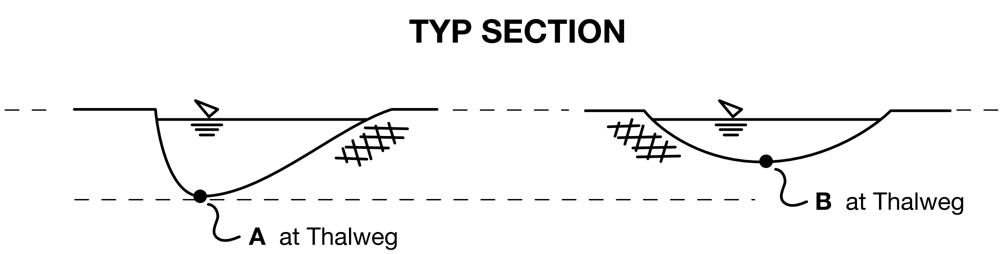


Figure 4-4 Field Slope Measurement of Non-Uniform Flow Channels Section View

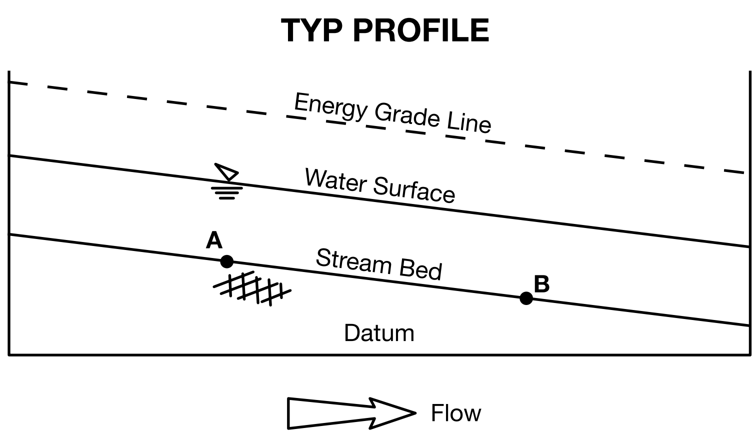


In both uniform and non-uniform channels, the engineer may need to apply discretion in how the gradient reaches are assessed and/or combined to best represent the channel hydraulic conditions, and where the thalweg is located.

4-2.1.1 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 should 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 should be measured between the two points where the bed or top of water points were located.

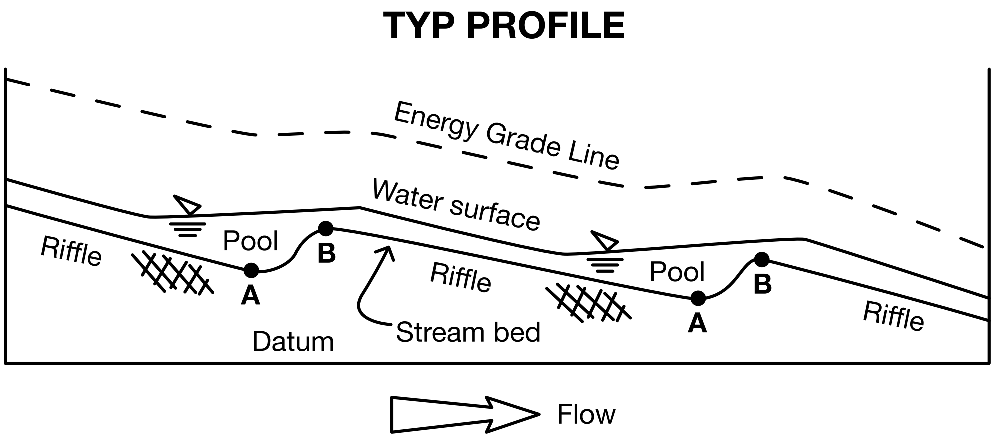
Figure 4-5 Field Slope Measurement of Uniform Flow Channels Profile View



4-2.1.2 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 should 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.

4-2.1.3 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 should 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 PEO for more information regarding more complex analyses.

4-3 Roadside Ditch Design Criteria

Roadside ditches are generally located alongside uncurbed roadways with the primary purpose of conveying runoff away from the roadway. Ditches shall be designed to convey the 10-year recurrence interval with a 0.5-foot freeboard (from the ditch design WSEL to the bottom of the pavement subgrade or ditch spill) and a maximum side slope of 2 horizontal (H):1 vertical (V) (Figure 4-7).

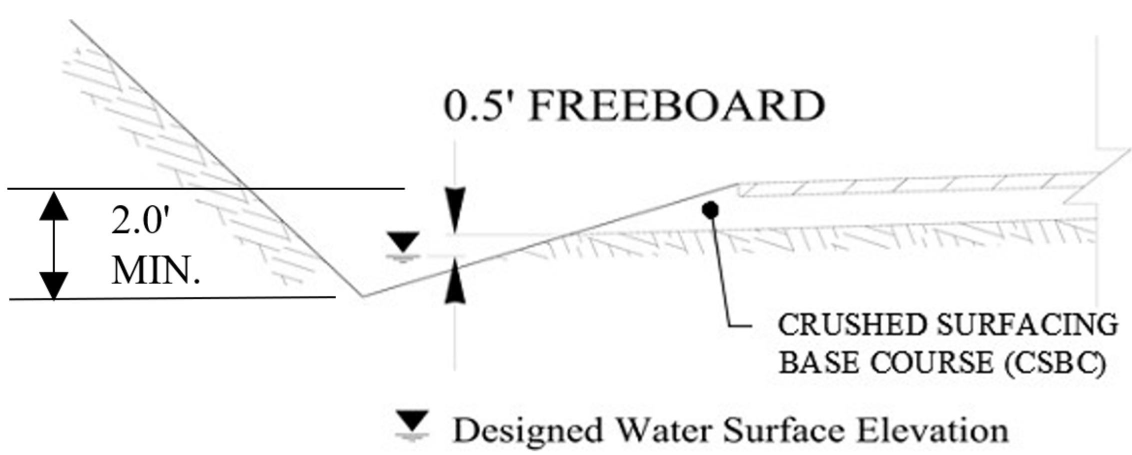
The preferred cross section of a ditch is trapezoidal; however, a "V" ditch that meets the design requirements can also be used where ROW is limited. In those cases where the grade is flat, preventing adequate freeboard, the depth of channel should still be sufficient to remove the water without saturating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

If the freeboard is less than 0.50 foot, a deviation is required, unless there is a strong justification by the designer for 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.

The WSDOT *Design Manual* also requires a 2-foot minimum ditch depth from the edge of pavement to the bottom of the ditch. If the depth is less than 2 feet, justification shall be included in the project design documentation. A *Design Manual* deviation will not be required as long the *Hydraulics Manual* requirement is met.

Figure 4-7 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 so 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.

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

Critical flow tends to occur when passing through an excessive contraction, either vertical or horizontal, before the water is discharged into an area where the flow is not restricted. A characteristic of critical depth flow is often a series of surface undulations over a very short stretch of channel. The 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 should 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 4-2 presents Manning's roughness coefficients for stream channels. Table 4-3 presents Manning's roughness coefficients for highway channels and swales with maintained vegetation.

Table 4-1 References for Manning's Roughness Coefficients

Category of Surface	Surfaces Included	Source
Open channel and pipe	Closed conduits pipes Pavement gutter Man-made channels	HEC-22
River, stream, and culvert design for aquatic organism passage	Rigid channel Minor streams Floodplains Major streams Alluvial beds Sand beds Gravel beds Cohesive soils Composite roughness value	HDS-6 HEC-26 (when required for aquatic organism passage) HEC-22 Chow V.T. 1959[a]
Channel lining	Rigid channel Unlined channel Grass Gravel Riprap Gabion	HEC-15
Storm sewer conduit[b]	Concrete pipe Metal pipe Polyethylene pipe PVC pipe	HEC-22
Street and gutter	Concrete gutter Asphalt Concrete pavement	HEC-22
Maintained vegetation	Grass	HEC-15 Chow V.T. 1959[c]

Notes:

a. See Table 4-2 below.
b. For storm sewer pipes 24 inches or less in diameter, use n = 0.013.
c. See Table 4-3 below.

Table 4-2 Manning's Roughness Coefficients for Stream Channels

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

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

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

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

Note:
Values shown are for velocities of 2 and 6 ft/s.

4-6 Riverbank Stabilization

Because of the abundance of watercourses in Washington State, and the legacy of highway placement along and across their corridors, stabilization of part of the river cross section or alignment is often necessary to 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.

4-6.1 *Streambank Protection*

Extensive guidance exists for numerous techniques for bank protection, from riprap to revegetation. Many techniques recommended in Pacific Northwest rivers incorporate large woody material (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 *Riprap for Bank Stabilization*

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

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

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

4-6.2.1 Riprap Sizing for Bank Protection

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

Once the hydraulic designer has completed the analysis in this section, the hydraulic designer should consider the certainty of the velocity value used to size riprap along with the importance of the facility. Alternatively, the riprap sizing for roadside channel bank protection can be determined using the procedure outlined in HEC-15. For additional guidance on riprap design, HEC-23, Volume 1 and Volume 2. Manning's formula or computer programs compute the hydraulic capacity of a riprap-lined channel. The appropriate n-values are shown in Table 4-4.

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

Type of Rock Lining[a]		n (Small Channels)[b]	n (Large Channels)
Spalls	$D_{50} \leq 0.5$ ft	0.035	0.030
RESP Class A	0.5 ft $> D_{50} < 1.0$ ft	0.040	0.035
RESP Class B	1.0 ft $\geq D_{50} < 1.8$ ft	0.042	0.037
RESP Class C	1.8 ft $\geq D_{50} < 2.3$ ft	0.045	0.040

Notes:

a. See the *Standard Specifications*.

b. Small channels can be loosely defined as less than 1,500 cfs.

Using Manning's equation, the hydraulic designer can determine the slope, depth of flow, and side slopes of the channel required to carry the design flow. The PEO, using this information, can then determine the required minimum D_{50} stone size with Equation 4-3.

$$D_{50}=C_R\, d\, S_o \qquad (4\text{-}3)$$

where:

D_{50} = particle size of gradation, ft, of which 50% by weight of the mixture is finer

C_R = riprap coefficient (see Table 4-5)

D = depth of flow in channel, ft

S_o = longitudinal slope of channel, ft/ft

B = bottom width of trapezoidal channel, ft (see Table 4-5)

Table 4-5 Riprap Coefficients

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

Note:

Angular rock should be used for new bank protection as it is better at interlocking and providing a stable slope. Rounded rock is unstable and shall not be used for new bank protection. The coefficients have been provided only to verify if native material is a sufficient size to resist erosion. Rounded and sub-angular rock use in new designs should be limited to streambeds and banks.

4-6.2.1.1 Example 1: Riprap Sizing for Bank Protection

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

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

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

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

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

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

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

From Table 4-4, "spalls" would provide adequate protection for a D_{50} of 0.5 foot or less in this channel. If the present streambed has rock that exceeds the calculated D_{50}, then select the appropriate type of rock lining per Table 4-4 to match the existing rocks in the streambed.

4-6.2.1.2 Example 2: Riprap Sizing for Bank Protection

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

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

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

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

$$\mathrm{C_R = 14.5}$$

$$\mathrm{D_{50} = 14.5\ (5.75\ ft)\ (0.01) = 0.8\ 3ft}$$

In this case, from Table 4-4, RESP Class A would be appropriate. Because the roughness coefficient noted in Table 4-4 for RESP Class A is n = 0.040, the PEO may recalculate the depth and velocity to get a more exact answer but this would only change the normal depth slightly and would not affect the choice of bank protection. In some cases, on very high-velocity rivers or rivers that can transport large rocks downstream, even

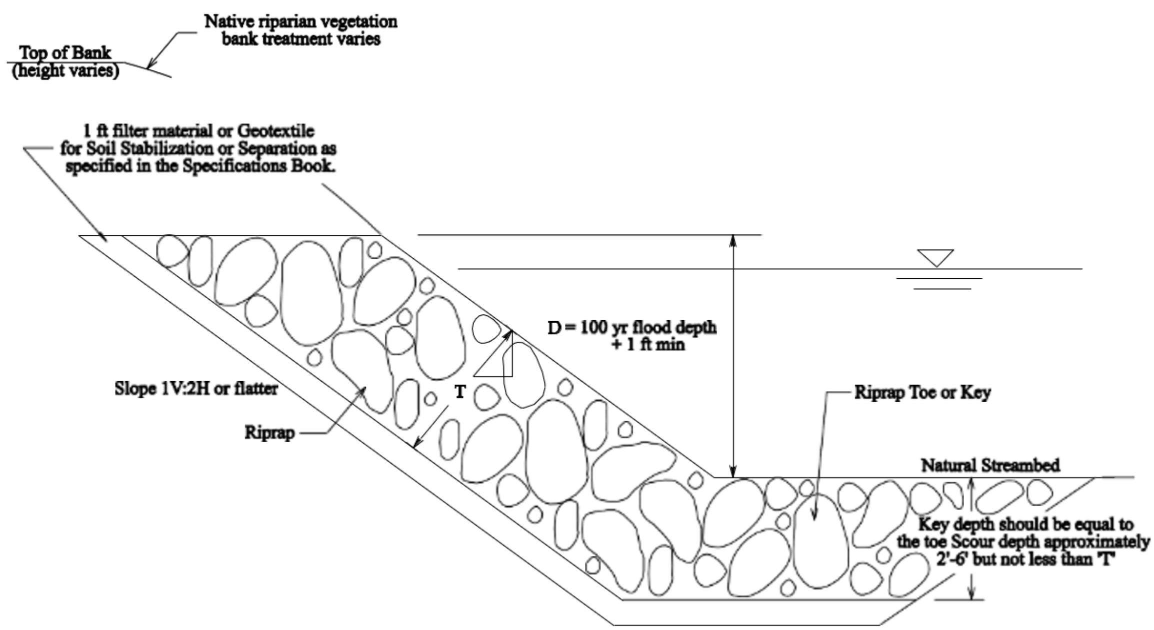
RESP Class C may not be adequate to control erosion and specially sized riprap may need to be specified in the contract. The RHE, State Hydraulics Office, and HQ Materials Laboratory are available for assistance in writing a complete specification for special riprap.

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

4-6.2.2 Placement of Riprap Bank Protection

Once the type of riprap has been selected from Table 4-4, the next step is to determine the appropriate installation. Several factors affect the placement of riprap including the type of filter material best suited for the project site, the thickness of riprap placement, and the depth to key riprap to prevent undermining. Figure 4-8 illustrates a typical cross section of a riprap bank protection installation.

Figure 4-8 Typical Cross Section of Riprap Bank Protection Installation



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

The proper selection of a filter material is critical to the stability of the original bank material in that it aids in preventing scour or sloughing. Prior to selecting a filter material, the hydraulic designer should first consult with the Region Materials Engineer and the

RHE to determine if there is a preference. In areas of highly erodible soil (fine, clay-like soils), the State Hydraulics Office should be consulted, and an additional layer of sand may be required. For additional guidance selecting the appropriate filter material, see HEC-15.

The thickness of riprap placed (shown as T in Figure 4-8) depends on which type of riprap was selected: quarry spalls, light loose riprap, or heavy loose riprap. Riprap thickness is 2.0 feet for RESP Class A, 2.5 feet for RESP Class B, 3.0 feet for RESP Class C, and 1.0 foot for quarry spalls. Care should be taken during construction to ensure that the range of riprap sizes, within each group, is evenly distributed to keep the riprap stable. Riprap is usually extended to 1 foot above the 100-year flood depth of the water as shown in Figure 4-8. However, if severe wave action is anticipated, it should extend farther up the bank.

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

4-6.3 *Channel Stabilization*

Channel stabilization, as opposed to bank stabilization, involves controlling and maintaining the channel cross section, alignment, and gradient, for some given length of the stream. There can be several reasons to stabilize a channel. At WSDOT, it is often to protect transportation infrastructure such as a culvert or roadway embankment. Some channel stabilization may also be used for fish habitat or passage. The major types of channel stabilization are concrete or rock linings, weirs, dams, and grade-control structures. see Chapter 7 and Chapter 10 for more details.

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

Because this topic is so broad and because there is existing guidance, we refer designers to the following references for details:

- HEC-23, Volume 1 and Volume 2
- *Integrated Streambank Protection Guidelines* (ISPG; WDFW 2002)