Chapter 8

CHANNELS

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8.1   Introduction

Open channels, whether natural or man-made, must have a free surface that is exposed to atmospheric pressure. For any closed conduit (pipe, box, or arch) to operate as an open channel, there must be an air space between the water surface and the inside top of the conduit. In other words, a closed conduit operating as an open channel can only flow partially full. Open channel hydraulics are always used to design these highway facilities:

- pavement gutters,
- roadside ditches,
- water quality swales,
- stream or other channel changes and reconstruction, and
- bridges.

The following facilities function under either pressure or open channel flow. Open channel flow principles are often used to evaluate the hydraulic performance of:

- storm drainage systems, and
- culverts.

Open channels are of two types, natural or artificial. Natural channels are:

- usually stream channels with their size and shape determined by natural forces,
- typically compound in cross-section with a main channel for conveying low flows and a floodplain to transport flood flows, and
- usually shaped geomorphologically by the long term effects of the sediment load and water discharge which they convey.

Artificial channels include roadside channels and ditches, irrigation canals and ditches, drainage canals and ditches, water quality swales, and closed conduits. Artificial channels are:

- constructed channels with regular geometric cross-sections, and
- unlined, or lined with artificial or natural material to protect against erosion.

8.2   Policy and Practice

General policies of the Federal government and ODOT pertaining to hydraulics are discussed in **Chapter 3**. Agency practices specific to channels include:

- coordination with other Federal, State, and local agencies concerned with water resources

- planning and management will have high priority in the design of channels,
- safety of the general public is an important consideration in the selection of the cross-sectional geometry of artificial drainage channels,
- the design of artificial drainage channels or other facilities shall consider the frequency and type of maintenance expected and make allowance for access of maintenance equipment and personnel,
- a stable channel is the goal for all channels that are located on highway right-of-way or that impact highway facilities,
- environmental impacts of channel modifications, including disturbance of fish habitat, wetlands, and riparian areas must be minimized as much as practicable, and
- the channel design shall be checked using a range of discharges based on class of roadway, consequences of traffic interruption, flood hazard risks, economics, potential damage to private property, and local site conditions.

Exceptions to design practices should be approved by the ODOT Geo-Environmental Section's Engineering and Asset Management Unit.

8.3   Design Criteria

Policies and practice define a definite course of action to guide and determine present and future decisions. Design criteria are developed to implement the specific policies and practice. Design criteria for channels follow.

8.3.1   Large Natural and Artificial Channels

The following criteria apply to natural and artificial channels designed to convey more than 50 cubic feet per second. They apply to both new channels and revisions to existing channels.

- Channels conveying stormwater runoff are designed and checked using the discharge recurrence intervals listed in **Chapter 3**. Higher design or check discharges may be justified for critical installations where considerable danger to the public or expense could occur as a result of a failure.
- Channels conveying regulated flow are designed to convey the operating discharges. These discharges are provided by irrigation districts, the U.S. Bureau of Reclamation, etc. These channels are checked using the discharge recurrence intervals listed in **Chapter 3** if they convey stormwater runoff.
- Channels are evaluated for bankfull flow if it can occur.
- The cross-sectional shape, meander, pattern, roughness, sediment transport, and slope of a relocated channel should mimic to the existing conditions insofar as practicable. Some means of energy dissipation may be necessary when existing conditions cannot be duplicated.

- Bank stabilization should be provided, where appropriate, as a result of any channel encroachment or realignment. This may include both the upstream and downstream banks as well as the local site.
- Features such as dikes and levees associated with channel modifications should have a 16-foot minimum top width to allow access for maintenance equipment. Turnaround points should be provided no further than 900 feet apart and at the end of any such feature. Concurrence on the minimum top width and turnaround point spacing should be obtained from the jurisdiction responsible for maintaining the dikes and levees.
- Dikes and levees should have a minimum of 1 foot, or two velocity heads, of freeboard above the design flow elevations.
- Levee, dike, or roadway embankment side slopes acting as channel banks should not have side slopes in excess of the angle of repose of the soil and/or lining. Side slopes should be 1V:1-1/2H or flatter in the case of loose riprap lining.
- The effects of wave action, increased flow velocities around bends, etc. should be considered in the channel design, if applicable.

Regulatory agency standards and criteria may also apply to the channel, and the design should satisfy these requirements. Irrigation ditch and canal crossings are almost always subject to U.S. Bureau of Reclamation or irrigation district standards and criteria. Additional guidance on large channel design is included in the remainder of this chapter and **Chapter 15**, Bank Protection.

8.3.2 Small Natural and Artificial Channels

Small natural and artificial channels are designed to convey less than 50 cubic feet per second. This includes roadside and most drainage ditches, water quality swales, channel reconstruction upstream and downstream from small culverts and bridges, and gutters. Gutters are the smallest artificial channels. Most gutters are designed to convey less than 2 cubic feet per second.

Small natural channels should match, as much a practicable, the natural channel curvature, slope, hydraulic roughness, and cross-section. This is not required for small artificial channels if they are properly protected from erosion or scour. The channel should be stable, both in vertical and horizontal alignment. Biological elements should be incorporated where possible to enhance habitat functions. The biological elements may also be used to provide channel stability if they will provide adequate performance and durability.

Gutters and roadside ditches are almost always within the clear zone alongside the highway. Ditches within the clear zone typically are shallow with relatively flat side slopes. These facilities are designed using criteria based on traffic safety as well as hydraulic characteristics. These facilities are designed to standards and criteria in the current ODOT "Highway Design Manual", the most recent ODOT "Standard Drawings," guidance in this chapter, and criteria in **Chapter 13**, Storm Drainage.

Roadside ditches, drainage ditches, and most small channels outside of the clear zone are usually designed with deeper sections and steeper side slopes than facilities in the clear zone. Hydraulic characteristics often govern the design of these ditches. Design criteria for smaller artificial channels are listed below. Additional guidelines for small channel design are in the remainder of this chapter.

- Roadside ditches and permanent ditch linings should be designed using the design discharge recurrence intervals listed in **Chapter 3**.
- Temporary ditch linings should be designed for the 2-year flow.
- If a ditch is relocated or realigned, the cross-sectional shape, roughness, and slope should conform to the existing conditions insofar as practicable.
- Side slopes of ditches outside of the clear zone should not exceed the angle of repose of the soil and/or lining. Side slopes should be 1V:1-1/2H or flatter in the case of loose riprap lining.
- Flexible linings should be designed according to the method of permissible unit shear stress.
- The design flow should be confined within the banks of the ditch. In cases where there are no defined ditch banks, such as flow alongside the toe of an embankment fill, the design flow should be confined to ODOT right-of-way.

Water quality swales are designed to the criteria and guidelines in **Chapter 14**, Water Quality, supplemented by the guidelines in this chapter.

8.4   Documentation - Channel Design Studies

The results from typical ODOT channel studies are summarized in this subsection. Documentation may need to be sealed by a professional engineer registered in Oregon, as discussed in **Chapter 3**.

8.4.1   Documentation – Large Natural and Artificial Channels

The Preliminary Hydraulics Recommendations, Hydraulics Reports, and distribution list for large channel studies generally follow the bridge and large culvert documentation guidelines in **Chapter 4**. The difference is, the recommendations and report addresses a large channel instead of a bridge or large culvert. Specific information related to channels includes the following:

- Hydrologic and hydraulic data for design, and check floods if the channel conveys stormwater, as listed in **Chapter 3**.
- Hydrologic and hydraulic data for the operating flow, if applicable.
- Hydrologic and hydraulic data for the bankfull flow, if it occurs.
- A summary of the design standards from other agencies which influence the design, and documentation showing the proposed design complies with the standards.

- Cross-sections of the new, existing, or realigned channels, showing dimensions, Manning's n values, and flow depths.
- Channel bottom profiles and water surface profiles of the new, existing, or realigned channels.
- Hydraulic data tables for the new, existing, or realigned channels listing flows, water depths, water surface elevations at critical locations, and velocities. Sample hydraulic data tables for bridges and culverts are shown in **Chapter 4**. A similar data table is recommended for large channels.
- A description of recommended highway embankment and channel erosion protection.
- Recommendations about maintenance and access for maintenance.
- The printout from a step-backwater analysis, showing velocity and flow depth, energy, and channel bottom shear at sections of both the modified channel and the adjacent natural channels both upstream and downstream.

8.4.2   Documentation - Small Natural and Artificial Channels

The documentation and supporting data guidelines for small culverts in **Chapter 4** are recommended as a model for small channel studies. The small channel study will address a small channel instead of a small culvert. Specific information related to small channels includes the following.

- Applicable hydrology.
- Manning's "n" value used in design.
- Design and check flow depths and velocities.
- A description or drawing of the channel, including location, critical elevations, slope, bottom width, depth, side slopes, and recommended lining.

8.5   Definitions

The Glossary in this manual provides definitions of many terms used in open channel hydraulics.

8.6   Equations

Analysis of open channel flow in both natural and artificial channels uses the basic principles of fluid mechanics -- energy, continuity, and momentum. These principles are described by mathematical relationships. This section discusses the equations expressing these relationships.

8.6.1   Energy Equations

Energy relationships in open channel hydraulics are often described using the term "head." Head

represents energy per unit weight of fluid which has dimensions of length and is therefore a linear measurement. Commonly used heads are defined in the Glossary.

Energy Equation - The energy equation expresses conservation of energy in open channel flow. The energy equation should only be applied between two cross-sections at which the streamlines are nearly straight and parallel so that vertical accelerations can be neglected. Written between an upstream open channel Cross-section 1 and a downstream Cross-section 2, the energy equation is:

$$\mathbf{h}_1 + \alpha_1 \left(\frac{\mathbf{V}_1^2}{2\mathbf{g}} \right) = \mathbf{h}_2 + \alpha_2 \left(\frac{\mathbf{V}_2^2}{2\mathbf{g}} \right) + \mathbf{h}_L \qquad \text{(Equation 8-1)}$$

Where:

h_1 and h_2 are the upstream and downstream stages, respectively in feet

α = velocity distribution coefficient

V = mean velocity in feet per second

g = gravitational acceleration, 32.2 feet per second squared

h_L = energy head loss between Cross-sections 1 and 2 due to minor losses and/or friction loss, in feet

The terms in the energy equation are illustrated graphically in Figure 8-1. The energy equation states that the total energy head at the upstream cross-section is equal to the total energy head at the downstream cross-section plus the intervening energy head loss. The terms used in the energy equation are as follows:

Stage - The stage (h) is the sum of the elevation head (z) at the channel bottom and the pressure head, y.

$\qquad h = z + y$ (Equation 8-2)

Velocity Head - The velocity head is calculated as follows:

$$h_{velocity} = \alpha \frac{V^2}{2g}$$ (Equation 8-3)

Where:

$h_{velocity}$ = velocity head in feet

α = velocity distribution coefficient (see Equation 8-5)

V   = mean velocity in feet per second

G   = gravitational acceleration, 32.2 feet per second squared

Figure 8-1 Terms in the Energy Equation
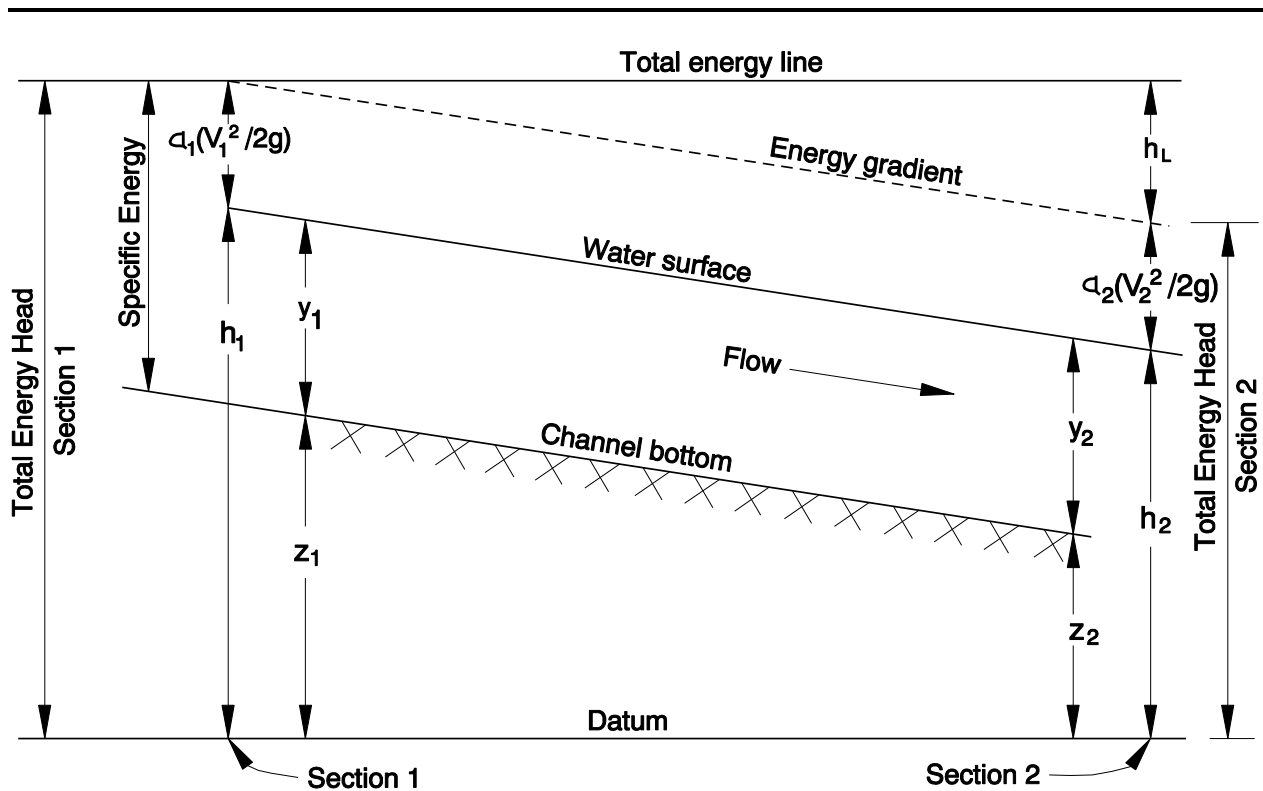
α   = Velocity distribution coefficient
g   = Gravitational acceleration
h   = Stage elevation above datum
h_L = Energy head loss
y   = Flow depth, typically called pressure head
z   = Channel bottom elevation above
      datum, typically called elevation head

Specific Energy - Specific energy (E) is typically expressed as a linear measurement relative to the channel bottom called specific energy head. If the channel is not too steep (slope less than 10 percent) and the streamlines are nearly straight and parallel, E becomes the sum of the pressure and velocity heads:

$$E = y + h_{velocity}$$ (Equation 8-4)

Where:

        E = specific energy head in feet
        y = pressure head in feet (this is flow depth in an open channel)
        $h_{velocity}$ = velocity head (see Equation 8-3)

Velocity Distribution Coefficient - Velocities are not uniformly distributed in a channel section because the top of the flow is bounded by a free surface causing negligible friction drag, and the wetted perimeter is bounded by the channel sides or the overbanks which may impart considerable friction drag. Velocity distributions for typical cross-section shapes are shown in the popular textbook by Ven Te Chow titled "Open-Channel Hydraulics."

As a result of this nonuniform velocity distribution, the actual velocity head of an open channel is usually greater than the average velocity head computed as $(Q / A_t)^2 / 2g$ where A_t is total waterway cross-section area. The average velocity head calculated by the preceding equation is multiplied by the velocity distribution coefficient (α) to more accurately estimate the actual velocity head.

Velocity distribution coefficients should be considered in the water surface profile calculations when they significantly affect the velocity head. This most often occurs in waterways having hydraulically efficient main channels and relatively constricted overbank areas. In addition, this coefficient should be considered when the water surface profile is to be calculated with a high degree of accuracy. Almost all modern step-backwater analysis programs calculate and use this coefficient.

Velocity distribution coefficients are often ignored or considered to be 1.0 when analyzing channels without overbank areas, such as conduits, ditches, canals, etc. They are also ignored in applications where accurate hydraulic estimates are not necessary.

The velocity distribution coefficient is also useful when determining the highest probable velocity of waterborne debris or ice. The velocity of these materials in the center of the main channel may be significantly greater than the average velocity in the cross-section. These higher velocities can be estimated by multiplying the average velocity in the section by the velocity distribution coefficient. Conversely, the velocity distribution coefficient can also be used to estimate the slower velocities debris or ice will have in less hydraulically efficient overbank areas.

There are several methods to determine the velocity distribution coefficient. The coefficient for natural channels can be calculated using the method in the US Corps of Engineers HEC-RAS step

backwater program. It is based on the conveyance in three flow elements: left overbank, right overbank, and channel. It can be written in terms of conveyance and area as follows:

The velocity coefficient (α) is computed based on the conveyance in the three flow elements: left overbank, right overbank, and channel. It can also be written in terms of conveyance and area as in the following equation:

$$a = \frac{(A_1)\left[\dfrac{K_{lob}^3}{A_{lob}^2} + \dfrac{K_{ch}^3}{A_{ch}^2} + \dfrac{K_{rob}^3}{A_{rob}^2}\right]}{K_1^3} \qquad \text{(Equation 8-5)}$$

Where:

A_1	= total flow areas of cross-section in square feet
A_{lob}, A_{ch}, A_{rob}	= flow areas of left overbank, main channel, and right overbank, respectively, in square feet
K_1	= total conveyance of cross-section in cubic feet per second
K_{lob}, K_{ch}, K_{rob}	= conveyance of left overbank, main channel, and right overbank, respectively, in cubic feet per second

Calculating the velocity distribution coefficient using Equation 8-5 can be time consuming if done by hand. An approximate coefficient based on typical velocity distributions is adequate for most hand calculation purposes. Typical velocity distribution coefficients, based on the January 1956 publication by Steponas Kolupaila "Methods of determination of the kinetic energy factor" in The Port Engineer, Calcutta, India, Volume 5, Number 1, are:

- regular channels, flumes, spillways: 1.10 minimum, 1.15 average, 1.20 maximum,
- natural streams and torrents: 1.15 minimum, 1.30 average, 1.50 maximum,
- rivers under ice cover: 1.20 minimum, 1.50 average, 2.00 maximum, and
- river valleys, overflooded: 1.50 minimum, 1.75 average, 2.00 maximum.

Minor Losses - Flow contraction or expansion between cross-sections results in energy losses. These losses are often called "minor losses" and they are a function of the difference in velocity heads between upstream Cross-Section 1 and downstream Cross-Section 2, as follows:

$$h_L = C_c \Delta h_{velocity} = C_c \left[\left(\frac{\alpha_2 V_2^2}{2g}\right) - \left(\frac{\alpha_1 V_1^2}{2g}\right)\right] \qquad \text{(Contraction head loss, Equation 8-6)}$$

$$h_L = C_e \Delta h_{velocity} = C_e \left[\left(\frac{\alpha_1 V_1^2}{2g}\right) - \left(\frac{\alpha_2 V_2^2}{2g}\right)\right] \qquad \text{(Contraction head loss, Equation 8-7)}$$

Where:

h_L = minor loss between bounding cross-sections in feet

$\Delta h_{velocity}$ = change in velocity heads between bounding cross-sections

Contraction and expansion coefficients are as follows;

- No transition: $C_c = 0.0$ and $C_e = 0.0$
- Gradual transition: $C_c = 0.1$ and $C_e = 0.3$
- Typical transition at bridge: $C_c = 0.3$ and $C_e = 0.5$
- Abrupt transition: $C_c = 0.6$ and $C_e = 0.8$

Note: Contraction and expansion losses are always zero or a positive value.

Friction Loss - Friction between the flow and the wetted perimeter causes friction loss. Between two cross-sections, the friction loss is calculated as follows:

$$h_F = LS_f \qquad \text{(Equation 8-8)}$$

Where:

h_F = friction loss between bounding cross-sections in feet

L = reach length in feet. Sometimes weighted reach lengths are used (see Equations 8-22 and 8-23).

S_f = friction slope (see Equations 8-9 or 8-10)

Note: Friction loss is always a positive value.

Friction Slope - The friction slope is also the energy grade line slope. It is expressed as the friction loss per unit length of channel, and it can be calculated by several methods. Two commonly used methods are:

- Average conveyance equation

$$S_f = \left(\frac{Q_u + Q_d}{K_u + K_d} \right)^2 \qquad \text{(Equation 8 - 9)}$$

- Geometric mean friction slope equation

$$S_f = (S_{fu}S_{fd})^{1/2} \qquad \text{(Equation 8-10)}$$

Where:

S_f = friction slope is feet per foot

Q_u, Q_d = discharge at the upstream and downstream cross-sections, respectively, in cubic feet per second

K_u, K_d = conveyance at the upstream and downstream cross-sections, respectively, in cubic

feet per second

S_{fu}, S_{fd} = friction slope at the upstream and downstream cross-sections, respectively, in foot per foot

Note: The friction slope is equal to the bed slope during relatively uniform flow in nearly uniform channels where expansion and contraction losses are negligible.

8.6.2 Continuity Equation

The concept of continuity results from the principle of conservation of mass. The mass of fluid is the same passing through all sections of a stream, per unit of time, if the rate of flow is constant.

Continuity Equation - The continuity relationship is expressed by the continuity equation as follows:

$$Q = A_1 V_1 = A_2 V_2 \qquad \text{(Equation 8-11)}$$

Where:

    Q = discharge in cubic feet per second
    A = cross-sectional area of flow in square feet
    V = mean cross-sectional velocity in feet per second
    Subscripts 1 and 2 refer to successive cross-sections along the flow path.

8.6.3 Manning's Equations

Normal flow occurs in an open channel when the energy gradient, water surface, and bed are parallel. In nature, normal flow typically occurs in long prismatic channels such as canals. Normal flow rarely occurs in natural channels. In the analysis of natural waterways, however, it is assumed that normal flow occurs in the more uniform reaches of the channel. As a result, normal flow equations such as the Manning's formula are used extensively in hydraulic modeling.

Manning's Equation - The mean velocity, V, can be computed with Manning's equation for a given depth of flow in an open channel with a steady, uniform flow:

$$V = \left(\frac{1.486}{n}\right) R^{2/3} S^{1/2} \qquad \text{(Equation 8-12)}$$

Where:

V	= mean velocity in feet per second
n	= Manning's roughness coefficient
R	= hydraulic radius = A / P, in feet
A	= cross-sectional area in square feet
P	= wetted perimeter in feet

S   = slope of the energy gradeline in feet per feet

Note: For steady uniform flow, S = channel slope

The selection of Manning's n is generally based on observation; however, considerable experience is essential in selecting appropriate n values. Selecting Manning's n values for natural channels is discussed in Subsections 8.10.2 and 8.10.4. Ranges of n values for various channels and floodplains are listed in Appendix A. Manning's n can also be calculated for normal flow if the water surface elevation, and either flow or velocity are known, using the following equations:

$$n = \frac{1.486\,(R^{2/3}\,S^{1/2})}{V}$$, if velocity is known   (Equation 8-13)

$$n = \frac{1.486\,(A\,R^{2/3}\,S^{1/2})}{Q}$$,if flow is known   (Equation 8-14)

Manning's Equation for Discharge - The continuity equation can be combined with Manning's equation to obtain Manning's equation for discharge:

$$Q = \left(\frac{1.486}{n}\right) A\,R^{2/3}\,S^{1/2}$$ (Equation 8-15)

Where:

        Q = discharge in cubic feet per second

A unique flow depth occurs during steady, uniform flow for a given channel geometry, slope, roughness, and discharge Q. It is called normal depth, and it can be calculated for natural stream channels and many artificial channel shapes using the procedures in the remainder of this chapter.

Conveyance Equation - The first terms of Equation 8-15 can be combined to determine the channel conveyance, K. The following equation is used:

$$K = \left(\frac{1.486}{n}\right) A\,R^{2/3}$$ (Equation 8-16)

Where:

        K = conveyance in cubic feet per second

Equation 8-15 can then be written as:

$$Q = K\,S^{1/2}$$ (Equation 8-17)

The conveyance represents the carrying capacity of a stream cross-section based upon its geometry

and roughness characteristics alone, and it is independent of the streambed slope. The concept of channel conveyance is useful when computing the distribution of overbank flood flows in the stream cross-section and the flow distribution through the opening in a proposed stream crossing. It is also used to determine the velocity distribution coefficient, α (see Equation 8-5).

8.7   Supercritical, Subcritical, and Critical Flows

Open channel flow can be classed as supercritical, subcritical, or critical. Supercritical flows:

- are influenced mainly by inertial forces, and are often described as torrents or rapids,
- have flow depths less than critical depth,
- occur on slopes steeper than the critical slope,
- have Froude numbers greater than one, and
- small water surface disturbances are always swept downstream in supercritical flow, and the flow characteristics at a cross-section are controlled by flow characteristics at upstream cross-sections.

Subcritical flows:

- are mainly influenced by gravity forces, and are often called tranquil,
- have flow depths greater than critical depth,
- occur on slopes flatter than the critical slope,
- have Froude numbers less than one, and
- small water surface disturbances such as waves can travel both upstream and downstream, and flow characteristics at a cross-section are controlled by flow characteristics at downstream cross-sections.

Critical flows:

- are an unstable condition rarely encountered in nature,
- flow is almost always in either the super or subcritical regime, and it passes through critical flow when it makes a transition from one regime to the other.

Relative to subcritical flows, supercritical flow occurs on steeper slopes, has shallower flow depths, and faster velocities.

8.7.1   Froude Number

The Froude number (F_r) is an important dimensionless parameter in open channel flow. It represents the ratio of inertial forces to gravitational forces. This expression for Froude number applies to channels of any cross-sectional shape.

$$F_r = \frac{V}{\left(\dfrac{gd}{\alpha}\right)^{0.5}}$$

(Equation 8-18)

Where:

α = velocity distribution coefficient (see Equation 8-5)
V = mean velocity = Q / A, in feet per second
g = acceleration of gravity (32.2 feet per second squared)
d = hydraulic depth = A / T, in feet
A = cross-sectional area of flow in square feet
T = channel top width at the water surface in feet
Q = total discharge in cubic feet per second

Note: The hydraulic depth is equal to the flow depth for rectangular channels.

8.7.2   Critical Depth

At a constant discharge, flows at different depths have different specific energy heads. The minimum specific energy head occurs at a depth called critical depth at which the Froude number is one. Critical depth is also the depth of maximum discharge when the specific energy head is held constant. When flow is at critical depth, the velocity head is equal to half the hydraulic depth. The general expression for flow at critical depth is:

$$\frac{\alpha Q^2}{g} = \frac{A^3}{T}$$

(Equation 8-19)

Where:

α = velocity distribution coefficient (see Equation 8-5)
Q = total discharge in cubic feet per second
g = gravitational acceleration, 32.2 feet per second squared
A = cross-sectional area of flow in square feet
T = channel top width at the water surface in feet

Equation 8-19 must be satisfied, no matter what the shape of the channel, when flow is at critical depth. The slope is classified as a mild slope if the normal depth is greater than critical depth, and the slope is called a steep slope if normal depth is less than critical depth. Thus, uniform flow is subcritical on a mild slope and supercritical on a steep slope.

Critical depths for closed conduits and many artificial channel shapes are discussed Subsection 8.14.6. Critical depth charts for common prismatic shapes are in Appendix B.

8.8   Transition Sections and Rapidly Varied Flow

Transition sections occur when there are changes in discharge, roughness, cross-section, or slope and the flow regime does not change as water passes through the transition section. In other words, there is subcritical flow upstream, through, and downstream of the transition. Transition sections with supercritical flow are similar. There is supercritical flow downstream, through and upstream of the transition.

Rapidly varied flow occurs when there are changes in discharge, roughness, cross-section, or slope and the flow changes from one regime to another. The most common occurrences are hydraulic drops and hydraulic jumps. Hydraulic drops occur when subcritical flow turns into supercritical flow. Typical locations are the upstream edge of constrictive bridge openings, sections of channel where a flatter slope upstream breaks to a steeper slope downstream, and sections where a hydraulically rough upstream channel changes to a hydraulically smooth downstream channel.

Hydraulic jumps occur when supercritical flow turns into subcritical flow. Locations of hydraulic jumps can be at the downstream edge of constrictive bridge openings, channels where a steeper upstream section changes to a flatter downstream section, and channels where a hydraulically smooth upstream channel changes to a hydraulically rough downstream channel.

Understanding transition sections and rapidly varied flow is important in channel design because drops and jumps often cause energy loss and create turbulence. Special features may be needed in the turbulent areas to protect from scour or erosion.

Transition sections and areas of rapidly varied flow cannot be analyzed by single-section methods. Energy losses other than friction loss also need to be considered. The analysis can be done by hand methods as discussed in Section 8.16. One, two, or three dimensional computer programs can also be used.   The step-backwater programs HEC-RAS and WSPRO mentioned in the following subsection have the ability to analyze channel reaches with transition sections and rapidly varied flow. Hydraulic analysis methods are discussed in the following section.

8.9   Hydraulic Analysis Methods

The hydraulic analysis typically determines the depth and velocity that given discharge will have in a channel of known geometry, roughness, and slope. The flow depths and velocities are necessary for the design or analysis of channels, channel linings, and highway drainage structures.
Flow in open-channels is complex. The velocity vector of flow at any point in the channel typically has three components, as follows:

- a longitudinal component horizontal and parallel to the channel centerline,
- a lateral component horizontal and transverse to the channel centerline, and

- a vertical component.

Three dimensional models analyze flow velocity in all three of these directions with respect to time. These methods are highly complex and seldom used. Two dimensional models analyze flow velocity and water surface elevation in the longitudinal and lateral directions with respect to time. These models are typically used in special situations such as:

- wide floodplains with multiple openings, particularly where embankments are skewed to the flow direction,
- floodplains with significant variations in roughness or complex geometry such as ineffective flow areas, flow around islands, or multiple channels,
- sites where more accurate flow patterns and velocities are needed to design better and more cost-effective countermeasures such as riprap along embankments and/or abutments,
- tidally affected river crossings and crossings of tidal inlets, bays, and estuaries,
- high risk or sensitive locations where potential losses and liability costs are high, and
- analyzing the effects of sedimentation and scour due to channel changes.

Two-dimensional hydraulic models are complex and directions on their use are beyond the scope of this manual. It is recommended that computer programs such as US Corps of Engineers RMA-2V or the FHWA's Finite Element Surface Water Modeling System: Two Dimensional Flow in a Horizontal Plane - FESWMS-2DH (FESWMS) be used. FESWMS is recommended for crossings of rivers and floodplains it supports both super and subcritical flow analysis and can analyze weirs (roadway overtopping), culverts, and bridges.

The Surface Water Modeling System (SMS) developed by the Engineering Computer Graphics Laboratory at Brigham Young University in cooperation with the U.S. Army Corps of Engineers Waterways Experiment Station and the FHWA can be used to develop the finite element mesh and associated boundary conditions necessary for RMA-2V and FESWMS. The solution files from RMA-2V or FESWMS, which contain water surface elevation, velocity, or other functional data at each node of the mesh, can be read into SMS to generate vector plots, color-shaded contour plots, time variant curve plots, and dynamic animation sequences.

Open channel flow hydraulic analysis is simplified by assuming one-dimensional flow. Lateral and vertical velocity vector components are ignored and flow is assumed to occur only in the longitudinal direction. This assumption is the basis of many of the most frequently used hydraulic modeling methods. The two hydraulic modeling methods presented in this manual, the single-section analysis and the step-backwater method are one-dimensional models.

8.9.1   Single-Section Method

The single-section method (slope-area method) is based on the cross-sectional geometry, hydraulic roughness, and energy gradeline slope of a single waterway cross-section. This method is simply a solution of Manning's equation for the normal depth of flow given the discharge. It implicitly assumes the existence of steady, uniform flow; however, uniform flow rarely exists in either

artificial or natural channels. The results obtained by assuming steady uniform flow are understood to be approximate and general, but they provide a satisfactory solution to many practical problems. As a result, this method is often used to determine hydraulic characteristics of waterways with relatively uniform cross-section, roughness, and slope. Examples are canals, ditches, gutters, and closed-conduits such as culverts and storm drains. The single-section method can also be used to estimate highwater elevations for bridges that do not constrict the flow on streams with relatively uniform cross-section. The step-backwater method or other procedures that consider energy losses in addition to friction are preferred in situations where uniform or nearly uniform flow does not occur.

Use of the single-section method to determine a stage-discharge curve is discussed in Section 8.11. Use of the single-section method to analyze flow in artificial channels is discussed in the remainder of this chapter.

8.9.2   Step-Backwater Method

The step-backwater method is used to compute water surface profiles. It is often used in bridge hydraulic design and flood studies. The method is particularly useful to determine how far upstream or downstream water surface elevations are affected by a bridge, culvert, or other encroachment into the floodplain.

The step-backwater method is generally more accurate than the single section method. It is seldom done by a pencil-and-paper analysis because it requires extensive iterative calculations. Computer programs such as the U.S. Corps of Engineers "Hydraulic Engineering Center River Analysis System" (HEC-RAS), or the Federal Highway Administration's "WSPRO - Step Backwater and Bridge Hydraulics" program in the HYDRAIN package are often used. This chapter contains a general discussion of step-backwater modeling. Specific discussion of step-backwater models, instructions, and example problems are included in the user's manuals for the various programs.

8.9.3   Calibration

Regardless of the method, the model should be calibrated using historical highwater marks and/or gaged streamflow data to ensure that it accurately represents local conditions. The following parameters, in order of preference, may be adjusted to calibrate the hydraulic model: Manning's n, slope, discharge, and cross-section. Proper calibration is essential if accurate results are to be obtained.

It is good practice to find two or more people who have observed the site for at least twice as many years as the calibration flood recurrence interval. Typically two or more people can be found who have seen a site for at least ten years, and they can be asked about the 2 and 5-year events. The two-year event occurs one out of every two years, on the average. The five year event occurs one in five years, on the average.

Witnesses give conflicting accounts in some instances. Additional people need to be interviewed or

other information should be used to verify the hydraulic history of the site. An example of a question to these observers follows.

"My hydraulic model predicts water will just cover the top of the culvert inlet once a year during one of two years, on the average. It also predicts water will overtop the driveway at least once during one of every 5 years, on the average. Does this match the flooding you observed?"

The model would be adjusted depending on the observer statements. The model "n" value, discharge, or other characteristics could be adjusted if the predicted flooding occurs more or less often than the observed flooding.

Sometimes a gage station is on the stream and the peak discharges during notable floods can be estimated fairly accurately. Witnesses can often provide detailed accounts of water surface elevations during these floods. The hydraulic model can be adjusted so it produces a water surface elevation similar to the observed during the discharge estimated for the flood.

8.10 Hydraulic Modeling

One of the first steps in hydraulic analysis is to model the waterway. The choice of analysis method depends on the type of information needed, the importance of the hydraulic structure, and the sensitivity of the area to flooding. As examples, a roadside ditch design may only need a single-section analysis if it had a fairly uniform slope, roughness, and cross-section, a large bridge that constricts a floodplain in a rural area may justify a step-backwater flow analysis, and a large bridge that affects flood elevations in a city may justify a two-dimensional hydraulic study. Elements of typical single-section and step-backwater hydraulic models are described in this section.

8.10.1 Cross-Sections

Stream cross-sectional geometry is defined by lateral distance and ground elevation coordinates which locate individual ground points. The cross-section is taken normal to the flow direction along a single straight line where possible. It may be necessary to use a cross-section with intersecting straight lines which are perpendicular to the flow path, i.e. a "dog-leg" section, in wide floodplains or bends where flow lines are not parallel. It is especially important to plot the cross-section on paper or electronically to reveal any inconsistencies or errors.

Cross-sections should be located to be representative of the subreaches between them. Major breaks in bed profile, abrupt changes in roughness or shape, control sections such as waterfalls, bends and contractions, or other abrupt changes in channel slope or conveyance will require that cross-sections are taken at shorter intervals. Cross-sections representing changes in channel shape and roughness are shown in Figure 8-2. Additional discussion about cross-section location is in **Chapter 6**.

8.10.2 Manning's n Value Selection

Manning's n is affected by many factors and its selection in natural channels depends heavily on engineering experience. Pictures of channels and flood plains for which the discharges and channel properties have been measured and Manning's n values have been calculated are in USGS Water-Supply Paper 1849, published in 1967, by Harry H. Barnes, Jr. titled "Roughness Characteristics of Natural Channels." Additional pictures and formulae for calculating roughness values are in FHWA Report No. FHWA-TS-84-204, published in 1984, by G. J. Arcement, Jr. and V. R. Schneider, titled "Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Flood Plains." These publications are available from the FHWA website: http://www.fhwa.dot.gov/bridge/elibrary.htm Manning's n values for artificial channels are more easily defined than n values for natural channels. Appendix A lists typical n values for natural and artificial channels.

8.10.3 Cross-Section Subdivision

The cross-section may need to be divided into subsections with vertical boundaries where there are abrupt lateral changes in geometry and/or roughness. The subdivision points should be carefully chosen so that the distribution of flow or conveyance is nearly uniform within each subdivision. Dividing the cross-section will allow the conveyance of each subdivision to be calculated separately. This is necessary to calculate the velocity distribution factor (Equation 8-5) and the weighted reach lengths (Equations 8-22 and 8-23). It is also needed to prevent "switchback", as discussed in Subsection 8.10.5. The three cross-sections shown in Figure 8-2 are shown in more detail in Figure 8-3 to illustrate waterway subdivision.

Cross-Section 1 is shown in Figure 8-3a. One side of this channel is deep and has no vegetation. The other side is almost as deep and has light vegetation that does not retard flood flows. Flow velocities are expected to be fairly uniform throughout the section, and as a result, the section will not be subdivided and a single n value will be used. A Manning's n value from Appendix A could be used for this natural stream channel.

Cross-Section 2 is shown in Figure 8-3b. There are no significant changes in geometry or roughness which would cause one section of the waterway to have significantly different flow velocities than the other sections. As a result, the section will not be subdivided and a single composite Manning's n value will be calculated by method shown in the next subsection.
Cross-Section 3 is shown in Figure 8-3c. There are changes in waterway geometry and roughness in different sections of the channel that cause differences in flow velocities. Flow in the deep smooth main channel is expected to have higher velocities than flow over the wide, hydraulically rough, and flat floodplains. As a result, the cross-section is subdivided at the channel banks. It should be noted that Cross-Section 3 would also be subdivided at the channel banks due to changes in the waterway geometry even if there was no difference in the n values or velocity between the overbanks and the main channel.

Guidance on the subdivision of cross-sections, calculation of the velocity distribution coefficient, as well as many other aspects of open-channel hydraulics are described in the text by Jacob Davidian titled "Computation of Water-Surface Profiles in Open Channels." The text comprises Chapter A-15 of Book 3 of the 1984 USGS publication titled "Techniques of Water-Resources Investigations of the United States Geological Survey."
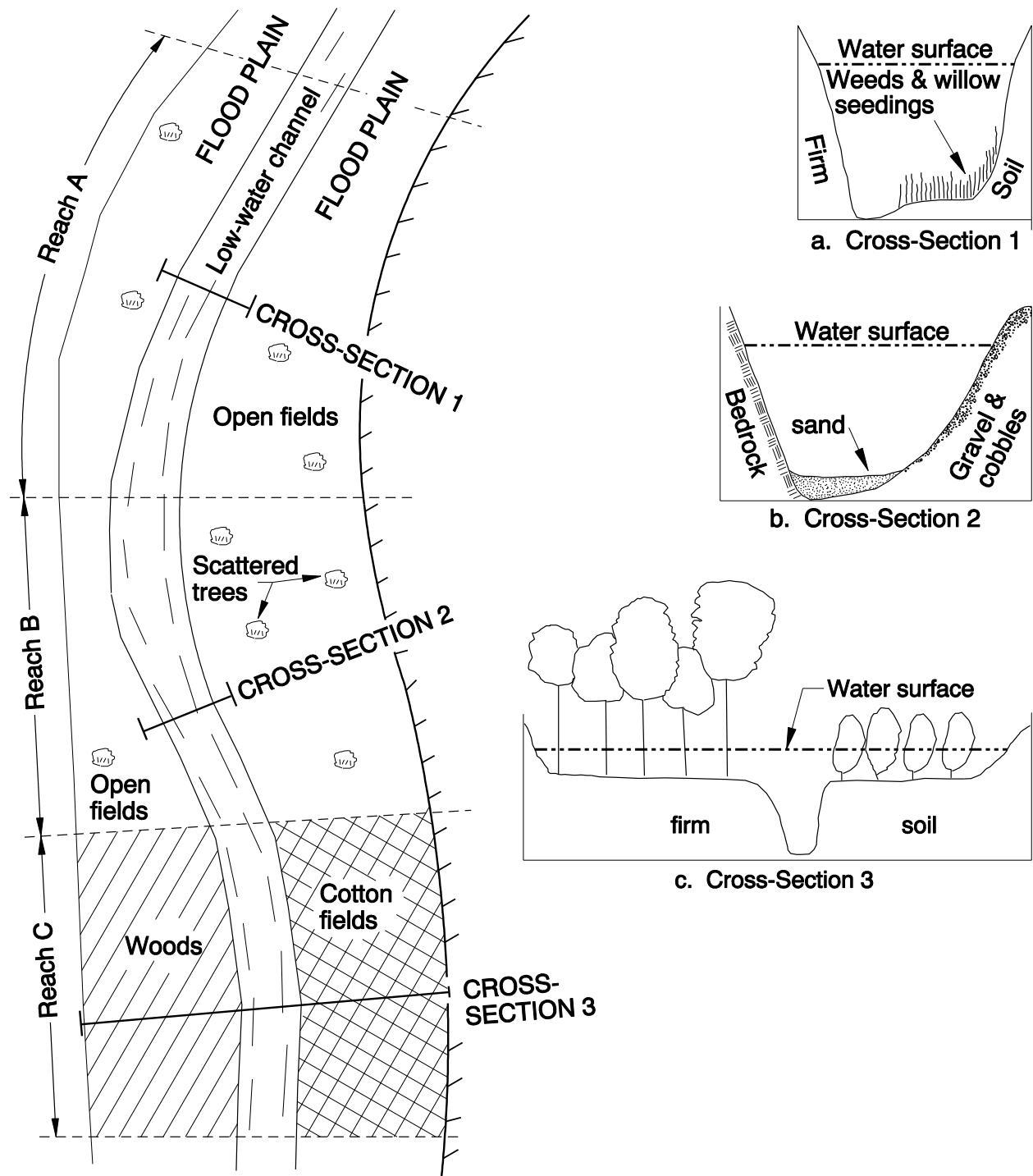
Figure 8-2   Reach and Cross-Section Location

No subdivisions
n
ft
16
Water surface
12   Weeds & willow
seedings
8
Firm   Soil
4
0
0   25   50   75   100 ft

a. Cross-Section 1

No subdivisions
n composite
n₁   n₂   n₃
ft
16
Water surface
12
8
Bedrock   sand   Gravel & cobbles
4
0
0   25   50   75 100 125 150 ft

b. Cross-Section 2

Subdivision
subsection 2
subsection 1   subsection 3
n₁   n₂   n₃
ft
30
25   Water surface
20
15
10
5   firm   soil
0
0   250   500   750   1000 ft

c. Cross-Section 3

Figure 8-3   Waterway Subdivision

8.10.4 Composite Roughness Coefficients

Occasionally, portions of the cross-section wetted perimeter will have different roughness coefficients, and the differences in roughness will not cause a significant variation in channel velocities. This often occurs in channels with the bottom and sides of different materials. In these cases, the waterway is not subdivided and a single composite roughness coefficient is calculated. The composite Manning's n value can be calculated by the following equation:

$$n_c = \left(\frac{\sum (n_i^{3/2} \, P_i)}{P} \right)^{2/3}$$ (Equation 8-20)

  i=1

Where:

    n_c = composite roughness coefficient, n
    P = wetted perimeter of entire waterway in feet
    n_i = roughness coefficient for subsection i
    P_i = wetted perimeter of subsection i in feet

Cross-Section 2 shown in Figure 8-3b is an example of this type of waterway. The channel banks are composed of sections with bedrock, gravel, and cobbles, and the channel bottom is sand. These materials have different Manning's n values n_1, n_2, and n_3 which correspond to sections of the wetted perimeter P_1, P_2, and P_3. Based on Equation 8-20, the composite Manning's n value is calculated by the following equation:

$$n_{composite} = \left[\frac{(n_1^{3/2} \, P_1) + (n_2^{3/2} \, P_2) + (n_3^{3/2} \, P_3)}{(P_1 + P_2 + P_3)} \right]^{2/3}$$ (Equation 8-21)

8.10.5 Switchback Phenomenon

A plot of the stage discharge curve may have the irregular "switchback" shape if a cross-section is improperly subdivided, as shown in Figure 8-4a. A switchback results when the calculated discharge decreases with an associated increase in elevation. This occurs when, with a minor increase in water depth, there is a large increase of wetted perimeter. Simultaneously, there is a corresponding small increase in cross-sectional area which causes a net decrease in the hydraulic radius from the value it had for a lesser water depth. A discharge is computed which is lower than the discharge based on the lower water depth due to the combination of the slightly larger cross-sectional area and the significantly lower hydraulic radius. More subdivisions within such cross-sections should be used to avoid the switchback.

This phenomenon can occur in any type of conveyance computation, including the step-backwater method. Computer logic can be seriously confused if a switchback were to occur in any cross-section being used in a step-backwater program, and it is possible an erroneous water surface profile could be calculated. For this reason, the cross-section should always be subdivided with respect to both roughness and geometric changes. Note that the actual n value, itself, may be the same in adjacent subsections, as shown in Figure 8-4b. In this figure, overbank areas are subdivided to prevent switchback.

8.10.6 Reach Lengths

Curving stream channels may have different reach lengths between cross-sections in the main channel and overbanks, as shown in Figure 8-5. To simplify the hydraulic analysis, a weighted reach length (L) is used in the hydraulic equations. Two methods of calculating L are as follows:

- Conveyance-weighted reach length, L:

$$L = \frac{\left(L_{lob}K_{lob} + L_{ch}K_{ch} + L_{rob}K_{rob}\right)}{\left(K_{lob} + K_{ch} + K_{rob}\right)}$$ (Equation 8-22)

Where:

L_{lob}, L_{ch}, L_{rob} = flow distance between sections in the left overbank, main channel, and right overbank, respectively, in feet

K_{lob}, K_{ch}, K_{rob} = conveyance in the left overbank, main channel, and right overbank, respectively, of the section with the unknown water surface elevation, in cubic feet per second

- Discharge-weighted reach length, L:

$$L = \frac{\left(L_{lob}Q_{lob} + L_{ch}Q_{ch} + L_{rob}Q_{rob}\right)}{\left(Q_{lob} + Q_{ch} + Q_{rob}\right)}$$ (Equation 8-23)

Where:

L_{lob}, L_{ch}, L_{rob} = flow distance between cross-sections in the left overbank, main channel, and right overbank, respectively, in feet

Q_{lob}, Q_{ch}, Q_{rob} = arithmetic averages of flows in the left overbanks, main channels, and right overbanks, respectively, of the bounding cross-sections in cubic feet per second. For example: $Q_{lob} = (Q_{lob\,(Section\,1)} + Q_{lob\,(Section\,2)}) / 2$

Note: An iterative solution is used. The conveyances or discharges are assumed at the unknown section, the weighted reach length is calculated based on the previous assumption, and the

Discharge

a. Switchback

n = 0.03   n = 0.03   n = 0.03
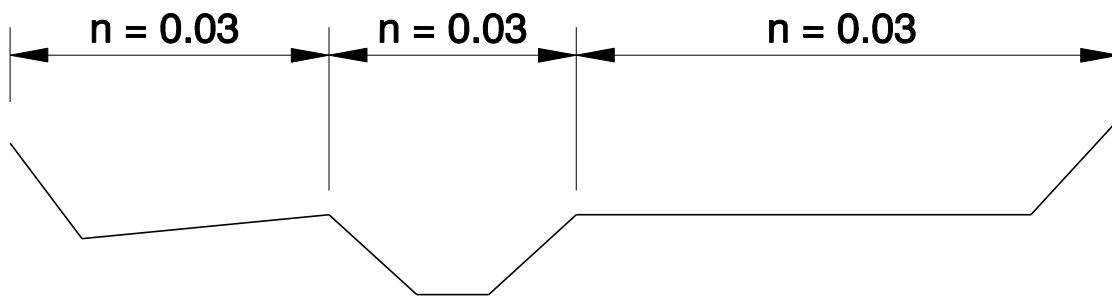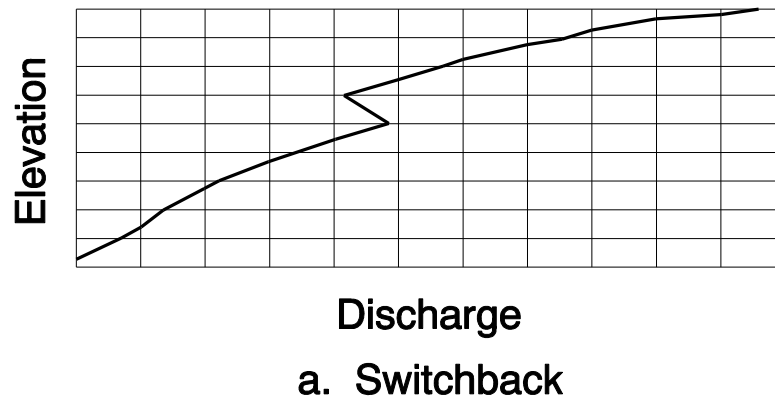
b. Cross-section subdivision to avoid swithcback

Figure 8-4   Switchback

Downstream
Cross-Section

$L_{lob} < L_{ch}$

$L_{rob} < L_{ch}$

Edge of Flow

Length of Flow on
Left Overbank, L_{lob}

Length of Flow on
Right Overbank, L_{rob}

Edge of Flow

Upstream
Cross-Section

Channel Bank

Channel Bank

Length of Flow in Channel, L_{ch}

Edge of Flow

Length of Flow on
Left Overbank, L_{lob}

Channel Bank

Length of Flow in
Channel, L_{ch}

Channel Bank

Length of Flow on
Right Overbank, L_{rob}

Edge of Flow

Downstream
Cross-Section

Upstream
Cross-Section

$L_{lob} > L_{ch} > L_{rob}$

Figure 8-5   Varying Reach Lengths Between Sections

conveyances or discharges are recalculated using the adjusted reach length. These steps are repeated until the assumed and calculated conveyances or discharges match each other. These calculations are cumbersome if done by hand. Use of a step-backwater computer program is recommended.

8.11 Stage-Discharge Curve

A stage-discharge curve is a graphical relationship of streamflow depth or elevation versus discharge at a specific cross-section on a stream. A stage-discharge curve is shown in Figure 8-8 to illustrate the example problem. The stage-discharge curve can be determined by single-section analysis if uniform or nearly uniform flow is present. The step-backwater method should be used if non-uniform or varied flow occurs. The stage-discharge curve can be determined as follows.

Step 1 - Select the typical cross-section at or near the location where the stage-discharge curve is needed. This is the only cross-section for a single-section analysis. Additional sections will be needed upstream or downstream from the location where the stage-discharge curve is needed in a step-backwater analysis, as follows:

- at least three sections are needed downstream if subcritical flow occurs, or,
- at least three sections are needed upstream if supercritical flow occurs.

Step 2 - Subdivide cross-section(s) and assign n-values to subsections as described in Subsection 8.10.3.

Step 3 - Estimate the energy grade line slope at the section for a single-section analysis, or at the starting section for a step-backwater analysis. The slope of the surveyed streambed or water surface can be used since uniform flow is assumed.

Step 4 - Select a range of incremental water surface elevations at the cross-section and calculate the discharge at each elevation using Manning's equation for discharge if a single-section analysis is made. Total discharge at each elevation is the sum of the discharges through each subsection if the cross-section is subdivided. The wetted perimeter should be measured along the solid boundary of the cross-section and not along the vertical water interface between subsections when determining hydraulic radius for subdivided cross-sections.

Calculate the water surface at the cross-section in question for several incremental discharges if a step-backwater analysis is used. Start the analysis at the furthest downstream section if there is subcritical flow. Do the opposite if there is supercritical flow.

Step 5 - Plot the stage-discharge curve after the discharge has been calculated at several incremental elevations, or the elevations have been calculated for incremental discharges. This plot can be used to determine the water surface elevation corresponding to the discharges of interest.

Single-section stage-discharge curves can be calculated by hand tabulation, as shown in the example. Stage-discharge curves calculated by the step-backwater method, especially in the case of stream channels, typically use output from a computer program such as HEC-RAS or WSPRO.

The transverse velocity variation in any channel cross-section is a function of subsection geometry and roughness, and it may vary considerably from one stage and discharge to another. It is important to know this variation for purposes of designing scour control measures and locating relief openings in highway fills, for example. The best method of establishing transverse velocity variations is by current meter measurements. If this is not possible, the single-section or step-backwater method can be used by dividing the cross-section into subsections of relatively uniform roughness and geometry. It is assumed that the energy grade line slope is the same across the cross-section so that the total conveyance K_t of the cross-section is the sum of the subsection conveyances. The total discharge is then $K_t S^{1/2}$ and the discharge in each subsection is proportional to its conveyance. The velocity in each subsection is obtained from the continuity equation, $V = Q / A$. HEC-RAS or WSPRO also list velocities for multiple subsections or the total cross-section.

There may be locations where a stage-discharge relationship has already been determined using measured flows and stages, such as at gaging stations. These stage-discharge relationships are commonly called "rating curves." They are available for most stations from the operators. Measured stage-discharge curves will yield more accurate estimates of water surface elevation and should take precedence over the analytical methods described above.

8.11.1 Example Problem - Stage-Discharge Curve

This example illustrates use of a stage-discharge curve to determine the tailwater elevation at a culvert outlet. The single-section method will be used.

Given:
        The culvert outlet is at Channel Station 0+98.4, as shown in Figure 8-6a. The invert elevation is 729.00 feet.

        A typical channel cross-section was surveyed at Channel Station 4+46 and it is shown in Figure 8-6b. It is Cross-Section A.
        The 25-year peak discharge (Q_{25}) is 175 cubic feet per second.

The stream profile is surveyed. The streambed slope through the culvert site is 0.0027 foot per foot.

Find:

Tailwater elevation at the culvert outlet during the 25-year event.

Step 1 - The downstream channel geometry is fairly uniform and it is at a constant slope. Near uniform flow occurs. The single cross-section at Station 4+46 represents the downstream channel.

Step 2 - The section at Station 4+46 is divided into subsections using guidelines in Subsection 8.10.3. The subdivisions are due to both geometry and varying roughness. The subsections are given roughness values based on vegetation and terrain using values in Appendix A. Subsection 1 is an overbank area with light brush and trees: n = 0.06. Subsection 2 is the main channel, and it is clean, straight, and it has a few weeds and rocks: n = 0.035. Subsection 3 is an overbank area with scattered brush and dense weeds: n = 0.05. The cross-section data is tabulated as follows:

Step 3 - The energy grade line slope is assumed to be the slope of the stream bottom as represented by a line drawn through the high points on the channel bottom profile. There is a 2.16 foot drop in this 800 foot long stream reach, and the slope (S) is calculated to be 0.0027 feet per foot.

Step 4 - A range of flow elevations are arbitrarily chosen. In this example, the selected elevations are at increments of approximately 1 foot. The area (A) and wetted perimeter (P) are calculated from geometric formulae or measured from a cross-section plot drawn to scale for each subsection at each elevation increment. The wetted perimeter is considered to be the perimeter of the subsection where water contacts a solid surface. The hydraulic radius (R) is calculated for each subsection at each elevation increment as follows: R = A / P.

The flow (Q) and velocity (V) in each subsection is calculated by Manning's Equations 8-15 and 8-12, respectively. The flows in each of the subsections are added together to get the total flow in the waterway. The areas of each subsection are added together to get the total waterway area. The total discharge is divided by the total waterway area to calculate the average flow velocity in the waterway. Typically, the calculations are tabulated. Tabular calculations for three of the five elevation increments are shown in Figure 8-7.

Step 5 - The tabulated discharge and elevation values are plotted to determine the stage-discharge curve. The stage discharge curve at Cross-Section A is shown in Figure 8-8. The water surface elevation at the section is 730.81 feet during the 50-year event.

Cross-Section A is 348 feet downstream from the culvert outlet, and the estimated water surface slope is 0.0027 feet per foot. The water surface elevations at Cross-Section A are adjusted to represent the tailwater elevations at the outlet as follows:

$TW_{Outlet\ 50-yr}$ = (0.0027 x 348 feet) + 730.81 = 731.75 feet ⟵ Answer

---

Table of Cross-Section Data (Section "A" at Sta. 4+46)

Distance (feet)	Elevation (feet)	n-value
0.00	733.50	0.06
8.0	731.20	0.06
40.0	730.81	0.035
45.0	727.00	0.035
50.0	727.00	0.035
53.0	730.61	0.05
78.0	729.40	0.05
103.0	731.00	0.05
108.0	734.00	0.05

a) Stream Profile

b) Cross-Section A
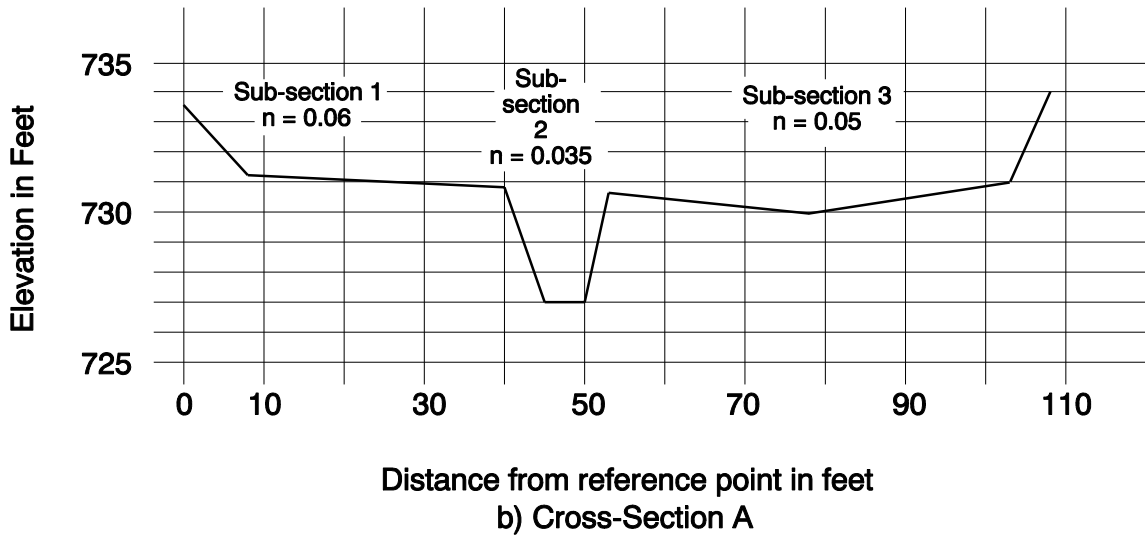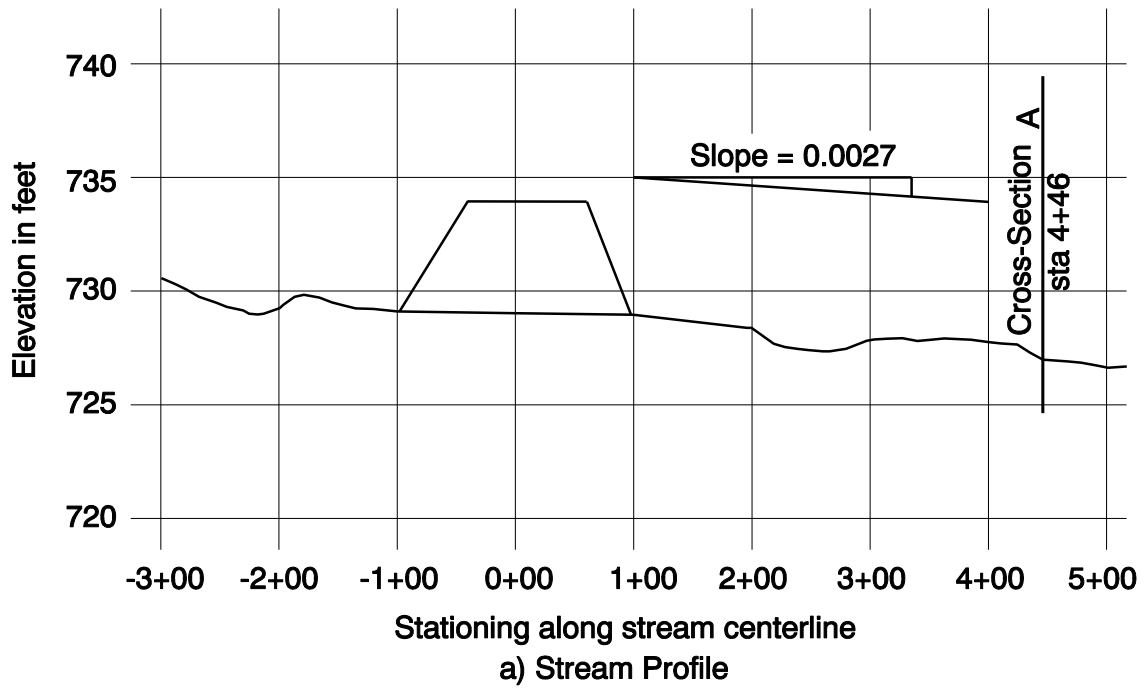
Figure 8-6   Site Data

Elevation = 728.0			Slope = 0.0027				
Sub-section ID	I	II	III	IV	V	VI	Totals/Average
Area (square feet)		6.0					6.0
Wetted Perimeter (feet)		7.9					
Hydraulic Radius (feet)		0.76					
$R^{2/3}$		0.83					
n	0.060	0.035	0.050				
Q = (cubic feet a second)		11					11
Sub-section Velocity (feet per second)		1.8					1.8

Elevation = 729.0			Slope = 0.0027				
Sub-section ID	I	II	III	IV	V	VI	Totals/Average
Area (square feet)		14					
Wetted Perimeter (feet)		11					
Hydraulic Radius (feet)		1.3					
$R^{2/3}$		1.2					
n	0.060	0.035	0.050				
Q = (cubic feet a second)		37					37
Sub-section Velocity (feet per second)		2.6					2.6

Elevation = 730.0			Slope = 0.0027				
Sub-section ID	I	II	III	IV	V	VI	Totals/Average
Area (square feet)		25	6.6				32
Wetted Perimeter (feet)		15	22				
Hydraulic Radius (feet)		1.7	0.30				
$R^{2/3}$		1.4	0.45				
n	0.060	0.035	0.050				
Q = (cubic feet a second)		77	4.6				82
Sub-section Velocity (feet per second)		3.1	0.70				2.6

Figure 8-7　Tabular Stage-Discharge Calculations
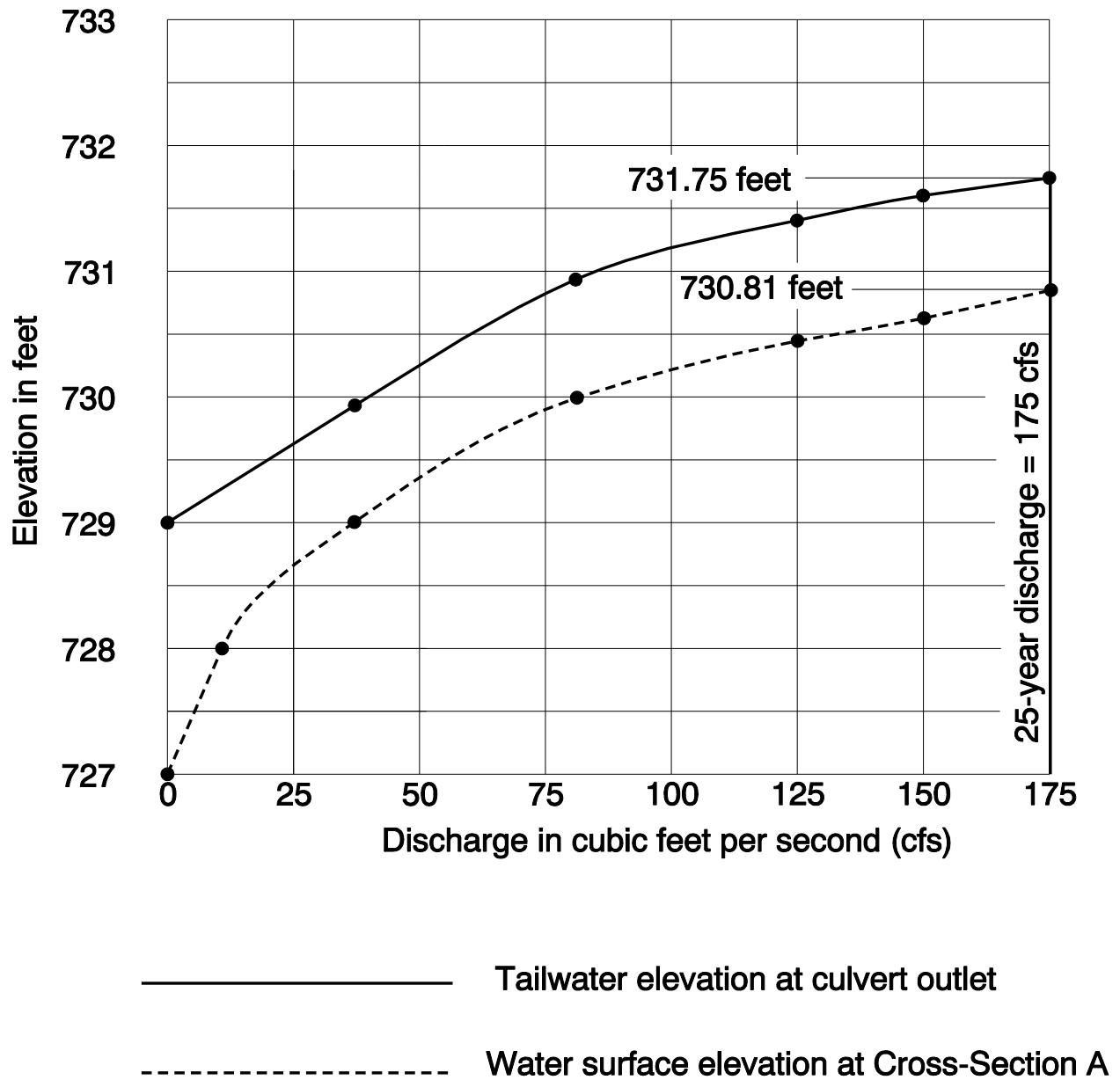
Figure 8-8   Stage-Discharge Curves

8.12 Profile Computation

Water surface profile computation requires a beginning value of water surface elevation, depth, or energy grade line slope at the initial or starting section. These beginning values are often called boundary conditions. The analysis proceeds section-by-section upstream in the case of subcritical flow, or downstream in supercritical flow. Boundary conditions are an approximation of the water surface elevation or slope at the initial section, and they are usually determined by the following methods.

- The water surface or channel bottom slope at the initial section is assumed to be the energy grade line slope at that location. This assumption can be used if there is uniform or nearly uniform flow at the initial section. Often the initial section is deliberately located in an area of nearly uniform flow so this assumption can be used.

- Critical depth is assumed to occur at the initial section. This assumption is usually made if there is a rapidly varied flow such as a hydraulic drop or jump and flow passes through critical depth at the initial section. Sometimes the initial section is located just upstream or downstream from a rapids, cataract, or waterfall so this assumption can be used.

- A range of reasonable water surface elevations or depths are assumed at the initial section and water surface profiles are computed upstream or downstream to the desired location using the same discharge. These profiles should converge toward the same elevation or depth at the cross-section where the stage-discharge relationship is desired, as shown in Figure 8-9a.

   Note: Estimating the boundary conditions introduces unavoidable error into the hydraulic model, and this convergence indicates that this error does not affect the water surface elevations at the study site. If the profiles do not converge, as shown in Figure 8-9b, the error in estimating the boundary conditions does affect the results, and more cross-sections are needed in the model. The model can be extended downstream or upstream as shown in Figure 8-9c. This is the method of adding sections that most often produces the desired results. Satisfactory results can also be obtained in some instances by adding one or more intermediate cross-sections between the sections in the original model, as shown in Figure 8-9d.

- Water surface elevations of a river, lake, ocean, or other body of water can be used as boundary conditions if they are located upstream or downstream. Elevation data are often available from published sources such as Flood Insurance Studies.

8.13 Curbs and Gutters

Gutter flow hydraulics are an essential part of storm drain system design. Gutter hydraulics can be determined by several methods, such as equations, nomographs, and computer programs. Equations and nomographs are discussed in this section.

Computer programs to calculate gutter flow are available from public and proprietary sources. The most common gutter shapes can be analyzed using the FHWA VISUAL URBAN program. This software is based on the FHWA Hydraulic Engineering Circular No. 22 "Urban Drainage Design Manual" (HEC-22). Irregular shaped gutters and the Valley Gutter can be analyzed using the U.S. Corps of Engineers HEC-RAS step backwater program.

Gutters should be designed to convey the 10-year event and limit the maximum flow widths to the values listed in **Chapter 13**, Storm Drainage. **The rational or unit hydrograph methods discussed in Chapter 7 are used to estimate runoff from the pavement and side drainage areas adjacent to the gutters.**

8.13.1 Gutter Types and Dimensions

A pavement gutter is defined as the section of pavement next to the curb which conveys water during a storm runoff event. Gutter cross-sections usually have a triangular shape with the curb forming the near-vertical leg of the triangle. The gutter may have a straight cross slope or a cross slope composed of two straight lines. Shallow swale gutters typically have V-shaped or circular sections and are often used in paved median areas on roadways with inverted crowns. Curbs and gutters are shown on Figure 8-10 and ODOT Standard Drawings RD700, RD701, and RD720. Dimensions for ODOT gutters are listed in Table 8-1.

The ODOT Curb and Gutter and Mountable Curb and Gutter provide a composite gutter cross-section. The composite section has a smooth concrete apron adjacent to the curb called a depressed gutter section. The apron edge near the curb is slightly depressed because the apron has a steeper cross slope than the adjacent pavement. These are the most hydraulically efficient curb and gutter sections. Using these curbs may result in the most economical design since fewer inlets are usually required to drain the pavement.

Another gutter type is the extruded curb. It has an asphalt concrete or Portland cement concrete curb and the pavement forms the gutter. The cross slope of the gutter is the cross-slope of the pavement. These gutters are not as hydraulically efficient as those with depressed sections. Extruded curbs are often used to keep highway drainage on the pavement and prevent it from eroding fill slopes.

Figure 8-9   Convergence

EXTRUDED CURBS

SECTION
P.C. CONCRETE

Finish surfacing

P.C. conc. curb

Norm. rock slope

SECTION
ASPHALT CONCRETE

Finish surfacing

Asph. conc. curb

Norm. rock slope

h = 6" normal
4" when run of curb or any
part thereof is placed under guardrail

CURB AND GUTTER

6" Var. 18" or 24"

MOUNTABLE CURB AND GUTTER

Var. 18" or 24" specify in plans

6"

6"

STANDARD CURB

Surf. of pvmt.

Batter 6:1

LOW PROFILE MOUNTABLE CURB

Surf. of pvmt.

4"   1"

12"

MONOLITHIC CURB & SIDEWALK

Var. (6' Standard)

Dummy joint ¼" deep "V" groove

Surface of Pavement

6" max.

Sl. 2%

VALLEY GUTTER

5%   5%

7"

20"

40"

MOUNTABLE CURB

6"

Surf. of pvmt.

Batter 6:1

① Slope 8% norm. Use 5% slope for gutter width greater than 24". Slope 4% at ramps. Vary slope as reqd. for dr.

② Norm. edge of shldr. or face of guardrail

● Curb exposure "E" = 6" to 9"

Figure 8-10   Curb and Gutter Types and Dimensions

Type	Configuration			Notes
	Curb Height (inches)	Depressed Gutter Width (inches)	Gutter Cross Slope (percent)	
Standard Curb	6-9	N/A	Same as Pavement	See Note 1.
Curb and Gutter	6-9	18-24	8	
		more than 24	5	
Mountable Curb	6-9	N/A	Same as Pavement	See Note 1.
Low Profile Mountable Curb	3	N/A	Same as Pavement	Recommended for rural highways or freeways. See Note 1.
Mountable Curb and Gutter	6-9	18-24	8	
		more than 24	5	
Extruded Curb	4-6	N/A	Same as Pavement	See Note 1.
Monolithic Curb and Sidewalk	6-9	N/A	Same as Pavement	See Note 1.

Table 8-1   ODOT Curbs and Gutters

Note 1: Gutter grades more than or equal to 0.3 percent recommended. Do not use where gutter grades are less than 0.2 percent.

8.13.2 Curb and Gutter Flow Equations

Channel capacities, average velocities, and flow widths for gutter types can be calculated using a modified form of the Manning equation. This modified equation is necessary to compute flow in a triangular channel because the hydraulic radius in the conventional equation does not adequately describe a gutter cross-section. Cross-sections of typical gutters are shown in Figure 8-11.
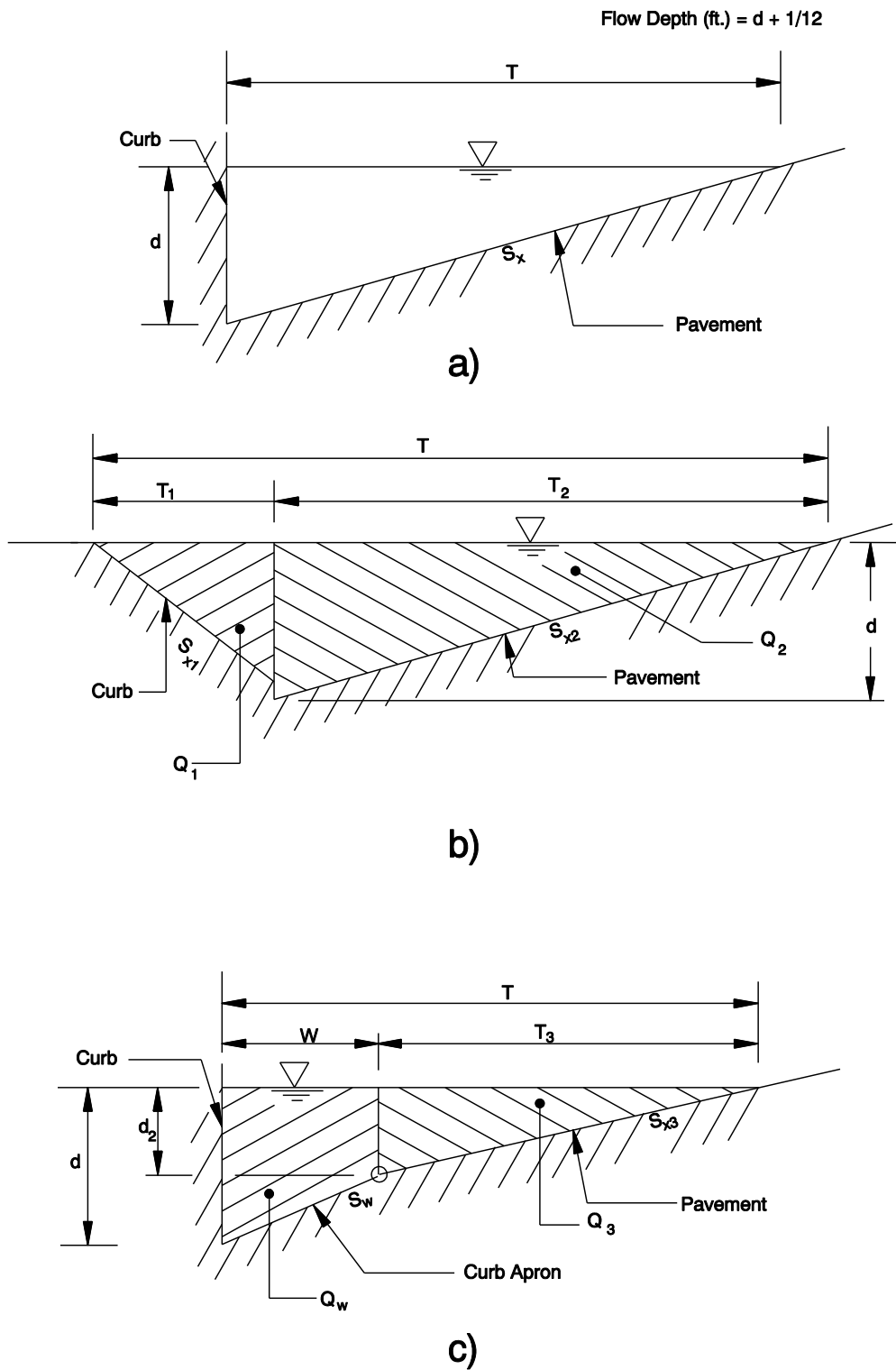
Flow Depth (ft.) = d + 1/12

a)

b)

c)

Figure 8-11   Curb and Gutter Cross-Sections

The terms in the gutter flow equations follow:

Q	Total gutter flow (cubic feet per second)
Q_1, Q_2, Q_3	Partial gutter flow (cubic feet per second)
Q_W	Depressed gutter flow (cubic feet per second)
n	Manning's "n" value (see Appendix A)
$S_X, S_{X1}, S_{X2}, S_{X3}$	Gutter cross slope (foot per foot)
S	Longitudinal gutter slope (foot per foot)
T, T_1, T_2, T_3	Flow width in gutter (feet)
V	Average gutter flow velocity (feet per second)
d	Flow depth at curb (feet)
d_2	Flow depth at break in cross slope (feet)
S_W	Depressed gutter cross slope (foot per foot)
W	Depressed gutter width (feet)

These curbs and gutters have a straight cross slope (S_X) and a curb face that is near vertical, as shown in Figure 8-11a:

- Portland Cement Concrete Extruded Drainage Curb,
- Asphalt Concrete Extruded Drainage Curb,
- Monolithic Curb and Sidewalk,
- Standard Curb, and
- Mountable Curb (with 6:1 batter).

These formulae can be used:

$$Q = \left(\frac{0.56}{n}\right) S_X^{1.67} \, S^{0.5} \, T^{2.67} \qquad \text{(Equation 8-24)}$$

$$T = \left(\frac{nQ}{0.56 \, S_X^{1.67} \, S^{0.5}}\right)^{0.375} \qquad \text{(Equation 8-25)}$$

$$V = \frac{Q}{A} = \frac{1.12}{n} S^{0.5} \, S_X^{0.67} \, T^{0.67} = \frac{2Q}{T^2 S_X} \qquad \text{(Equation 8-26)}$$

The Low Profile Mountable Curb has two gutter cross slopes (S_{X1} and S_{X2}), as shown in Figure 8-11b. These formulae can be used:

$$Q = Q_1 + Q_2 = \left(\frac{0.56}{n}\right)\left(S^{0.5}\right)\left(S_{X1}^{1.67} \, T_1^{2.67} + S_{X2}^{1.67} \, T_2^{2.67}\right) \qquad \text{(Equation 8-27)}$$

$$T = T_1 + T_2 = \left[\frac{Q_1 n}{0.56\, S^{0.5}\, S_{X1}^{1.67}}\right]^{0.375} + \left[\frac{Q_2 n}{\left(0.56\, S^{0.5}\, S_{X2}\right)^{1.67}}\right]^{0.375}$$

(Equation 8-28)

$$V = \frac{Q}{A} = \frac{2Q}{\left(T_1^2\, S_{X1} + T_2^2\, S_{X2}\right)}$$

(Equation 8-29)

$$d = T_2 S_{X2}$$

(Equation 8-30)

The Curb and Gutter has a near vertical curb and two gutter cross slopes (S_{X3} and S_W), as shown in Figures 8-11c. These formulae are derived to calculate discharges for the Curb and Gutter. They can be used to approximate discharges in the Mountable Curb and Gutter.

$$Q_3 = \left(\frac{0.56}{n}\right) S_{X3}^{1.67}\, S^{0.3}\, T_3^{2.67} = \left(\frac{0.56}{n\, S_{X3}}\right) S^{0.5}\, d_2^{2.67}$$

(Equation 8-31)

$$Q_w = \left(\frac{0.56}{n S_W}\right) S^{0.5}\, (d^{2.67} - d_2^{2.67})$$

(Equation 8-32)

Where:

$$d = T_3 S_{X3} + W S_W$$

(Equation 8-33)

$$d_2 = T_3 S_{X3}$$

(Equation 8-34)

$$Q = Q_3 + Q_w = \left(\frac{0.56}{n}\right)\left(S^{0.5}\right)\left[\left(\frac{d_2^{2.67}}{S_{X3}}\right) + \frac{\left(d^{2.67} - d_2^{2.67}\right)}{S_W}\right]$$

(Equation 8-35)

$$V = \frac{Q}{A} = \frac{2Q}{(T_3^2\, S_{X3} + W^2\, S_W + 2WT_3 S_{X3})} = \frac{2Q}{\dfrac{w\,(d + d_2) + d_2^2}{S_{X3}}}$$

(Equation 8-36)

8.13.3 Gutter Flow Examples Using Equations

The following examples illustrate the use of the gutter flow equations.

8.13.3.1 Example 1: Determine Discharge and Velocity for Low Profile Mountable Curb

Calculate the discharge, average velocity, and flow depth in a gutter alongside an ODOT Low Profile Mountable Curb with a flow width of 6 feet, as shown in Figure 8-12.

Given:

S = longitudinal gutter slope = 0.01 foot per foot,

S_{X1} = 3 inches / 12 inches = 0.25 inch per inch = 0.25 foot / foot (Std. Dwg. RD 700)

S_{X2} = pavement cross slope 0.025 foot per foot,

T = $T_1 + T_2 = 6$ feet, and

n = 0.016 ([Appendix A](#)).

Need: Q = discharge in cubic feet per second,

        V = average velocity in feet per second, and

        d = maximum flow depth in feet.

Step 1 - Determine T_1 and T_2

$$S_{X1}\, T_1 = d - \frac{1}{12}$$

$$d = S_{X2}T_2$$

$$\therefore S_{X1}\, T_1 = S_{X2}\, T_2 - \frac{1}{12}$$

(1) $0.25T_1 = 0.025T_2 - 0.083$

(2) $0.25T_1 - 0.025T_2 = -0.083$

Divide equation (2) above by 0.025

(1) $10T_1 - T_2 = -3.3$

(2) $T_1 + T_2 = 6$ see "Given"

Add equation (1) + (2) as shown below

(1) $10T_1 - T_2 = -3.3$

(2) $\underline{T_1 + T_2 = 6}$

       $11T_1 = 2.7$

$$T_1 = \frac{2.7}{11} = 0.25 \text{ feet} \quad \leftarrow$$

Solve for T_2 as follows:

$T_2 = 6 - T_1$

$T_2 = 6 - 0.25 = 5.75$ feet $\leftarrow$

Step 2 - Determine d using Equation 8-30

d = $T_2 S_{X2}$ = (5.75)(0.025) = 0.14 feet = 1.7 inches ← Answer

Note: Flow is contained within 4-inch high curb and assumptions in Step 1 are valid. The Step 1 assumptions would not be valid if the flow is over the curb and onto the sidewalk. This should always be checked before proceeding to the next step.

Step 3 - Determine Q using Equation 8-27 and V using Equation 8-29

$$Q = \left(\frac{0.56}{n}\right)\left(S^{0.5}\right)\left(S_{X1}^{1.67} T_1^{2.67} + S_{X2}^{1.67} T_2^{2.67}\right)$$

$$Q = \left(\frac{0.56}{0.016}\right)(0.01^{0.5})(0.25^{1.67} 0.25^{2.67} + 0.025^{1.67} 5.75^{2.67})$$

Q = 0.80 cubic feet per second ← Answer

$$V = \frac{2Q}{T_1^2 S_{X1} + T_2^2 S_{X2}}$$

$$V = \frac{2(0.80)}{\left[(0.25^2)(0.25) + (5.75^2)(0.025)\right]} = 1.9 \text{ feet per second} \leftarrow \text{Answer}$$
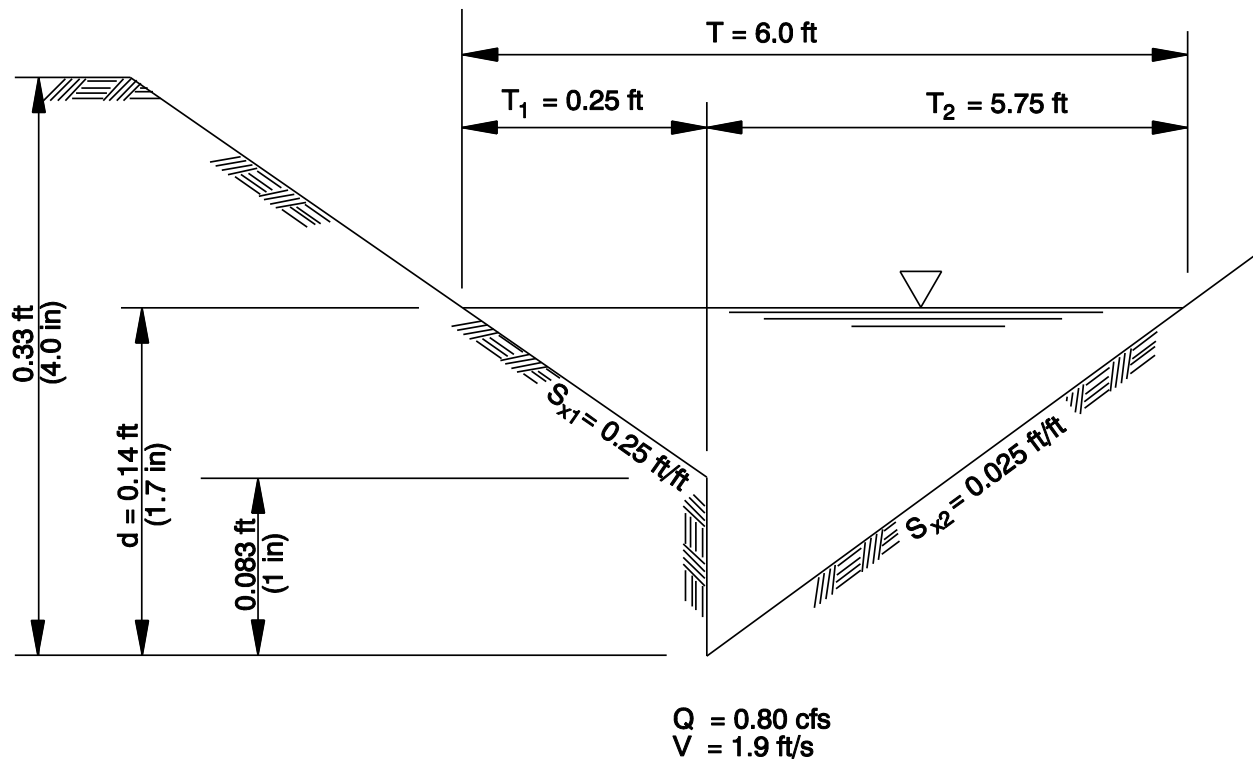
Figure 8-12   Gutter Cross-Section for Example 1

8.13.3.2   Example 2: Determine Flow Width and Depth for Curb and Gutter

Calculate the flow width and depth adjacent to an ODOT Curb and Gutter during a 1.0 cubic foot per second discharge, as shown in Figure 8-13. A 24-inch wide apron will be used. Verify the flow depth will not be higher than the 7-inch tall curb.

This is a trial and error solution. Steps 1 and 2 are repeated as necessary to calculate an answer with the desired accuracy.

Given:

S	= longitudinal gutter slope = 0.001 foot per foot,
S_W	= 0.080 foot per foot for the 24-inch apron (See Std. Dwg. RD 700),
S_{X3}	= 0.02 foot per foot,
W	= 24 inches = 2 feet
Q	= 1.0 cubic foot per second, and
n	= 0.014 (Appendix A).

curb height   = 7 inches

Need:

T = flow width in gutter in feet, and

d = maximum flow depth in feet.

Step 1 -   Determine flow depths, d_2 and d, using Equations 8-33 and 8-34

Assume T_3 = 6 feet

then

$d_2 = T_3 \, S_{X3} = (6)(0.02) = 0.12$ foot

$d = T_3 S_{X3} + W S_W = (6)(0.02) + (2.0)(0.080) = 0.28$ foot

Step 2 -   Calculate flow (Q) for flow depths determined in Step 1 using Equation 8-35

$$Q = \left(\frac{0.56}{0.014}\right)\left(0.001^{0.5}\right)\left(\frac{0.12^{2.67}}{0.02}\right) + \left(\frac{0.28^{2.67} - 0.12^{2.67}}{0.080}\right)$$

$$Q = [(40)\,(0.032)](0.17) + \left(\frac{0.030}{0.080}\right)$$

Q = 0.70 cubic feet per second …. too low

assume T_3 = 7 feet

then $d_2 = (7)(0.02) = 0.14$ foot

d = (7)(0.02) + (2.0)(0.080) = 0.30 foot

Q = 0.90 cubic feet per second …..too low

assume T_3 = 7.5 feet

then $d_2 = (7.5)(0.02) = 0.15$ ft.

d = (7.5)(0.02) + (2.0)(0.080) = 0.31 foot

Q = 1.0 cubic foot per second ←

$T = T_3 + W = 7.5 + 2.0 = 9.5$ feet

$d = 0.31$ foot $= 3.7$ inches $\Leftarrow$ Answer

Since the curb exposure (7 inches) is greater than the flow depth at curb (3.7 inches), the gutter flow will not overtop the curb.
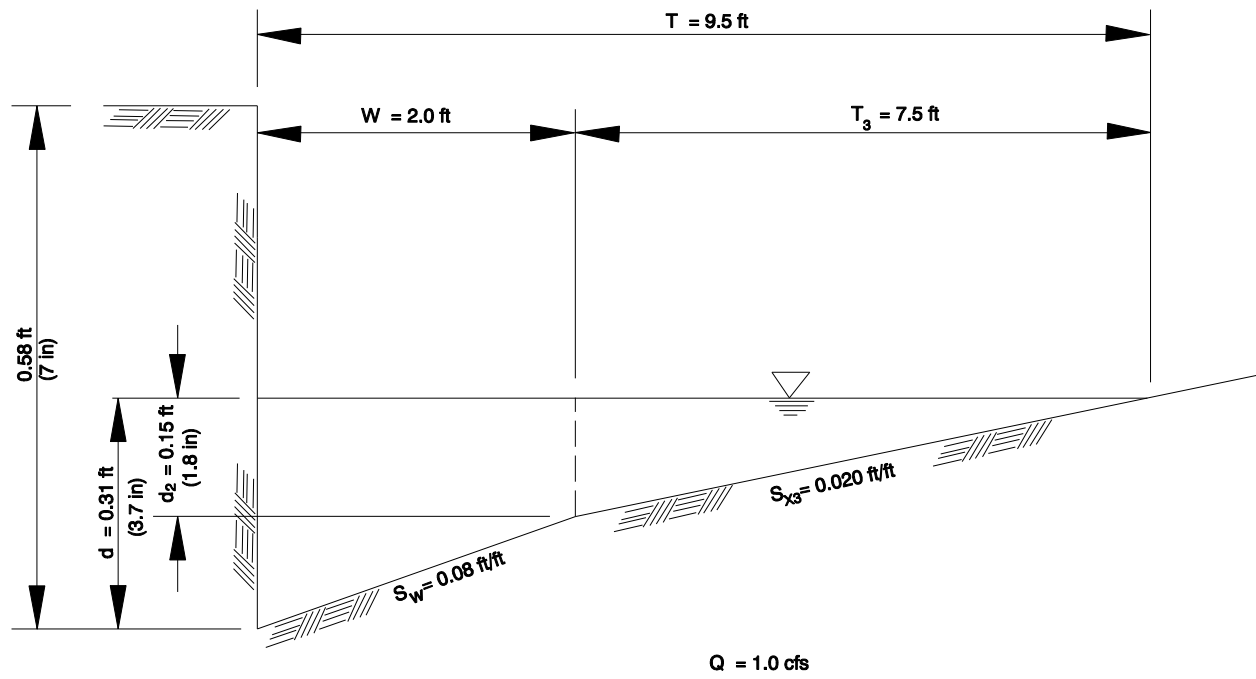
Figure 8-13   Gutter Cross-Section for Example 2

8.13.4 Curb and Gutter Flow Nomographs

Curb and gutter flow properties can be estimated using nomographs. Flow properties in a gutter with a near vertical curb face and a single gutter cross slope can be analyzed using Figure 8-14 for discharge and depth, and Figure 8-15 for average velocity. These gutters are illustrated in Figure 8-11a and they are:

- Portland Cement Concrete Extruded Drainage Curb,
- Asphalt Concrete Extruded Drainage Curb,
- Monolithic Curb and Sidewalk,
- Standard Curb, and
- Mountable Curb (with 6:1 batter).

Discharge and flow depth can be estimated in a gutter with a two gutter cross slopes, such as the Low Profile Mountable Curb, using Figure 8-16. This gutter is shown in Figure 8-10b.

Discharges and flow depths in a gutter with a near vertical curb and two gutter cross slopes can be estimated using Figure 8-17 and 8-18. This gutter is shown in Figure 8-10c. The nomograph provides an accurate estimate of flow properties in the Curb and Gutter. It provides an approximation for the Mountable Curb and Gutter.

The procedures and example calculations are illustrated on the nomographs.
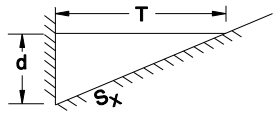
$$Q = \frac{0.56}{n} S_X^{1.67} S^{0.5} T^{2.67}$$

$$d = TS_X$$

Example: Given:
n = 0.016, S_X = 0.03 ft/ft
S = 0.04 ft/ft, T = 6 ft
Find:
Q = 2.4 cfs
d = (6)(0.03) = 0.18 ft
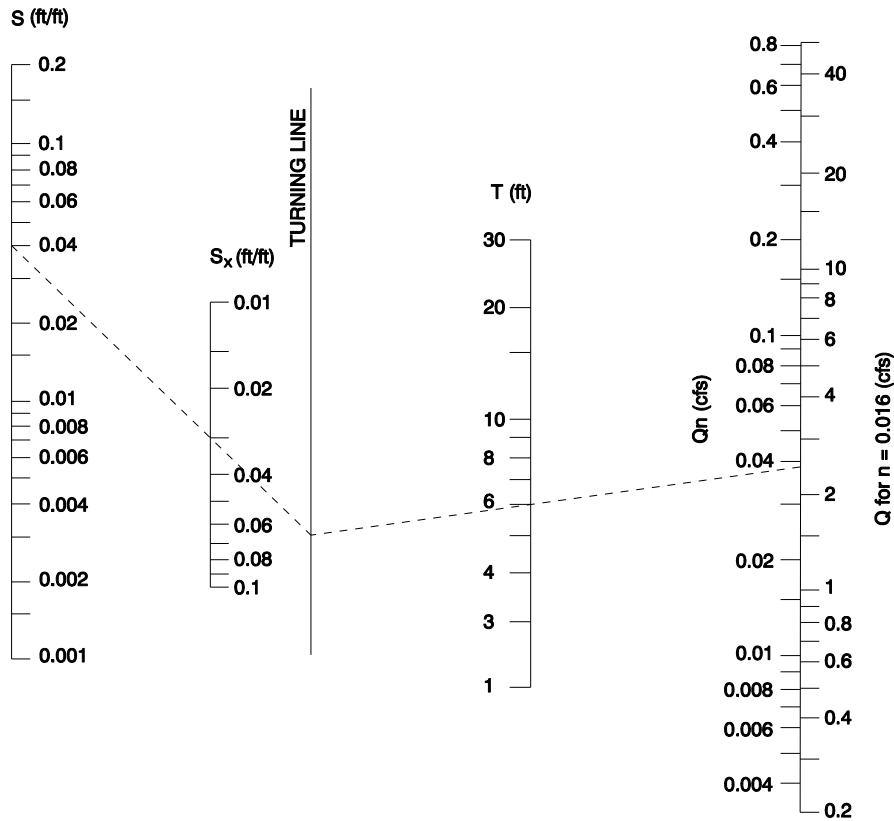
S (ft/ft)

S = longitudinal gutter slope (ft/ft)

S_X = gutter cross-slope (ft/ft)

T = flow width in gutter (ft)

Q = total gutter flow (cfs)

n = Manning's "n"

d = flow depth at curb (ft)

Example: Given:
n = 0.012, S_X = 0.03 ft/ft
S = 0.04 ft/ft, T = 6 ft
Find:
Q_n = 0.038 cfs
$Q = \frac{Q_n}{n} = \frac{0.038}{0.012} = 3.2$ cfs

**Figure 8-14   Discharge Nomograph for Gutter with
Near Vertical Curb and Single Gutter Cross Slope**
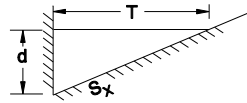
$$V = \frac{1.12}{n}S^{0.5}S_X^{0.67}T^{0.67}$$

Example: Given:
S = 0.12 ft/ft
S_X = 0.015 ft/ft
T = 6 ft
n = 0.016
Find:
V = 2.0 ft/s

S (ft/ft)

- 0.004
- 0.005
- 0.006
- 0.008
- 0.01

T (ft)

S_X (ft/ft)

- 0.1
- 0.08
- 0.06
- 0.04
- 0.03
- 0.02
- 0.01

- 0.02
- 0.03
- 0.04
- 0.05
- 0.06
- 0.08
- 0.1
- 0.2

Turning Line

Vn (cfs)

V for n = 0.016 (cfs)

0.3
0.2

0.1
0.08
0.06
0.04
0.03
0.02

20
15
10
9
8
7
6
5
4
3
2
1

20
15
10
9
8
7
6
5
4
3
2

V   = average gutter flow velocity (ft/s)

S   = longitudinal gutter slope (ft/ft)

S_X = gutter cross-slope (ft/ft)

T   = flow width in gutter (ft)

n   = Manning's "n"

Example: Given:
S   = 0.02 ft/ft
S_X = 0.015 ft/ft
T   = 6 ft
n   = 0.013
Find:
Vn = 0.032 ft/s
$V = \frac{Vn}{n} = \frac{0.032}{0.013} = 2.5$ ft/s

**Figure 8-15   Velocity Nomograph for Gutter with
Near Vertical Curb and Single Gutter Cross Slope**
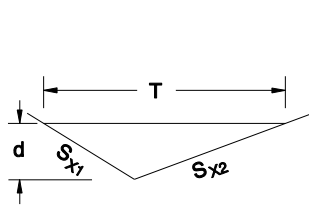
$$Q = \frac{0.56}{n} S^{1.67} S^{0.5} T^{2.67}$$

$$S_E = S_{X1} S_{X2}/(S_{X1} + S_{X2})$$

$$d = TS_E$$

Example: Given:
n = 0.016, S_{X1} = 0.50 ft/ft, S_{X2} = 0.02 ft/ft
S = 0.04 ft/ft, T = 6 ft
Find:
S_E = (0.50)(0.02)/(0.50 + 0.02)
S_E = 0.019
Q = 1.2 cfs
d = (6)(0.019) = 0.11 ft
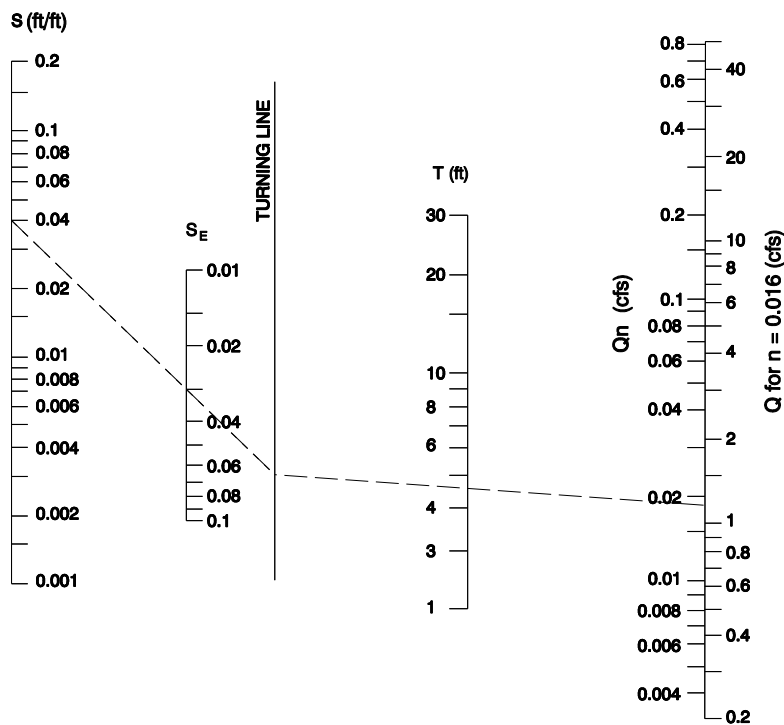
S = longitudinal gutter slope (ft/ft)

S_{X1}, S_{X2} = gutter cross-slope (ft/ft)

S_E = equivalent gutter cross-slope (ft/ft)

T = flow width in gutter (ft)

Q = total gutter flow (cfs)

n = Manning's "n"

d = flow depth at curb (ft)

Example: Given:
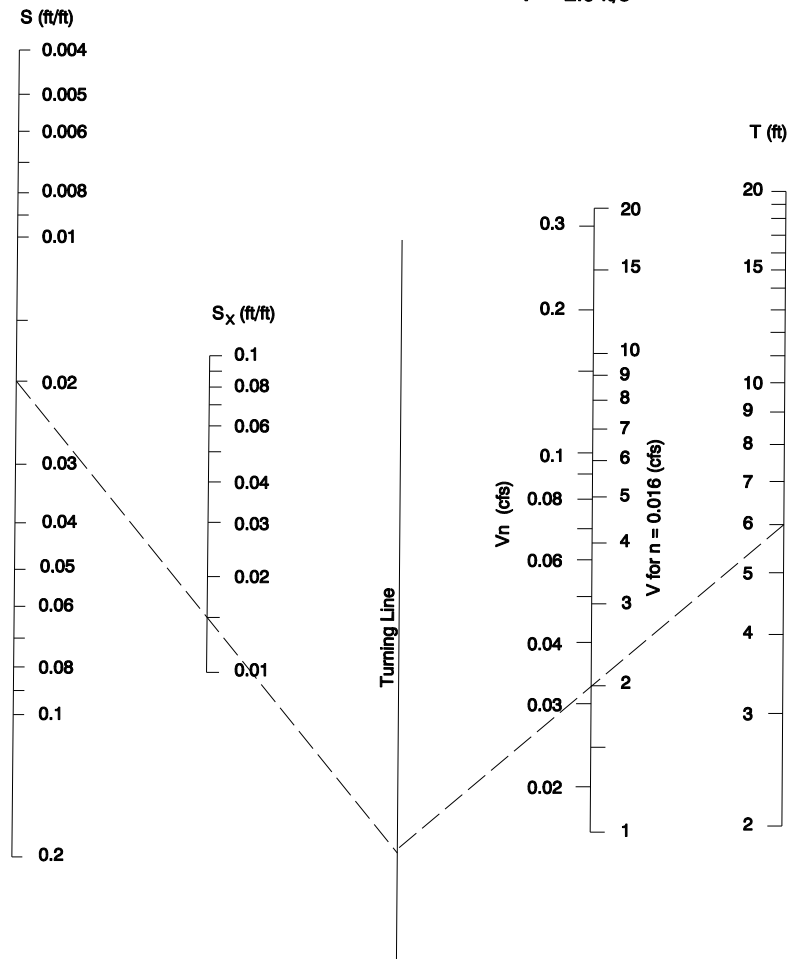n = 0.012, S_{X1} = 0.50 ft/ft,
S_{X2} = 0.02 ft/ft, S = 0.04 ft/ft
T = 6 ft
Need:
S_E = (0.50)(0.02)/(0.50 + 0.02)
S_E = 0.019 ft/ft
Q_n = 0.019 cfs
$Q = \frac{Q_n}{n} = \frac{0.019}{0.012} = 1.6$ cfs

**Figure 8-16   Discharge Nomograph for
Gutter with Two Cross Slopes**

To determine discharge in gutter with composite cross slopes, find Q_S using T_S and S_X. Then, use Figure 8-18 to find E_O. The total discharge is $Q = Q_S /(1 - E_O)$, and $Q_w = Q - Q_S$
$d = WS_W + T_S S_X$

Example: Given:
          $n = 0.016$, $S_W = 0.08$ ft/ft, $S_X = 0.02$ ft/ft
          $T = 6$ ft, $W = 1.5$ ft, $S = 0.04$ ft/ft
Find:
          $T_S = T - W = 6 - 1.5 = 4.5$ ft
          $Q_S = 0.59$ cfs
          $E_O = 0.67$ (from Figure 8-18)
          $Q = 0.59/(1 - 0.67) = 1.8$ cfs
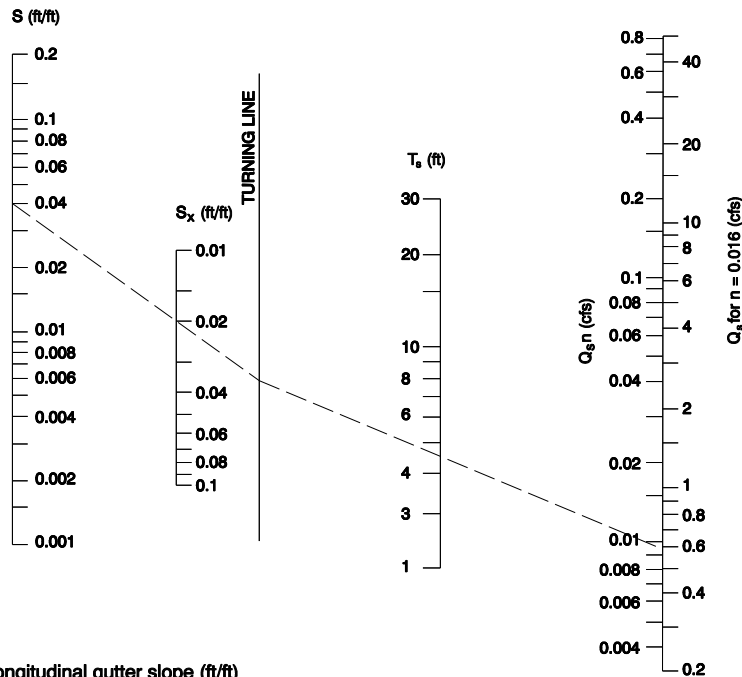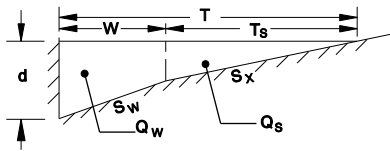
S = longitudinal gutter slope (ft/ft)

S_W = depressed gutter cross slope (ft/ft)

S_X = gutter cross-slope (ft/ft)

T = flow width in gutter (ft)

T_S = flow width outside of depressed gutter (ft)

W = depressed gutter flow width (ft)

Q = total gutter flow (cfs)

Q_S = flow outside of depressed gutter (cfs)

Q_W = depressed gutter flow (cfs)

E_O = ratio of depressed gutter flow to total gutter flow

Example: Given:
           $n = 0.012$, $S_W = 0.08$ ft/ft, $S_X = 0.02$ ft/ft
           $T = 6$ ft, $W = 1.5$ ft, $S = 0.04$ ft/ft
Find:
           $T_S = T - W = 6 - 1.5 = 4.5$ ft
           $Q_S n = 0.0093$ cfs
           $Q_S = \dfrac{Q_S n}{n} = \dfrac{0.0093}{0.012} = 0.77$ cfs

           $E_O = 0.67$ (from Figure 8-18)
           $Q = 0.77/(1 - 0.67) = 2.3$ cfs

Figure 8-17   Discharge Nomograph for Gutter
with Near Vertical Curb and Two Gutter Cross Slopes

Example: Given:
W = 1.5 ft, T = 6 ft, S_X = 0.02 ft/ft, S_W = 0.08 ft/ft

Find:
W/T = 1.5/6 = 0.25 S_W/S_X = 0.08/0.02 = 4
E_O = 0.67
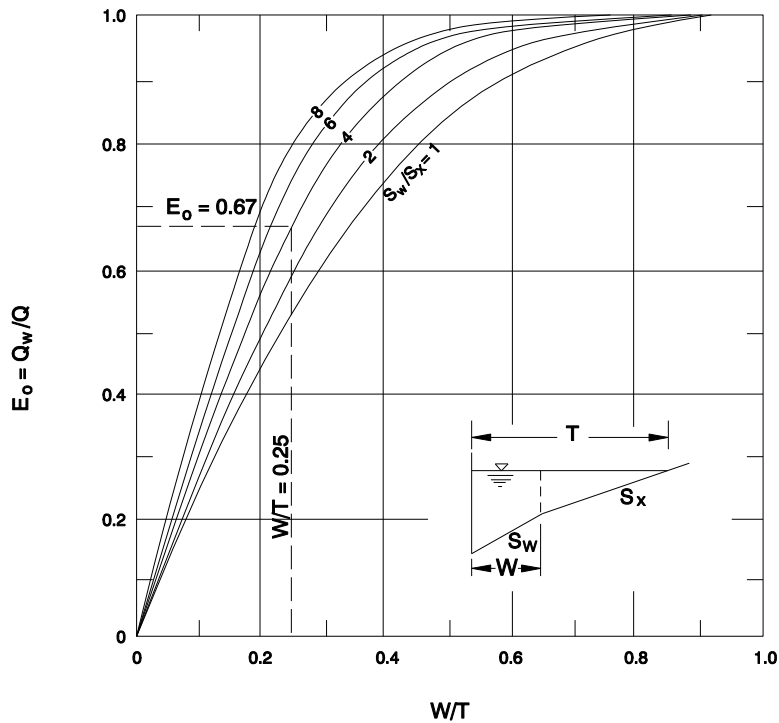
E_O = ratio of depressed gutter flow to total gutter flow

Q   = total gutter flow (cfs)

Q_W = depressed gutter flow (cfs)

W   = depressed gutter width (ft)

T   = flow width in gutter (ft)

S_W = depressed gutter cross slope (ft/ft)

S_X = gutter cross slope (ft/ft)

Figure 8-18   Ratio of Depressed Gutter Flow to Total Gutter Flow

8.14 Closed Conduits

Determining the hydraulic properties of closed conduits is essential to evaluate and design culverts, storm drains, and many other hydraulic structures. Many techniques can be used, such as equations, hydraulic element charts, nomographs, and computer programs. Equations, hydraulic element charts, and nomographs are discussed in this chapter.

A comprehensive set of nomographs for common conduit and channel shapes is in the Federal Highway Administration (FHWA) publication Hydraulic Design Series No. 3 "Design Charts for Open-Channel Flow." The publication is available from the following FHWA website http://www.fhwa.dot.gov/engineering/hydraulics/

Many computer programs to calculate closed conduit hydraulics are available from public and proprietary sources. The most common conduit shapes can be analyzed using the FHWA VISUAL URBAN program. This software is based on FHWA Hydraulic Engineering Circular No. 22 "Urban Drainage Design Manual" (HEC-22).

8.14.1 Equations for Full Flow in Closed Conduits

The hydraulic properties of a conduit flowing full, but not under surcharge or pressure flow, are useful tools in hydraulic design. The Manning's equation can be used to calculate the velocity and discharge in conduits flowing full, but not under pressure flow. The Manning's equation can also be used to estimate approximate discharges and velocities in conduits under slight surcharge such as storm drains conveying check discharges. In this case the pressure flow component of the total flow is considered insignificant and it is ignored. Usually the Manning's equation is adequate for these estimates. Accurate hydraulic analyses of critical applications or systems under significant pressure flow should use other methods, such as the Hazen-Williams formula.

Output and input variables of Manning's equations for full flow in conduits follow:

V_{FULL} = velocity in full conduit in feet per second
Q_{FULL} = discharge in full conduit in cubic feet per second
D_{FULL} = diameter of circular conduit needed to convey full flow, in feet
S_f = friction slope of full or slightly surcharged circular conduit, in feet per foot
n = Manning's roughness coefficient
R = hydraulic radius = A / P, in feet
D = diameter of circular pipe, in feet
A = cross-sectional area of full conduit in square feet
P = inside circumference of conduit in feet
S = slope of energy gradeline in foot per foot (S = conduit slope for steady uniform flow.)
V = velocity in conduit flowing full in cubic feet per second

Flow areas and hydraulic radii for many common closed conduits are listed in Appendix B. Use of these values will simplify calculations.

Manning's Roughness Coefficients – In Manning's equations the roughness of the wetted perimeter is represented by the Manning's roughness coefficient. Roughness values for most conduits are listed in Appendix A. Many of the listed values are for culverts and storm drains with a constant roughness coefficient throughout their wetted perimeter. An example is a typical new circular concrete pipe. The roughness throughout the wetted perimeter has the same value, and the Appendix lists a typical value of 0.013.

The appendix also lists Manning's coefficients for many conduits with varying roughness on their wetted perimeters. An example is a corrugated metal pipe with a paved invert. The roughness varies because the conduit has a smooth concrete bottom and relatively rough corrugated sides. The Appendix lists several roughness values for different flow depths ranging from 0.013 to 0.021.

The Appendix does not list roughness values for all conduits. An example is a box culvert with smooth reinforced concrete sides and a relatively rough channel bottom of cobbles and gravels. This conduit has varying roughness coefficients, and it can be analyzed using a composite Manning's roughness value, n_c. This method is also used in the analysis of natural stream channels, as discussed previously in this chapter. n_c is calculated by dividing the wetted perimeter (P) into parts (N), each with an individual roughness coefficient (n_i) and use of Equation 8-20. The following formula is used:

$$n_c = \left[\frac{\sum (n_i^{3/2} P_i)}{P} \right]^{2/3} \qquad \text{(Equation 8-37)}$$

$$i = 1$$

Where:

        n_c = composite Manning's roughness value for wetted perimeter
        P = wetted perimeter of entire conduit in feet
        P_i = wetted perimeter of the individual subdivision of the entire wetted perimeter
        n_i = Manning's coefficient of roughness for the individual subsection

Example: The composite roughness coefficient n_c is needed for a box culvert with a 2.0-foot wide bottom. The roughness of the gravels and cobbles on the culvert bottom is similar to typical ODOT Class 25 loose riprap with a roughness coefficient of 0.070. The sides are formed concrete with a roughness coefficient of 0.013. The estimated flow depth is 0.5 foot.

$$n_c = \left[\frac{\left(0.013^{3/2} \times 0.5\right) + \left(0.070^{3/2} \times 2.0\right) + \left(0.013^{3/2} \times 0.5\right)}{\left(0.5 + 2.0 + 0.5\right)} \right]^{2/3} = 0.55 \leftarrow$$

The composite Manning's roughness value of the culvert is 0.055 at a flow depth of 0.5 foot.

Manning's Equation for Full Flow Velocity in Conduits - The mean velocity, V, can be computed with Manning's equation for a conduit flowing full using the following formula:

$$V_{FULL} = \left(\frac{1.486}{n} \right) R^{2/3} S^{1/2}$$ (Equation 8-38)

Manning's Equation for Full Flow Discharge in Conduits - The continuity equation can be combined with Manning's equation to obtain Manning's equation for discharge:

$$Q_{FULL} = \left(\frac{1.486}{n} \right) A R^{2/3} S^{1/2}$$ (Equation 8-39)

Manning's Equation to Determine n Value – Manning's n

value can be calculated for a conduit under full flow, if either velocity or discharge is known, using one of the following versions of the Manning's equation:

$$n = \frac{1.486 \, (R^{2/3} \, S^{1/2})}{V_{FULL}} \text{ if velocity is known}$$ (Equation 8-40)

$$n = \frac{1.486 \, (A \, R^{2/3} \, S^{1/2})}{Q_{FULL}} \text{ if discharge is known}$$ (Equation 8-41)

Manning's Equation for Full Flow in Circular Conduits – A version of the Manning's equation can be used to calculate full flow discharge in circular conduits. Another version can be used to calculate the diameter needed to convey full flow. The equations follow.

$$Q_{Full} = \left(\frac{0.464}{n} \right) D^{8/3} S^{1/2}$$ (Equation 8-42)

$$D_{Full} = \left(\frac{2.16Qn}{S^{1/2}} \right)^{3/8}$$ (Equation 8-43)

Manning's Equation to Determine Friction Slope for Circular Conduits – Friction loss (H_f) is the energy required to overcome barrel roughness and is the most significant energy loss in a closed conduit system. It is particularly significant for surcharged or pressure flow. Friction loss is directly related to the velocity in the pipe; therefore, the higher the velocity, the greater the friction loss and vice versa. The friction loss can be calculated for circular conduits flowing full using Equations 8-44 and 8-45. These equations calculate the friction slope for conduits flowing full, and they estimate the friction slope for a conduit flowing with slight surcharge under pressure flow.

$$H_f = S_f L \hspace{8cm} \text{(Equation 8-44)}$$

Where:

H_f = friction loss in feet
S_f = friction slope in feet per foot
L = conduit length in feet

The friction slope can be calculated using a version of the Manning's equation:

$$S_f = \frac{V^2 n^2}{(2.21)(R^{4/3})} \hspace{6cm} \text{(Equation 8-45)}$$

8.14.2 Nomographs for Full Flow in Closed Conduits

Nomographs can also be used to estimate full flow in conduits. Full flow hydraulic characteristics for circular concrete and corrugated metal pipes are shown in Figures 8-19 and 8-20, respectively. Examples of typical applications are shown on the figures. Nomographs for full flow in many common conduit shapes are in the FHWA publication listed at the beginning of this section.

Full flow nomographs can also be used to estimate friction loss in pipes under slight surcharge. This is often needed to evaluate a storm drain system during a check discharge, as shown in the following example.

8.14.2.1   Example – Estimate Friction Loss in Pipe Under Slight Surcharge

A storm drain is designed to have the smallest possible conduits providing free surface flow during the 10-year event. The pipes are sized to be 75 to 95 percent full during this design storm. Local regulations require that the system be checked for the 50-year event. It is necessary to verify that water is contained within the system and does not flow out of manholes or inlets. One task in this analysis is to determine the friction loss in a section of 100-foot long 24-inch diameter corrugated metal pipe during a 15 cubic feet per second check discharge.

Given:

Q_{50} = 20 cubic feet per second
n   = 0.024 (corrugated metal)

L   = 100 feet

Need:

H_f = friction loss in feet

Step 1 - The nomograph in Figure 8-20 is used to determine the friction slope. A line is drawn between the 15 cubic feet per second discharge on the DISCHARGE scale, through the 24-inch diameter on the PIPE DIAMETER scale, to the SLOPE scale. The friction slope is read from the SLOPE scale, and it is 0.015 foot per foot, as shown on Figure 8-20.

S_f = 0.015 foot per foot

Step 2 - The friction loss is calculated using Equation 8-44, as follows:

H_f = (0.015)(100) = 1.5 feet ← Answer

EXAMPLE #1)

GIVEN: Q = 6 CFS
        S = 0.01 FOOT PER FOOT
NEED: SMALLEST DIAMETER
        PIPE THAT WILL FLOW
        PARTIALLY FULL
ANSWER: 15 - INCH DIAMETER
        PIPE IS SMALLEST SIZE
        FLOWING PARTIALLY
        FULL

EXAMPLE #2)

GIVEN: PIPE FLOWS FULL, BUT
        NOT UNDER PRESSURE
        FLOW:
        D = 36 INCHES
        S = 0.002 FOOT PER FOOT
NEED: FULL FLOW DISCHARGE,
        Q_{FULL} AND VELOCITY,
        V_{FULL}
ANSWER:
        Q_{FULL} = 29 CFS
        V_{FULL} = 4.2 FT/S
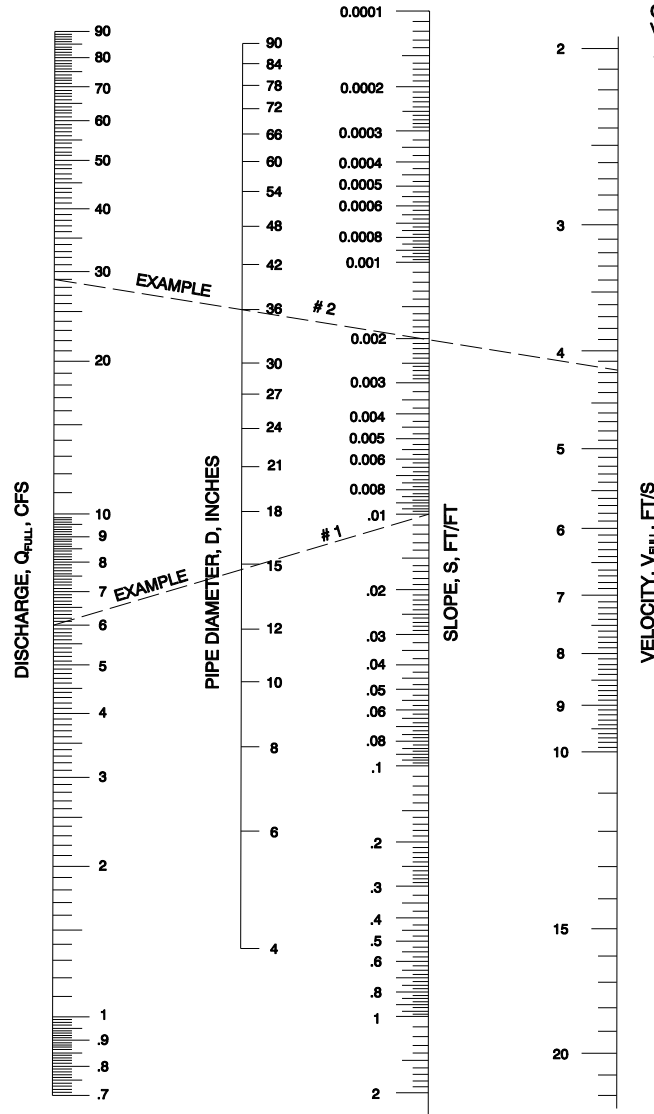
n = 0.013

Figure 8-19   Capacity and Velocity Nomograph
for Circular Concrete Pipes Flowing Full

EXAMPLE
GIVEN: Q = 15 CFS
           D = 24 INCHES
NEED: ESTIMATE OF FRICTION
           SLOPE IN FEET PER FOOT
ANSWER: FRICTION SLOPE = 0.015 FOOT
           PER FOOT
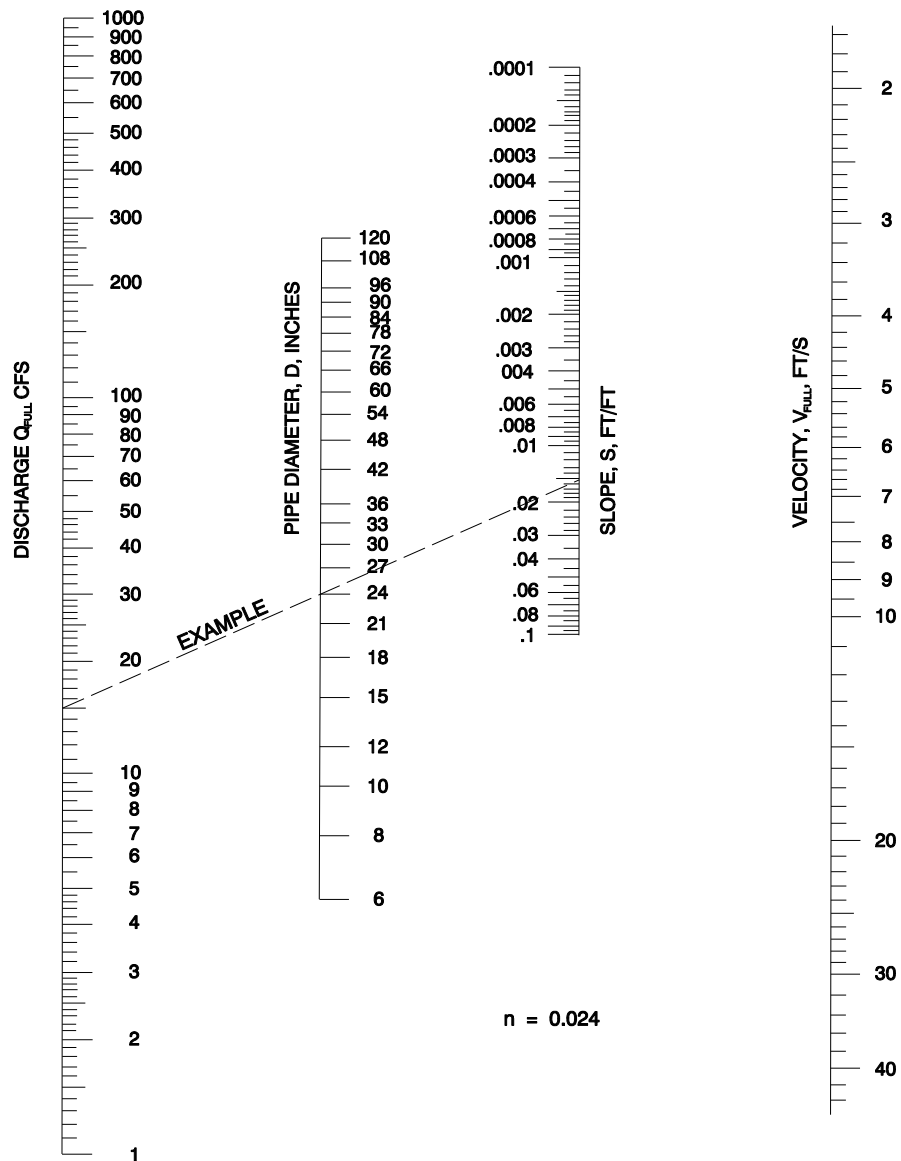
**Figure 8-20   Capacity and Velocity Nomograph
for Circular Corrugated Metal Pipes Flowing Full**

8.14.3 Equations for Normal Flow in Partially Full Conduits

The hydraulic properties of normal flow are a useful hydraulic analysis tool. Normal flow properties of open-channel discharge in closed conduits can be determined by many methods. These procedures can also be used for open-channels with shapes similar to partially full conduits, such as flumes and lined prismatic canals. Normal flow occurs:

- in prismatic conduits such as pipes or boxes where the cross-section and hydraulic roughness does not vary significantly between upstream and downstream sections,
- during open-channel flow,
- where the depth of flow is the same in upstream and downstream sections,
- where the flow streamlines are fairly parallel to each other and to the long axis of the conduit, and
- where turbulence such as hydraulic jumps and drops do not occur.

Manning's equations for velocity (Equation 8-12) and discharge (Equation 8-15) can be used with the continuity equation (Equation 8-11) to analyze normal flow in conduits. This method can be used for open channels of any shape. It is most useful for odd or irregular shapes that can not be analyzed by other procedures. It involves an iterative solution where the depth of flow is assumed and the flow area, hydraulic radius, and discharge are calculated. The iterations are repeated using various flow depths until the calculated discharge matches the desired discharge. In the last step the discharge is divided by the flow area to determine the average velocity. An example of this procedure using Manning's equation follows this subsection.

Irregular shapes often have varying hydraulic roughness throughout their cross-section. An example would be a box culvert with a hydraulically rough fish ladder on one half of the bottom, a smooth concrete surface on the other half of the bottom, and brick sides. A composite Manning's n values based on Equation 8-37 would be used when analyzing this conduit.

Normal velocity in circular pipes can be approximated within $\pm$ 5 percent using the following version of the Manning equation. It is sufficiently accurate for many purposes.

$$V_n = 0.863 \frac{\left(S^{0.366}\right)\left(Q^{0.268}\right)}{\left(n^{0.732}\right)\left(D^{0.048}\right)}$$

(Equation 8-46)

Where:

V_n = normal velocity for partial flow in feet per second
S = channel slope in foot per foot
Q = flow rate in cubic feet per second
N = Manning's roughness coefficient (Appendix A
D = pipe diameter in feet

8.14.3.1 Example – Manning's Equation to Determine Normal Flow Properties

Normal flow velocity needs to be determined for an elliptical pipe laid with its long axis horizontal.

Given:

> 14-inch rise by 23-inch span horizontal elliptical concrete pipe,
> pipe slope is 0.005 foot per foot,
> discharge is 1.0 cubic foot per second, and
> Manning's n = 0.013 from Appendix A.

Need:

> average flow velocity in pipe assuming normal flow

The procedure is to assume a flow depth and solve for the discharge. The process is repeated until the discharge is close to 1.0 cubic feet per second. The average velocity is determined by dividing the discharge by the flow area. Intermediate steps in these calculations require the hydraulic radius and flow area. These values are typically measured from a drawing of the conduit cross-section.

Assumed Flow Depth, d (feet)	Flow Area, A (square feet)	Hydraulic Radius, R (feet)	Discharge, Q $\left(\dfrac{1.486}{n}\right) A\,R^{2/3}\,S^{1/2}$ (cubic feet per second)
0.33	0.445	0.225	1.33
0.26	0.323	0.179	0.829
0.29	0.373	0.197	1.02
0.285	0.370	0.195	1.00

$$V = \frac{Q}{A} = \frac{1.00}{0.370} = 2.7 \text{ feet per second} \quad \leftarrow \quad \text{Answer}$$

8.14.4 Nomographs for Normal Flow in Partially Full Conduits

Nomographs can be used to determine the hydraulic properties of partially full conduits. Nomographs for many common conduit shapes are in the FHWA publication listed at the beginning of this section.

8.14.5 Hydraulic Elements Charts for Normal Flow in Partially Full Conduits

Normal flow properties can be determined by the hydraulic element charts in Appendix B for circular pipes, pipe-arches, semicircular arches, and reinforced concrete boxes. Although this method is easy to use, it has some limitations. First, the method can only be used for the conduit shapes represented by charts. The iterative solution shown previously is recommended for conduits not shown on the charts. Second, the hydraulic element method gives accurate results if the conduit has a constant roughness value throughout its entire wetted perimeter at full flow, such as a corrugated metal pipe. This method gives approximate results for conduits with varying roughness coefficients such as box culverts with gravel covered bottoms or corrugated metal pipes with paved inverts. The iterative solution method is recommended for accurate analysis of these conduits.

8.14.5.1   Example – Hydraulic Element Chart to Determine Normal Flow Properties in a Circular Pipe

Hydraulic properties such as depth and velocity are needed for a 40 cubic foot per second discharge through a 48-inch diameter circular corrugated metal pipe on a 0.6 percent grade.

Given:

> D = 48 inches = 4.0 feet = pipe diameter
> S = 0.6 / 100 = 0.006 foot per foot = pipe slope
> q = 40 cubic feet per second
> n = 0.023 from Appendix A for a 48-inch diameter corrugated metal pipe

Need:

> d = flow depth in feet
> V = average flow velocity in feet, and
> a = flow area in square feet

Step 1 – Determine one of the following proportional ratios:

q / Q_{FULL} will be selected as the proportional ratio since q is known. Q_{FULL} is calculated by Equation 8-42 as follows:

$$Q_{FULL} = \left(\frac{0.464}{0.023}\right)(4.0^{8/3})(0.006^{1/2}) = 63 \text{ cubic feet per second}$$

$$\frac{d}{D_{FULL}} \text{ or}$$

$$\frac{a}{A_{FULL}}$$

$$\frac{q}{Q_{FULL}} = \frac{40}{63} = 0.63$$

Step 2 - Locate the known proportional value from Step 1 on the appropriate axis of the hydraulic elements chart, as shown in Figure 8- 21. This would be a value of 0.63 on the horizontal axis for q / Q_{FULL} ratio of 0.63. Extend a line upward from the known proportional value on the axis to the curve representing the known proportional value. This will be the q / Q_{FULL} curve.

Step 3 - Extend a horizontal line from the intersection point of the line and curve determined in the previous step. This line will intercept one of the axes and several of the curves. In this example, the horizontal line intersects the d / D_{FULL} axis at a ratio of 0.57. Draw vertical lines from the intersection points on the various curves to the axes that represent these curves. The vertical lines from the a / A_{FULL}, v / V_{FULL}, and r / R_{FULL} curves intercept the horizontal axis at ratios of 0.58, 1.05, and 1.08, respectively.

Step 4 - Calculate desired values by multiplying the full flow values by the corresponding proportional values. As an example: v = (V_{FULL})(v / V_{FULL}). In some cases the

$$\frac{q}{Q_{FULL}}$$

$$\frac{v}{V_{FULL}}$$

continuity equation (Equation 8-11) can also be used.   In the example:

$$A_{FULL} = \pi r^2 = (3.142)(2 \text{ feet})^2 = 12.5 \text{ square feet}$$

$$a = (A_{FULL})\left(\frac{a}{A_{FULL}}\right) = (12.5)(0.58) = 7.3 \text{ Square feet} \leftarrow \text{ Answer}$$

$$d = (D_{FULL})\left(\frac{d}{D_{FULL}}\right) = (2)(0.57) = 1.1 \text{ feet} \leftarrow \text{ Answer}$$

Using the continuity equation:

$$v = \frac{q}{a} = \frac{40}{7.3} = 5.5 \text{ feet per second} \leftarrow \text{ Answer}$$

8.14.6 Critical Depth in Conduits

Critical depth (d_c) is defined as the depth for which the specific energy (sum of the flow depth and velocity head) of a given discharge is at a minimum. A slight change in specific energy can result in a significant rise or fall in the water depth when flow is at or near critical depth. Because of this, critical depth is an unstable condition and it rarely occurs for any distance along a water surface profile.

Flow at critical depth, however, is a common occurrence. Flow always passes through critical depth during transitions between flow regimes, such as the transition between the subcritical flow regime, where normal depth is greater than critical depth, and the supercritical flow regime, where normal depth is less than critical depth. This typically occurs when flow passes through a contraction, either horizontal or vertical, such as a culvert inlet, outlet, storm sewer manhole, etc. A detailed discussion of the relationship between critical depth and specific energy is beyond the scope of this manual. The designer should refer to any open-channel flow reference text for further information.

Critical depth is a useful tool in hydraulic analysis because it can be used to determine the type of flow profile inside the conduit and at the outlet. Critical depth is dependent on the shape of the conduit and the discharge, and it can easily be shown on a chart. Critical depth charts for most common conduit shapes are in Appendix B.

Critical depth can be calculated for conduit shapes that are not included in the charts. The following method can be used to determine critical depth for open channels of any shape. It is based on the principle that critical depth occurs during critical flow. It involves an iterative solution where the depth of flow is assumed, the flow area is calculated, the top width of the flow is determined, and the discharge at critical flow calculated. The procedure is repeated until the critical flow based on the assumed depth equals the discharge of interest. The flow depth at critical flow is critical depth.

The following equation is used:

$$Q_C = \left(\frac{A^3 g}{T} \right)^{1/2}$$

Where:

Q_C = discharge at critical depth in cubic feet per second
g = acceleration of gravity (32.2 feet per second squared)
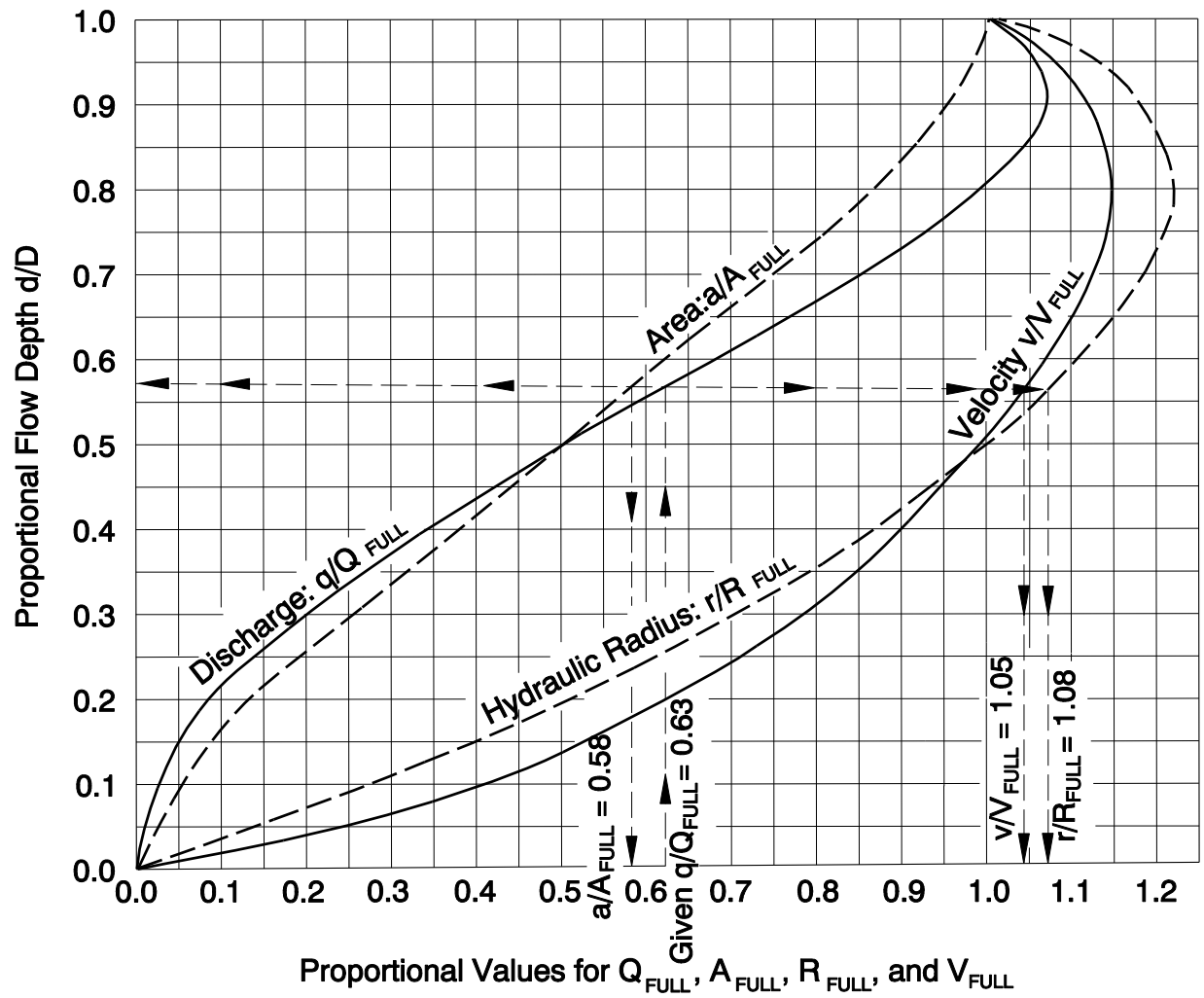A = flow area in square feet
T = top width of flow in feet

Figure 8-21   Use of Hydraulic Elements Chart

Example: Critical depth is needed for a trapezoidal shaped culvert during a 1200 cubic feet per second discharge.

Given:

   trapezoidal channel has a 9-foot wide bottom width and 1V:2H side slopes, and
   Q = Q_C = 1200 cubic feet per second.

Need:

   D_C = critical depth in feet

Assume a depth and solve for Q_C. Repeat process until Q_C is close to 1200 cubic feet per second. Areas and top widths are typically measured from a drawing of the conduit cross-section. A programmable calculator is strongly recommended for the calculations.

Assumed d (feet)	A (square feet)	T (feet)	$\left(\dfrac{A^3 g}{T}\right)^{1/2}$ (cubic feet a second)
3	45	21	370
6	126	33	1400
5.2	101	30	1050
5.56	112	31.2	1200

The critical depth for the given channel and discharge is approximately 5.56 feet ← Answer

Critical depth can be determined for a circular channel using Appendix B, or it can be approximated as follows:

$$D_C = 0.420\left(\frac{Q^{1/2}}{D^{1/4}}\right)$$ (Equation 8-48)

Where:

   D_C = critical depth in a circular section in feet
   Q = flow in cubic feet per second
   D = diameter in feet

Example: Find critical depths for a 3-foot diameter pipe with flows of 18 and 180 cubic feet per second.

Given:

   D = 3 feet
   Q_1 = Q_{C1} = 18 cubic feet per second and Q_2 = Q_{C2} = 180 cubic feet per second

Need:

       D_{C1} and D_{C2} in feet

Using Equation 8-48:

$$D_{C1} = 0.420\left(\frac{18^{1/2}}{3^{1/4}}\right) = 1.3 \text{ feet for a flow of 18 cubic feet per second}$$

$$D_{C2} = 0.420\left(\frac{180^{1/2}}{3^{1/4}}\right) = 4.3 \text{ feet for a flow of 180 cubic feet per second}$$

The critical depth for a discharge of 18 cubic feet per second is 1.3 feet ← Answer

The critical depth for a discharge of 180 cubic feet per second is 4.3 feet. The maximum critical depth that can occur is 3 feet because this is the diameter of the pipe. Greater critical depths cannot occur because the conduit will not be conveying open-channel flow. As a result, the critical depth of 4.3 feet is called a "calculated critical depth." Although the calculated critical depth cannot occur, it is useful because it indicates the conduit is flowing full with a surcharge.

Critical depth, and consequently critical flow, do not occur during an 180 cubic foot per second discharge in a 3-foot diameter pipe ← Answer

Critical depth can be calculated for a rectangular channel, such as a box culvert, using Appendix B, or it can be approximated as follows:

$$D_C = \left(\frac{0.176\,Q}{S}\right)^{2/3} \qquad\qquad\qquad\qquad \text{(Equation 8-49)}$$

Where:

       D_C = critical depth in feet
       Q = flow in cubic feet per second
       S = span of box section in feet

Example: Calculate critical depth in a box culvert with a 15-foot span by 10-foot rise during a discharge of 140 cubic feet per second.

Given:

       S = 15 feet
       Q = 140 cubic feet per second

Need:

       D_C in feet
Using Equation 8-49:

$$D_C = \left[\frac{(0.176)(140)}{15}\right]^{2/3} = 1.4 \text{ feet} \leftarrow \text{ Answer}$$

Critical depth can be calculated for a triangular channel, such as a flume, using the following approximation:

$$D_C = 0.757 \left(\frac{Q}{Z_1 + Z_2}\right)^{2/5} \qquad \text{(Equation 8-50)}$$

Where:

D_C = critical depth in feet
Q = flow in cubic feet per second
Z_1 = slope of channel side in feet per feet
Z_2 = slope of opposite channel side in feet per feet

Example: Find the critical depth in a triangular shaped channel with 1V:1.75H side slopes during a 880 cubic foot per second discharge

Given:

$Z_1 = Z_2 = 1.75$
Q = 880 cubic feet per second

Need:

D_C in feet

Using Equation 8-50:

$$D_C = 0.757 \left(\frac{880}{1.75 + 1.75}\right)^{2/5} = 6.9 \text{ feet} \leftarrow \text{ Answer}$$

Critical depths can be determined for pipe-arches and semicircular arches using charts in Appendix B.

8.15 Artificial Channels

Roadside or median ditches or channels, ditches leading from highway drainage systems to outfalls, canals, or other ditches conveying flow through the highway right-of-way are the artificial channels most often encountered in highway design. These artificial channels typically have trapezoidal, rectangular, or triangular cross-sections, and they are in bare earth or lined with grass, riprap, or other protective linings. Design discharge recurrence intervals for these smaller artificial channels are summarized in **Chapter 3** and Section 8.3.2. Design procedures are presented in this section for:

- channels with a prismatic cross-section conveying uniform or nearly uniform flow,
- 50 cubic foot per second or less discharge, and
- 10 percent or flatter channel bottom grade.

Procedures in the latest version of FHWA Hydraulic Engineering Circular Number 11, "Design of Riprap Revetment" can be used to design channels with flows greater than 50 cubic feet per second. Methods in FHWA Hydraulic Engineering Circular Number 15, "Design of Roadside Channels with Flexible Linings" can be used to design channels with grades greater than 10 percent. Both publications are available from the FHWA website: http://www.fhwa.dot.gov/bridge/elibrary.htm
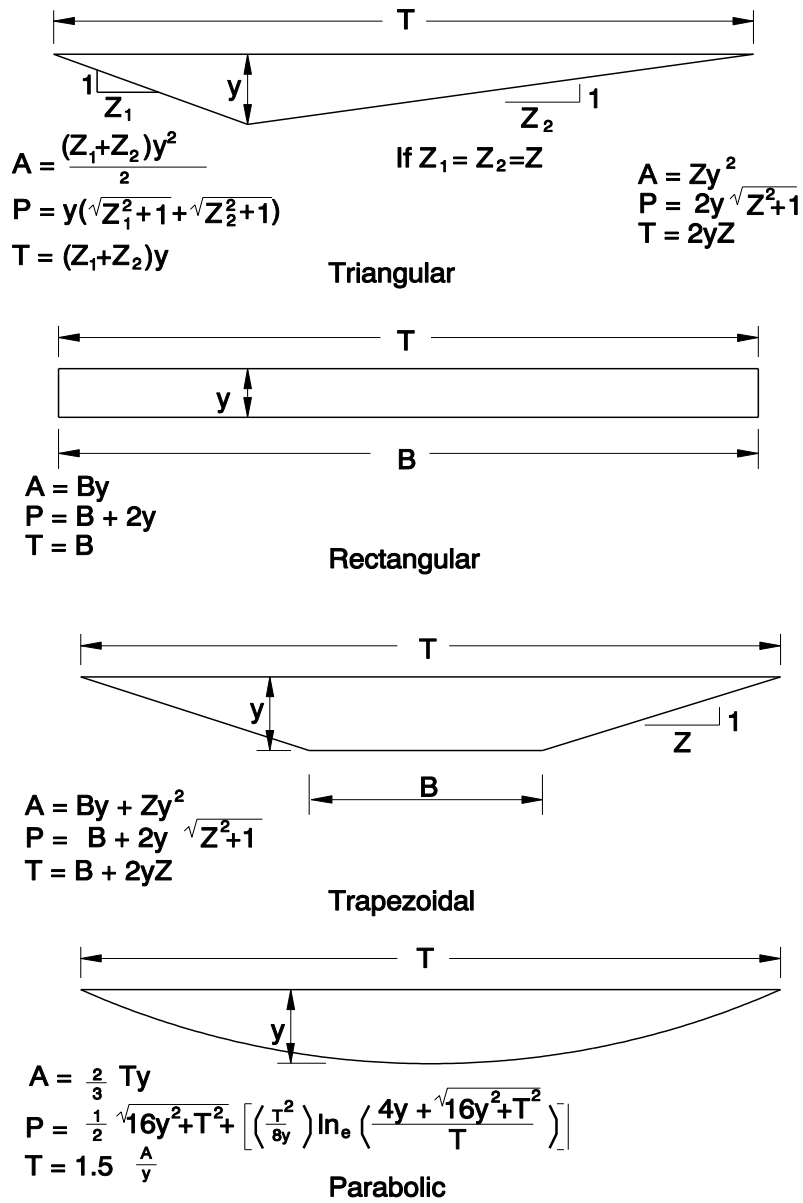
8.15.1 Artificial Channel Hydraulics

Artificial channels are composed of prismatic shapes where uniform or nearly uniform flow occurs. They also have transition sections where the cross-sections, profiles, surface roughness, or other properties change, and non-uniform or turbulent flow occurs.

Channel sections with non-uniform flow can be analyzed using many methods. Regardless of the procedure, transition and other energy losses need to be considered. The FHWA WSPRO module in HYDRAIN or the US Corps of Engineers HEC-RAS programs are often used.

Hydraulic characteristics of prismatic channel sections with uniform or nearly uniform flow are typically calculated by single section analysis using versions of Manning's equation (Equations 8-12 and 8-15). These procedures are discussed in this section. Single section analysis can also be done using the HYCHL module in the FHWA HYDRAIN software package, or the FHWA VISUAL URBAN program.

Versions of Manning's equation relate the average flow velocity or discharge to channel properties, such as cross-sectional flow area (A), wetted perimeter length (P), energy gradeline slope (S), and the hydraulic roughness (n). Cross-sections of triangular, rectangular, trapezoidal, and parabolic channels are shown in Figure 8-22 with formulae to calculate A, P, and the top width of the flow (T). It is usually assumed S is equal to the slope of the channel bottom if uniform or nearly uniform flow. Artificial channel Manning's n values are listed in Appendix A.

Flow depths, discharges, and other hydraulic properties of rectangular or trapezoidal channels can be estimated using the nomograph in Figure 8-23. Hydraulic properties of triangular channels can be analyzed using the procedures for gutters in Section 8.13. Nomographs for many channel shapes are in the FHWA HDS #3 publication mentioned at the beginning of Section 8.14.

Triangular

$$A = \frac{(Z_1+Z_2)y^2}{2}$$
$$P = y(\sqrt{Z_1^2+1}+\sqrt{Z_2^2+1})$$
$$T = (Z_1+Z_2)y$$

If $Z_1 = Z_2 = Z$

$$A = Zy^2$$
$$P = 2y\sqrt{Z^2+1}$$
$$T = 2yZ$$

Rectangular

$$A = By$$
$$P = B + 2y$$
$$T = B$$

Trapezoidal

$$A = By + Zy^2$$
$$P = B + 2y\sqrt{Z^2+1}$$
$$T = B + 2yZ$$

Parabolic

$$A = \frac{2}{3}Ty$$
$$P = \frac{1}{2}\sqrt{16y^2+T^2}+\left[\left(\frac{T^2}{8y}\right)\ln_e\left(\frac{4y+\sqrt{16y^2+T^2}}{T}\right)\right]$$
$$T = 1.5\frac{A}{y}$$

A = Area of flow in square feet
P = Wetted perimeter in feet
T = Top width in feet
B = Bottom width in feet
y = Depth of flow in feet

Figure 8-22   Channel Geometries

8.15.2 Grassed Channel Linings

Grassed channel Manning's n values are related to the retardance class of the lining, the length of the hydraulic radius, and the channel slope, as shown in Figures 8-24 through 8-28. The retardance classification of various grass linings are summarized in Table 8-2. Retardance is related to the type of grass in the lining and whether it is cut or uncut. In general, cut grass has more retardance than uncut grass of the same height.

Information about the height of the grass before and after mowing can be provided by maintenance personnel, and a description of the grass type can be provided by the designer. The height of uncut grass can be estimated by measuring the height of similar types of uncut grasses in the project vicinity. Typically, the height of the grass during the season of peak runoff is used in the channel design.

Table 8-2 Retardance Classes of Grass Channel Linings
(Adapted from FHWA HEC-15)

Retardance Class	Lining
A	Uncut grass, excellent stand, average of 30 to 36 inches tall.
B	Uncut grass, good stand, average of 11 to 24 inches tall.
C	Uncut grass, fair stand, average of 10 to 48 inches tall.
C	Uncut grass, good stand, average of 6 to 18 inches tall.
C	Cut grass, good stand, average of 6 inches tall.
D	Uncut grass, good to excellent stand, average of 3.6 to 6 inches tall.
D	Cut grass, good to very good stand, 2.4 inches tall.
E	Cut grass, good stand, average of 1.5 inches tall.

Note: Retardance classes are for green and generally uniform cover with flows of 50 cubic feet per second or less. Shorter heights within each retardance class apply to denser grass stands or ground covers.

Example 1:

$$\frac{y}{B} = \frac{1.1}{3} = 0.37$$

Given: y = 1.1 ft
       n = 0.032
       B = 3 ft
       S = 0.05 ft/ft
       Z = 2
Need: Q

From Figure: $\frac{(Q)(n)}{(B^{8/3})(S^{1/2})} = 0.375$

$Q = 0.375 \ \frac{(B^{8/3})(S^{1/2})}{(n)} \quad 0.375 \ \frac{(3^{8/3})(0.05^{1/2})}{(0.032)}$

Q = 49 cfs ←

Figure 8-23   Rectangular and Trapezoidal Channel Capacity Nomograph
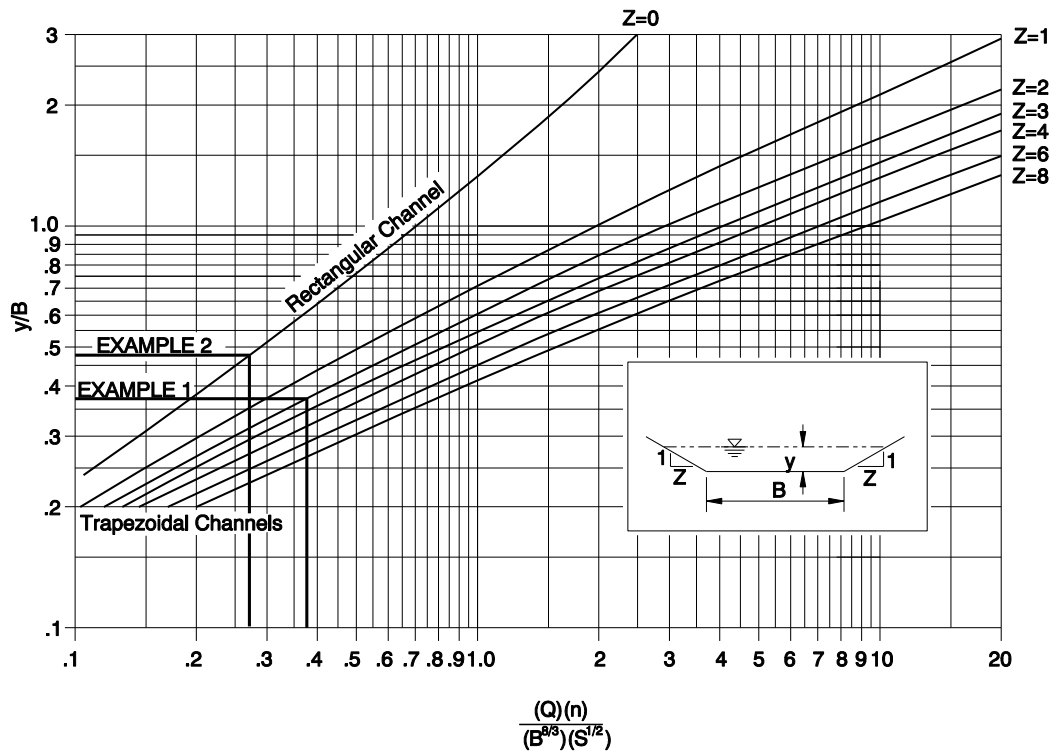
Example 2:

Given: Q = 120 cfs
      n = 0.015
      B = 6 ft
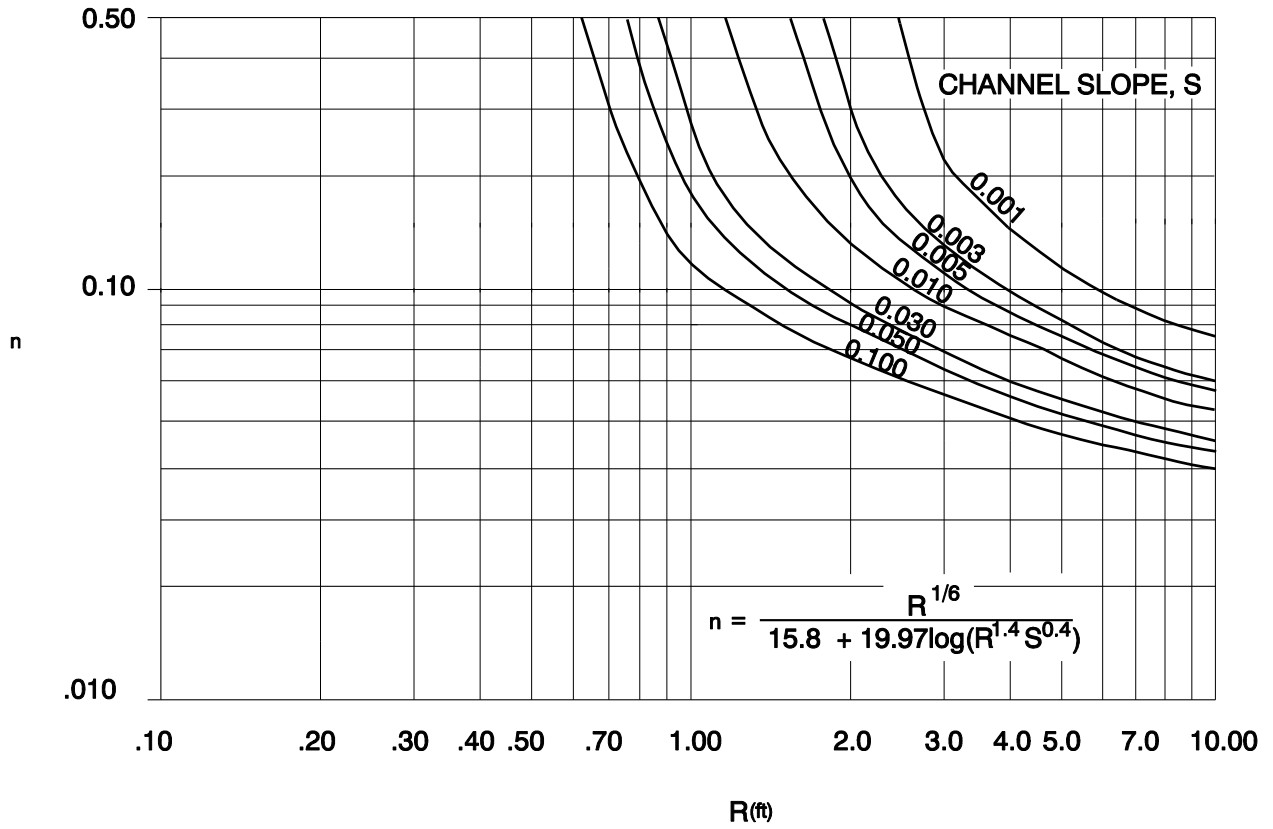   S = 0.003 ft/ft
      Z = 0
Need: y

Q = discharge in cubic feet per second
n = Manning's roughness coefficient
B = channel bottom width in feet
y = flow depth in feet
S = channel slope in feet per foot

$$\frac{(Q)(n)}{(B^{8/3})(S^{1/2})} = \frac{(120)(0.015)}{(6^{8/3})(0.003^{1/2})} = .28$$

From Figure, y/B = 0.48

y = (y/B)(B) = (0.48)(6) = 2.9 ft ←

The graph shows the equation:

$$n = \frac{R^{1/6}}{15.8 + 19.97\log(R^{1.4}\,S^{0.4})}$$

with channel slope S curves labeled 0.001, 0.003, 0.005, 0.010, 0.030, 0.050, 0.100.

n = Manning's roughness coefficient
R = hydraulic radius in feet
S = channel slope in feet per foot

Figure 8-24   Manning's n Versus Hydraulic Radius, R, for Class A Vegetation
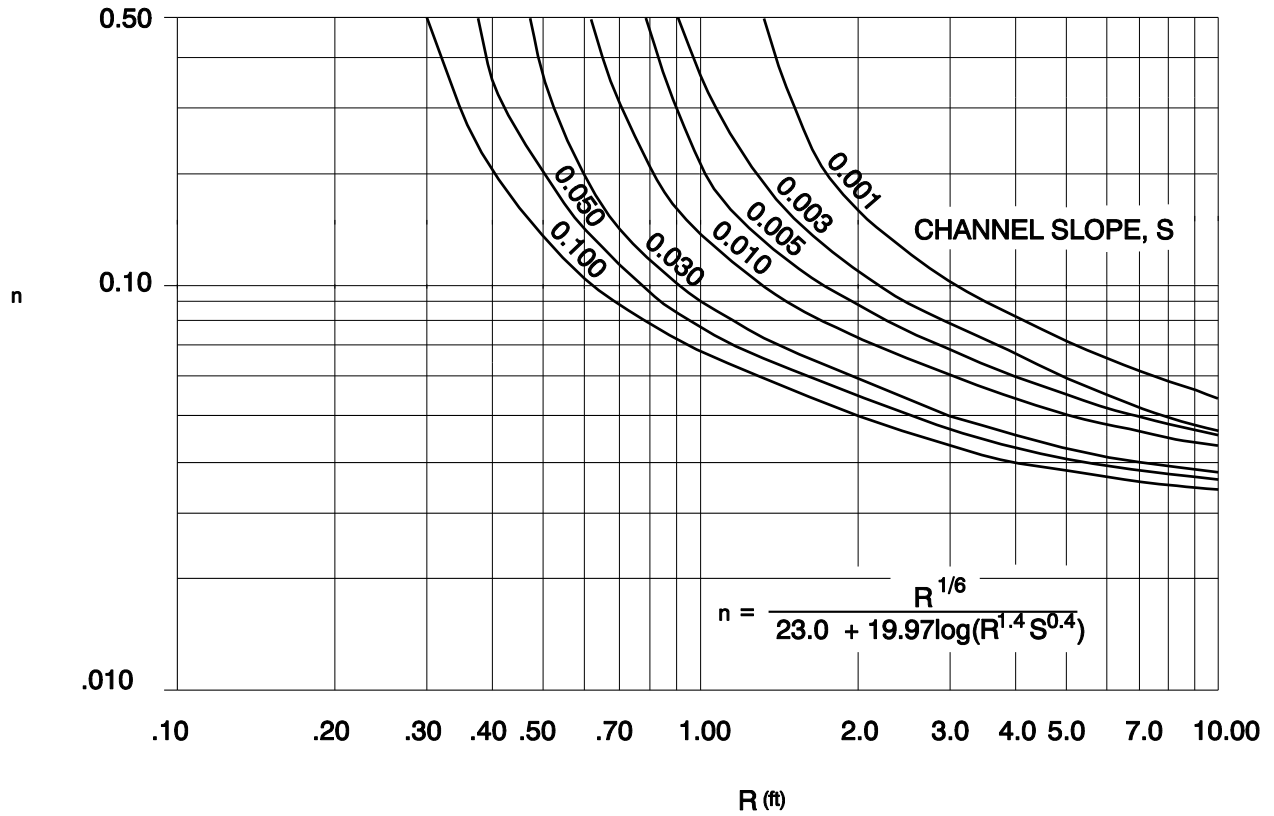
$$n = \frac{R^{1/6}}{23.0 + 19.97\log(R^{1.4} S^{0.4})}$$

n = Manning's roughness coefficient
R = hydraulic radius in feet
S = channel slope in feet per foot

Figure 8-25   Manning's n Versus Hydraulic Radius, R, for Class B Vegetation

The chart shows Manning's n (vertical axis, from .010 to 0.50) versus R (ft) (horizontal axis, from .10 to 10.00). Curves are labeled for CHANNEL SLOPE, S with values: 0.100, 0.050, 0.030, 0.010, 0.005, 0.003, 0.001.

Example: n = 0.073, R = 0.92 feet

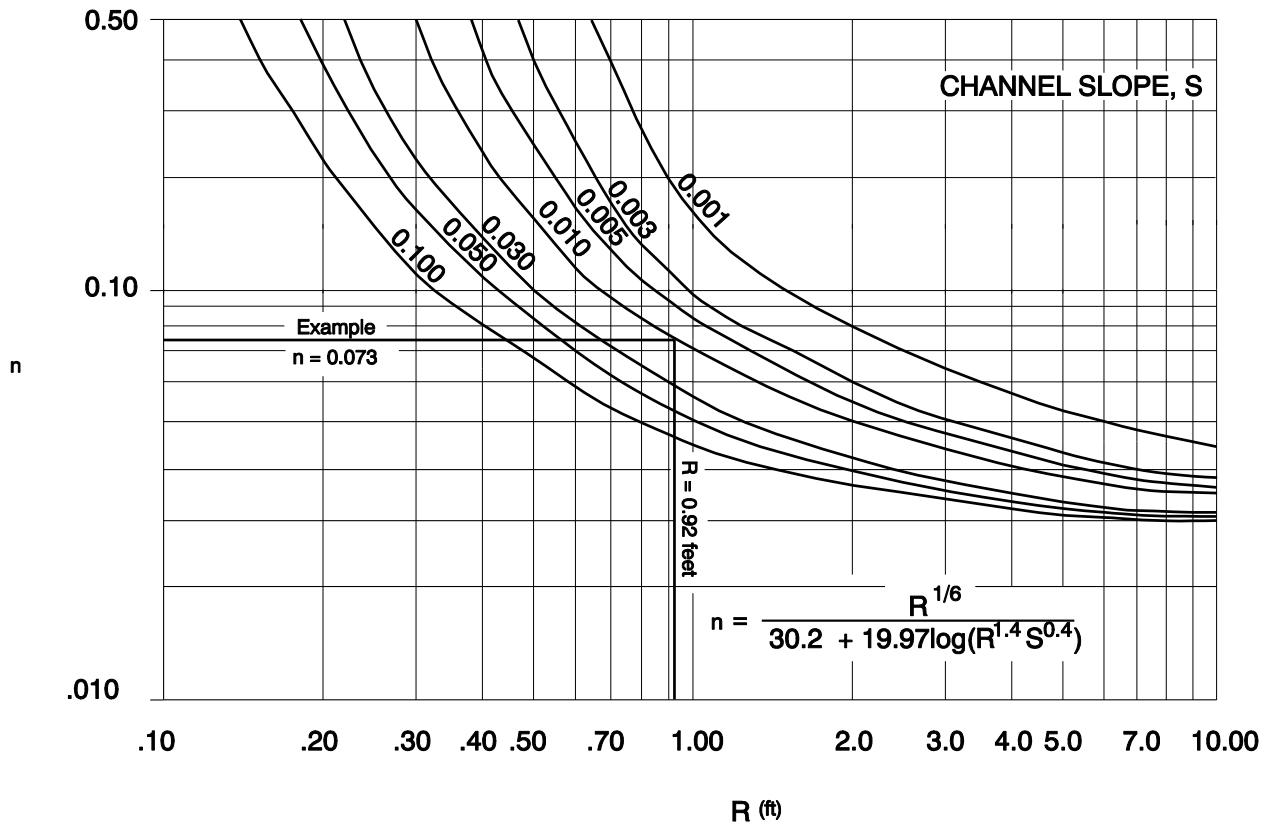$$n = \frac{R^{1/6}}{30.2 + 19.97\log(R^{1.4}\,S^{0.4})}$$

n = Manning's roughness coefficient
R = hydraulic radius in feet
S = channel slope in feet per foot

Figure 8-26   Manning's n Versus Hydraulic Radius, R, for Class C Vegetation

$$n = \frac{R^{1/6}}{34.6 + 19.97\log(R^{1.4} S^{0.4})}$$

n = Manning's roughness coefficient
R = hydraulic radius in feet
S = channel slope in feet per foot

Figure 8-27   Manning's n Versus Hydraulic Radius, R, for Class D Vegetation

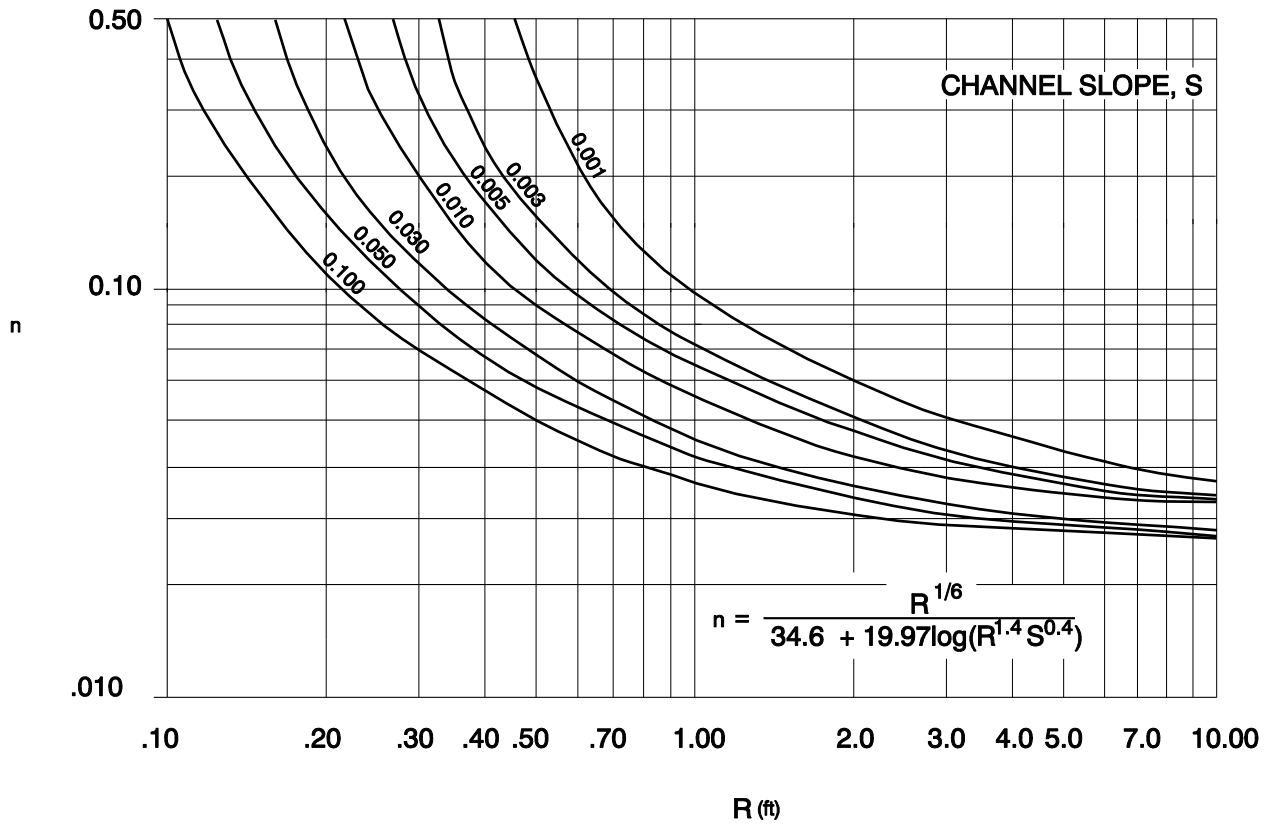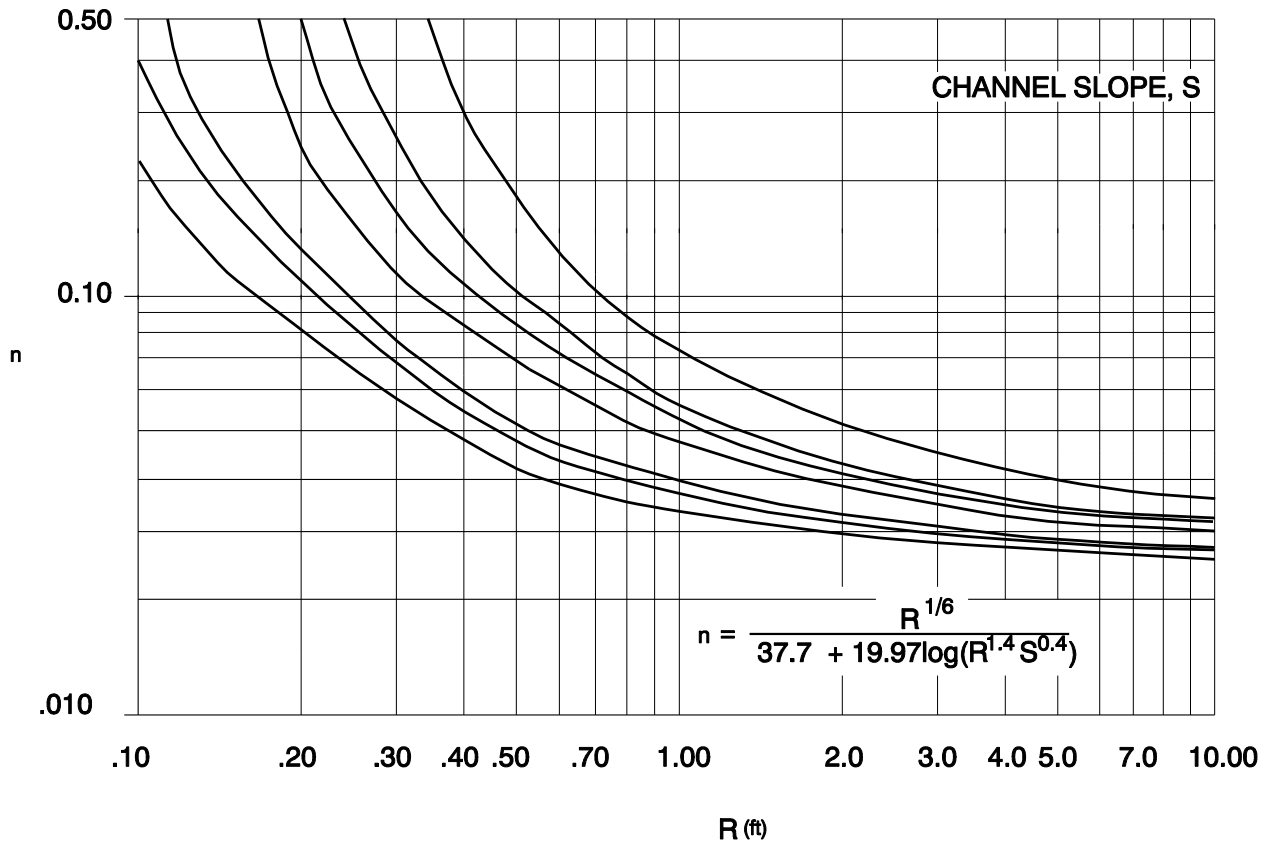$$n = \frac{R^{1/6}}{37.7 + 19.97\log(R^{1.4}S^{0.4})}$$

n = Manning's roughness coefficient
R = hydraulic radius in feet
S = channel slope in feet per foot

Figure 8-28   Manning's n Versus Hydraulic Radius, R, for Class E Vegetation

8.15.2.1   Example Problem - Artificial Channel Hydraulics

The hydraulic characteristics of a trapezoidal channel with two different linings are determined in this example. The linings are:

- ODOT Class 50 loose riprap, and
- a good stand of mowed grass, 6 inches tall after mowing and 10 inches tall before mowing.

Given:

$\quad$ S = channel slope = 0.01 foot per foot

$\quad$ B = channel bottom width = 2.6 feet

$\quad$ Z = 3 for a 1V : 3H side slope

$\quad$ y = maximum allowable flow depth = 1.6 feet

$\quad$ n = 0.070 (see Appendix A Table 3)

Find:

$\quad$ Q = channel capacity

$\quad$ V = average flow velocity

Solution 1:   Riprap Lining

Step 1 - Calculate and determine input variables for Manning's and continuity equations (Equations 8-12 and 8-11). Geometric formulae for trapezoidal channels are shown in Figure 8-22. Input to Manning's equation is:

$$A = By + Zy^2 = (2.6)(1.6) + (3)(1.6^2) = 12 \text{ square feet}$$

$$P = B + 2y(Z^2 + 1)^{1/2} = 2.6 + (2)(1.6)[(3^2 + 1)^{1/2}] = 13 \text{ feet}$$

$$R = \frac{A}{P} = \frac{12}{13} = 0.92 \text{ feet}$$

Step 2 - Calculate average velocity based on Manning's equation.

$$V = \left(\frac{1.486}{n}\right) R^{2/3} S^{1/2} = \left(\frac{1.486}{0.07}\right)(0.92)^{2/3}(0.01)^{1/2} = 2.0 \text{ feet per second} \leftarrow \text{ Answer}$$

Step 3 - Discharge is calculated using continuity equation as follows:

$$Q = (V)(A) = (2.0)(12) = 24 \text{ cubic feet per second} \leftarrow \text{ Answer}$$

Solution 2: Grass Lining

Step 1 - Determine channel roughness (n). Channel roughness is related to retardance of the grass lining. Maintenance forces say the grass will not be mowed during the wet season so hydraulic properties will be based on the "before mowing" grass height of 10 inches. Based on Table 8-2, this grass is in Retardance Class C. S = 0.01 foot per foot, R = 0.92 foot, and A = 12 square feet (see Solution 1). As shown in Figure 8-26, n = 0.073.

Step 2 - The nomograph in Figure 8-23 will be used to determine discharge. An example similar to this problem is shown on the Figure.

$$\frac{y}{B} = \frac{1.6}{2.6} = 0.61$$

From Figure 8 - 23, for $\frac{y}{B} = 0.61$ and Z = 3 :

$$1.35 = \frac{Qn}{B^{8/3}\,S^{1/2}}$$

$$\therefore \quad Q = \frac{(1.35)(B^{8/3})(S^{1/2})}{n}$$

$$Q = \frac{(1.35)(2.6^{8/3})(0.01^{1/2})}{0.073} = 24 \text{ cubic feet per second} \leftarrow \text{ Answer}$$

Step 3 - The continuity equation will be used to determine velocity.

$$V = \frac{Q}{A} = \frac{24}{12} = 2.0 \text{ feet per second} \leftarrow \text{ Answer}$$

8.15.3 Flow in Bends

The water surface elevation at the outside of a channel bend is higher than it is on the inside of the bend. This superelevation should be considered in channel design to assure there is adequate freeboard. The difference in water surface elevations between the outside and inside of a bend can be calculated by the following formula:

$$\Delta y = \frac{(V^2)(T)}{(g)(R_C)}$$
(Equation 8-51)

Where:

Δy = difference in water surface elevation between the inner and outer banks of the channel in the bend in feet

V = average velocity in feet per second

T = surface width of the channel in feet

g   = gravitational acceleration (32.2 feet per second squared)
R_c = radius to the centerline of the channel in feet

This equation is valid for subcritical flow. The water surface elevations at the outside and inside of the bend are $\Delta y/2$ higher and lower, respectively, than the water surface at centerline.

Changes in water surface elevation around bends during supercritical flow are difficult to predict by equations. Often it is necessary to construct a model of the design, route water through it, and measure the differences in water surface elevations. The modeling results are used to predict the actual flow characteristics.

8.15.4 Shear Stresses on Channel Linings

Hydrodynamic forces created by water flowing in a channel cause shear stresses on the channel sides and bottom. The lining will be displaced and erosion may occur if the hydrodynamic forces create shear stresses great enough to initiate movement of the lining material. Consequently, to assure the channel will be stable, the maximum shear stress caused by the design flow must be less than the maximum shear stress the lining can resist without particle movement. Maximum shear stress in a straight trapezoidal channel occurs near the center of the channel bottom, as follows:

$$\tau_y = \gamma yS \qquad\qquad\qquad\qquad\qquad\qquad\qquad\qquad \text{(Equation 8-52)}$$

Where:

τ_y = maximum shear stress on channel bottom in pounds per square foot
γ = unit weight of water (62.4 pounds per cubic foot)
y = maximum flow depth in feet
S = average bed or energy gradeline slope in feet per foot

Shear stresses on the channel sides are used to design channels with different materials on the sides and bottom. These stresses can be calculated using the shear stress ratio from Figure 8-29 and the following equation:

$$\tau_s = K_1 \tau_y \qquad\qquad\qquad\qquad\qquad\qquad\qquad\qquad \text{(Equation 8-53)}$$

Where:

τ_s = maximum shear stress on channel side in pounds per square foot
K_1 = ratio of maximum stress on channel side to maximum stress on channel bottom, as determined from Figure 8-29
τ_y = maximum shear stress on channel bottom, as determined from Equation 8-52
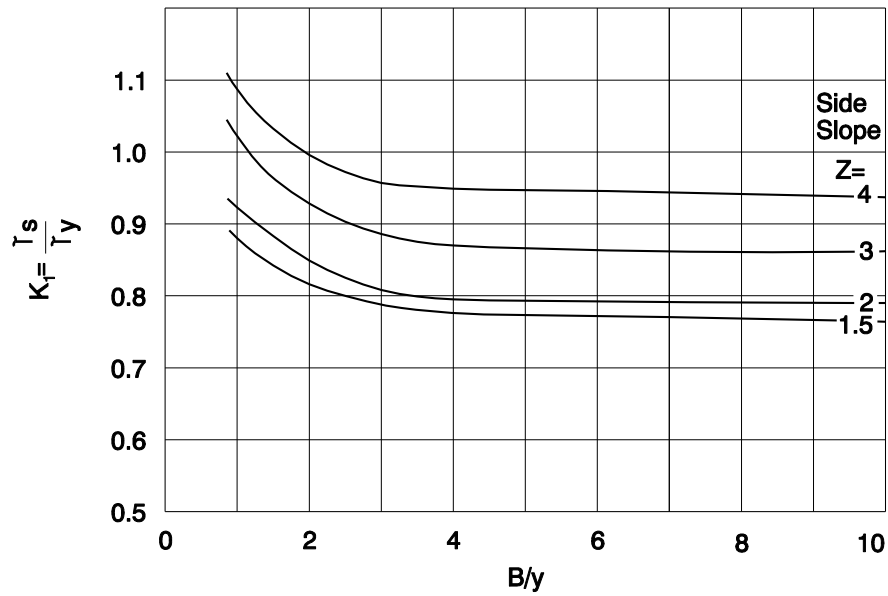
Υ_s = maximum shear stress on channel sides in pounds per square foot

Υ_y = maximum shear stress on channel bottom in pounds per square foot

K_1 = factor

B = channel bottom width in feet

y = depth of flow in feet
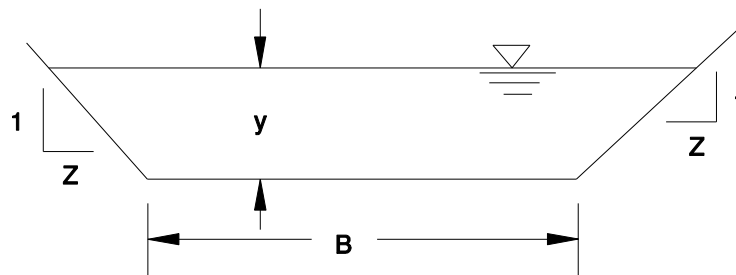
Z = channel side slope as shown

Figure 8-29   Ratio of Channel Side Shear Stress to Bottom Shear Stress, K_1

Flow in a channel with a bend creates a higher shear stress on the channel lining than the same flow in a straight channel. Areas of higher shear stress are shown in Figure 8-30. The increased shear stress persists downstream from the bend a distance L_p as shown in the figure, and L_p can be calculated as follows:

$$L_p = \frac{0.604\,R^{7/6}}{n_b}$$ (Equation 8-54)

Where:

L_p = length of increased shear stress downstream from the point of tangency (P.T) in feet
R = hydraulic radius in feet
n_b = Manning's roughness of the channel bend

The initial upstream point on the bend where shear stresses start to increase is difficult to estimate. Often it is assumed the point of tangency is the upstream limit of the area affected by increased scour. This is the shown on Figure 8-30.

The maximum shear stress due to the bend is related to the ratio of the radius of the bend to the bottom width of the channel, as shown in Figure 8-31. The maximum shear stress is calculated as follows:

$$\tau_b = K_b\,\tau_y$$ (Equation 8-55)

Where:

τ_b = maximum shear stress due to bend in pounds per square foot
K_b = function of R_c and B as shown in Figure 8-31
τ_y = maximum shear stress on channel bottom in pounds per square foot as calculated by Equation 8-52
R_c = radius of bend to the centerline of the channel in feet
B = bottom width of channel in feet

Note: In almost all situations, the greatest shear stresses are on the channel bottom rather than the channel sides, and τ_y more than τ_s. In some instances the greatest shear stresses are on the channel sides and , τ_s more than τ_y. In these cases, τ_s should be used in Equation 8-55 instead of τ_y.

8.15.4.1   Example Problem - Shear Stress in Straight and Bent Channels

Lining shear stresses need to be calculated for a prismatic trapezoidal channel with a straight section and a 90 degree bend having a centerline radius of 14.8 feet. The lining is a good stand of grass 6 inches (mowed) to 10 inches (unmowed) tall. This is the grass lined channel for the example problem in Subsection 8.15.2.1.

Given:

γ = unit weight of water (62.4 pounds per cubic foot)
y = maximum flow depth = 2.6 feet
S = channel slope = 0.01 foot per foot
R_C = radius of bend to the centerline of the channel = 14.8 feet
B = channel bottom width = 2.6 feet

Need:

τ_y = maximum shear stress on channel bottom in pounds per square foot
τ_b = maximum shear stress due to bend in pounds per square foot

Step 1 - Calculate maximum shear stress on channel bottom in straight section using Equation 8-52

$$\tau_y = \gamma y S = (62.4)(2.6)(0.01) = 1.6 \text{ pounds per square foot} \leftarrow \text{Answer}$$

**Step 2 -** Compute maximum shear stress in the bend using Figures 8-30 and 8-31 with Equation 8-55.

$$\frac{R_C}{B} = \frac{14.8}{2.6} = 5.7 \quad \text{From Figure 8-31, } K_b = 1.45$$

$$\tau_b = K_b \, \tau_y = (1.45)(1.6) = 2.3 \text{ pounds per square foot} \leftarrow \text{Answer}$$
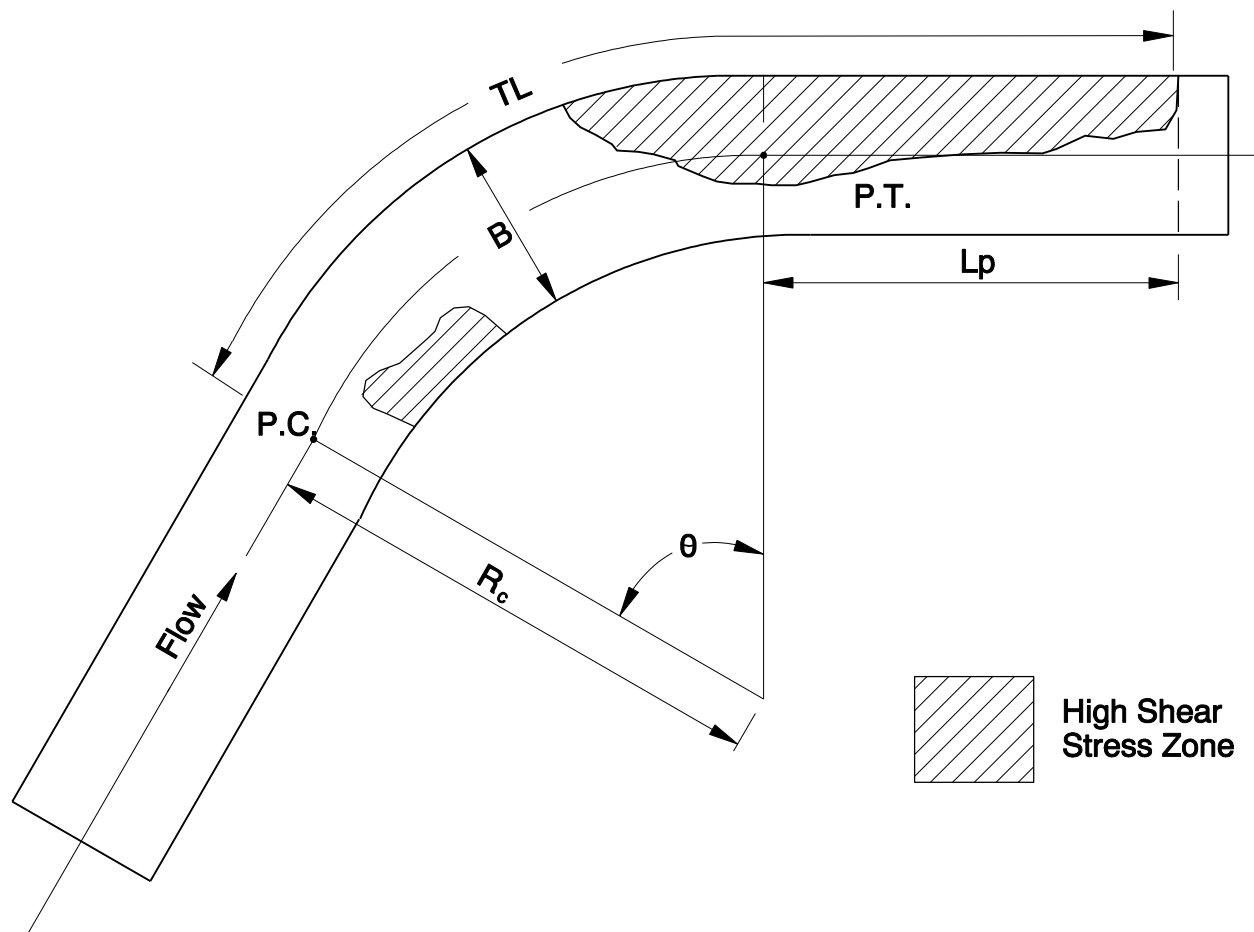
TL   = Total channel length subject to increased shear
Lp   = Length of increased shear stress downstream
         from the point of tangency in feet
P.C. = Point of curvature
P.T. = Point of tangency
R c   = Radius of bend in feet
B   = Bottom width of channel in feet
θ   = Bend angle

Figure 8-30   Shear Stress Distribution in Channel Bend

$$\Upsilon_{b} = K_{b}\Upsilon_{y}$$
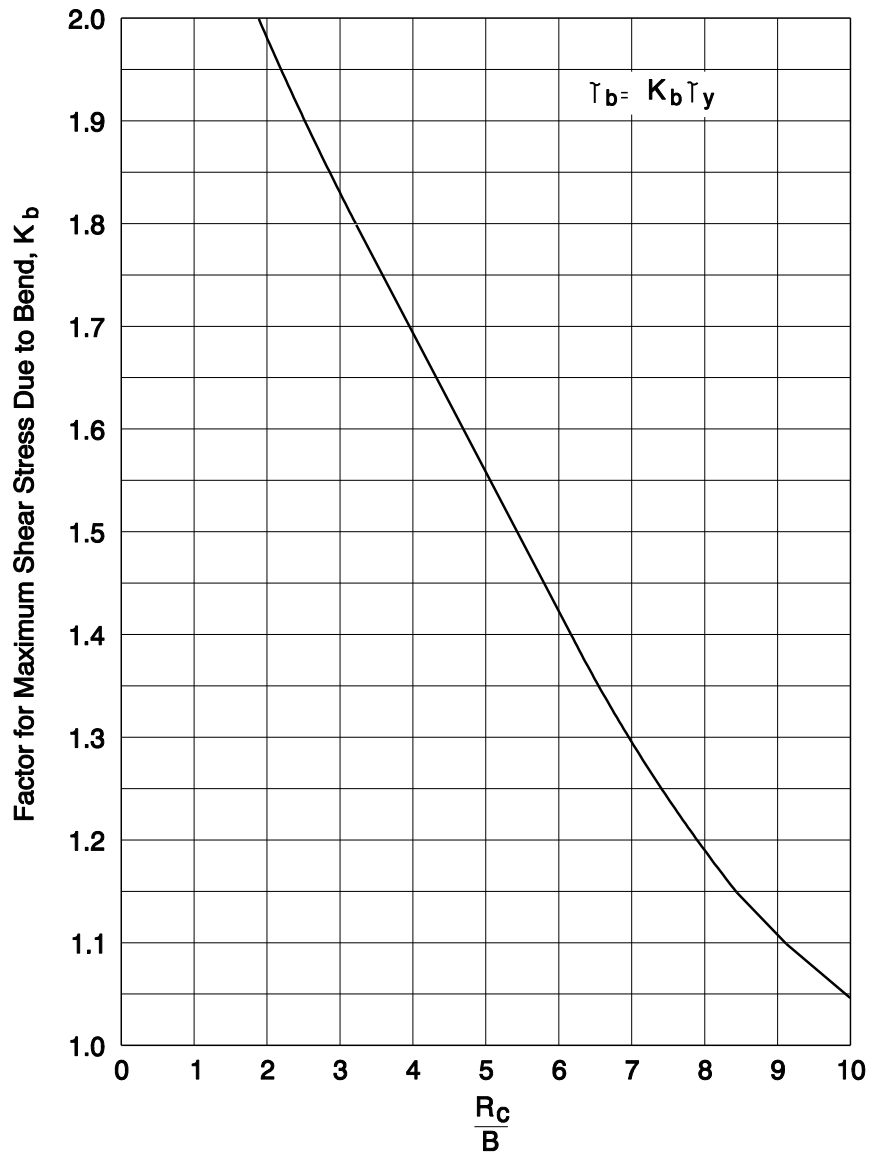
Υ_{b} = maximum shear stress due to channel bend in pounds per square foot

Υ_{y} = maximum shear stress on bottom of straight channel in pounds per square foot

K_{b} = factor ($\Upsilon b/\Upsilon y$) to account for increased shear in bend

R_{c} = radius of curvature of channel centerline in feet

B = channel bottom width in feet

Figure 8-31 K_b Factor for Maximum Shear Stress Due to Channel Bend

8.15.5 Permissible Shear Stresses on Channel Linings

Sometimes a channel does not have a lining and the flow contacts bare soil. This is acceptable if some erosion can be tolerated and the flow velocities are low. The permissible shear stresses for non-cohesive soils are related to the mean diameter of the soil particles, as shown in Figure 8-32. The permissible shear stresses of cohesive soils are related to the Plasticity Index (P.I.), as shown in Figure 8-33. Soil particle size and plasticity information is available from soil tests. These properties can be estimated for native soils using the Engineering Index Property tables in the applicable National Resources Conservation Service (NRCS) Soil Survey. These surveys are available for most counties and populated areas. They are published by the NRCS and available in many libraries. They are also accessible from the NRCS website: http://www.or.nrcs.usda.gov/pnw_soil/index.html

Temporary channel linings are typically designed to withstand shear during the 2-year event, as discussed in Section 8.3.2. They are intended to provide scour and erosion protection until vegetation is established on the channel sides and bottom. Many of these temporary linings, such as straw with net, are truly temporary because they decompose quickly as the underlaying vegetation becomes established. Other linings, such as some types of plastic and fiberglass roving, are semi-permanent. They do not decompose quickly, and they continue to add strength to the channel lining for many years after the vegetation is established.
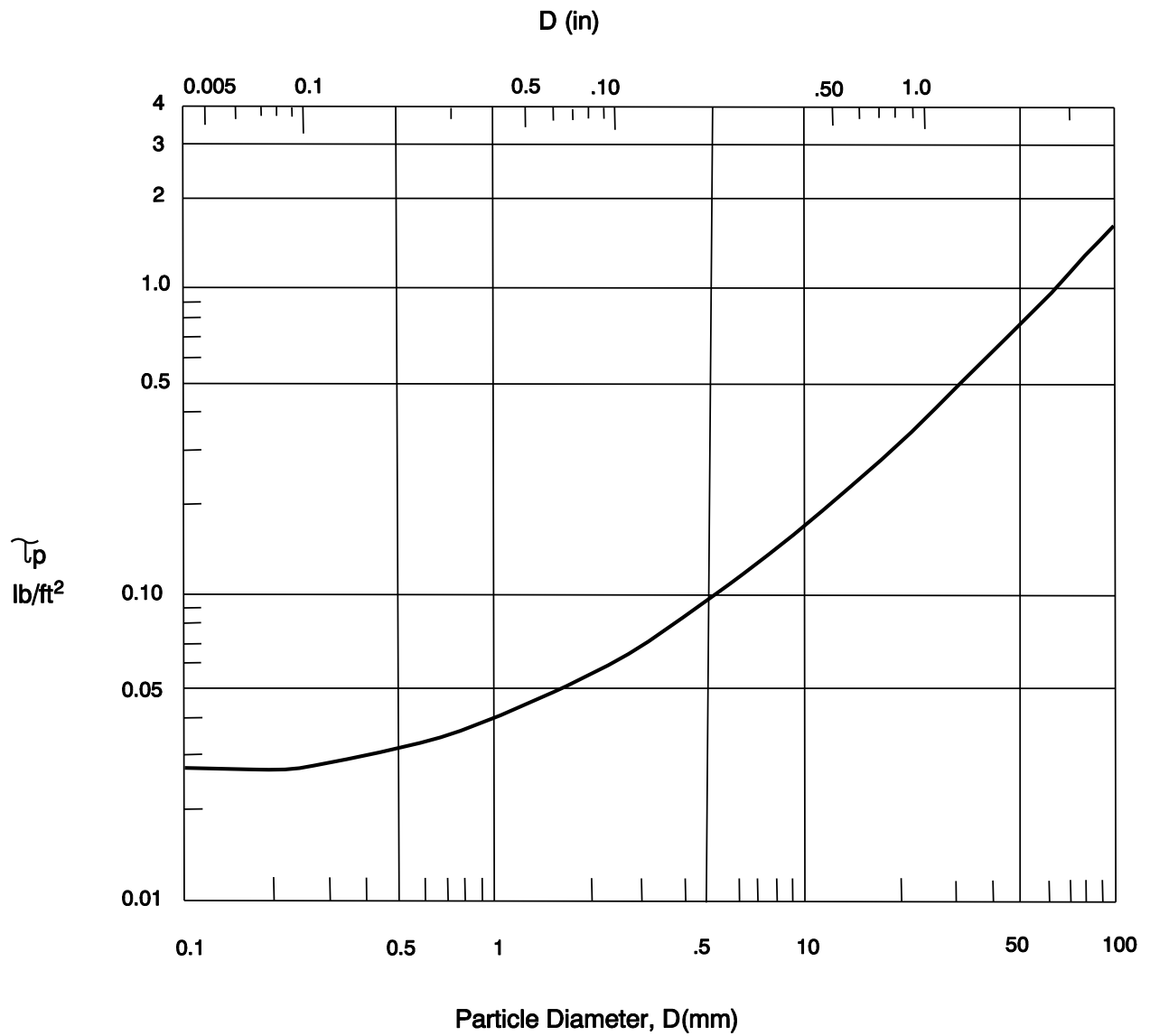
D (in)

Particle Diameter, D(mm)

Figure 8-32   Permissible Shear Stress for Non-Cohesive Soils
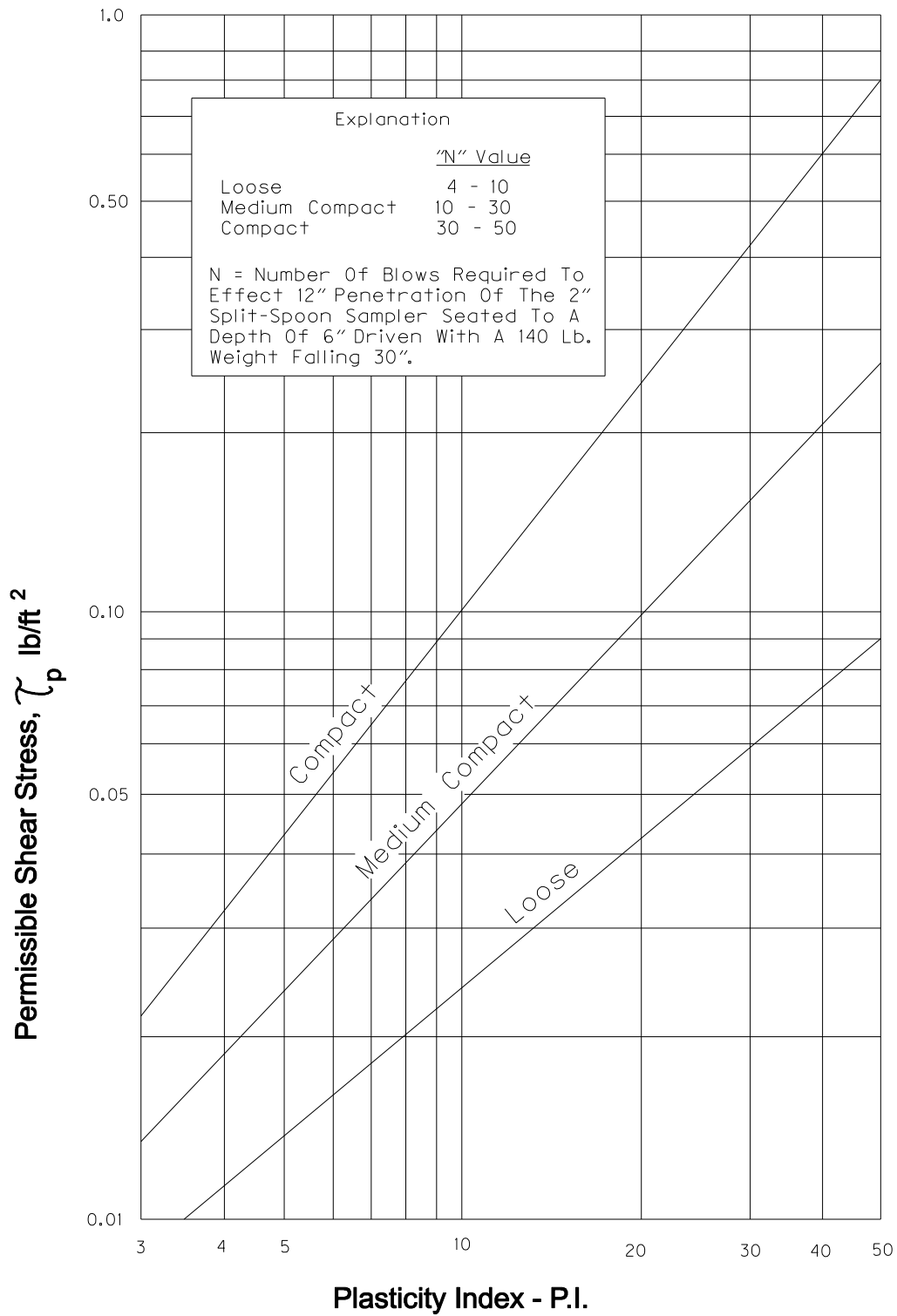
Figure 8-33 Permissible Shear Stress for Cohesive Soils

A more durable temporary lining may be desirable in some circumstances to provide erosion

protection if the vegetal lining fails or does not become fully established. Examples are a location where a scour resistant vegetation layer may not be fully established for some time, or a critical installation where a lining failure could be expensive, endanger public safety, or harm adjacent property.

Permanent channels are typically lined with vegetation or rock riprap. A lining is selected which will withstand the design event, and different lining materials have varying abilities to resist hydraulic shear stresses. The maximum permissible unit shear stresses for many temporary and permanent linings are summarized in Table 8-3.

The shear stresses a grass lining can withstand are related to the retardance class of the grasses. Taller grasses that provide more retardance also withstand greater shear stresses. The retardance classes of cut and uncut grasses of various heights are listed in Table 8-2.

Riprap channel linings should use angular rock produced to ODOT standards. Channel banks as steep as 1-1/2H: 1 V are allowed if the underlying soils will be stable, unless other design standards require flatter side slopes. The riprap or large stone size needed to provide stability on the channel bottom is calculated as follows:

$$\left(D_{50}\right)_{bottom} = \frac{\tau_y}{4.0}$$ (Equation 8-56)

Where:

$(D_{50})_{bottom}$ = mean effective diameter of riprap on channel bottom in feet

τ_y = maximum shear stress on channel bottom using Equation 8-52 (for channel bends, use maximum shear stress on channel bottom in a bend, τ_b, using Equation 8-55)

The mean effective diameters of ODOT loose riprap are:

- Class 50 = 0.59 feet
- Class 100 = 0.71 feet
- Class 200 = 1.01 feet
- Class 700 = 1.45 feet
- Class 2000 = 2.06 feet

The maximum allowable shear stresses for ODOT riprap on the channel bottom are listed in Table 8-3.

Side slope stability must also be considered for gravel or riprap linings with side slopes steeper than 3 Horizontal : 1 Vertical. This analysis is done by comparing the shear force ratio of the channel sides and bottom to the tractive force ratio of the channel sides and bottom.

Table 8-3 Permissible Shear Stresses for Linings			
Lining Category	Lining Type	Underlying Soil	Permissible Shear Stress in Pounds per Square Foot
Bare Soil		Non-Cohesive	See Figure 8-33
		Cohesive	See Figure 8-34
Temporary*	Woven Paper Net		0.15
	Jute Net		0.45
	Fiberglass Roving:		
	Single		0.60
	Double		0.85
	Straw with Net		1.45
	Curled Wood Mat		1.55
	Synthetic Mat		2.00
Vegetal	Class A Retardance		3.7
	Class B Retardance		2.1
	Class C Retardance		1.0
	Class D Retardance		0.60
	Class E Retardance		0.35
Riprap	ODOT Class 50		2.36
	ODOT Class 100		2.84
	ODOT Class 200**		4.04
	ODOT Class 700**		5.80
	ODOT Class 2000**		8.24
Other	6-Inch Thick Gabions	***	35
	4-Inch Thick Geoweb	***	10
	Soil Cement (8% cement)	***	more than 45
	Dycel w/o Grass	***	more than 7.0
	Petraflex w/o Grass	***	more than 32
	Armorflex w/o Grass	***	12-20
	Erikamat w/3-in Asphalt	***	13-16
	Erikamat w/1-in Asphalt	***	less than 5
	Armorflex Class 30 with Longitudinal and Lateral Cables, No Grass	***	more than 34
	Dycell 100, Longitudinal Cables, Cells Filled with Mortar	***	less than 12
	Concrete Construction Blocks w/Granular Filter Underlayer	***	more than 20
	Wedge-shaped Blocks w/Drainage Slot	***	more than 25

*Some "temporary" linings become permanent when buried.
**Riprap backing required.
***Shear stress values are for linings on silty clay to silty sand (SC-SM) with AASHTO classification A-4(0).

The minimum riprap size needed to provide stability on channel sides is calculated as follows:

$$\left(D_{50}\right)_{sides} = \frac{\left(K_1\right)\left(D_{50}\right)_{bottom}}{K_2}$$ (Equation 8-57)

Where:

$(D_{50})_{sides}$ = minimum mean effective riprap diameter on channel sides in feet

K_1 = ratio of maximum shear stress on channel side to maximum shear stress channel bottom, as determined from Figure 8-29

$(D_{50})_{bottom}$ = minimum mean effective riprap diameter on channel bottom, in feet, from Equation 8-56

K_2 = ratio of tractive force on channel side to tractive force on channel bottom, as shown in Figure 8-34

8.15.6 Channel Lining Selection

Selection of an artificial channel lining involves many factors, such as hydraulic capacity, ease of maintenance, aesthetics, and most importantly, the ability of the lining to resist erosion. This subsection describes a method to select a lining to resist erosion. The channel lining should have a permissible shear stress (τ_p) which is higher than the shear stresses in the channel to prevent erosion. As applicable, the lining needs to resist shear stresses at the following locations:

- maximum shear stress on channel bottom (τ_y),
- maximum shear stress on channel side(τ_s), and
- maximum shear stress due to bends in the channel (τ_b).

8.15.6.1 Example Problem - Lining Selection

An ODOT Class 50 loose riprap lining is proposed for the prismatic channel with a bend. It must be verified whether or not this lining is adequate.

Given:

S = channel slope = 0.012 feet per foot

B = channel bottom width = 3 feet

Z = 2 for a 1V: 2H side slope

Q = design discharge = 18 cubic feet per second

R_C = channel radius of curvature = 25 feet

n = 0.070 (Table 3 of Appendix A)

γ = unit weight of water = 62.4 pounds per cubic foot

Find:

Minimum mean effective diameter (D_{50}) needed to resist displacement. D_{50}(minimum) must be less than 0.59 feet. This is the mean effective diameter for ODOT Class 50 loose riprap.

Step 1 - Determine maximum shear stress on channel bottom in straight section.

Using Figure 8-23:

$$\therefore y = 0.47B = (0.47)(3) = 1.4 \text{ feet}$$

From Figure with $Z = 2 : \dfrac{y}{B} = 0.47$

From Equation 8-52:

$$\frac{(Q)(n)}{(B^{8/3})(S^{1/2})} = \frac{(18)(0.070)}{(3^{8/3})(0.012^{1/2})} = 0.61$$

$$\tau_y = \gamma yS = (62.4)(1.4)(0.012) = 1.0 \text{ pounds per square foot} \leftarrow$$

Step 2 - Determine maximum shear stress on channel side in straight section.

Using Figure 8 - 29 with $Z = 2$, and $\dfrac{B}{y} = \dfrac{3}{1.4} = 2.1$:

$$K_1 = 0.88$$

Using Equation 8-53:

$$\tau_S = K_1\tau_y = (0.88)(1.0) = 0.88 \text{ pounds per square foot} \leftarrow$$

Step 3 - Determine maximum shear stress in bend. The channel bottom shear stress is greater than the channel side shear stress. The bottom shear stress will be used in the bend shear stress calculation. Using Figures 8-30 and 8-31 with Equation 8-55:

$$\frac{R_C}{B} = \frac{25}{3} = 8.3 \text{ feet} \quad \text{From Figure 8 - 31, } K_b = 1.6$$

$$\tau_b = K_b\tau_y = (1.6)(1.0) = 1.6 \text{ pounds per square foot} \leftarrow$$

Step 4 - Determine riprap size on channel bottom.

Using Equation 8-56 with $\tau_y = 1.6$ pounds per square foot (see Step 3):

$$(D_{50})_{bottom} = \frac{\tau_y}{4.0} = \frac{1.6}{4.0} = 0.40 \text{ feet} \leftarrow$$

Step 5 - Determine riprap size on channel sides.

$$\frac{B}{y} = \frac{3}{1.4} = 2.1 \text{ and } Z = 2. \text{ Using Figure 8 - 29 : } K_1 = 0.85$$

Using Figure 8-34 with Z = 2: $K_2 = 0.73$

Using Equation 8-57 with the channel bottom shear stress in the bend:

$$\left(D_{50}\right)_{sides} = \frac{(K_1)\left[(D_{50})_{bottom}\right]}{K_2} = \frac{(0.85)(0.40)}{0.73} = 0.47 \text{ feet } \leftarrow \text{ Answer}$$

The mean effective diameter needed to resist displacement is 0.47 feet. This is less than the 0.59 foot mean effective diameter of ODOT Class 50 loose riprap. Therefore, Class 50 riprap is verified to be adequate.

Side Slope Angle in Degrees
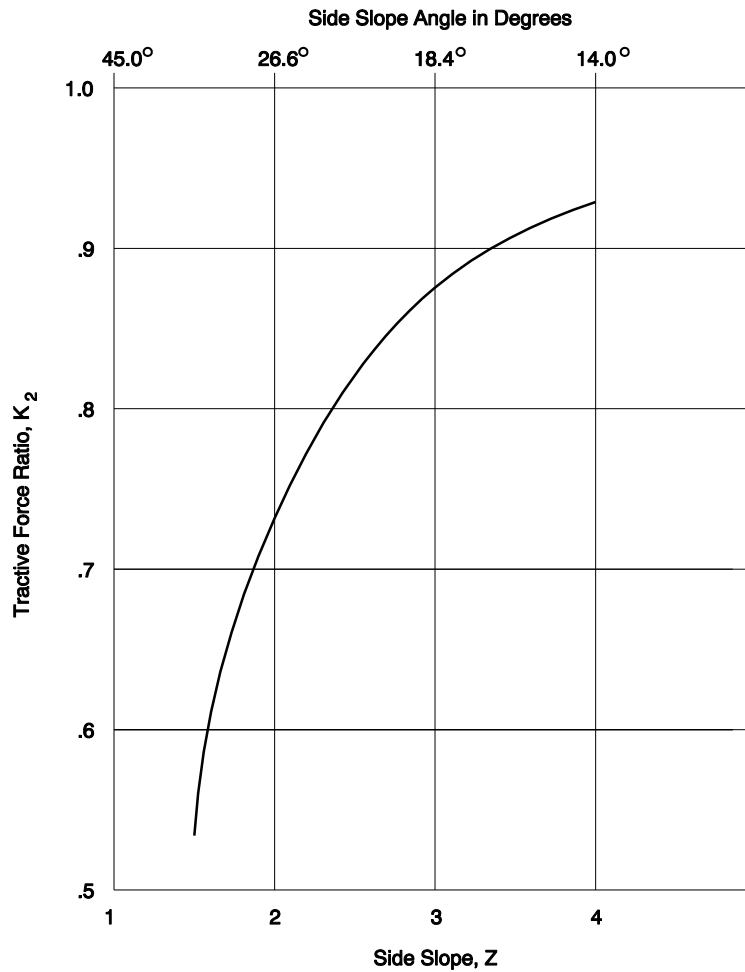
NOTE:
1) Chart valid for riprap that meets ODOT
   Specifications (angle of repose = 41°)
2) K_2 for Z = 1-1/2 is 0.53
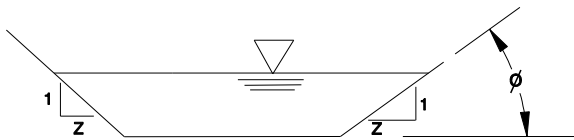
$$K_2 = \sqrt{1 - \frac{\sin^2\phi}{\sin^2\theta}}$$

K_2 = ratio of tractive force on channel side
       to tractive force on channel bottom
ϕ = angle of side slope as shown in degrees
θ = angle of repose in degrees (See Note 1)

Figure 8-34   Tractive Force Ratios for Channel Side Slopes

8.16   Changes in Channel Cross-Section and Slope

Changes in cross-section between upstream and downstream channels, where the slope does not vary, are often called transition sections. Changes in channel slope, or gradient, between upstream and downstream channels are often called slope or grade breaks. In some cases a channel can have a transition section and a slope break where both the cross-section geometry and slope change.

The hydraulic characteristics of flow, such as velocity and depth, are related to the slope and cross-section of the channel. As a result, there is always a change in flow characteristics at transition sections and slope breaks. Sudden changes in flow depth and velocity may occur, and these changes may be accompanied by turbulence. The channel design may need to consider the effects of turbulence in these areas. Extra freeboard may be needed to keep the water in the channel, and additional erosion protection may be needed to prevent scour.

Flow transitions are detected by determining the flow regimes in the upstream and downstream channel reaches, as follows:

Step 1 - Determine normal depths in the upstream and downstream channels using single-section analyses.

Step 2 - Calculate the Froude numbers for the channels to determine the flow regimes, either supercritical or subcritical.

Step 3 - Compare flow regimes. A hydraulic jump or drop will occur, and significant turbulence may result if the upstream and downstream Froude numbers are in different regimes. Lesser amounts of turbulence will occur if the upstream and downstream Froude numbers differ and are in the same regime.

Hydraulic jumps and drops can be avoided in some cases by designing the upstream channels and downstream channels to have Froude numbers as similar as possible and both within the same regime. Significant turbulence and resulting contraction or expansion head losses may be prevented in many cases by avoiding sudden transitions from one channel configuration to another. Limited experimental data suggests that a contraction with 30 degrees or less taper (measured between tapered face and stream centerline) is significantly more efficient than an abrupt (90 degree) transition. The same experimental data shows that an expansion with 2:1, 3:1 or 4:1 tapers are much more efficient than an abrupt transition, and the 4:1 taper is significantly more efficient than the 2:1 taper. (A 4:1 taper, for example, would occur when both sides of a 20 foot wide channel contract into a 10 foot wide channel within a 20 foot long transition section.)

Water surface profiles through simple transitions where the cross-section geometry changes and the channel slope is constant can be approximated by hand calculations using the energy

equations. This method is best suited for transitions where expansion and contraction losses predominate and boundary friction losses are minor. These calculations are described in detail in the FHWA Hydraulic Design Circular No 14 "Hydraulic Design of Energy Dissipators for Culverts and Channels." This publication is available from the FHWA website http://www.fhwa.dot.gov/engineering/hydraulics/library_arc.cfm?pub_number=13 This publication also discusses other aspects of transition section design, such as minimizing standing waves and scour and erosion protection.

Water surface profiles through long and gradual transitions must consider boundary friction losses. These sections, as well as more complex situations where there are channel slope breaks, are typically analyzed using a step-backwater computer program. Several programs are discussed previously in this chapter. Transition sections and slope breaks are discussed in many texts on hydraulics such as Ven Te Chow's "Open-Channel Hydraulics."

8.17 Channel Changes

Channel changes are sometimes required to reduce flood hazards to highways or to relocate channels displaced by the highway. A channel change should be avoided when possible without adding unreasonable costs to the project. Alternate solutions which minimize the disruption of the natural channel should be given thorough consideration.

Step-backwater modeling using computer programs such as the FHWA's WSPRO or the U.S. Corps of Engineers HEC-RAS is recommended for channel changes. Both the existing and proposed channels should be modeled. These analyses provide detailed information about velocities, water surface elevations, and energy, and they are a convenient means to compare the hydraulic aspects of channel modifications. The hydraulic modeling should include all proposed biological enhancements, such as fish rocks, woody debris, and vegetation.

Channel changes may have impacts which could damage fish habitat, reduce the quality of the stream side environment, and increase flow velocities as a result of reductions in channel roughness and an increase in channel slope. Disrupting channel stability may cause erosion or deposition problems after construction. These problems may occur for considerable distances upstream and downstream from the channel change. Other impacts from channel changes include the removal of vegetation from stream banks leaving raw cut-slopes that require riprap protection or concrete linings. These impacts usually reduce the aesthetics of a natural channel and may increase water temperature and flow velocities.

The following guidelines should be used when designing a channel change:

1.   Design the channel to be stable under design flow conditions. A riprap blanket will usually provide sufficient protection. Generally riprap protection will only be needed along the

outside bends of curves and along any highway embankments which form a channel bank. A sinuous channel typically requires more protection to be stable than does a straight channel.

2.   Contact the appropriate regulatory agencies such as the Oregon Department of Fish and Wildlife, National Marine Fisheries Service, Corps of Engineers, etc, early in the design. These agencies may recommend the appropriate enhancement features necessary to mitigate the undesirable effects of the channel change. Typical enhancement features are replanting streamside vegetation, placing fish rocks, and including vegetation and woody debris in the bank stabilization.

3.   Design a channel with curvature, and when feasible, attempt to match the meander pattern, stage-discharge relationship, slope, and cross-section of the existing channel. Field surveys are required to determine the cross-section of the existing channel and the slope of the natural stream. This survey data is also used to calculate the stage-discharge relationship for the existing channel.

4.   The channel change should maintain the energy conditions of the existing channel during the design event. The energy gradeline (EGL) and hydraulic gradeline (HGL) of the existing and realigned channels should be plotted and compared. The HGL and EGL of the original and realigned channels should match as closely as possible. The HGL shows the potential energy of the stream above a known datum plane and usually can be considered coincident with the water surface elevation at any given section. The EGL shows the total available energy of a stream above a known datum plane and is the sum of the potential energy and the kinetic energy.

As an example, if a channel change were designed for a small creek having a 500 cubic foot per second design flow, numerous trapezoidal sections could be found which convey this discharge. The proper size channel would maintain about the same energy relationships as the existing channel under design conditions.

8.17.1 Channel Change Example

This example illustrates a channel change designed by hand calculations. It is intended to illustrate the procedure. This channel change would normally be designed using a step-backwater hydraulic analysis. The modified channel would normally include extensive biological components. These are omitted from the example for clarity.

A highway project requires replacing a 225-foot long reach of DeLores Creek with a 230-foot long channel change and also requires an 8-foot extension to each end of the existing 7-foot span by 4-foot rise reinforced concrete box culvert (RCBC). The proposed channel change and RCBC lengthening are shown on Figure 8-35. The existing creek bottom (thalweg) and a water surface

profile 1000 feet upstream and 1000 feet downstream are shown on Figure 8-36. The water surface profile represents the 2 February 1989 flood. The largest known flood at the project site occurred during 22 December 1964. The December 1964 highwater elevation of 2947.47 feet (NGVD) was determined from information provided by Mr. John Oldtimer, telephone number (541) 468-1234, a 50-year resident of the area. The channel Manning's n value is estimated to be 0.04.

The channel change will be designed to convey the December 1964 flood and maintain the energy conditions of the existing channel. The peak discharge and average velocity of the 1964 flood will be estimated. The following procedure is used to design the channel change.

Step 1 - Draw a straight line through the channel bottom points at stations "D" 11 and "D" 26 and identify the elevations of these two points on this line as shown on Figure 8-37. This line should be approximately parallel to the water surface profile also shown on Figure 8-37. (It is recommended that the water surface profile be used to determine the slope for highly irregular channel bottom profiles.) The estimated mean slope is shown below:

$$\text{Mean Slope} = \frac{(2949.9 \text{ - } 2941.5)}{(26 \text{ - } 11)(100)} = 0.0056 \text{ feet per foot}$$

Step 2 - Draw a straight line through the December 1964 highwater mark which is parallel to the mean slope as shown on Figure 8-37.

Step 3 - Locate and identify on Figure 8-37 the downstream and upstream controls which will be used to determine the mean channel cross-section.

Step 4 - Sketch the mean cross-section as shown on Figure 8-37. The mean cross-section represents a cross-section at the control sections. This is also the recommended cross-section for the channel change. Energy conditions of the existing channel will be maintained since the channel will provide the same cross-section and slope as the existing channel.

Step 5 - Plot the channel change section on the plan view, as shown on Figures 8-35. Plot the typical channel change cross-section, as shown on Figure 8-38.

The estimated 1964 peak discharge and average velocity are calculated by Manning's equation for discharge and the continuity equation as shown below. These equations are presented in Section 8.6.

$$Q = \frac{1.486}{n} (A)(R^{2/3})(S^{1/2})$$

$$A = \left(\frac{7+7+3.4+3.4}{2}\right)(1.7) = 17.68 \text{ Square feet}$$

$$P = 7 + 2(1.7^2 + 3.4^2)^{1/2} = 14.60 \text{ feet}$$

$$R = \frac{A}{P} = \frac{17.68}{14.60} = 1.21 \text{ feet}$$

$$Q = \left(\frac{1.486}{0.04}\right)(17.68)(1.21^{2/3})(0.0056^{1/2}) = 56 \text{ cubic feet per second}$$

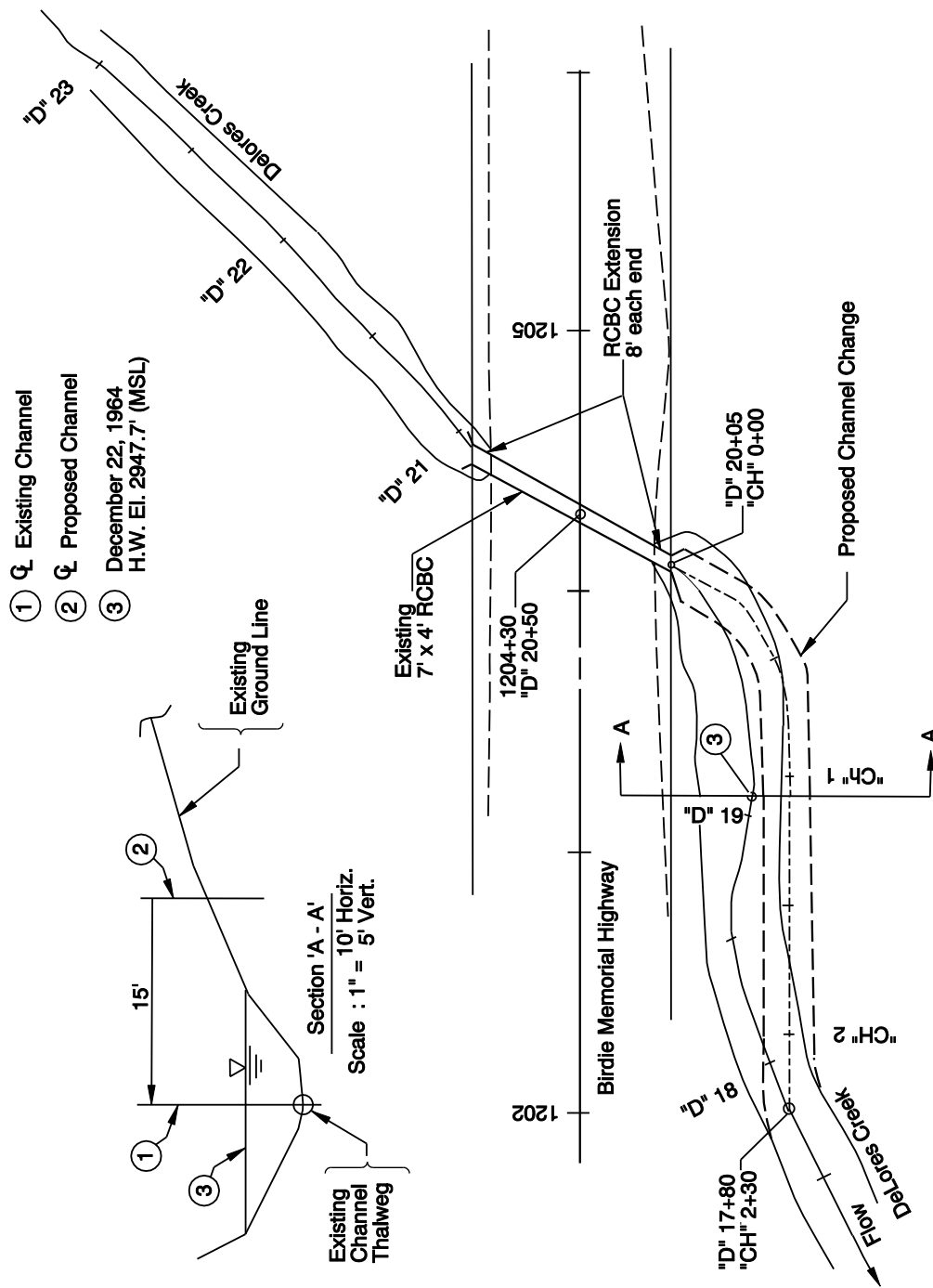$$V = \frac{Q}{A} = \frac{56}{17.68} = 3.2 \text{ feet per second}$$
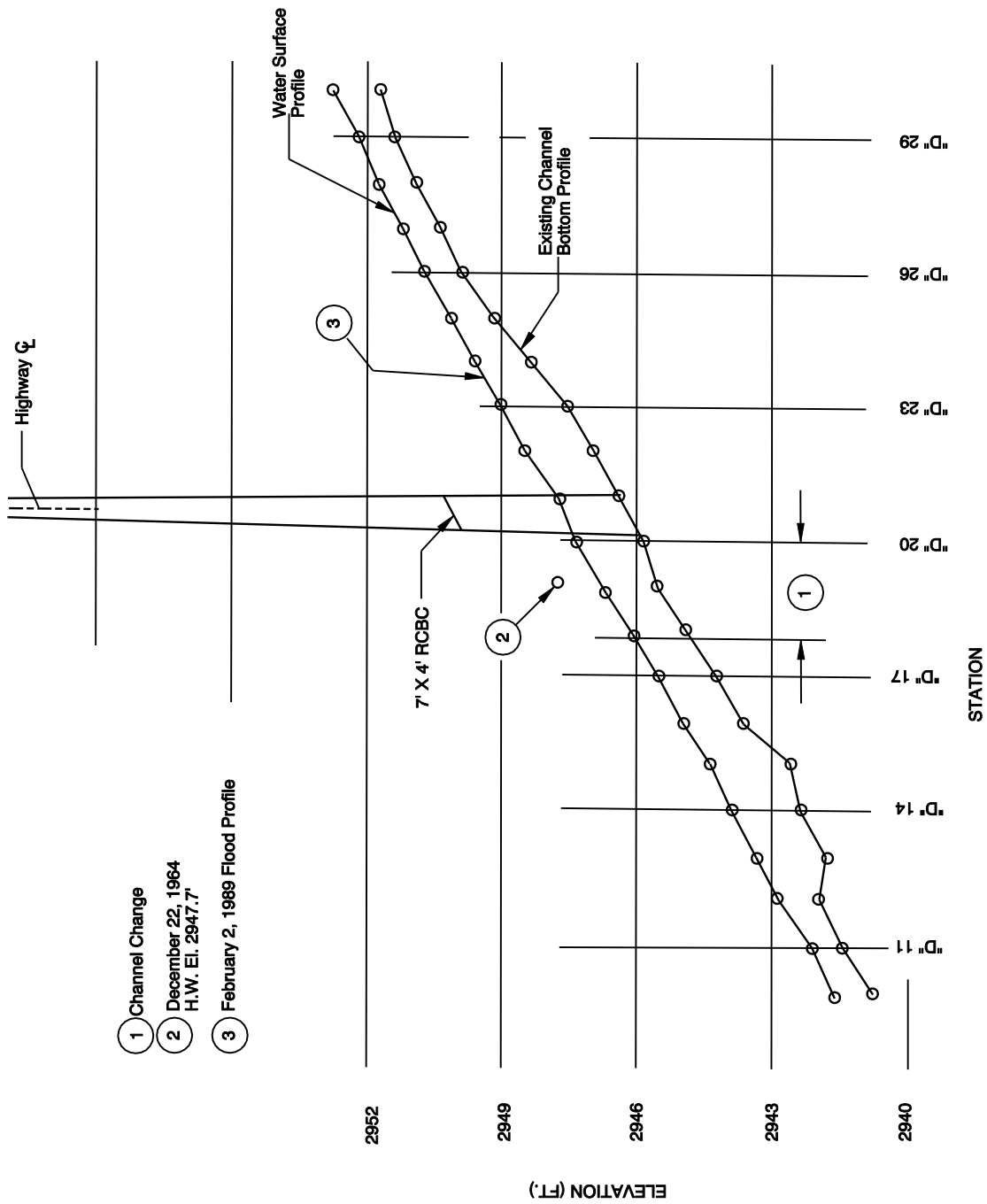
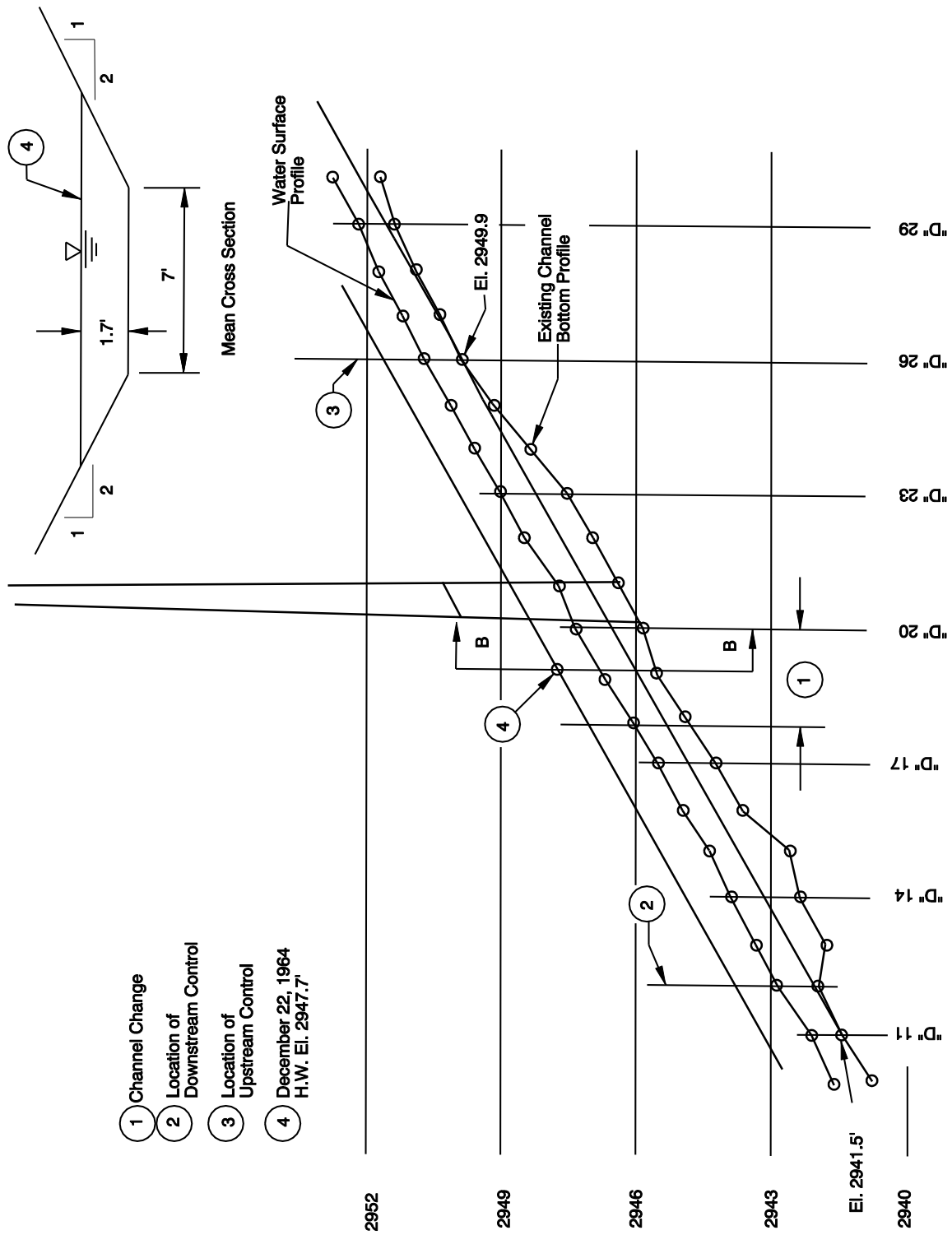**Figure 8-35   Plan View**

Figure 8-36   Existing Channel Profile
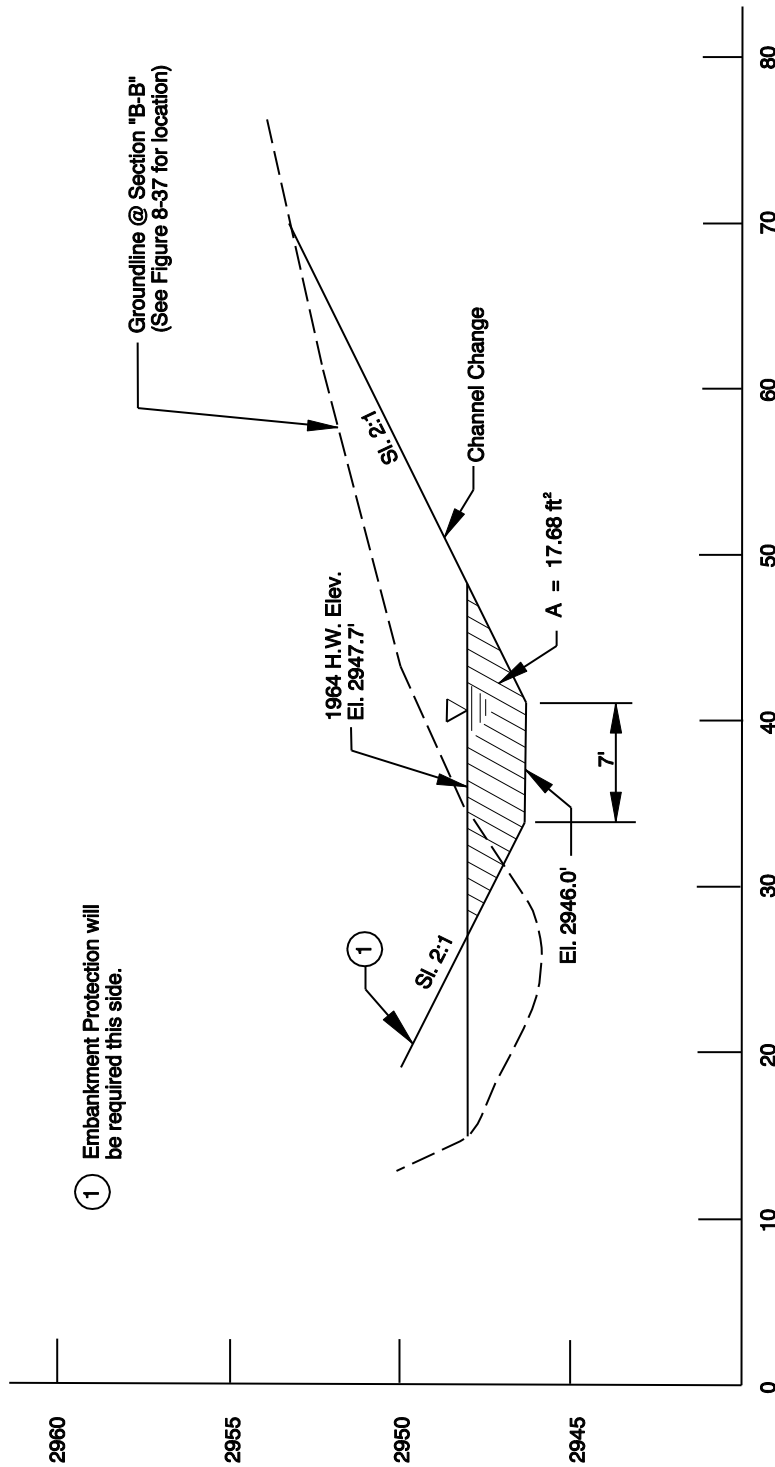
Figure 8-37   Proposed Channel Profile

Figure 8-38   Channel Change Cross-Section

8.18 Design Procedures

The design procedure for all types of channels has common elements as well as substantial differences. This section will outline a process for assessing a natural stream channel and a more specific design procedure for roadside channels.

8.18.1 Natural Stream Channel Procedure

A stream channel analysis is often part of a culvert or bridge hydraulic design. In general, the objective is to convey the water along or under the highway in an environmentally acceptable manner that will not cause damage to the highway, stream, or adjacent property. An assessment of the existing channel is usually necessary to determine the potential for problems that might result from a proposed action. The detail of the study should be commensurate with the risk associated with the action and with the environmental sensitivity of the stream and adjoining flood plain. The following step-by-step procedure outlines many of the steps in a typical modification of a natural channel or larger artificial channel.

Step 1 - Assemble available data (see **Chapter 6**) and design criteria (see **Chapter 3** and Subsection   8.3.1).

Step 2 - Determine the level of assessment. Information may be required to:

- show compliance with floodway and floodplain land use regulations,
- apply for permits,
- prepare a Hydraulic Report if a significant structure such as a bridge or culvert is involved,
- assure regulatory agencies that fish passage is provided, and
- provide needed data for designs. Generally, structures with the most potential for causing damage are studied in greater detail.

Step 3 - Determine type of hydraulic analysis. Examples are:

- Qualitative assessment. These are generally used for low-risk designs and preliminary estimates.
- Single-section analysis. Sometimes used for structures over prismatic channels such as irrigation canals where contraction scour does not need to be calculated.
- Step-backwater analysis. Used in most structural analyses.

Step 4 - Obtain additional survey information such as streambed profiles and cross-sections, elevations of flood-prone property, etc (see **Chapter 6**).

Step 5 - Determine hydrology (see **Chapter 7**).

Step 6 -   Perform hydraulic analysis.

- Select representative cross-section (single-section method) or cross-sections (step-backwater method).
- Select appropriate n values (both methods) and reach lengths (step-backwater method).
- Calculate energy slope (single-section method) or starting water surface slope or elevations (step-backwater method).
- Compute stage-discharge relationship (single-section method) or profiles (step-backwater method).
- Calibrate model if historical flows and corresponding high water elevations are known.

Step 7-   Perform stability analysis if needed. This is beyond the scope of this manual. A useful reference is the latest FHWA Hydraulic Engineering Circular Number 20 "Stream Stability at Highway Structures."

Step 8 -   Design channel change and/or bank protection.

- Calculate and compare hydraulic characteristics of existing and modified channels.
- Design bank protection. (see **Chapter 15**)

Step 9 -   Prepare report and file with background information (see **Chapter 4**).

8.18.2 Artificial Channel Procedure

An artificial channel is an open channel which typically conveys surface runoff through or from the highway right-of-way. A generalized procedure for the design of roadside and median channels follows. A discussion of design procedures including equations and examples is included in Section 8.15. Although many projects may require different tasks, many of the following design steps will be applicable.

Step 1 -   Assemble available data (see **Chapter 6**) and design discharge recurrence intervals (see **Chapter 3** and Subsection 8.4.2).

Step 2 -   Establish a preliminary drainage plan.

- Prepare plans and profiles of the existing and proposed channels. Include any constraints on design such as highway and road locations, right-of-way boundaries, culverts, utilities, etc. An example of a plan and profile is shown in Figure 8-39.
- Determine and plot on the plan the locations of natural basin divides and channel outfalls. Include all areas contributing runoff to the drainage system, both on and off of the highway right-of-way.

Step 3 - Obtain or establish typical ditch cross-section.

- Provide adequate depth in roadside ditches to drain the subbase and prevent freeze-thaw damage.
- Select channel side slopes considering design policy, safety, maintenance, economics, soil, aesthetics, and access.

Step 4 - Determine initial channel grades. Plot initial grades on the plan and profile. Slopes of roadside ditches in cuts are usually controlled by highway grades. Use the following guidelines when establishing initial grades.

- Provide minimum grade of 0.3% to minimize ponding and sediment accumulation, if possible.
- Identify and avoid features which may influence or restrict grade such as utilities.

Step 5 - Locate outfall and perform a preliminary hydrologic estimate to determine flow to outfall. (Detailed design of the outfall occurs at a later stage in the design process.)

Step 6 - Identify any adverse impacts on downstream properties from changes in peak flow or runoff volume. Compliance with Oregon drainage law (see **Chapter 2**) and the hydraulic capacity of the downstream receiving channel should be considered. In addition, any flow diversions should consider the water rights of affected parties. Mitigation measures may be needed to reduce impacts to an acceptable level. Mitigation measures may include:

- increasing capacity and adequately protecting the banks of the downstream channel,
- installing an energy dissipator or other control structure to reduce excessive outlet velocities,
- providing flow detention,
- diverting flow to another outfall where increased flow does not cause problems, or
- obtaining drainage easements.

Step 7 - Identify need for water quality treatment (see **Chapter 14**).

Step 8 - Size ditch to convey flow. Typically, the flow in a ditch is related to the time of concentration (T_c) of the runoff reaching the ditch. The T_c, however, is not known unless the flow time can be estimated in upstream ditches, and flow time is related to the size and lining of the upstream ditch. As a result, ditches are usually designed in segments, one after the other, proceeding downstream. Segments typically include ditches which convey approximately the same flow, and a new segment starts where there is a significant change of flow.

Typically T_c and the resultant design flow are calculated for a ditch segment and the ditch is sized and lining selected. Then T_c and the resultant flow are calculated for the adjacent

downstream ditch section, and those ditches are designed. The procedure is repeated segment-by-segment until the outfall is reached. Steps in the sizing of ditches follow.

- Compute the design discharge at the downstream end of the channel segment (see **Chapter 7**).
- Estimate liner type and roughness of channel during period of peak runoff. Calculate depth of flow using preliminary estimates of ditch cross-section and Manning's equations or nomograph. One or more of these changes may be needed if flow is deeper than maximum allowable flow depth including freeboard:

  - increase bottom width,
  - make channel side slopes flatter,
  - make channel slope steeper,
  - provide smoother channel lining, and/or
  - install drop inlets and a parallel storm drain pipe beneath the channel   to supplement channel capacity.

Step 9 - Check adequacy of lining. Calculate maximum shear stress on lining including effects of bends in the channel.

- Compare maximum to permissible shear stress for all linings except riprap. Reduce maximum shear stress by one or more of the following if maximum shear stress is greater than allowable:

  - enlarging channel, or
  - changing the channel slope,
  - flattening the bank side slopes, or
  - increasing the radius of curvature if the channel has bends.

  Increase the allowable shear stress by changing the lining material if reducing the shear stress is impractical.

- Calculate D_{50} needed to resist displacement for riprap linings. Compare D_{50} needed to D_{50} provided. Reduce the D_{50} needed by the measures previously listed. Increase the D_{50} of the liner if these measures are not practical.

Step 10 - Verify adequacy of ditch size. Verify that the depth of flow in the ditch is within allowable limits if liner roughness or ditch cross-section is changed.   Adjust ditch parameters listed in Step 9 above if depth is excessive. Verify that the liner is adequate for revised ditch, if needed.

Step 11 - Design a temporary ditch lining, if needed. Consider whether or not a stand of grass with complete coverage can be quickly established before high flows occur. Liner

materials which quickly decompose may be adequate if a stand of grass with full coverage will grow before high flows occur. Conversely, a more permanent liner may be needed if high flows occur before the grass is established.

Step 12 - Check channel transitions where there are changes in cross-section, slope, discharge, or roughness. Transitions may require special design considerations, such as:

- gradual transitions to minimize turbulence, and
- adequate erosion protection and channel capacity, including freeboard. A hydraulic analysis using energy equations may be needed to determine flow depths and velocities.

Step 13 - Design outfall improvements, detention, and water quality treatment facilities, as needed. The ditches and liners designed in Steps 8 and 9 should be used to determine the T_c and the resultant design flows.
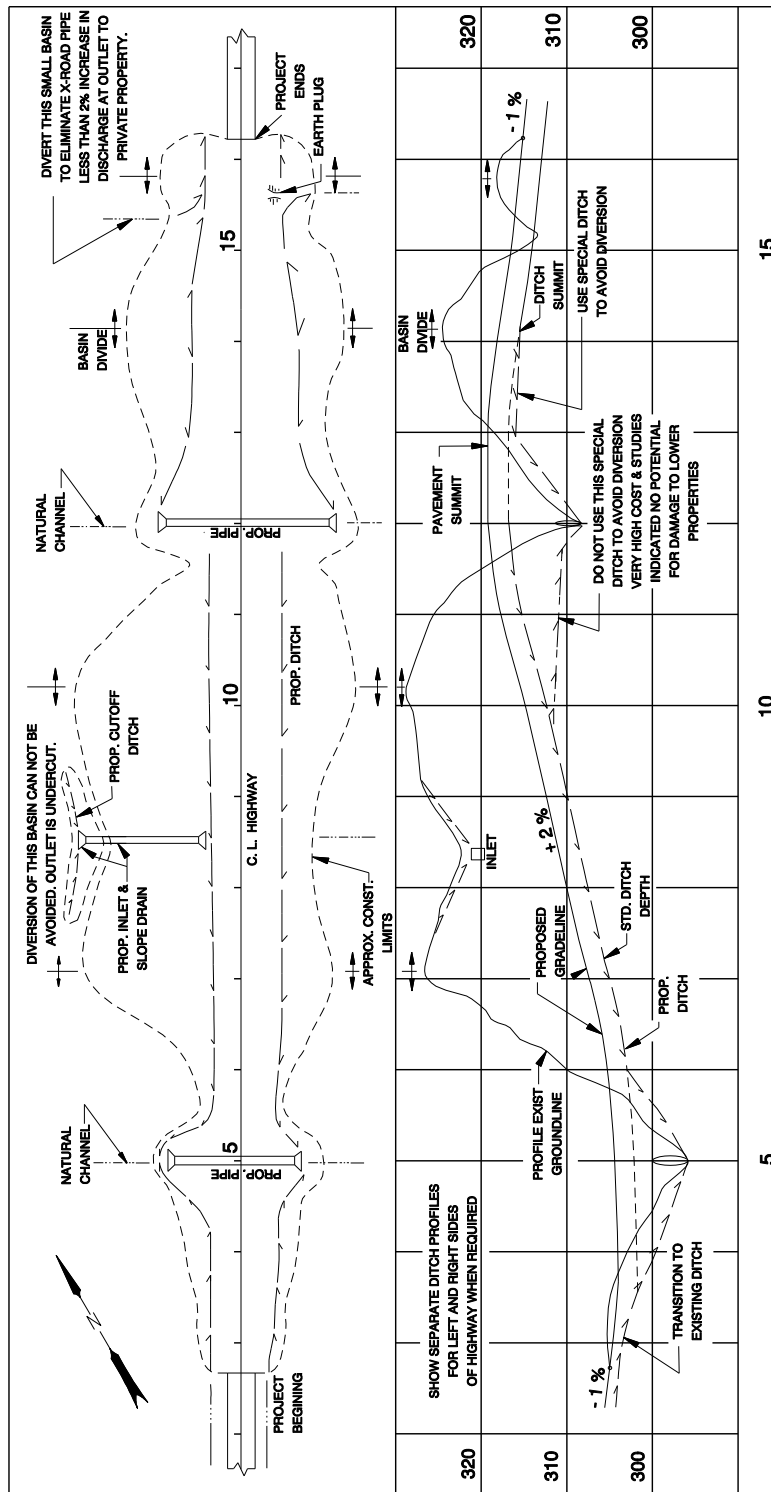
Figure 8-39   Roadside Drainage Plan and Profile