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# Chapter 7 — Ditches and Channels

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# **Chapter 7 - Ditches and Channels**

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## **7.1 Introduction**

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The **function\*** of ditches and open channels is to convey stormwater runoff from, through, or around roadway rights-of-way without damage to the highway, to the open channel, to other components of the highway system, or to adjacent property. Culverts and storm drains can be used for the same purposes. Open channels may be natural or constructed. In either case, the water surface is exposed to the atmosphere, and the gravity force component in the direction of motion is the driving force. Open channels are free to overflow their banks, and cannot develop pressure flow, as can closed conduits such as circular pipes. However, closed conduits flow as open channels when the water surface is below the crown of the conduit, and the design concepts of this chapter apply to closed conduits flowing partly full.

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## 7.2 Design Policy

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Following are Federal, Commonwealth of Virginia, and Virginia Department of Transportation (VDOT) design policies related to channel design.

### 7.2.1 Federal Policy

Channel designs and/or designs of highway facilities that impact channels should satisfy the policies of the Federal Highway Administration applicable to floodplain management. Federal Emergency Management Agency (FEMA) floodway regulations and Corps of Engineers (COE) wetland restrictions for permits should also be satisfied.

### 7.2.2 Commonwealth of Virginia Policy

#### 7.2.2.1 Adequate Receiving Channels

The Virginia Erosion and Sediment Control Regulations, Minimum Standard 19 (VESCR MS-19 <<http://www.dcr.virginia.gov/>>) requires that properties and waterways, downstream from new development sites, shall be protected from sediment deposition, erosion, and damage due to increases in the volume, velocity, and peak flow rate of stormwater runoff for the stated frequency storm of 24-hour duration. Design criteria for adequate channels are summarized in Section 7.3.2.

### 7.2.3 VDOT Policy

The following statements represent VDOT goals for ditch and channel design:

- Coordination with other Federal, State and local agencies concerned with water resources planning has high priority in the planning of highway facilities
- Safety of the general public is an important consideration in the selection of the cross-sectional geometries of artificial drainage channels
- The design of artificial drainage channels or other facilities should consider the frequency and type of maintenance expected and make allowance for maintenance access
- Stability 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 channel stability, should be assessed
- The range of design channel discharges should be selected by the designer based on class of roadway, consequences of traffic interruption, flood hazard risks, economics, and local site conditions
- Wherever possible, encroachment into streams should be avoided and encroachment onto flood plains should be minimized to the fullest extent practical
- Whenever natural channels must be relocated or otherwise modified, the extent of channel reach and degree of modification should be the minimum necessary to

provide compatibility of the channel and roadway, and will incorporate any necessary natural channel design and/or stream restoration

A thorough analysis of the stream's morphology and environment shall be conducted and documented in addition to the economic and engineering alternatives available for the particular location.



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## 7.3 Design Criteria

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### 7.3.1 Roadside Ditches and Channel Classifications

In this chapter, ditches and channels are classified as:

#### 7.3.1.1 Minor Channels (Roadside Ditches)

Minor channels collect sheet flow from the highway pavement or right-of-way and convey that flow to collection points in larger channels or pipes. Flows are generally 50 cubic feet per second (cfs) or less in minor channels. Minor channels usually parallel the highway embankment and are within the highway right-of-way. See Section 7.3.3 for minor channel criteria.

#### 7.3.1.2 Major Channels (Drainage Channels)

Major channels collect drainage from minor channels, pipe systems, and off-site areas, and convey that flow to an adequate discharge point on- or off-site. Flows are generally greater than 50 cfs in major channels. See Section 7.3.4 for major channel criteria.

#### 7.3.1.3 Natural Channels

Natural channels are formed through geomorphologic activity, including erosion and sedimentation. Generally meandering and irregular in cross-section, natural channels may convey any flow rate. See Section 7.3.5 for natural channel criteria.

### 7.3.2 Adequate Receiving Channels

Minimum Standard 19 (Virginia Erosion and Sediment Control Handbook) for adequate receiving channels establishes the following design criteria for all channels.

Concentrated stormwater runoff leaving a development site must be discharged directly into an adequate well-defined, natural or man-made off-site receiving channel, pipe, or storm sewer system. If there is no adequate off-site receiving channel or pipe, one must be constructed to convey stormwater to the nearest adequate channel. Newly constructed channels shall be designed as adequate channels.

An adequate channel is defined as follows: (1) A natural channel, which is capable of conveying the runoff from a 2-year storm without overtopping its banks or eroding after development of the site in question, (2) A previously constructed man-made channel shall be capable of conveying the runoff from a 10-year storm without overtopping its banks, and bed or bank erosion shall not occur due to a 2-year storm, (3) Pipes and storm sewer systems shall contain the 10-year storm.

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A receiving channel may also be considered adequate at any point where the total contributing drainage area is at least 100 times greater than the drainage area of the development site in question; or, if it can be shown that the peak rate of runoff from the site during a 2-year storm will not be increased after development.

Runoff rate and channel adequacy must be verified with acceptable engineering calculations. Refer to Chapter 6, Hydrology, for peak discharge methods.

If an existing off-site receiving channel is not an adequate channel, the applicant must choose one of the following options:

- Obtain permission from downstream property owners to improve the receiving channel to an adequate condition. Such improvements should extend downstream until an adequate channel section is reached.
- Develop a site design that will not cause the pre-development peak runoff rate from a 2-year storm to increase when outfall is into a natural channel, or will not cause the pre-development rate from a 10-year storm to increase when outfall is to a man-made channel. Such a design may be accomplished by enhancing the infiltration capability of the site or by providing on-site stormwater detention measures. The pre-development and post-development peak runoff rates must be verified by engineering calculations.
- Provide a combination of channel improvement, stormwater detention, or other measures which are satisfactory to prevent downstream channel erosion.

All channel improvements or modifications must comply with all applicable laws and regulations.

Increased volumes of unconcentrated sheet flows which may cause erosion or sedimentation of adjacent property must be diverted to an adequate outlet or detention facility.

Outfall from a detention facility shall be discharged to an adequate channel. Outlet protection and/or energy dissipation should be placed at the discharge point as necessary.

### **7.3.3 Minor Channels (Roadside Ditches)**

Minor channels are normally V-shaped and sometimes trapezoidal in cross section and lined with grass or a protective lining. They are usually designed to convey the 10-year discharge, and to resist erosion from the 2-year discharge. Higher design discharges may be necessary when the channel intercepts off-site drainage.

Special design ditches are designed for storm frequencies appropriate to the functional classification of the roadway and the risk involved when the design capacity is exceeded.

A secondary function of a roadside ditch is to drain subsurface water from the base of the roadway to prevent saturation and loss of support for the pavement or to provide a positive outlet for subsurface drainage systems such as pipe underdrains.

The alignment, cross-section, and grade of roadside ditches are usually constrained largely by the geometric and safety standards applicable to the project. These ditches should accommodate the design runoff in a manner that assures the safety of motorists and minimizes future maintenance, damage to adjacent properties, and adverse environmental, or aesthetic effects. A sample roadside ditch plan and profile is shown in Figure 7-1. The VDOT Method for Design of Roadside Ditch Linings (Section 7.5.2.2) is recommended for use on minor channels.

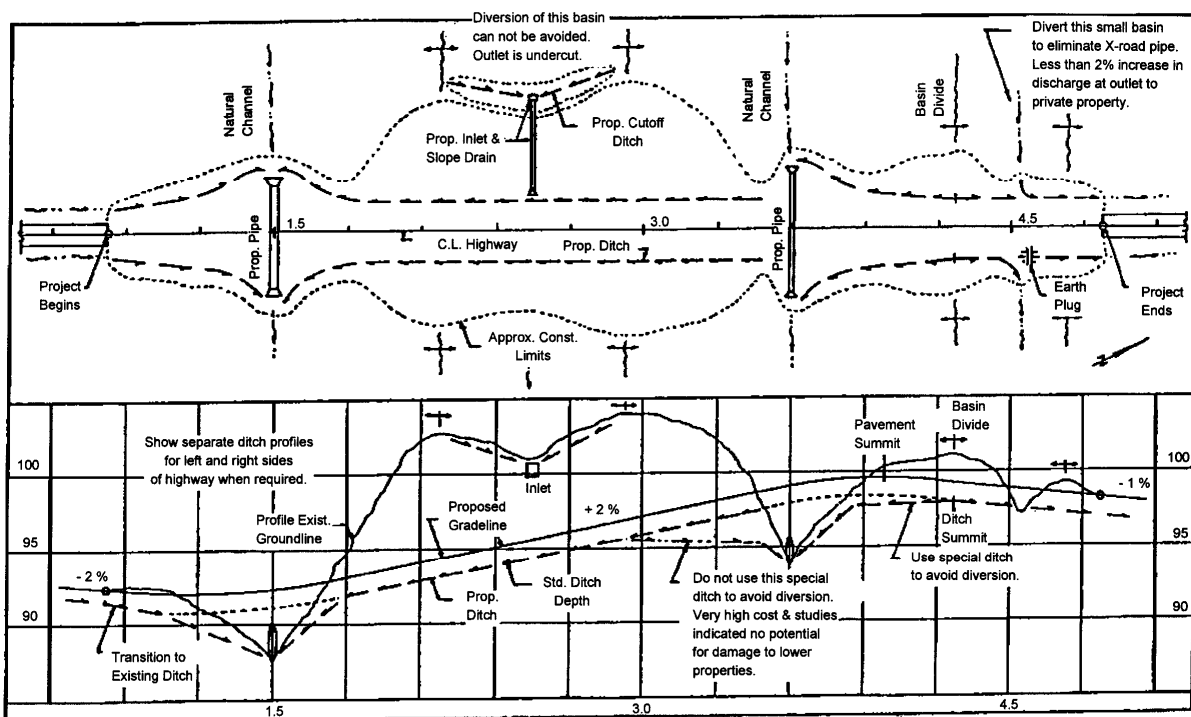


Figure 7-1. Sample Roadside Ditch Plan and Profile

### 7.3.4 Major Channels

Major channels may be within or outside the highway right-of-way. The same design criteria apply as for minor channels. Conveyance is usually based on the 10-year storm but may be greater based upon risk. Erosive protection is based on the 2-year storm. Major channels are usually trapezoidal in cross section. One foot or more of freeboard is recommended for larger channels where the consequences of overtopping are significant. The consequences of failure are usually more severe for major channels;

therefore, a higher level of engineering analysis and design is usually justified for major channels.

### 7.3.5 Natural Channels

The hydraulic effects of floodplain encroachments should be evaluated over a full range of frequency-based peak discharges from the 2-year through the 500-year recurrence intervals on any major highway facility, as deemed necessary by the Department. The hydrologic and hydraulic analysis procedure and required documentation for such situations are more fully described in Chapter 12, Bridge and Structure Hydraulics. If the floodplain encroachment is located in a FEMA or other officially delineated floodplain, no increase in the established natural 100-yr. flood level will be permitted either up or downstream. It should be noted that the Department's criteria is more stringent than FEMA's in this instance. In situations where no FEMA or other officially delineated floodplain exists, it will be acceptable to increase the level of the 100-yr flood event not to exceed one foot up or downstream, provided such increase does not adversely impact adjacent properties, buildings, etc. If an increase in the 100-yr flood level will cause such adverse impact then no increase shall be permitted. The Department's State Hydraulics Engineer must approve exceptions to either of the above criteria.\*

If relocation of a stream channel is unavoidable, the cross-sectional shape, meander, pattern, roughness, sediment transport, and slope should conform to the existing conditions as far as practical. Some means of energy dissipation may be necessary when existing conditions cannot be duplicated.

Streambank stabilization should be provided when appropriate, because of any stream disturbance such as encroachment and should include both upstream and downstream banks as well as the local site.

Relocation of major streams is complex, and special expertise in river mechanics engineering and appropriate natural channel and/or stream restoration design may be necessary.

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## 7.4 Design Concepts

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### 7.4.1 Minor Channels (Roadside Ditches)

#### 7.4.1.1 General

Design discharges (peak flows) should be determined by the Rational Method as defined in Chapter 6, Hydrology.

Velocity should be based on normal depth computed using Manning's equation. Manning's equation requires information on the ditch geometry, such as side slopes, the longitudinal grade, and the appropriate Manning's n-value.

Ditch side slopes should not exceed the angle of repose of the soil and/or lining and should usually be 2H:1V or flatter. See Section 7.4.6 for further discussion on channel linings. Figure 7-1 shows a sample plan and profile for a roadside ditch.

#### 7.4.1.2 Design Considerations

Roadside ditch design involves both capacity and erosion resistance. A trial-and-error process may be necessary to obtain the optimum design. More information on roadside ditch design procedures is contained in Section 7.5.

The VDOT method for design of roadside ditch linings is recommended for use on minor channels. Consideration should be given to ditch bends, steep slopes, and composite linings, which are further defined in Section 7.4.6, HEC-11, and HEC-15. The riprap design procedures described in HEC-15 are for minor channels having a design discharge of 50 cfs or less. When the design discharge exceeds 50 cfs, the design procedures presented in HEC-11 should be followed for riprap-lined channels. HEC-15 may be used for design of larger channels with linings other than riprap.

### 7.4.2 Major Channels

#### 7.4.2.1 General

Design analysis of both natural and artificial channels proceeds according to the basic principles of open channel flow (see Chow, 1959<sup>\*</sup>; Henderson, 1966). The basic principles of fluid mechanics, continuity, momentum, and energy can be applied to open channel flow with the additional complication that the position of the free surface is usually one of the unknown variables. The determination of this unknown is one of the principal problems of open channel flow analysis and it depends on quantification of the flow resistance. Natural channels display a much wider range of roughness values than do artificial channels.

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### 7.4.2.2 Flow Classifications

The classifications of open channel flow are summarized as follows:

*Steady Flow (Rate of flow remains constant with time)*

1. Uniform Flow (Velocity and depth of flow remain constant over length)
2. Non-uniform Flow (Velocity and depth of flow vary over length)
  - Gradually Varied Flow
  - Rapidly Varied Flow

*Unsteady Flow (Rate of flow varies with time)*

1. Unsteady Uniform Flow (rare)
2. Unsteady Non-uniform Flow
  - Gradually Varied Unsteady Flow
  - Rapidly Varied Unsteady Flow

The steady uniform flow class and the steady non-uniform flow class are the most common types of flow treated in highway engineering hydraulics. However, uniform flow is rare in natural channels.

## 7.4.3 Natural Channels

### 7.4.3.1 Stream Morphology

#### 7.4.3.1.1 Introduction

The form assumed by a natural stream, which includes its cross-sectional geometry as well as its plan-form, is a function of many variables for which cause-and-effect relationships are difficult to establish. The stream may be graded or in equilibrium with respect to long time periods, which means that on the average it discharges the same amount of sediment that it receives, although there may be short-term adjustments in its bed-forms in response to flood flows. On the other hand, the stream reach of interest may be aggrading or degrading as a result of deposition or scour in the reach, respectively. The plan-form of the stream may be straight, braided, or meandering. These complexities of stream morphology can be assessed by inspecting aerial photographs and topographic maps for changes in slope, width, depth, meander form and bank erosion with time.

A qualitative assessment of the river response to proposed highway facilities is possible through a thorough knowledge of river mechanics and accumulation of engineering experience. The FHWA publications "Stream Stability at Highway Structures" (HEC-20)\* and "River Engineering for Highway Encroachments" (HDS-6) provide additional

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and more detailed information on making such assessments . Both publications can be accessed and/or downloaded from the FHWA's Internet web site at [http://www.fhwa.dot.gov/engineering/hydraulics/library\\_listing.cfm](http://www.fhwa.dot.gov/engineering/hydraulics/library_listing.cfm) .

### 7.4.4 Channel Analysis

#### 7.4.4.1 General

The hydraulic analysis of a channel determines the depth and velocity at which a given discharge will flow in a channel of known geometry, roughness and slope. The depth and velocity of flow are necessary for the design or analysis of channel linings and highway drainage structures.

Two methods are commonly used in hydraulic analysis of open channels. The single-section method is a simple application of Manning's equation to determine tailwater rating curves for culverts or to analyze other situations in which uniform or nearly uniform flow conditions can be assumed.

The step-backwater method is used to compute the complete water surface profile in a stream reach to evaluate the unrestricted water surface elevations for bridge hydraulic design or to analyze other gradually varied flow problems in streams.

- A. Where the 100-year discharge for a particular site is less than 500 cfs, the site is generally considered to be a minor drainage installation. A single cross section analysis, using nomographs and charts, usually provides an acceptable level of hydraulic analysis. Documentation requirements are satisfied by supplying all requested information on VDOT standard hydraulic computation forms.
- B. Where the 100-year discharge for a particular site is 500 cfs or more, the site is generally considered to be a major drainage installation. The method of hydraulic analysis and the level of documentation must conform to hydrology analysis (H&HA) outline provided in Chapter 12, Bridge and Structure Hydraulics. This type of analysis often requires water surface profile calculations such as those provided by the HEC-2, HEC-RAS, or WSPRO computer models. However, other methods of analysis, which provide the necessary data for proper documentation, may be approved for use.

The single-section method will generally yield less reliable results than the step-backwater method because it requires more judgment and assumptions. In many situations, however, the single-section method is all that is justified. In minor drainage channels such as roadside ditches, the single section method is adequate, except in the case of special design channels or critical locations.

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The step-backwater method should be used for important major channels, where an accurate definition of the water surface profile is needed. The basic principles of open channel hydraulics are applicable to all drainage channels, as well as culverts and storm drains. The variable is the level of detail required in design, which depends on the risks of damage or loss of life caused by a failure of the facility.

#### 7.4.4.2 Equations

The following equations are those most commonly used to analyze open channel flow. The detailed use of these equations in analyzing open channel hydraulics is discussed in Section 7.5.

##### 7.4.4.2.1 Specific Energy

Specific energy ( $E$ ) is defined as the energy head relative to the channel bottom. If the channel slope is less than 10 percent and the streamlines are nearly straight and parallel (so that the hydrostatic assumption holds), the specific energy ( $E$ ) becomes the sum of the depth and velocity head:

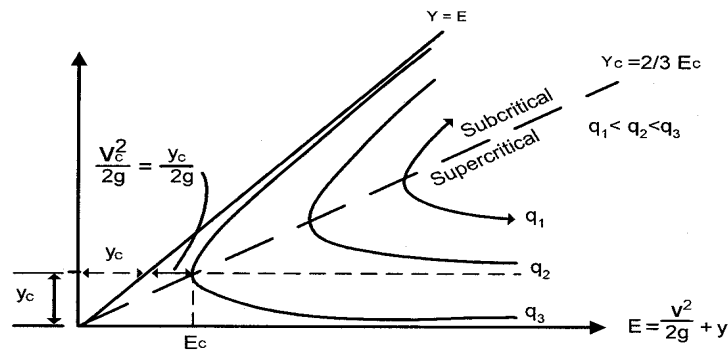
$$E = d + \alpha \frac{V^2}{2g} \quad (7.1)$$

Where:

- $d$  = Depth of flow, ft
- $\alpha$  = Velocity distribution coefficient (see Equation 7.2)
- $v$  = Mean velocity, fps
- $g$  = Gravitational acceleration, 32.2 ft/s<sup>2</sup>

When specific energy is plotted against depth of flow, a curve with a minimum specific energy results, as shown in Figure 7-2. At the minimum specific energy, the depth is called critical depth. Depths above critical depth are subcritical, and below critical depth are supercritical. The velocity distribution coefficient is usually assumed to have a value of one for turbulent flow in prismatic channels but may be significantly different than one in natural channels.





Note:  $y = d$  in Equation 7.1

Figure 7-2. Specific Energy Diagram for Rectangular Channels

#### 7.4.4.2.2 Velocity Distribution Coefficient

Due to the presence of a free surface and due to friction along the channel boundary, the velocities in a channel are not uniformly distributed across the channel cross section. Because of non-uniform distribution of velocities in a channel section, the velocity head of an open channel is usually greater than the average velocity head computed as  $(Q/A_t)^2/2g$ . A weighted average value of the velocity head is obtained by multiplying the average velocity head, above, by a velocity distribution coefficient ( $\alpha$ ) defined as:

$$\alpha = \frac{\sum_{i=1}^n \frac{K_i^3}{A_i^2}}{\frac{K_t^3}{A_t^2}} \quad (7.2)$$

Where:

- $K_i$  = Conveyance in subsection (see Equation 7.8)
- $K_t$  = Total conveyance in section (see Equation 7.8)
- $A_i$  = Cross-sectional area of subsection,  $\text{ft}^2$
- $A_t$  = Total cross-sectional area of section,  $\text{ft}^2$
- $n$  = Number of subsections

#### 7.4.4.2.3 Total Energy Head

The total energy head is the specific energy head plus the elevation of the channel bottom with respect to some datum. A plot of the energy head from one cross section to the next defines the energy grade line.

\* Rev 9/09

**7.4.4.2.4 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 and is defined as:

$$F_r = \frac{V}{\sqrt{\frac{gH_D}{\alpha}}} \quad (7.3)$$

Where:

- $\alpha$  = Velocity distribution coefficient
- $V$  = Mean velocity =  $Q/A$ , fps
- $g$  = Gravitational acceleration\*, 32.2 ft/s<sup>2</sup>

$$H_D = \text{Hydraulic depth} = \left( \frac{A}{T} \right), \text{ ft}$$

(For rectangular channels the hydraulic depth is equal to the flow depth.)

- $A$  = Cross-sectional area of flow, ft<sup>2</sup>
- $T$  = Channel top width at the water surface, ft
- $Q$  = Discharge, cfs

This expression for Froude number applies to channel flow at any cross section. The Froude number is useful in determining the flow regime for water surface profiles.

- $F_r < 1$ , Subcritical Flow
- $F_r = 1$ , Critical Flow
- $F_r > 1$ , Supercritical Flow

**7.4.4.2.5 Critical Flow**

Critical flow occurs when the specific energy is a minimum for a given flow rate (see Figure 7-2). The variation of specific energy with depth at a constant discharge shows a minimum in the specific energy at a depth called critical depth at which the Froude number has a value of one. Critical depth is the depth of maximum discharge when the specific energy is held constant. During critical flow the velocity head is equal to one-half of the hydraulic depth. The general expression for flow at critical depth is:

$$\frac{\alpha Q^2}{g} = \frac{A^3}{T} \quad (7.4)$$

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\* Rev 9/09

Where:

- $\alpha$  = Velocity distribution coefficient
- Q = Discharge, cfs
- g = Gravitational acceleration, 32.2 ft/s<sup>2</sup>
- A = Cross-sectional area of flow, ft<sup>2</sup>
- T = Channel top width at the water surface, ft

When flow is at critical depth, Equation 7.4 must be satisfied, regardless of the shape of the channel.

### **7.4.4.2.6 Subcritical Flow**

Depths greater than critical occur in subcritical flow. The Froude number is less than one for subcritical flow. In this state of flow, small water surface disturbances can travel both upstream and downstream, and the control is always located downstream.

### **7.4.4.2.7 Supercritical Flow**

Depths less than critical depth occur in supercritical flow. The Froude number is greater than one. Small water surface disturbances are always swept downstream in supercritical flow, and the location of the flow control is always upstream.

### **7.4.4.2.8 Continuity Equation**

The continuity equation is the statement of conservation of mass in fluid mechanics. For the special case of one-dimensional, steady flow of an incompressible fluid, it assumes the simple form:

$$Q = A_1 \cdot V_1 = A_2 \cdot V_2^* \quad (7.5)$$

Where:

- Q = Discharge, cfs
- A = Cross-sectional area of flow, ft<sup>2</sup>
- V = Mean cross-sectional velocity, fps (which is perpendicular to the cross section)

Subscripts 1 and 2 refer to successive cross sections along the flow path.

### **7.4.4.2.9 Manning's Equation**

For a given depth of flow in an open channel with a steady, uniform flow, the mean velocity (V) can be computed using Manning's equation:

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\* Rev 9/09

$$V = \frac{1.486}{n} R^{\frac{2}{3}} S^{\frac{1}{2}} \quad (7.6)$$

Where:

- V = Velocity, fps
- n = Manning's roughness coefficient
- R = Hydraulic radius = A/P, ft
- A = Flow area, ft<sup>2</sup>
- P = Wetted perimeter, ft
- S = Slope of the energy grade line, ft/ft

The selection of Manning's n is generally based on observation; however, considerable experience is essential in selecting appropriate n-values. The selection of Manning's n is discussed in Section 7.4.4.3.2. The range of n-values for various types of channels and floodplains is given in Appendix 7D-1 and 7D-2.

The continuity equation can be combined with Manning's equation to obtain the steady, uniform flow discharge as:

$$Q = \frac{1.486}{n} A R^{\frac{2}{3}} S^{\frac{1}{2}} \quad (7.7)$$

For a given channel geometry, slope, roughness, and a specified discharge (Q), a unique value of depth occurs in steady, uniform flow. It is called normal depth. At normal depth, the slope of the energy grade line, the hydraulic grade line, and the channel slope are the same. Normal depth is computed from Equation 7.7 by expressing the area and hydraulic radius in terms of depth. The resulting equation may require a trial-and-error solution. See Section 7.5 for a more detailed discussion of the computation of normal depth.

If normal depth is greater than critical depth, the channel slope is classified as a mild slope. On a steep slope, the normal depth is less than the critical depth. Thus, uniform flow is subcritical on a mild slope and supercritical on a steep slope.

#### **7.4.4.2.10 Conveyance**

In channel analysis, it is often convenient to group the channel properties in a single term called the channel conveyance (K):

$$K = \frac{1.486}{n} A R^{\frac{2}{3}} \quad (7.8)$$

Then Equation 7.7 can be written as:

$$Q = K S^{\frac{1}{2}} \quad (7.9)$$

The conveyance represents the carrying capacity of a stream cross-section based upon its geometry and roughness characteristics alone and 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 ( $\alpha$ ).

### 7.4.4.2.11 Energy Equation

The energy equation (Bernoulli's Equation) expresses conservation of energy in open channel flow defined as energy per unit weight of fluid. This dimension of length is called energy head. The energy head is composed of potential energy head (elevation head), pressure head, and kinetic energy head (velocity head). These energy heads are scalar quantities, the sum of which gives the total energy head at any cross section. Written between an upstream open channel cross section designated 1 and a downstream cross section designated 2, the energy equation is:

$$h_1 + \alpha_1 \frac{V_1^2}{2g} = h_2 + \alpha_2 \frac{V_2^2}{2g} + h_L \quad (7.10)$$

Where:

- $h_1$  = Upstream water surface elevation, ft.
- $h_2$  = Downstream water surface elevation, ft.
- $\alpha$  = Velocity distribution coefficient
- $V$  = Mean velocity, fps
- $h_L$  = Head loss due to local cross-sectional changes (minor loss) plus friction loss, ft

The stage,  $h$ , is the sum of the elevation head ( $z$ ) at the channel bed and the pressure head, or depth of flow ( $y$ ); i.e.,  $h = z + y$ . The terms in the energy equation are illustrated graphically in Figure 7-3. The energy equation states that the total energy head at an upstream cross section is equal to the energy head at a downstream section plus energy head losses between two consecutive sections. The energy equation can only be applied between two cross sections at which the streamlines are nearly straight and parallel so that vertical acceleration can be neglected.

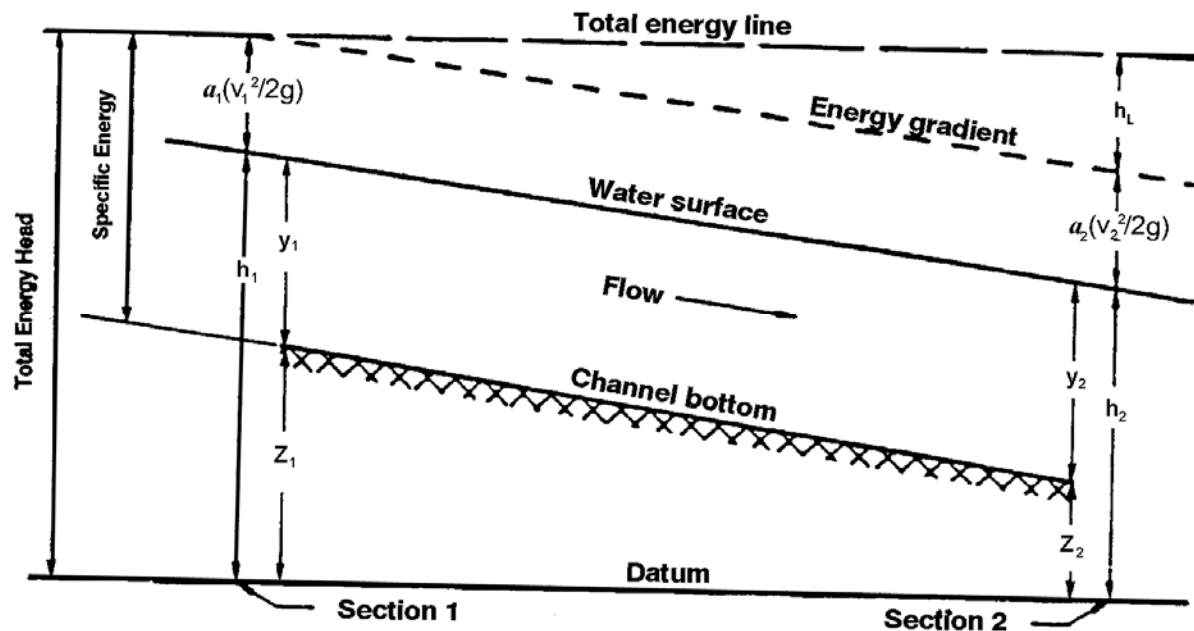


Figure 7-3. Terms in the Energy Equation

#### 7.4.4.3 Hydraulic Representation of Channels

The following sections describe the data needed to apply Manning's equation to the analysis of open channels.

##### 7.4.4.3.1 Cross Sections

The cross-sectional geometry of streams is defined by coordinates of lateral distance and ground elevation, which locate individual ground points. Individual cross sections are taken normal to the flow direction along a single straight line where possible, but in wide floodplains or bends it may be necessary to use intersecting straight lines to form a section; i.e., a "dog-leg" section. It is especially important to make a plot of the cross section to reveal any inconsistencies or errors.

Cross sections should be located to be representative of the sub-reaches between them. Stream locations with major breaks in bed profile, abrupt changes in roughness or shape, control sections such as free overfalls, bends and contractions, or other abrupt changes in channel slope or conveyance will require cross sections taken at shorter intervals in order to better model the changes in conveyance.

Cross sections should be subdivided with vertical boundaries where there are abrupt lateral changes in geometry and/or roughness as in the case of overbank flows. The conveyances of each subsection are computed separately to determine the flow

distribution and  $\alpha$ , and are then added to determine the total flow conveyance. The subsection divisions must be chosen carefully so that the distribution of flow or conveyance is nearly uniform in each subsection (Davidian, 1984). Selection of cross sections and vertical subdivision of a cross section are shown in Figure 7-4.

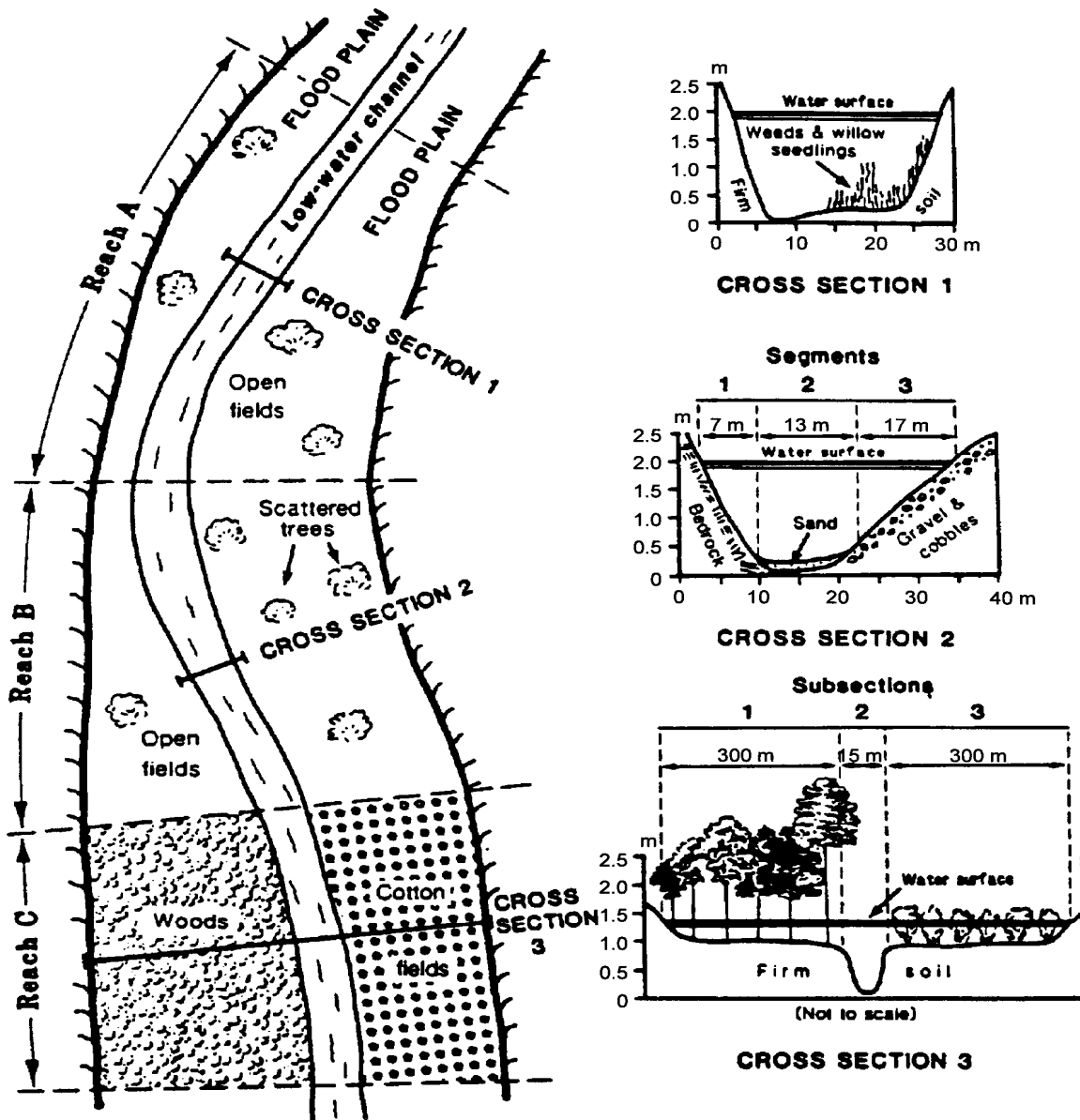


Figure 7-4. Hypothetical Cross Section Showing Reaches, Segments, and Subsections Used in Assigning n-Values

#### 7.4.4.3.2 Manning's n-Value Selection

Manning's  $n$  is affected by many factors and its selection, especially in natural channels depends heavily on engineering experience. Photographs of channels and flood plains, for which the discharge has been measured, and Manning's  $n$  has been calculated, are very useful (see Arcement and Schneider, 1984; Barnes, 1978). For situations lying outside the engineer's experience, a more regimented approach is presented (Arcement and Schneider, 1984). Once the Manning's  $n$ -values have been selected, it is highly recommended that they be verified or calibrated with historical highwater marks and/or gaged stream flow data.

Manning's  $n$ -values for artificial channels are more easily obtained than for natural stream channels. Refer to Appendix 7D-1 and 7D-2 for typical  $n$ -values for both man-made and natural stream channels.

#### 7.4.4.3.3 Calibration

For major channel analyses in existing channels, the equations should be calibrated with historical highwater marks and/or gaged stream flow data to ensure that they accurately represent local channel conditions. The following parameters, in order of preference, should be used for calibration: Manning's  $n$ , slope, discharge, and cross section. Proper calibration is essential if accurate results are to be obtained.

#### 7.4.4.3.4 Switchback Phenomenon

If the cross section is improperly subdivided, the mathematics of Manning's equation causes a switchback. 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 computed for a lesser water depth. With the combination of the lower hydraulic radius and the slightly larger cross-sectional area, a discharge is computed which is lower than the discharge based upon the lower water depth. More subdivisions within such cross sections should be used, and the divisions should be based on both vegetation and geometry, in order to avoid the switchback error. Figure 7-5 depicts an example of the switchback phenomenon.

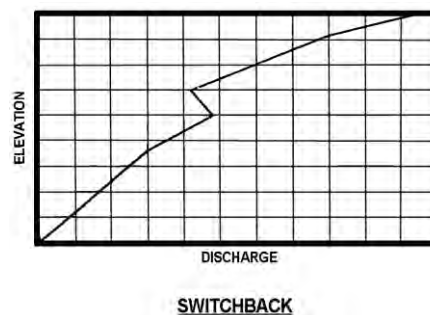


Figure 7-5. Example of Switchback Phenomenon



### 7.4.4.4 Single-Section Analysis

The single-section analysis method (slope-area method) is simply a solution of Manning's equation for the normal depth of flow, given the discharge and channel cross section properties including geometry, slope and roughness. It implicitly assumes the existence of steady, uniform flow; however, uniform flow rarely exists in either man-made or natural channels. Nevertheless, the single-section method is often used to design man-made channels for uniform flow as a first approximation, and to develop a stage-discharge rating curve in a natural channel for tailwater determination at a culvert or storm drain outlet. The single-section analysis method is used in the VDOT method for design of roadside ditch linings.

A stage-discharge rating curve is a graphical relationship of stream flow depth or elevation versus discharge at a specific point on a channel. This relationship should cover a range of discharges up to at least the base (100-year) flood. The stage-discharge curve procedure is discussed in Section 7.5.3.1.

Alternatively, a graphical technique such as those given in the appendices can be used for trapezoidal and prismatic channels. The best approach, especially in the case of natural channels, is to use a computer program such as FEMA's "Quick-2" software package.

In natural channels, the transverse variation of velocity in any cross section is a function of subsection geometry and roughness and may vary considerably from one stage and discharge to another. It is important to know this variation for purposes of designing erosion control measures and locating relief openings such as in highway fills. The single-section 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,  $V = Q/A$ .

### 7.4.4.5 Water Surface Profile Analysis

The step-backwater analysis is useful for determining unrestricted water surface profiles where a highway crossing or encroachment is planned, and for analyzing how far upstream the water surface elevations would be affected. Because the calculations involved in this analysis are tedious and repetitive, it is recommended that a computer program such as the Corps of Engineers HEC-RAS model be used.

#### 7.4.4.5.1 Water Surface Profile Methodology

When uniform flow cannot be reasonably assumed and, therefore, a single cross section cannot represent the channel segment, then an energy balance method must be used to compute the water surface profile (elevation). The energy equation is used in computing the water surface profile.

The method requires definition of the geometry and roughness of each cross section as discussed previously. Manning's n values can vary both horizontally and vertically across the section. Expansion and contraction head loss coefficients, variable main channel and overbank flow lengths and the method of averaging the slope of the energy grade line can all be specified.

The energy equation is derived from Equation 7.10 in Section 7.4.4.2.11.

$$d_1 + \alpha_1 \frac{V_1^2}{2g} + z_1 = d_2 + \alpha_2 \frac{V_2^2}{2g} + z_2 + h_L \quad (7.11)$$

Where:

- d = Depth of flow, ft
- V = Mean velocity, fps
- z = Elevation of flow line, ft
- $h_L$  = Total head loss, ft
- $\alpha$  = Velocity distribution coefficient\*

Equation 7.11 shows that the total head at Section 1 is equal to the total head at Section 2 and the energy (head) losses. Total energy losses include friction and minor losses.

$$h_L = h_f + h_o \quad (7.12)$$

Where:

- $h_L$  = Total head losses, ft.
- $h_f$  = Friction loss, ft.
- $h_o$  = Summation of minor losses, ft.

In most simple water surface profile calculations, minor losses are ignored. Therefore,  $h_L$  is assumed to be equal to  $h_f$ .

$$h_f = LS_A \quad (7.13)$$

Where:

- L = Length of channel segment, ft.
- $S_A$  = Average energy slope of channel segment, ft./ft.

$$S_A = \frac{S_1 + S_2}{2} \quad (7.14)$$

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\* Rev 9/09

Where:

- $S_1$  = Energy slope at Section 1, ft./ft.
- $S_2$  = Energy slope at Section 2, ft./ft.

The energy slope at a given cross section is computed using Equation 7.15 and is shown graphically in Figure 7-6.

$$S = \frac{Q^2 n^2}{2.25 R^{\frac{4}{3}} A^2} \quad (7.15)$$

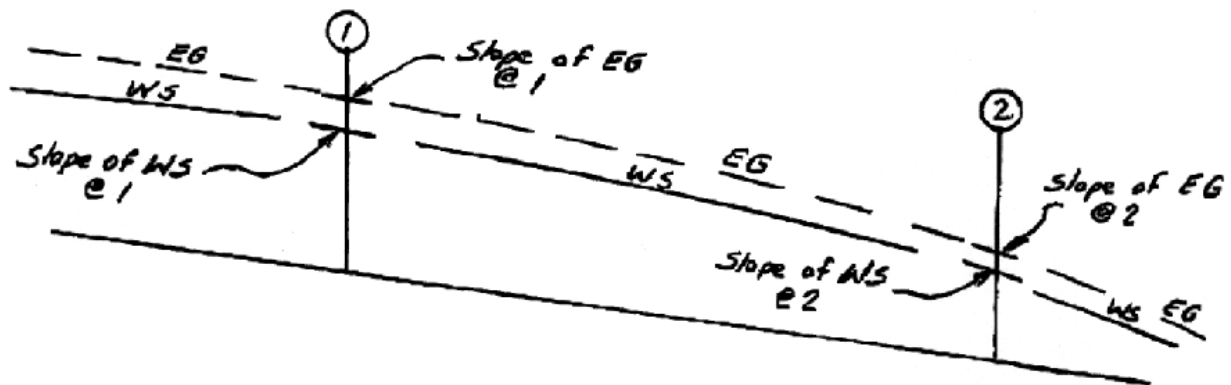


Figure 7-6. Energy Slope between Two Channel Sections

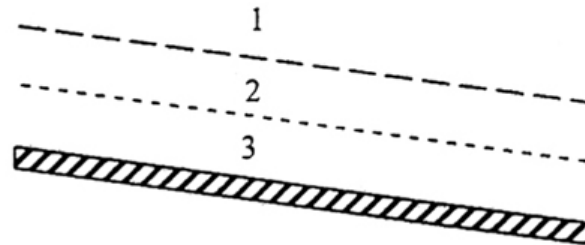
**7.4.4.5.2 Classifications of Backwater Profiles**

Figure 7-7 shows the notation for classifying water surface profiles and Figure 7-8 shows the types of possible flow profiles. Figure 7-7 and Figure 7-8 are from the USACE's Gary Brunner.

DEPTHS

Actual Depth	—————
Normal Depth	- - - - -
Critical Depth	- · - · - ·

REGIONS



SLOPES

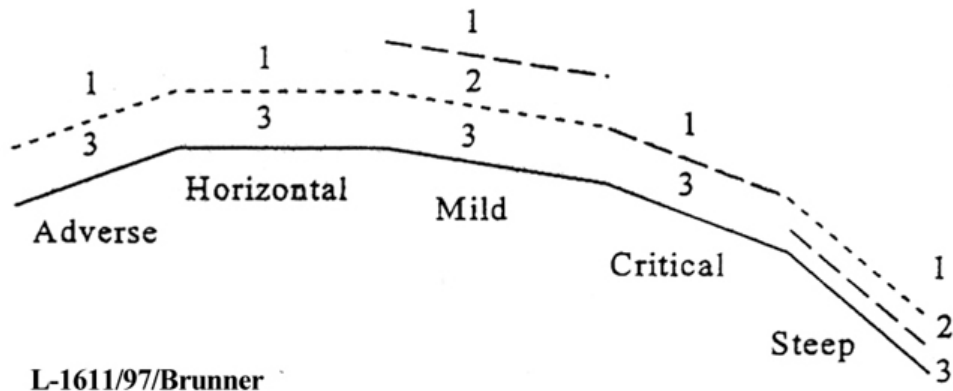
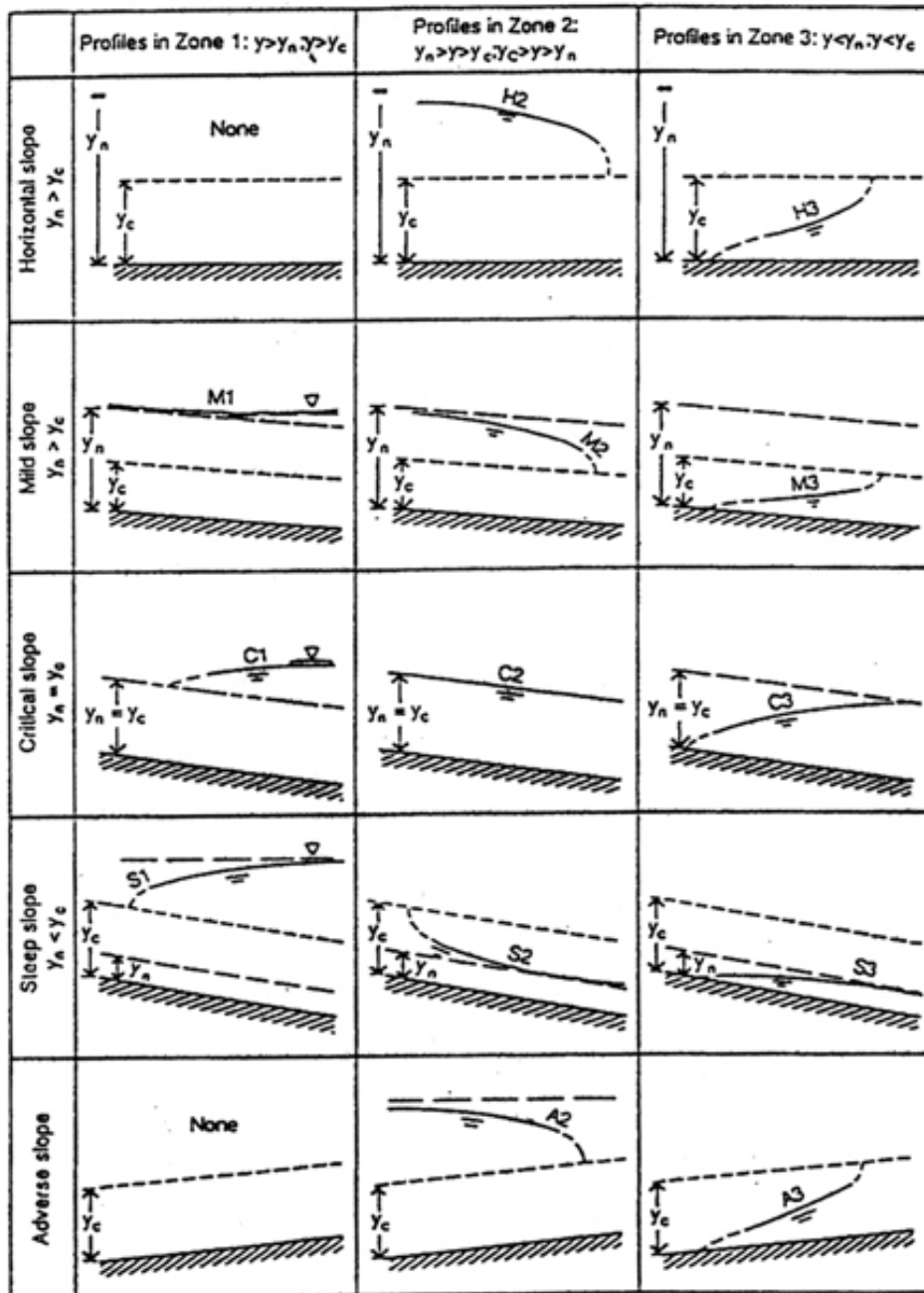


Figure 7-7. Notation for Classifying Water Surface Profiles

GRADUALLY VARIED FLOW



L-1611/97/Brunner

Figure 7-8. Types of Backwater Profiles

#### **7.4.4.5.3 Water Surface Profile Computations**

The references (Davidian, 1984 and USACE, 1986) are valuable sources of guidance on the practical application of the step-backwater method to roadway drainage problems involving open channels. These references contain guidance on cross section determination, location and spacing, and stream reach determination. The reference (USACE, 1986) investigates the accuracy and reliability of water surface profiles related to n-value determination and the survey or mapping technology used to determine the cross section coordinate geometry.

### **7.4.5 Water and Sediment Routing**

Water and sediment routing is a complex phenomenon, and a detailed discussion is beyond the scope of this manual. Information may be found in documentation of the BRI-STARS (Bridge Stream Tube Model for Alluvial River Simulation) Computer Model, (Molinas, 2000\*). The model is semi-two dimensional, and both energy and momentum functions are incorporated so that the water surface profile computation can be carried out through combinations of subcritical and supercritical flows without interruption. Another computer model for sediment routing is the Corps of Engineers HEC-6, "Scour and Deposition in Rivers and Reservoirs." (legacy software)

### **7.4.6 Ditch and Channel Protection**

A significant means of reducing erosion associated with roadways is through the use of properly designed ditches and ditch lining. Linings may be flexible such as vegetation, synthetic material; or riprap or linings may be rigid such as concrete. Erosion resistant vegetation should be used whenever possible and may in some locations require the use of either a temporary protective covering (VDOT Standard EC-2) or a permanent soil stabilization mat (VDOT Standard EC-3, Type A, or Type B). Flexible linings are generally less expensive than rigid linings and permit infiltration and filtering of pollutants. Flexible linings provide a lower flow capacity for a given cross-sectional area when compared to rigid linings. They also have correspondingly lower velocities than rigid linings.

Rigid linings are used on steep grades due to high velocities and may be used for areas where channel width is restricted and the higher flow capacity is needed. Rigid channels require channel protection or energy dissipation at the termination point to prevent scour due to the high outlet velocities. Rigid linings can be damaged or destroyed due to flow undercutting the lining at bends, joints or intersecting ditches where the flow is not contained within the lining. The design of channels with rigid lining should provide any design details that are needed to protect the undercutting of the lining and preserve the integrity of the ditch.

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\* Rev 9/09

Two methods are commonly applied to determine whether a channel is stable from an erosion standpoint: Permissible Velocity and Tractive Force. These methods are described in the following sections.

### 7.4.6.1 Permissible Velocity

Using the permissible velocity approach, the channel is assumed to be stable if the mean velocity is lower than the maximum permissible velocity. The application of the permissible velocity method is quite simple. The Manning equation is used to compute the flow velocity in the channel for the design storm. The flow velocity is then compared with the maximum permissible velocity for the channel bed and bank soils and the lining material.

If the computed velocity is less than the maximum permissible velocity, the channel should be stable. Corrections to the maximum permissible velocity may be applied based on flow depth and channel sinuosity.

The maximum permissible velocity (MPV) in a channel varies with the channel bed material and with the material being transported by the water. Clear water is the most erosive, and therefore its MPVs are the lowest for a given bed material (see Appendix 7D-6). When the water is transporting fine silts, the MPV is up to 100 percent higher than for clear water. For this reason, clear water being released from a settling facility such as a detention pond is often called “hungry water.”

For water carrying a courser material such as sand and gravel, the MPV may be higher or lower than for clear water. For this situation, refer to Appendix 7D-6 to obtain the MPV for the given channel bed material.

While the permissible velocity method is widely used, the tractive force (permissible shear) method provides a more physically based and realistic model for particle detachment and erosion processes (FHWA's HEC-15, “Design of Roadside Channels with Flexible Linings”). It is recommended that the permissible velocity method be used for roadside ditches and that the tractive force method be used for major channel analyses.

### 7.4.6.2 Tractive Force

The Tractive Force Method takes into account the physical factors of bed material, channel geometry, depth, and velocity of flow. The method is confined to non-cohesive materials for which the permissible tractive force is related to particle size and shape and the sediment load in the water. Particle diameters are based on the equivalent spherical volume and assume 75 percent of the mass is smaller by weight ( $D_{75}$ ). In the absence of a gradation curve, it may be assumed that the  $D_{75}$  stone size is equal to 1.96 times the  $D_{50}$  stone size.

The material on the side slope may establish the limiting condition for permissible tractive force, rather than the material on the bed. The resistance to movement of the material on the side slope is reduced by the downward sliding force due to gravity. The

ratio of critical shear on the side slope to critical shear on the bottom is expressed as factor  $K_1$ .

**Note:  $K_1$  in HEC-11 is the same variable as  $K_2$  in HEC-15.**

$$K_1 = \sqrt{1 - \frac{\sin^2 \theta}{\sin^2 \phi}} \quad (7.16)$$

Where:

- $\theta$  = Side slope angle (measured from the horizontal), degrees
- $\phi$  = Natural angle of repose of material under consideration (measured from the horizontal), deg.

**Note: The descriptions for  $\theta$  and  $\phi$  (above) are reversed in Appendix 7E-8.**  
(shown one way in HEC-11 and the other way in HEC-15)\*

The angles of repose for various sizes of non-cohesive materials can be determined from Appendix 7E-1.

The permissible tractive force for various non-cohesive soils is obtained from Appendix 7E-2. Appendix 7E-3 can be used to estimate the permissible tractive force for cohesive soils.

The average tractive force formula is:

$$\tau_o = 62.4RS_o \quad (7.17)$$

Where:

- $\tau_o$  = Average tractive force, lbs/ft<sup>2</sup>
- R = Hydraulic radius, ft.
- $S_o$  = Channel slope, ft/ft.

In channels whose width (B) to depth (d) ratio is 10 or more, the depth of flow (d) may be substituted for R, thus obtaining the maximum tractive force on the channel bed.

$$\tau_{\max} = 62.4dS_o \quad (7.18)$$

Where:

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\* Rev 9/09



d = Depth of flow, ft

A convenient form that can be used for manually performing this computation is found in Appendix 7B-3.

### 7.4.6.3 Geotextile Channel Linings

Geotextile materials designated as Standard EC-3 (Type A and B) Soil Stabilization Mat are used for protective linings in ditches. Standard EC-3 Soil Stabilization Mat is intended to be used as a protective ditch lining material to be applied when the design velocity exceeds the allowable velocity for Standard EC-2, Jute Mesh.

When the design velocity exceeds the allowable velocity for Standard EC-3, a paved (or riprap) lining is required.

The Standard EC-3 (Type C) Soil Stabilization Mat may be used as a protective slope lining for dry cut or fill slopes and wet cut slopes to stabilize the slope on which vegetation is being established. (See VDOT Road and Bridge Standards)

#### 7.4.6.3.1 Design Criteria, Geotextile Linings

Standard EC-3 Type A is to be employed where the 2-year design velocity in the ditch does not exceed 7\* feet per second (fps).

Standard EC-3 Type B is to be employed where the 2-year design velocity does not exceed 10 fps.

A Manning's n-value of 0.05 should be used with Standard EC-3.

Typically, the use of Standard EC-3 Type A should begin at the point where the flow velocity exceeds 4 fps (velocity is assumed to be for flow in an EC-2 lined channel) and change to EC-3, Type B at the appropriate point, until the design velocity exceeds 10 fps or until the velocity decreases so that the use of a ditch lining can be discontinued.

**Table 7-1. PROTECTIVE LININGS FOR DITCHES AND CHANNELS**

Type of Lining	Mannings "n" Value (Default)	Maximum Allowable Velocity (fps)
Bare Earth	0.03	Varies
VDOT EC-2	0.05	4.0
VDOT EC-3 Type A	0.05	7.0

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\* Rev 9/09

VDOT EC-3 Type B	0.05	10.0
Concrete	0.015	NA
VDOT Riprap	0.04-0.045	Based on Sheer Stress

#### 7.4.6.4 Riprap

Riprap is defined as a blanket of well-graded stone used to counteract the effects of erosion or scouring on channels, ditches, embankments, jetties, shorelines, and bridge substructure members such as abutments and piers. Riprap is usually described in terms of the size and/or weight of the stone whose volume makes up approximately 50 percent of the total mass. The size of the 50 percent stone is measured in terms of its equivalent mean spherical diameter (MSD) and is referred to as the stone's  $D_{50}$ . The weight of the 50 percent stone is referred to by its  $W_{50}$ . The Department has six standard riprap classifications whose 50 percent stone size and weights and recommended blanket thickness (T) are tabulated in Appendix 7D-3.

Additional information on VDOT standard riprap may be found in the Department's [Road and Bridge Specifications](#).

The designer shall specify on the plans the type of Riprap and the dimensions (length, width and depth) for placement. The quantity shall be computed using two (2) tons per cubic yard (148 lbs. per cu. ft.) for plan estimating purposes, unless otherwise specified by the District Administrator.

The following general note is to be copied on the plans when riprap is specified.

“The proposed riprap may be omitted by the Engineer if the slope upon which the plans designate riprap to be placed is found to meet the following criteria: The slope designated for placement of riprap is comprised of solid rock or closely consolidated boulders with soundness, size and weight equal to or exceed the specifications for the proposed riprap. If the slope is found to be comprised of material, which is coarser than the bedding aggregate filter blanket specified on the plans, the aggregate filter blanket may be deleted by the Engineer.”

##### 7.4.6.4.1 Riprap Dimensions and Weights

Appendix 7D-4 may be used as a guide to match certain rock dimensions to equivalent weights. This table is not to be used for acceptance or rejection of riprap material.

The approximate percentages of voids for all VDOT standard riprap classes is 25 percent for the estimation of quantities.

##### 7.4.6.4.2 Soils Survey

A soil survey is to be conducted through areas where a channel change is proposed and through embankment areas where riprap may be required. The plans or profile rolls for the regular soil survey will show the location of channel changes and the location where riprap will be required on the fill section.

Borings along the proposed channel change are to be taken at sufficient intervals to determine the type of material encountered along the slopes and in the bottom of the channel.

The borings made in the cut sections or in the borrow pits for construction of the fills are adequate to determine the type of material used in the fills. The test results on the material used in embankments or along channel changes where riprap is required should include the plastic and liquid limits of the minus No. 40 sieve and the grading or particle size of the total sample. This information should be submitted in the regular soil survey report.

The Project Inspector will visually examine the slope upon which the plans designate riprap to be placed. If the slope material appears coarser than the bedding aggregate specified the Project Inspector is to notify the District Material Engineer, through normal channels, for a more detailed investigation to determine the actual need for the bedding. If the slope is comprised of solid rock or closely consolidated boulders with soundness, size and weight equal to or exceeding the specifications for the proposed riprap, then the riprap may be deleted by the District Construction Engineer.

The quantities will be field adjusted, using the supplier's stone weight and the applicable percent of voids for the type or class of material used, to obtain the actual quantity.

### **7.4.6.4.3 Riprap Bedding**

Riprap shall be placed over an appropriate bedding material consisting of a geotextile and aggregate cushion layer in accordance with the following guidelines\*

- In the case of Class AI, I, and II riprap, the aggregate cushion is not required.
- In the case of Class III, Type I, and Type II riprap, the an intermediate aggregate cushion layer will be required which consists of a material of a size, gradation, and thickness as recommended by the Materials Division

The geotextile may only be omitted under the conditions discussed above or with the approval of the District Materials Engineer or District Drainage Engineer.

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**7.4.6.4.4 Major Channels**

Riprap is often used as slope protection for natural or man-made stream channels. The need for such slope protection is predicated on the fact that the native soil material or fill material may be displaced by design flows in the channel. The first step in the design process is to determine whether or not the fill or native material will be displaced. The tractive force method is usually employed to make this determination.

**7.4.6.4.5 Minor Channels**

VDOT normally does not permit the use of standard riprap for linings in standard roadside or median ditches due to its size and normal blanket thickness reducing the ditch cross section. In addition, hand placement, as opposed to end dumping, would probably be necessary in small channels. There are, however, situations in which the smaller riprap sizes such as Class A1 and I may be used to good advantage where special design small trapezoidal ditches such as those connecting culvert cross drains, outfalls, or the discharge point of standard design ditches, are required. In such situations, the Department requires that the riprap be sized in accordance with procedures presented in HEC-15. When riprap is used to line minor channels, the design for the channel cross section should allow for the thickness of the riprap and bedding layers without reducing the available flow section.

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## 7.5 Design Procedures and Sample Problems

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The following design procedures and examples pertain to minor channels, major channels, and natural channels. The procedures are similar, but become more detailed for the larger channel projects.

### 7.5.1 Documentation Requirements

These items establish a minimum requirement for all channels except roadside ditches. Also, see Chapter 3, Documentation. The following items used in the design or analysis should be included in the documentation file:

- Hydrology and stage discharge curves for the design, check floods and any historical water surface elevation(s)
- Cross section(s) used in the design water surface determinations and their locations
- Roughness coefficient assignments (n-values)
- Information on the method used for design water surface determinations
- Observed highwater, dates and discharges
- Channel velocity measurements or estimates and locations
- Water surface profiles through the reach for the design, check floods and any historical floods
- Design or analysis of materials proposed for the channel bed and banks;
- Energy dissipation calculations and designs
- Copies of all computer analyses.

### 7.5.2 Roadside Ditches and Minor Channels

#### 7.5.2.1 Roadside Ditch and Minor Channel Design Procedure

The following six basic design steps are normally applicable to minor channel design projects:

*Step 1: Establish a roadside plan*

- A. Collect available site data
- B. Obtain or prepare existing and proposed plan-profile layout including highway, culverts, bridges, etc.

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- C. Determine and plot on the plan the locations of natural basin divides and roadside ditch outlets and perform the layout of the proposed roadside ditches to minimize diversion flow lengths

An example of a roadside ditch plan/profile is shown in Figure 7-1.

*Step 2: Obtain or establish cross section and data*

- A. Provide ditch depth adequate to drain the subbase and minimize freeze-thaw effects using the standard underdrain outfall requirements that are appropriate for the project
- B. Choose ditch side slopes based on geometric design criteria including safety, economics, soil, aesthetics and access
- C. Establish bottom width of trapezoidal ditch
- D. Identify features which may restrict cross section design:
- Right-of-way limits
  - Trees or other environmentally-sensitive areas
  - Utilities
  - Existing drainage facilities

*Step 3: Determine initial ditch grades*

- A. Plot initial grades on plan-profile layout. (The roadside ditch grade in cut is usually controlled by the grade of the highway.)
- B. Provide desirable minimum grade of 0.5\* percent
- C. Consider influence of type of lining on grade
- D. Where possible, avoid features which may influence or restrict grade, such as utility locations

*Step 4: Check flow capacities and adjust as necessary*

- A. Compute the design discharge at the downstream end of a ditch segment (See Chapter 6, Hydrology)
- B. Set preliminary values of ditch size, roughness coefficient, and slope

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- C. Determine maximum allowable depth of ditch, including freeboard
- D. Check flow capacity using Manning's equation and the single-section analysis
- E. If ditch capacity is inadequate, possible adjustments are as follow:
  - Increase bottom width
  - Make channel side slopes flatter
  - Make channel slope steeper
  - Provide smoother channel lining
  - Install drop inlets and a parallel storm drain pipe beneath the channel to supplement channel capacity
- F. Provide smooth transitions at changes in channel cross sections
- G. Provide extra channel storage where needed to replace floodplain storage and/or to reduce peak discharge

*Step 5: Determine ditch lining/protection needed*

- A. Use the VDOT Method – Roadside Ditch Linings (Preferred Method - Section 7.5.2.2), or
- B. Use the method of allowable tractive force (HEC-15)

*Step 6: Analyze outlet points and downstream effects*

- A. Identify any adverse impacts such as increased flooding or erosion to downstream properties which may result from one of the following at the channel outlet:
  - Increase or decrease in discharge
  - Increase in velocity of flow
  - Concentration of sheet flow
  - Change in outlet water quality
  - Diversion of flow from another watershed
- B. Mitigate any adverse impacts identified in Step 6A. Possibilities include:
  - Enlarge outlet channel and/or install control structures to provide detention of increased runoff in channel
  - Install velocity control structures (energy dissipaters)
  - Increase capacity and/or improve lining of downstream channel
  - Install sedimentation/infiltration basins

- Install sophisticated weirs or other outlet devices to redistribute concentrated ditch flow
- Eliminate diversions which result in downstream damage and which cannot be mitigated in a less expensive fashion

### 7.5.2.2 VDOT Method for Design of Roadside Ditch Linings

The following computational procedure to determine the need for roadside ditch linings is recommended for use on roadside ditches and minor channels. It was developed by VDOT and has been used for many years.

Before the computational analysis can be performed, a soils report from the Materials Division is needed which specifies the type of soil that is found in the area of the ditch. The soil classification is then used with Appendix 7D-2 to determine the maximum allowable velocity for the native soil. Native soil is assumed for new ditches and for ditches with vegetation established for less than two years. When the maximum allowable velocity is exceeded, some type of ditch lining is needed.

*Step 1: Determine the section of each ditch where the following exist:*

- Steepest grade
- Highest flow or drainage area

*Step 2: Determine the longitudinal grade, discharge, and velocity for these sections, assuming no lining*

If the velocity is greater than the allowable velocity for the appropriate soil type, a ditch lining is needed and the following “General Design Procedure for Roadside Ditch Linings” should be used. If the velocity is less than the allowable velocity, no ditch lining is needed for this section of ditch. The same type of analysis should be used for the remainder of the ditches on the project.

#### 7.5.2.2.1 Design Procedure for Roadside Ditch Linings Using VDOT Method

Roadside ditch computations may be done using the format presented as Appendix 7B-1.

*Step 1: The ditch under investigation is divided into convenient segments of length. Usually 100-foot stations are used. The drainage area and runoff coefficient(s) for use in the Rational Method are determined for the first or most upstream segment of the ditch to be analyzed.*

*Step 2: Determine the time of concentration to the downstream end of the first segment of ditch. Using 2-year frequency rainfall intensity, determine the 2-year discharge by the Rational Method.*

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\* Rev 9/09



- Step 3: Determine the average longitudinal grade for the ditch segment under consideration.
- Step 4: Find the velocity for the 2-year frequency design discharge.
- Step 5: If the velocity is less than the maximum allowable velocity, no ditch lining is needed. If the velocity is more than the allowable velocity, ditch lining should be required. The usual progression of types of ditch lining used as velocities increase would begin with VDOT Standard EC-2, then EC-3 Type A, EC-3 Type B and lastly concrete lining (Standard PG-2A).
- Step 6: Determine the depth of flow for the 10-year frequency discharge to insure that the hydraulic capacity has not been exceeded.
- Step 7: Repeat the above steps for the next downstream segment of ditch. To calculate the discharge, add or accumulate the Rational Method CA values for each segment of the ditch contributing to the point of study. For rainfall intensity, use the rainfall intensity value from the previous segment minus 0.1 inch. This is a simplifying assumption or approximation of the actual time of concentration that is used for computational efficiency. If the computed Q value for any segment of ditch is found to be less than the preceding upstream segment, the Q should be held at the higher value of Q until a higher Q is calculated for a downstream segment.

### **7.5.2.2.2 Caveats to General Design Procedures for Roadside Ditch Linings**

The designer should be cautious in using this computational procedure to ensure that the factors for runoff coefficients, times of concentration, and drainage areas properly reflect the actual conditions.

The velocity and depth of flow calculated by this method is based upon uniform flow conditions. Abrupt changes in alignment or grade may cause significant deviations from uniform flow conditions and should be carefully evaluated. Design details must be provided that provide for erosion protection of the ditch in critical areas.

7.5.2.3 Roadside Ditch Lining Sample Problem

Given:

LOCATION: Lynchburg Area

ALLOWABLE VELOCITY: 2.0 fps (Bare Earth)

DITCH SLOPE:

STATION	SLOPE
1-2	0.5%
2-3	2.0%
3-4	3.0%
4-5	4.0%
5-6	4.0%

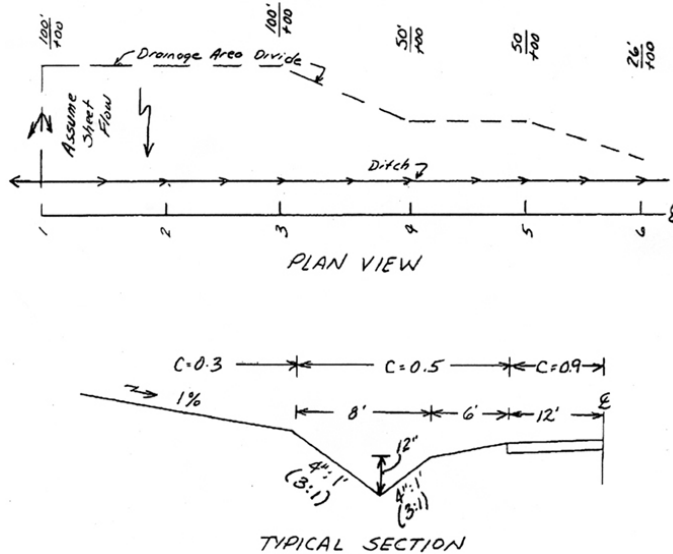


Figure 7-9. Roadside Ditch Protection Sample Problem, Plan and Section

## 7.5 - Design Procedures and Sample Problems

LD-268

LANE SIDE

CUT

MED.

FILL

PROJECT *DITCH PROTECTION*  
BY *EXAMPLE PROBLEM*  
DATE \_\_\_\_\_  
SHEET \_\_\_\_\_ of \_\_\_\_\_

STA. TO STA.	FLOW	CA			INCR.	ACC.	T <sub>r</sub>	I <sub>s</sub>	Q <sub>s</sub>	C or F	Slope Ft/Ft	ALLOW VEL.	Earth		Protective Lining				I <sub>10</sub>	Q <sub>10</sub>	DEP	REMARKS		
		WS/ CA	WS/ CA	WS/ CA									n=0.01	VEL	Q <sub>5</sub>	VEL	DEP	Q <sub>5</sub>					DEP	
1	2	12 .025	14 .016	14 .05	.09	.09	15	3.5	0.32	C	.005	2	1.2					4.8	.43	4	OK			
2	3	12 .025	14 .016	14 .05	.09	.18		3.4	0.61	C	.02	2	2.3											
												4		1.6	4.5			4.7	.85	5	EC-2 REQ'D START STA 2+00			
3	4	12 .025	14 .016	14 .017	.075	.26		3.3	0.85	C	.03	4		1.7	4.5			4.6	1.2	5	EC-2 REQ'D			
4	5	12 .025	14 .016	14 .017	.068	.32		3.2	1.02	C	.04	4		2.4	4.5			4.5	1.4	5	EC-2 REQ'D			
5	6	12 .025	16 .016	12 .008	.049	.37		3.1	1.15	C	.04	4		2.5	5			4.4	1.6	5.5	EC-2 REQ'D D=12"			

Figure 7-10. Worksheet (LD-268) for Calculation of Roadside Ditch Protection , Sample Problem

### 7.5.3 Major Channels

The following procedures are used to analyze flow in major channels.

#### 7.5.3.1 Single-Section Stage-Discharge Curve Procedure

The stage-discharge curve procedure is basically a single section analyses. The following steps outline the development of a stage-discharge curve:

- Step 1: Select the typical cross section at or near the location where the stage-discharge curve is needed
- Step 2: Subdivide cross section and assign n-values to subsections as described in Section 7.4.4.3.
- Step 3: Estimate water-surface slope. Since uniform flow is assumed, the average slope of the streambed is normally used.
- Step 4: Apply a range of incremental water surface elevations to the cross section.

*Step 5: Calculate each incremental elevation. Total discharge at each elevation is the sum of the discharges from each subsection at that elevation. In determining hydraulic radius, the wetted perimeter should be measured only along the solid boundary of the cross section and not along the vertical water interface between subsections.*

*Step 6: After the discharge has been calculated at several incremental elevations, a plot of stage versus discharge should be made. This plot is the stage-discharge curve and it can be used to determine the water-surface elevations corresponding to the design discharge or other discharges of interest.*

### **7.5.3.2 Water Surface Profile Procedure**

The water surface profile computations must be started at a point of a known or an assumed water surface elevation. This starting water surface elevation can be derived in one of three ways.

- Critical Depth ( $d_c$ ) - When changes in flow conditions or channel characteristics cause a transition from subcritical to supercritical flow, or supercritical to subcritical flow, the flow must, at some point, pass through critical depth. When the transition in flow is caused by a change in the slope of the channel bed, then critical depth can be assumed to occur at the change of grade point. Critical depth  $d_c$  can then be used to generate a starting elevation.
- Known Water Surface Elevation - When controls within the channel section dictate a certain stage (elevation)-discharge relationship, then the elevation generated by that control for the targeted discharge can be used as a starting elevation. Examples of controls within the channel section are a culvert under an embankment across the channel, a dam across the channel section, etc.
- Slope/Area (Normal Depth -  $d_n$ ) - Water Surface Profile computations can be started at a point in the channel where uniform flow can be assumed and, therefore, normal depth ( $d_n$ ) can be used in order to generate a starting elevation.

The following VDOT procedure for developing water surface profiles is recommended. A convenient design form can be found in Appendix 7B-2.

*Step 1: Determine flow type - subcritical or supercritical*

- A. If flow is supercritical, computations will proceed in a downstream direction from some known starting point.
- B. If flow is subcritical, computations will proceed in an upstream direction.

*Step 2: Starting point based on:*

- A. Critical depth ( $d_c$ )
- B. Known water surface elevation
- C. Slope/Area method ( $d_n$ )

*Step 3: On computation sheet, let subscript 1 reflect the values at the known cross section and subscript 2 reflect the values at the unknown cross section*

*Step 4: After determining the starting point and its water surface elevation, compute the Total Head for the starting cross section*

$$h_1 = d_1 + \frac{V_1^2}{2g} + z_1$$

*Step 5: For the channel segment between the starting point and the next cross section*

- A. Determine the Target Head, which is equal to the Total Head of the known cross section (starting point) (from step 4).
- B. Assume a depth ( $d_2$ ) at the unknown cross section.
- C. Compute area ( $A_2$ ), wetted perimeter ( $P_2$ ), hydraulic radius ( $R_2$ ) and velocity ( $V_2$ ) for the unknown cross section based on the assumed depth ( $d_2$ ).
- D. Compute velocity head for unknown cross section ( $V_2^2/2g$ ). Determine  $z_2$  (elevation) at unknown cross section and length ( $L$ ) between known and unknown cross section.
- E. Compute energy slope ( $S_1$ ) at the known cross section

$$S_1 = \frac{Q^2 n^2}{2.25 R_1^{\frac{4}{3}} A_1^2}$$

- F. Compute energy slope ( $S_2$ ) at the unknown cross section

$$S_2 = \frac{Q^2 n^2}{2.25 R_2^{\frac{4}{3}} A_2^2}$$

- G. Compute average energy slope ( $S_A$ ) between cross section

$$S_A = \frac{S_1 + S_2}{2}$$

H. Compute total head loss ( $h_L$ ) between cross sections

$$h_L = LS_A$$

I. Compute total head ( $h_2$ ) at the unknown cross section

$$h_2 = d_2 + \frac{V_2^2}{2g} + z_2$$

J. Solve equation:

Total Head at the upstream cross section = Total Head at the downstream cross section + head loss ( $h_f$  or  $h_L$ ).

Use trial and error method until the two sides of the equation balance within the permissible tolerance.

K. Once the equation has been satisfied and the depth ( $d_2$ ) established (Step 5j), then the previous unknown cross section becomes the known cross section for the next segment. Assume a depth ( $d_2$ ) for the new unknown cross section, compute properties ( $A_2$ ,  $P_2$ ,  $R_2$ ,  $V_2$ ) and repeat the procedure.

7.5.3.2.1 Water Surface Profile Sample Problem

GIVEN:  $Q = 100$  cfs  
 $n = 0.15$   
 Cross section shown in Figure 7-11

REQUIRED: Compute Water Surface Profile (WSP) between sections 1 and 3 with a tolerance of 0.2 feet.

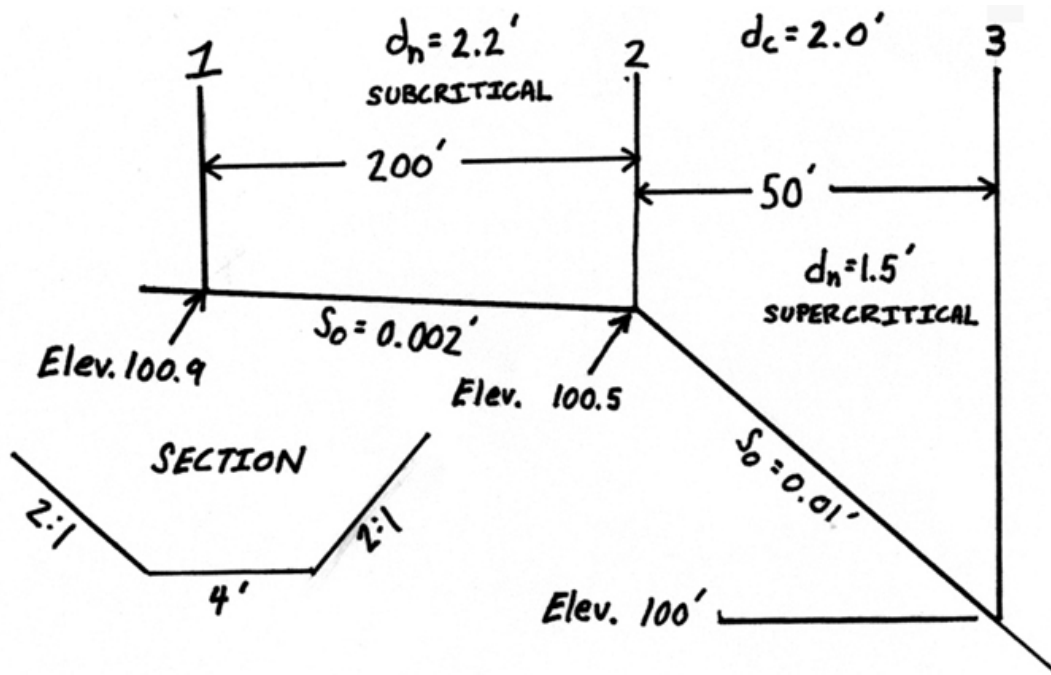


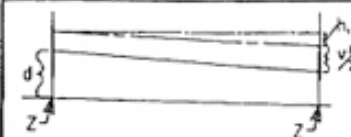
Figure 7-11. Water Surface Profile Sample Problem

- (1) Compute values for  $d_n$  and  $d_c$  at each section (use the nomographs in Appendix 7C). Determine elevation at each section. Determine subcritical or supercritical flow at each section.
- (2) Determine starting point and direction of computations.

Starting point = # 2 ( $d_c$ )

Compute from # 2 to # 3 (supercritical flow)

Compute from # 2 to # 1 (subcritical flow)



Worksheet For Non-Uniform Flow In Open Channels

STA.	$H_1$	depth $d_2$	X-Section Area of Station 2 $A_2$	Wetted Perimeter $P_2$	$A_2 + P_2$	$Q_2 + A_2$	$w + 2(32.2)$ $V_2$	Flow Line Elevation Distance Between Stations $Z_2$	$L$	$S_1$	$S_2$	$S_A$	$h_1$	$H_2$	Tolerance = $\pm 0.2'$ $H_{up} = H_{down} + h_1$
2	103.11	$2/d_c$	$A_1/16$	$P_1/12.99$	$R_1/1.24$	$V_1/6.25$	$V_1^2/2g$ 0.67 .61	$Z_1/100.5$	-	.00293					ASSUME WSP STARTS @ SEC. 2 WITH $d_c$ 2' ∴ WSE $100.5 + 2 = 102.5'$
3 →	103.11	$1.5/d_n$	10.5	10.71	0.98	9.52	1.41	100	50	.00293	.00932	.00612	0.31	102.91	$103.11 + 102.91/0.31 =$ $103.22$ OK ∴ WSE = $100 + 1.5 = 101.5'$
← 1	103.11	$2.2/d_n$	18.48	13.84	1.34	5.41	0.45	100.9	200	.00293	.00198	.00246	0.49	103.55	$103.55 = 103.11 + 0.49 =$ $103.6$ OK ∴ WSE = $100.9 + 2.2 = 103.1'$

Figure 7-12. Worksheet for Calculation of Non-Uniform Flow in Open Channels, Sample Problem

### 7.5.4 Natural Channels

#### 7.5.4.1 General

The analysis procedure for all types of channels has some common elements as well as some substantial differences. This section will outline a process for assessing a natural stream channel.

#### 7.5.4.2 Natural Channels Design Procedure

Usually the analysis of a natural channel is in conjunction with the design of a highway hydraulic structure such as a culvert or bridge or a longitudinal encroachment such as a highway embankment. In general, the objective is to convey the water along or under the highway in such a manner that does 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 level of detail of the studies necessary should be commensurate with the risk associated with the action, and with the environmental sensitivity of the stream and adjoining floodplain.

Although the following step-by-step procedure may not be appropriate for all possible applications, it does outline a process that will usually apply to natural channel design.



*Step 1: Assemble site data and project file*

A. Data Collection

- Topographic, site and location maps
- Roadway profile
- Photographs
- Field reviews
- Design data at nearby structures
- Gaging records
- Historic flood data and local knowledge
- Utilities, including existing drainage

B. Studies by other agencies

- Flood insurance studies
- Floodplain studies
- Watershed studies

C. Environmental constraints

- Floodplain encroachment
- Floodway designation
- Fish and wildlife habitat
- Commitments in review documents

D. Design criteria

*Step 2: Determine the project scope*

A. Determine level of assessment

- Stability of existing channel
- Potential for damage
- Sensitivity of the stream

B. Determine type of hydraulic analysis

- Single-section analysis
- Step-backwater analysis

C. Determine survey information needed

- Extent of streambed profiles
- Locations of cross-sections
- Elevations of flood-prone property
- Details of existing structures

- Properties of bed and bank materials

*Step 3: Evaluate hydrologic variables*

- A. Compute discharges for selected frequencies.
- B. Consult Chapter 6, Hydrology

*Step 4: Perform hydraulic analysis*

- A. Single-section analysis (7.4.4.4, 7.5.3.1)
  - Select representative cross section
  - Select appropriate n-values (Appendix 7D-1 and 7D-2)
  - Compute stage-discharge relationship
- B. Step-backwater analysis (7.5.3.2)
- C. Calibrate with known high water

*Step 5: Perform stability analysis*

- A. Geomorphic factors
- B. Hydraulic factors.
- C. Stream response to change

*Step 6: Design countermeasures*

- A. Criteria for selection
  - Erosion mechanism
  - Stream characteristics
  - Construction and maintenance requirements
  - Cost
- B. Types of countermeasures
  - Meander migration countermeasures
  - Bank stabilization
  - Bend control countermeasures
  - Channel braiding countermeasures
  - Degradation countermeasures
  - Aggradation countermeasures

C. For additional information

- HEC-20 Stream Stability
- Highways in the River Environment
- References

Step 7: Documentation (Section 7.5.1)

- Prepare report and file with background information

### 7.5.5 Riprap Channel Lining Design Procedure

In riprap lining design, it is first necessary to determine whether a lining is needed based upon the most appropriate method.

#### 7.5.5.1 Riprap Lining Design Method for Major Channels (HEC-11)

If it is determined that riprap lining protection is required for a major stream channel, the design procedures employed by the department are predicated directly on the FHWA publication, "Design of Riprap Revetment" (HEC-11). This publication can be accessed and/or downloaded from the FHWA's Internet web site at <http://isddc.dot.gov/OLPFiles/FHWA/009881.pdf>.

The riprap design procedure presented in HEC-11 is based on the equation:

$$D_{50} = C \left[ \frac{0.001V_a^3}{K_1^{1.5} \sqrt{d_{avg}}} \right] \quad (7.19)$$

Where:

- $D_{50}$  = Median riprap particle size, ft
- $C$  = Stone size correction factor
- $V_a$  = Average velocity in the main channel, fps
- $d_{avg}$  = Average flow depth in the main flow channel, ft

The equation for  $K_1$  is from HEC-11:\*

$$K_1 = \sqrt{1 - \frac{\sin^2 \theta}{\sin^2 \phi}} \quad (7.20)$$

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Where:

- $\theta$  = Side slope angle (measured from the horizontal), deg.
- $\phi$  = Natural angle of repose of material under consideration (measured from the horizontal), deg.

$$C = C_{sg} C_{SF} \quad (7.21)$$

Where:

- $C_{sg}$  = Adjustment factor for specific gravity of stone
- $C_{SF}$  = Adjustment factor for stability

$$C_{sg} = \frac{2.12}{(S_g - 1)^{1.5}}$$

Where:

- $S_g$  = Specific gravity of the rock riprap (Assume 2.65 for all VDOT standard riprap sizes.)

$$C_{SF} = \left( \frac{SF}{1.2} \right)^{1.5} \quad (7.22)$$

Where:

- SF = Stability Factor, See Appendix 7D-5, (Usually 1.2)

Notes:

1. Nomographs for the solutions of  $\phi$ ,  $K_1$ ,  $C$ , and  $D_{50}$  are found in Appendix 7E.
2.  $K_1$  in HEC-11 is the same variable as  $K_2$  in HEC-15.

Convenient forms that can be used for manually performing these computations are Appendices 7B-4 for standard VDOT riprap sizes and 7B-5 for non-standard VDOT riprap sizes.

**7.5.5.1.1 Riprap Lining Design for Major Channels (HEC-11) Sample Problem**

Given: Channel is in a relatively straight section of stream. Curve radius/channel width (RW) > 30 with uniform flow. Wave action and floating debris are not a consideration. The average velocity is 5.4 fps. The average depth is 2 ft. The side slope ratios are 2:1 (26.57°). The angle of repose for the riprap is 42°. The specific gravity of the riprap is 2.65.

Determine:

Size of required riprap slope protection and appropriate VDOT standard riprap.

Solution:

Step 1: Determine the stability factor (SF)

From Appendix 7D-5, select a stability factor (SF) of 1.2.

$$\text{Step 2: } C_{SF} = \left(\frac{SF}{1.2}\right)^{1.5} = \left(\frac{1.2}{1.2}\right)^{1.5} = 1$$

$$\text{Step 3: } C_{sg} = \frac{2.12}{(S_g - 1)^{1.5}} = \frac{2.12}{(2.65 - 1)^{1.5}} = 1$$

$$\text{Step 4: } C = C_{sg}C_{SF} = 1(1) = 1$$

$$\text{Step 5: } K_1 = \sqrt{1 - \frac{\sin^2 \theta}{\sin^2 \phi}} = \sqrt{1 - \frac{\sin^2 (26.57)}{\sin^2 (42)}} = 0.74$$

$$\text{Step 6: } D_{50} = C \left[ \frac{0.001V_a^3}{K_1^{1.5} \sqrt{d_{avg}}} \right] = 1 \left[ \frac{0.001(5.4)^3}{(0.74)^{1.5} \sqrt{2}} \right] = 0.17 \text{ ft.}$$

Step 7: Closest standard VDOT riprap size = Class AI ( $D_{50} = 0.8$  ft).

**7.5.5.2 Riprap Lining Design Method for Minor Channels (HEC-15)**

The following design procedures employed by the department are predicated directly on the FHWA publication, "Design of Roadside Channels With Flexible Linings" (HEC-15). This publication can be accessed and/or downloaded from the FHWA's Internet

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web site at <http://www.fhwa.dot.gov/engineering/hydraulics/pubs/05114/05114.pdf>.\*

Once it has been determined that the natural or backfill material is unstable, using either (1) the Tractive Force procedure, (2) a table of allowable velocities for specific soil types such as Appendix 7D-2 and 7D-6, or (3) engineering judgment, select a trial riprap size and proceed as follows:

*Step 1: Determine the permissible shear stress ( $\tau_p$ ) for the riprap size selected*

Use 3.2 lbs/ft<sup>2</sup> for Class AI Riprap and 4.4 lbs/ft<sup>2</sup> for Class I Riprap.

*Step 2: Select a trial flow depth range for the ditch configuration using the following table from HEC-15:*

**Table 7-2. Manning's n-Values for Depth Ranges**

Riprap Size	0-0.5 ft	0.5-2.0 ft	>2.0 ft
Class AI	0.104	0.069	0.035
Class I	--	0.078	0.040

*Step 3: Using the n-value for the selected trial flow depth range, calculate the actual depth of flow for the ditch configuration using the actual design discharge and ditch slope using an appropriate method such as Appendix 7C-3. If the calculated depth is within the trial depth range selected, proceed to the next step. If it is not, select another trial depth range and try again.*

*Step 4: Calculate the actual shear stress ( $\tau_o$ ):*

$$\tau_o = \gamma d S_o \quad (7.23)$$

Where:

- $\gamma$  = Unit weight of water (62.4 lbs/ft<sup>3</sup>)
- $d$  = Depth of flow, ft.
- $S$  = Average ditch flowline slope, ft./ft.

If  $\tau_o > \tau_p$ , the selected riprap size is too small. Choose the next larger size and try again.

If the ditch side slopes are steeper than 3:1, it is necessary to perform the additional calculations shown below:

\* Rev 9/09

Step 5: Determine the angle of repose for the riprap size determined above.

It should be noted that all VDOT standard riprap sizes are assumed to have angles of repose of  $42^\circ$ .

Step 6: Determine the ratio of maximum side shear to maximum bottom shear ( $K_1$ ) using Appendix 7E-7.

Step 7: Determine tractive force ratio ( $K_2$ ) from Appendix 7E-8.

Step 8: Calculate required  $D_{50}$  for side slopes ( $D_{50 \text{ side}}$ )

$$D_{50 \text{ side}} = \frac{K_1}{K_2} D_{50 \text{ bottom}}$$

From a practical standpoint, whatever riprap size is indicated for the ditch side slope should be used on the bottom as well.

### 7.5.5.2.1 Channel Stability Sample Problem - Tractive Force Calculation

Check the stability of channel's native material using tractive force calculation. Note that the "RIPRAP" computer program referenced in section 7.4.6.4.4 could be employed to perform this computation.\*

Given: A natural channel with a bed and banks of native materials composed of cobbles and pebbles. Mean diameter is 1.25 inches for the  $D_{75}$  size stone. The channel bottom width (B) is 10 ft. The longitudinal slope is 0.008 ft./ft. Its side slope is 2(h):1(v). The flow (Q) is 150 cfs at a depth (d) of 2 ft.

Determine whether the channel is stable for the indicated condition.

Solution:

Step 1: Determine the permissible shear stress ( $\tau_p$ )

From Appendix 7E-2, for a  $D_{75}$  particle diameter of 1.25 inches, read a permissible tractive force ( $\tau_p$ ) on the channel bottom of 0.5 lbs/ft<sup>2</sup>.

Step 2: For a side slope ratio of 2:1, the sine of the slope angle ( $\theta = 26.6^\circ$ ) is 0.5

Step 3: From Appendix 7E-1, for a particle diameter of 1.25 inches, read an angle of repose ( $\phi$ ) of  $40^\circ$

The sine of  $40^\circ = 0.643$

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$$\text{Step 4: } K_1 = \sqrt{1 - \frac{\sin^2 \theta}{\sin^2 \phi}} = \sqrt{1 - \frac{0.447^2}{0.643^2}} = 0.72$$

Step 5: Permissible tractive force on the side slopes ( $\tau_s$ )

$$\tau_s = K_1 \tau_p = 0.72(0.05) = 0.36 \text{ lbs/ft.}^2$$

Step 6: Compute the width to depth ratio

$$\frac{B}{d} = \frac{10}{2} = 5, \text{ which is less than 10, therefore use } \tau_o = 62.4RS_o$$

Step 7: Flow cross-sectional area

$$\begin{aligned} A &= Bd + zd^2 \\ &= 10(2) + 2(2)^2 \\ &= 28 \text{ ft.}^2 \end{aligned}$$

Step 8: Wetted perimeter

$$\begin{aligned} P &= B + 2d\sqrt{1 + z^2} \\ &= 10 + 2(2)\sqrt{1 + 2^2} \\ &= 18.94 \text{ ft.} \end{aligned}$$

Step 9: Hydraulic radius

$$R = \frac{A}{P} = \frac{28}{18.94} = 1.48 \text{ ft.}$$

Step 10: Compute the average tractive force

$$\begin{aligned} \tau_o &= 62.4RS_o \\ &= 62.4(1.48)(0.008) \\ &= 0.74 \text{ lbs/ft.}^2 \end{aligned}$$

Step 11: Compare the average tractive force of the channel to the allowable tractive force of the channel sides and bottom

$$\tau_o \text{ of } 0.74 > \tau_p \text{ (bottom) of } 0.5 \text{ lbs/ft}^2 \text{ or } \tau_s \text{ (side slope) of } 0.36 \text{ lbs/ft}^2$$

Native material is unstable. Channel protection is required.



**7.5.5.2.2 Riprap Lining Design for Minor Channels (HEC-15) Sample Problem**

Given: It has been determined that a special design ditch connected to culvert cross drain pipes will need a riprap lining. The ditch has a bottom width (B) of 2 ft, 2:1 side slopes ( $z = 2$ ), and a slope along the ditch line (S) of 0.005 ft/ft. The design discharge is 20 cfs.

Determine what size VDOT standard riprap will be required?

*Step 1: Try Class AI standard riprap,  $D_{50} = 0.8$  ft*

$$\tau_p = 3.2 \text{ lb/ft}^2$$

*Step 2: Assume a depth range of from 0.5-2.0 ft. with an  $n$ -value of 0.069*

*Step 3: Using Appendix 7E-3 for  $S = 0.005$  ft/ft,  $B = 2$  ft,  $z = 2$ ,  $Qn = 20 (0.069) = 1.38$ ,*

*Read  $d/B = 0.78$ .  $d = 0.78 (2) = 1.56$  ft.  $0.5 < 1.56 < 2.0$ ,*

*Therefore, the assumed  $n$ -value of 0.069 is acceptable.*

*Step 4: Determine the maximum shear stress*

$$\tau_{\max} = \gamma d S_o = 62.4(1.56)(0.005) = 0.49 \text{ lb/ft.}^2$$

*Step 5: Evaluate the maximum shear stress with the permissible shear stress*

$$\tau_{\max} < \tau_p,$$

$$0.49 < 3.2$$

Therefore, Class AI riprap is acceptable for the ditch bottom and side slopes of 3:1 or flatter. However, since the side slopes are 2:1, proceed with checking the side slope stability

*Step 6: Determine the angle of repose for Class AI riprap*

$$\phi = 42^\circ$$

*Step 7: Determine  $K_1$*

From Appendix 7E-7, for  $B/d = 2 / 1.56 = 1.28$  and  $z = 2$ , read  $K_1 = 0.9$

*Step 8: Determine  $K_2$*

From Appendix 7E-8 for  $z = 2$  and angle of repose =  $42^\circ$ , read  $K_2 = 0.73$ .

Step 9: Determine the  $D_{50}$  for the side slopes

$$D_{50 \text{ side}} = \frac{K_1}{K_2} D_{50 \text{ bottom}} = \frac{0.90}{0.73} (0.8) = 0.98 \text{ ft.}$$

Therefore, it would probably be best to use Standard Class I riprap ( $D_{50} = 1.1$  ft) for both the channel bottom and side slopes in lieu of the originally proposed Class AI ( $D_{50} = 0.8$  ft).

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## 7.6 References

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**Appendix 7A-1      Definitions and Abbreviations**

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**Abbreviations:**

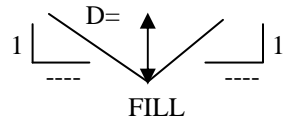
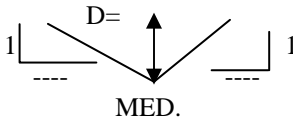
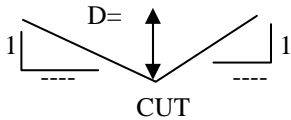
BRI-STARS	Bridge Stream Tube Model for Sediment Routing Alluvial River Simulation
FEMA	Federal Emergency Management Agency
FHWA	Federal Highway Administration
NTIS	National Technical Information Service
USCOE/USACE	United States Corps of Engineers
VDOT	Virginia Department of Transportation

SYMBOL	DEFINITION	UNITS
A	Cross-sectional area of flow	ft <sup>2</sup>
$\alpha$	Velocity distribution coefficient	-
C	Stone size correction factor	-
C <sub>sg</sub>	Adjustment to the specific gravity of stone	-
C <sub>SF</sub>	Adjustment to the stability factor	-
d	Depth of flow	ft
D <sub>50s</sub>	Required D <sub>50</sub> for side slopes	ft
d <sub>c</sub>	Critical depth	ft
d <sub>n</sub>	Normal depth	ft
d <sub>avg</sub>	Average flow depth in the main channel	ft
E	Specific energy	ft
F <sub>r</sub>	Froude Number	-
g	Acceleration due to gravity	ft/s <sup>2</sup>
h <sub>L</sub>	Total head loss due to local minor and friction losses	ft
h	Stage or head	ft
h <sub>f</sub>	Friction loss	ft
H <sub>D</sub>	Average hydraulic depth	ft
h <sub>o</sub>	Summation of minor losses	ft
K	Channel conveyance	-
L	Discharge-weighted or conveyance reach length	ft
n	Manning's roughness coefficient	-
P	Wetted perimeter	ft
Q	Discharge	cfs
R	Hydraulic radius	ft
S <sub>o</sub>	Channel slope	ft/ft
S	Slope of the energy grade line	ft/ft
S <sub>A</sub>	Average slope of the energy grade line	ft/ft
S <sub>g</sub>	Specific gravity of rock riprap	-
SF	Stability factor applied	-
T	Top width at the water surface	ft
$\tau_o$	Average tractive force	lbs/ft <sup>2</sup>
$\tau_{max}$	Maximum tractive force	lbs/ft <sup>2</sup>
$\tau_p$	Permissible shear stress	lbs/ft <sup>2</sup>
$\tau_s$	Side slope shear stress	lbs/ft <sup>2</sup>
TW	Tailwater depth above invert of culvert	ft
V	Mean velocity	ft/s
V <sub>a</sub>	Average velocity in main channel	ft/s
Z	Elevation head	ft
S <sub>f</sub>	Mean slope of the energy grade line	ft/ft
$\gamma$	Unit weight of water	lb/ft <sup>3</sup>
$\theta$	Side slope angle	deg.
$\phi$	Angle of repose of material	deg.

# Appendix 7B-1 LD-268 Roadside and Median Ditch Design Form

LD-268

\_\_\_ LANE  
\_\_\_ SIDE




PROJECT \_\_\_\_\_.  
BY \_\_\_\_\_.  
DATE \_\_\_\_\_.  
SHEET \_\_\_\_\_ of \_\_\_\_\_.

STA. TO STA.	FLOW	0.9 WS/ CA	0.5 WS/ CA	0.3 WS/ CA	CA		T <sub>c</sub>	I <sub>2</sub>	Q <sub>2</sub>	C or F	Slope Ft/Ft	ALLOW VEL.	Earth	Protective Lining			I <sub>10</sub>	Q <sub>10</sub>	DEP	REMARKS
					n=.03	n=.05							n=.015							
					VEL.	Q <sub>n</sub>							VEL	DEP	Q <sub>n</sub>	DEP				



Appendix 7B-2 Water Surface Profile Calculation Form



Worksheet For Non-Uniform Flow In Open Channels

STA.	$H_1$	$d_1$	$d_1 + \frac{V_1^2}{2g} + Z_1$	depth	X-Section Area of Station 2	Wetted Perimeter	$A_2 \div P_2$	$V_2$	$V_2^2 \div 2g$	Flow Line Elevation	Distance Between Stations	$S_1$	$S_2$	$S_1 + S_2$	$h_1$	$H_2$	$d_2 + \frac{V_2^2}{2g} + Z_2$	Tolerance = $\pm$	
					$A_2 \div P_2$	$P_2$	$R_2$	$A_2 \div V_2$	$V_2^2 \div 2g$	$Z_2$	L	$\frac{Q^2 \cdot n^2}{2.25 \cdot R_1^3 \cdot A_1^2}$	$\frac{Q^2 \cdot n^2}{2.25 \cdot R_2^3 \cdot A_2^2}$	$\frac{S_1 + S_2}{2}$	$h_1$	$H_2$	$H_{up} = H_{down} + h_1$		

Source:

Appendix 7B-3

CHANNEL STABILITY WORK SHEET

CHANNEL DATA

Q = \_\_\_\_\_(cfs)      P = \_\_\_\_\_(ft.)      Native Material  
 S<sub>o</sub> = \_\_\_\_\_(ft/ft)      R = \_\_\_\_\_(ft.)      D<sub>50</sub> = \_\_\_\_\_  
 d<sub>n</sub> = \_\_\_\_\_(ft.)      V<sub>n</sub> = \_\_\_\_\_(fps)      D<sub>75</sub> = \_\_\_\_\_  
 A = \_\_\_\_\_(ft<sup>2</sup>)      Side Slope = \_\_\_\_\_ :1      n = \_\_\_\_\_

STABILITY OF NATIVE MATERIAL

$\tau_o = 62.4 \cdot R \cdot S_o = 62.4 \cdot \underline{\hspace{2cm}} \cdot \underline{\hspace{2cm}} = \underline{\hspace{2cm}}$

$\tau_p \text{ Bed} = \underline{\hspace{2cm}}$  (Appendix 7E-2 or 3)

For D<sub>50</sub> = \_\_\_\_\_  $\phi = \underline{\hspace{2cm}}^\circ$  (Appendix 7E-1)

For D<sub>75</sub> = \_\_\_\_\_  $\phi = \underline{\hspace{2cm}}^\circ$  (Appendix 7E-9)

Side Slope = \_\_\_\_\_ :1     $\theta = \underline{\hspace{2cm}}^\circ$

$K_1 = [1 - (\sin^2 \theta / \sin^2 \phi)]^{0.5}$

$K_1 = [1 - (\sin^2 \underline{\hspace{2cm}}^\circ / \sin^2 \underline{\hspace{2cm}}^\circ)]^{0.5} = \underline{\hspace{2cm}}$

$\tau_s \text{ Side Slope (SS)} = \tau_p \text{ Bed} \cdot K = \underline{\hspace{2cm}} \cdot \underline{\hspace{2cm}} = \underline{\hspace{2cm}}$

$\tau_p \text{ Bed} ( \underline{\hspace{2cm}} ) (<) (=) (>) \tau_o ( \underline{\hspace{2cm}} )$

∴ Native Material on Bed is (stable) (unstable)

$\tau_s \text{ SS} ( \underline{\hspace{2cm}} ) (<) (=) (>) \tau_o ( \underline{\hspace{2cm}} )$

∴ Native Material on Side Slope is (stable) (unstable)

Source:      VDOT

Appendix 7B-4

RIPRAP DESIGN WORK SHEET  
FOR STANDARD VDOT RIPRAP SIZES ONLY

---

CHANNEL DATA

Q = \_\_\_\_\_(cfs)      P = \_\_\_\_\_(ft.)      n = \_\_\_\_\_  
S<sub>o</sub> = \_\_\_\_\_(ft/ft)      R = \_\_\_\_\_(ft.)  
d<sub>n</sub> = \_\_\_\_\_(ft.)      V<sub>n</sub> = \_\_\_\_\_(fps)  
A = \_\_\_\_\_(ft.<sup>2</sup>)      Side Slope = \_\_\_\_\_ :1

DETERMINE RIPRAP SIZE

φ = 42° Side Slope = \_\_\_\_\_ :1    θ = \_\_\_\_\_°

$$K_1 = [1 - (\sin^2 \theta / \sin^2 \phi)]^{0.5}$$

$$K_1 = [1 - (\sin^2 \text{_____}^\circ / \sin^2 42^\circ)]^{0.5} = \text{_____}$$

For Specific Gravity = 2.65 and Stability Factor = 1.2

$$D_{50} = 0.001 \cdot V_a^3 / (d_{avg}^{0.5} \cdot K_1^{1.5})$$

$$D_{50} = 0.001 \cdot \text{_____}^3 / (\text{_____}^{0.5} \cdot \text{_____}^{1.5})$$

D<sub>50</sub> Computed = \_\_\_\_\_

Note: All VDOT standard riprap (Class AI through Type II) is assumed to have a φ of approximately 42° and a Specific Gravity of 2.65. Therefore, the Computed D<sub>50</sub> should be adjusted by the Stability Correction Factor (C<sub>SF</sub>) (if any) to derive a Final D<sub>50</sub>. The VDOT standard class of riprap with the next higher D<sub>50</sub> should be specified.

Correction Factor For Stability Factor (SF) other than 1.2 (Default = 1.0)

$$C_{SF} = (SF / 1.2)^{1.5} = (\text{_____} / 1.2)^{1.5} = \text{_____}$$

$$\text{Final } D_{50} = C_{SF} \cdot \text{Computed } D_{50} = \text{_____} \cdot \text{_____} = \text{_____}$$

RIPRAP RECOMMENDATION: VDOT (Class) (Type) \_\_\_\_\_

Thickness (T) = \_\_\_\_\_" (2 • D<sub>50</sub> MSD minimum)

---

Source:      VDOT

Appendix 7B-5

RIPRAP DESIGN WORK SHEET  
FOR OTHER THAN VDOT STANDARD RIPRAP SIZES

CHANNEL DATA

Q = \_\_\_\_\_(cfs)                      P = \_\_\_\_\_(ft.)                      n = \_\_\_\_\_

S<sub>0</sub> = \_\_\_\_\_(ft/ft)                      R = \_\_\_\_\_(ft.)

d<sub>n</sub> = \_\_\_\_\_(ft.)                      V<sub>n</sub> = \_\_\_\_\_(fps)

A = \_\_\_\_\_(ft<sup>2</sup>)                      Side Slope = \_\_\_\_\_ :1

ASSUMED RIPRAP SIZE - D<sub>50</sub> = \_\_\_\_\_

VERIFY ASSUMED RIPRAP SIZE

φ = \_\_\_\_\_° (Appendix 7E-1)

Side Slope = \_\_\_\_\_ : 1      θ = \_\_\_\_\_°

K<sub>1</sub> = [1 - (sin<sup>2</sup> θ / sin<sup>2</sup> φ)]<sup>0.5</sup>

K<sub>1</sub> = [1 - (sin<sup>2</sup> \_\_\_\_\_° / sin<sup>2</sup> \_\_\_\_\_°)]<sup>0.5</sup> = \_\_\_\_\_

For Specific Gravity = 2.65 and Stability Factor = 1.2

D<sub>50</sub> = 0.001 • V<sub>a</sub><sup>3</sup> / (d<sub>avg</sub><sup>0.5</sup> • K<sub>1</sub><sup>1.5</sup>)

D<sub>50</sub> = 0.001 • \_\_\_\_\_<sup>3</sup> / ( \_\_\_\_\_<sup>0.5</sup> • \_\_\_\_\_<sup>1.5</sup>) = \_\_\_\_\_

D<sub>50</sub> Computed ( \_\_\_\_\_ ) (<) (=) (>) D<sub>50</sub> Assumed ( \_\_\_\_\_ )

Assumed D<sub>50</sub> is (correct) (incorrect)

Note: The above process of assuming a D<sub>50</sub> size, determining the natural angle of repose (φ) and computing a D<sub>50</sub> size should be repeated until the Assumed D<sub>50</sub> size equals the Computed D<sub>50</sub> size. Once the D<sub>50</sub> size determination has been made, it should be adjusted for the Specific Gravity Correction Factor C<sub>sg</sub> (if any) and the Stability Correction Factor (C<sub>SF</sub>) (if any) to derive a Final D<sub>50</sub>.

Correction Factor For Riprap Specific Gravity (S<sub>s</sub>) other than 2.65 (Default = 1.0)

C<sub>sg</sub> = 2.12 / (S<sub>s</sub> - 1)<sup>1.5</sup> = 2.12 / ( \_\_\_\_\_ - 1)<sup>1.5</sup> = \_\_\_\_\_

Correction Factor For Stability Factor (SF) other than 1.2 (Default = 1.0)

C<sub>SF</sub> = (SF / 1.2)<sup>1.5</sup> = ( \_\_\_\_\_ / 1.2)<sup>1.5</sup> = \_\_\_\_\_

Final Correction Factor = C = C<sub>sg</sub> • C<sub>SF</sub> = \_\_\_\_\_ • \_\_\_\_\_ = \_\_\_\_\_

Final D<sub>50</sub> = C • Computed D<sub>50</sub> = \_\_\_\_\_ • \_\_\_\_\_ = \_\_\_\_\_

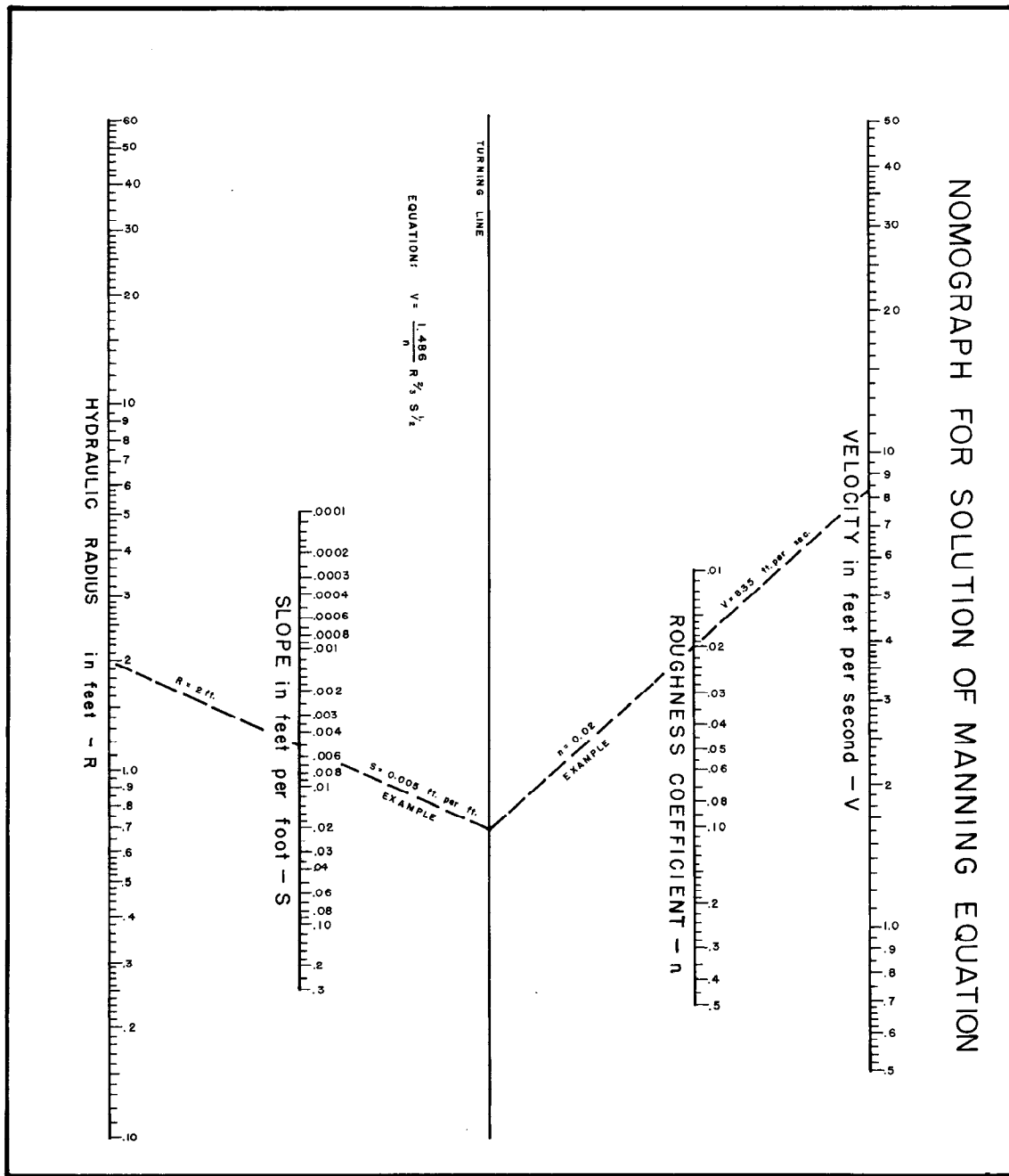
RIPRAP RECOMMENDATION: \_\_\_\_\_

Thickness (T) = \_\_\_\_\_" (2 • D<sub>50</sub> MSD minimum)

Source:            VDOT

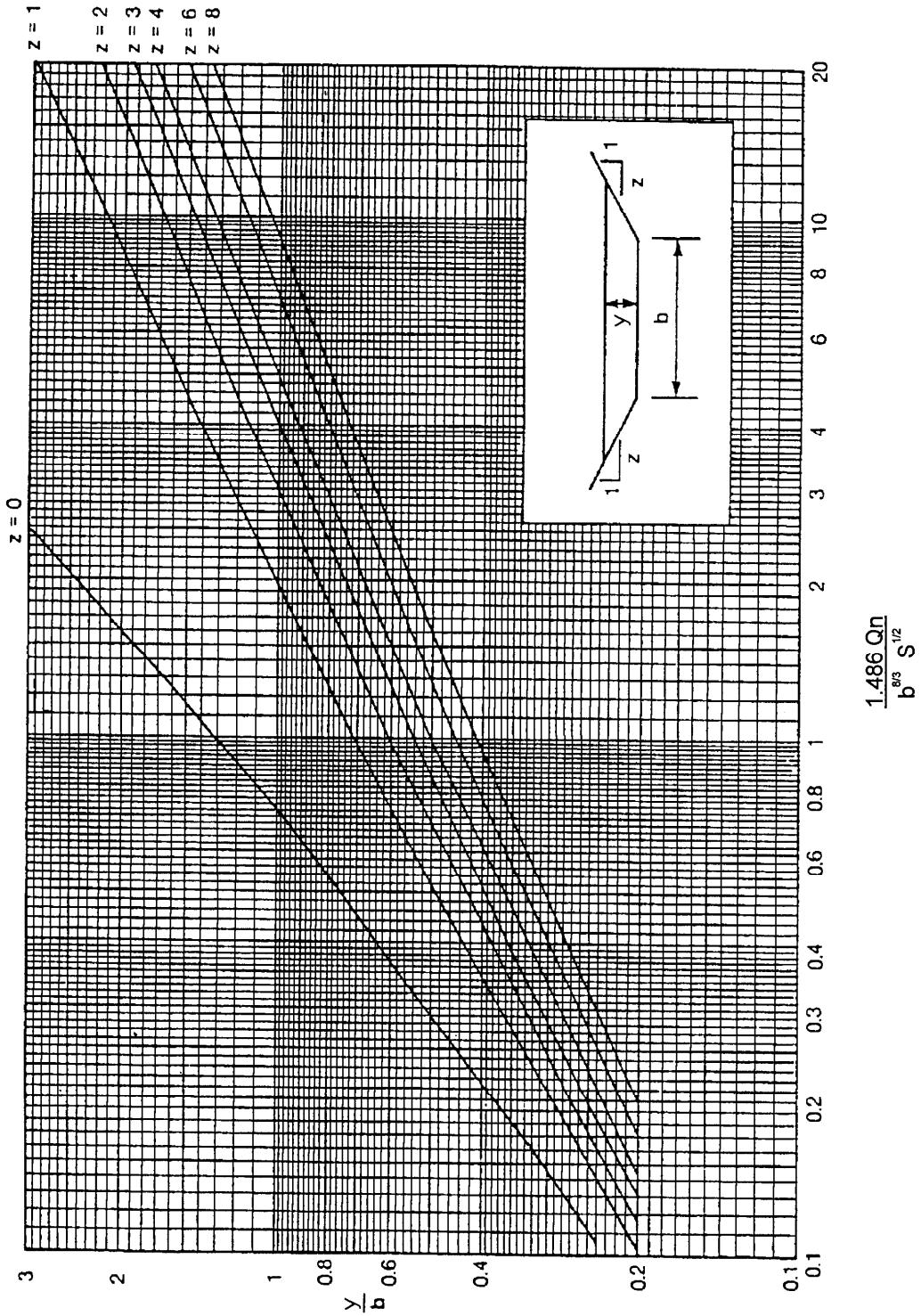
Appendix 7C-1

Nomograph for Solution of Manning's Equation



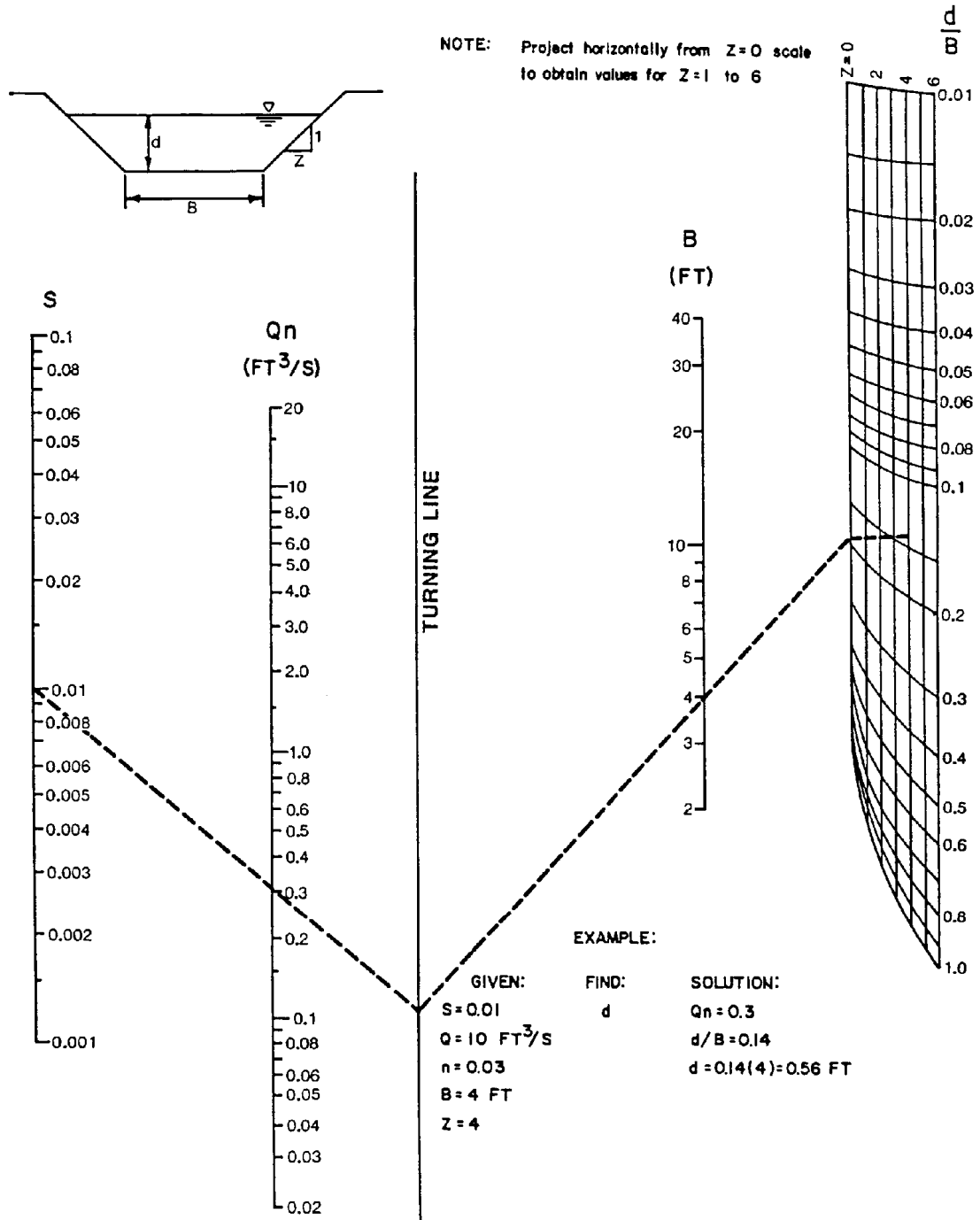
Source: VDOT

Appendix 7C-2 Trapezoidal Channel Capacity Chart



Source:

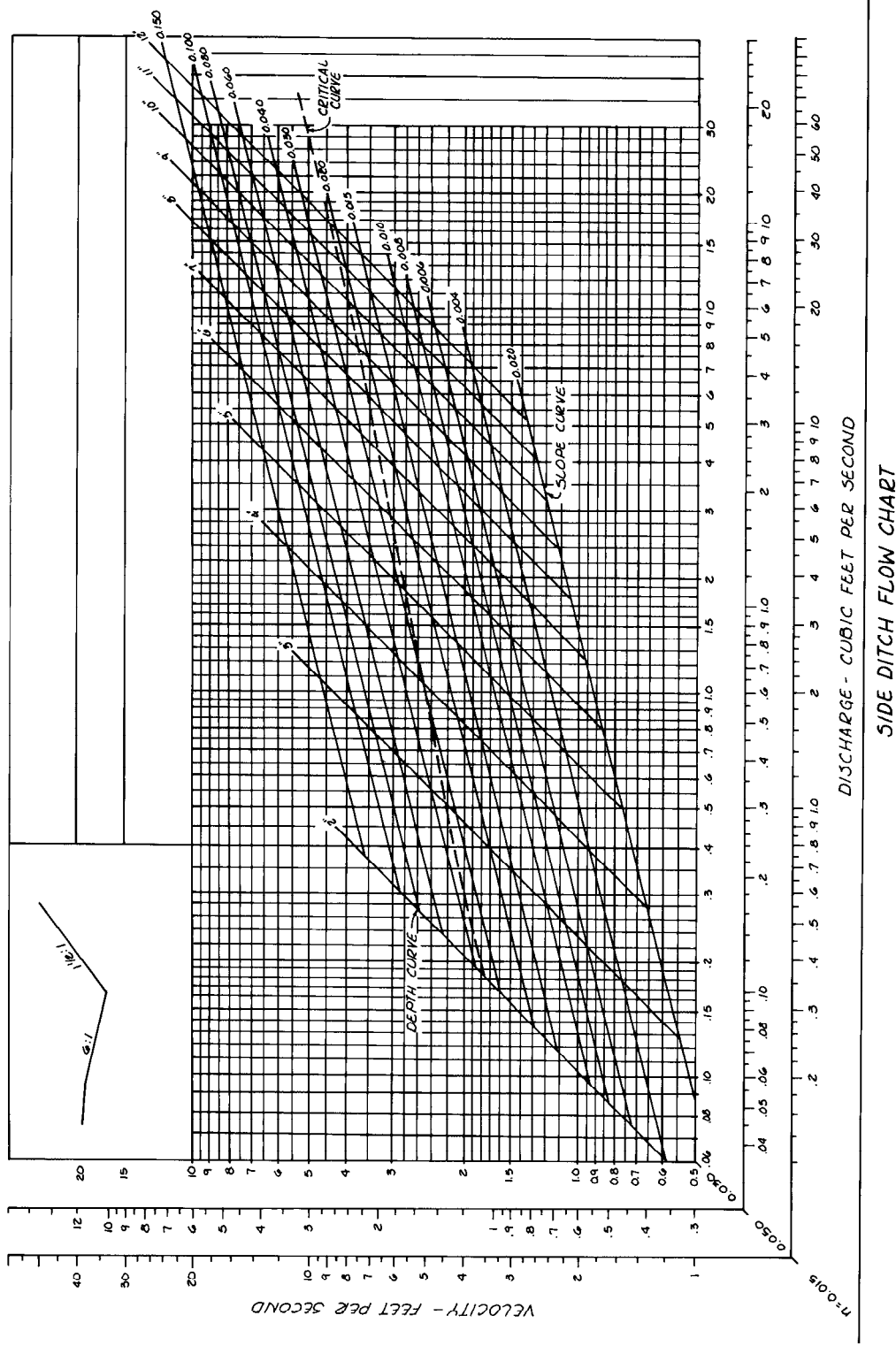
Appendix 7C-3 Nomograph for Solution of Normal Depth



Source: HEC-15 (Archived) 1988

Appendix 7C-4

Side Ditch Flow Chart  
(Side Slopes = 6:1, 1.5:1)



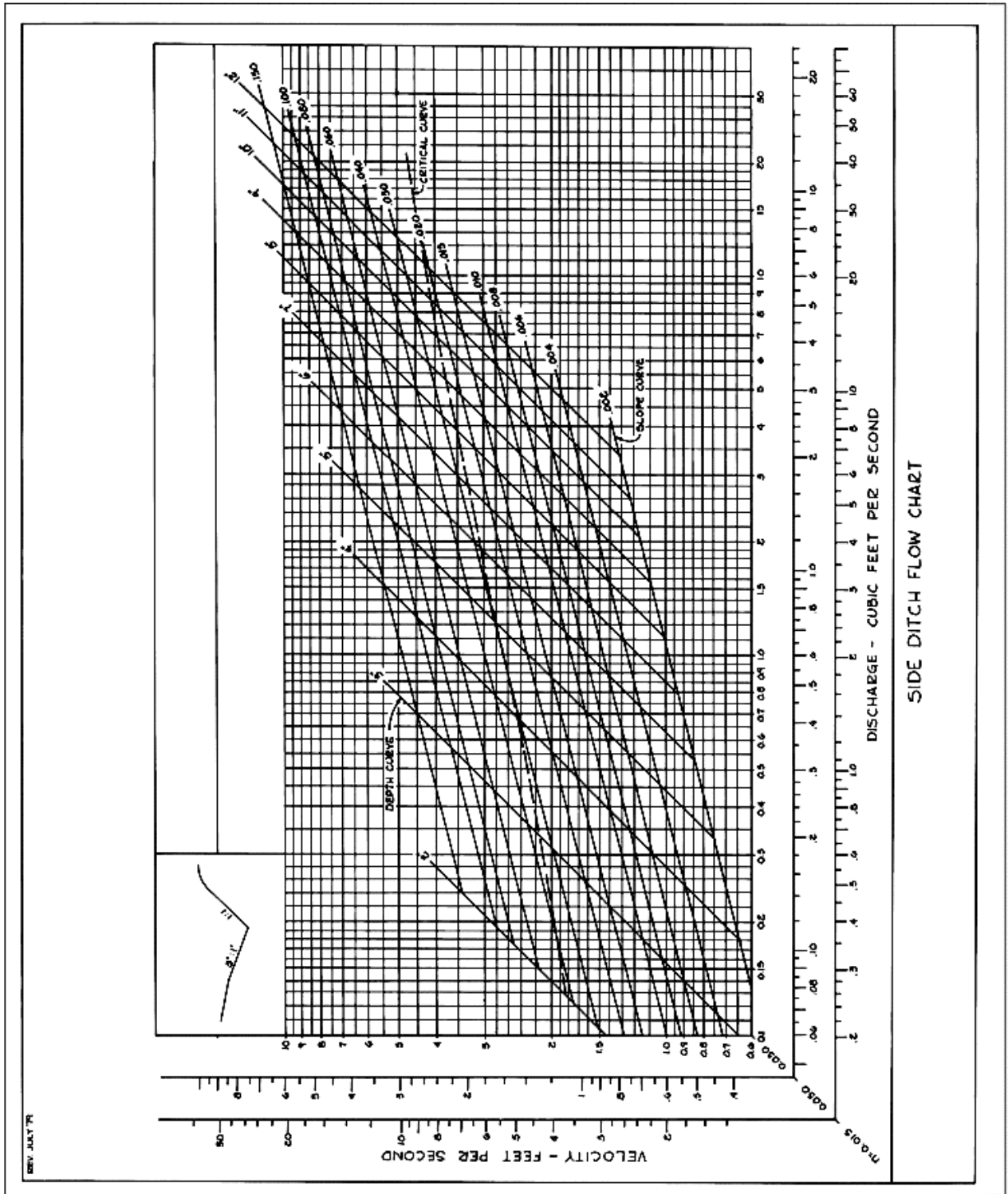
Source: VDOT



Appendix 7C-5

Side Ditch Flow Chart  
(Side Slopes = 4:1, 1:1)

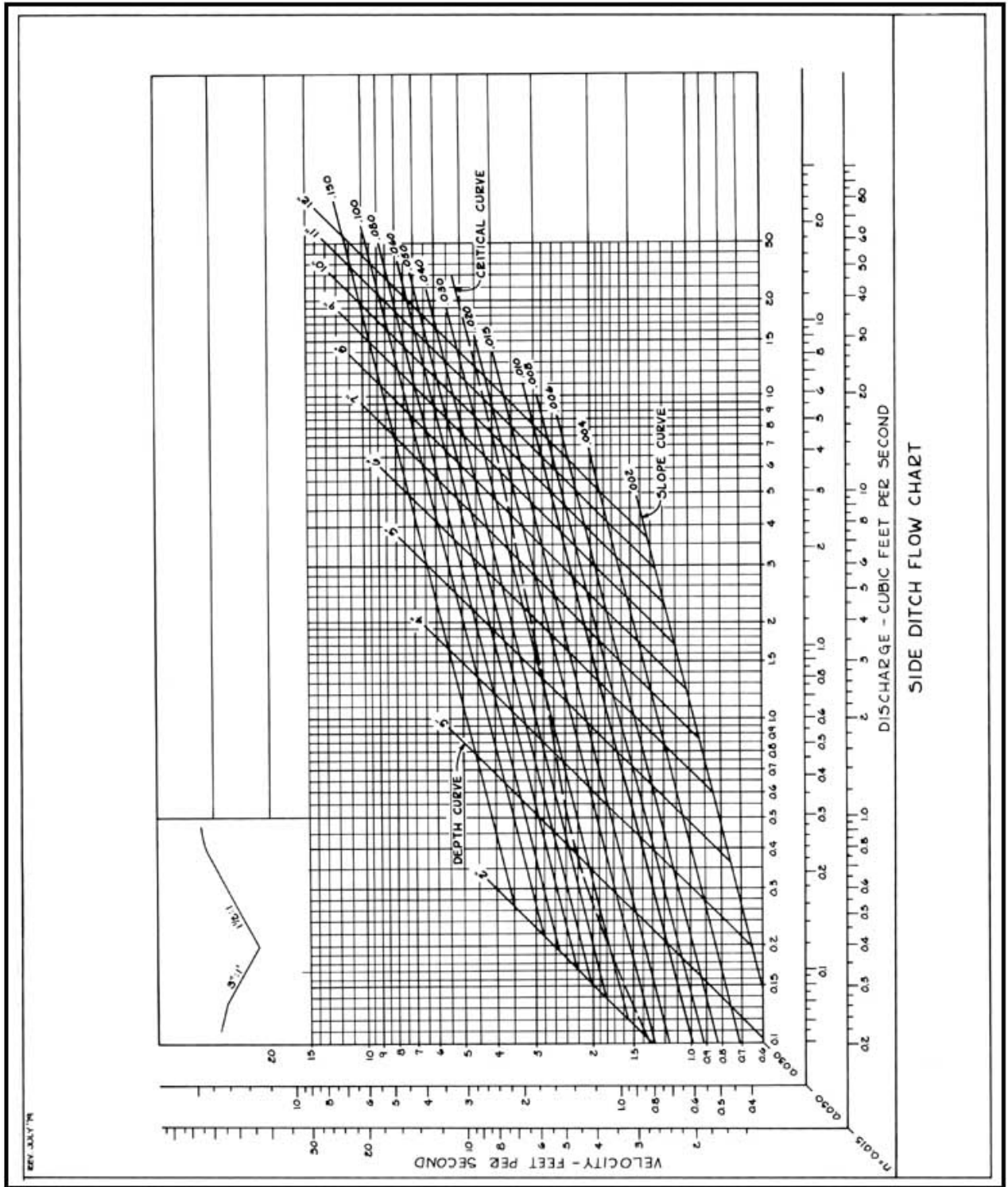
SD-2



Source: VDOT

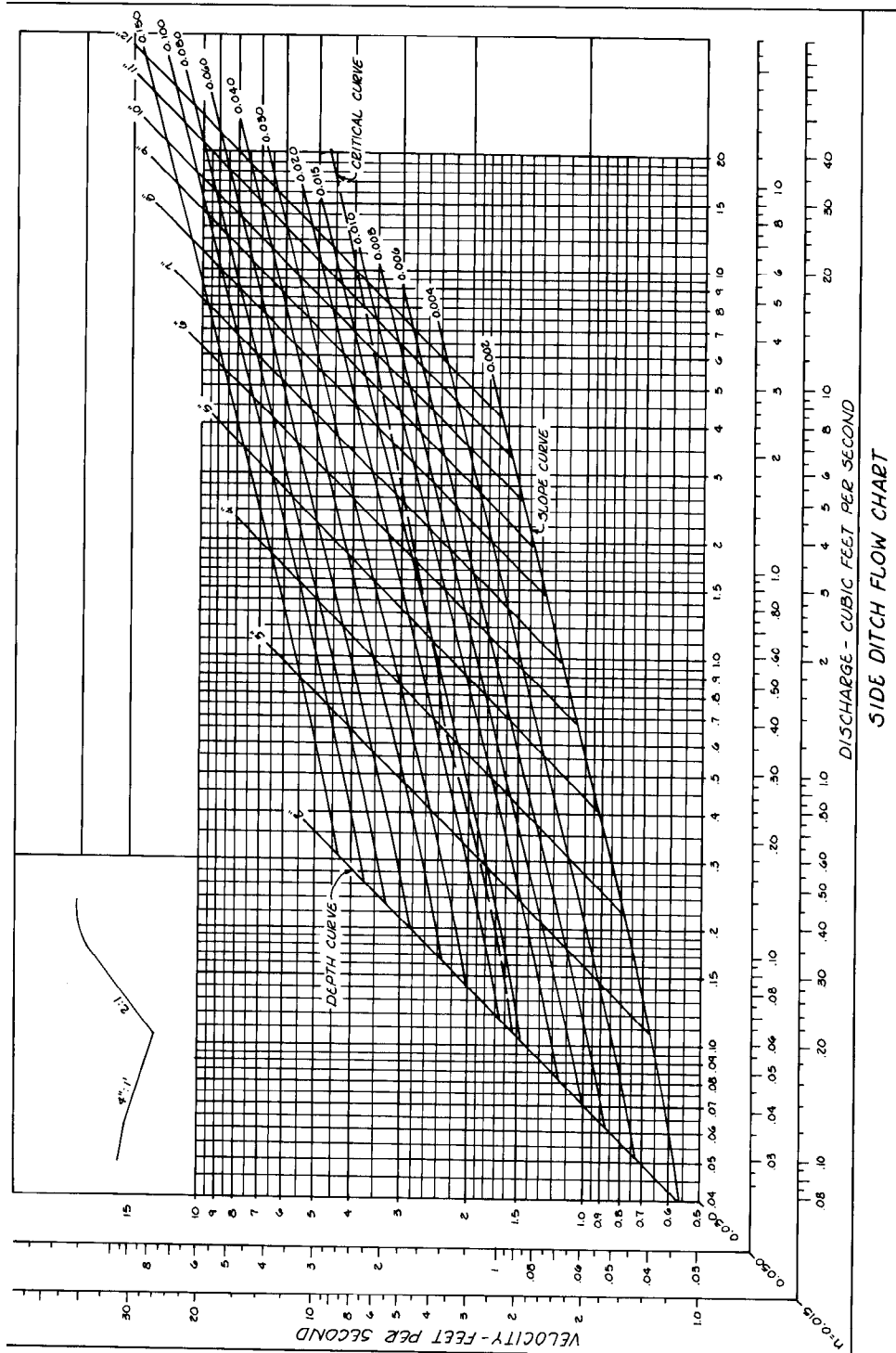
Appendix 7C-6

Side Ditch Flow Chart  
(Side Slopes = 4:1, 1.5:1)



Source: VDOT

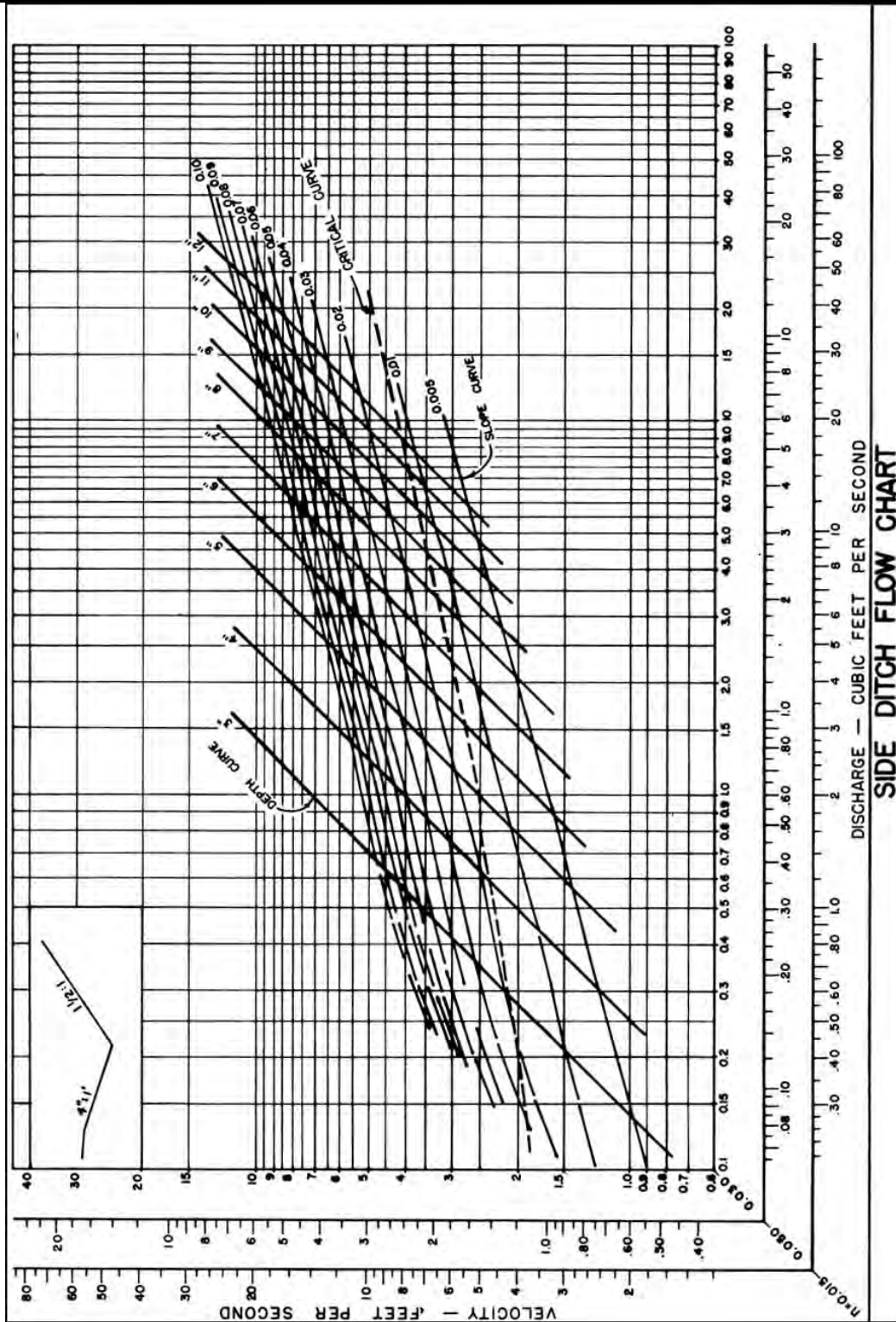
Appendix 7C-7 Side Ditch Flow Chart (Side Slopes = 3:1, 2:1)



Source:

Appendix 7C-8

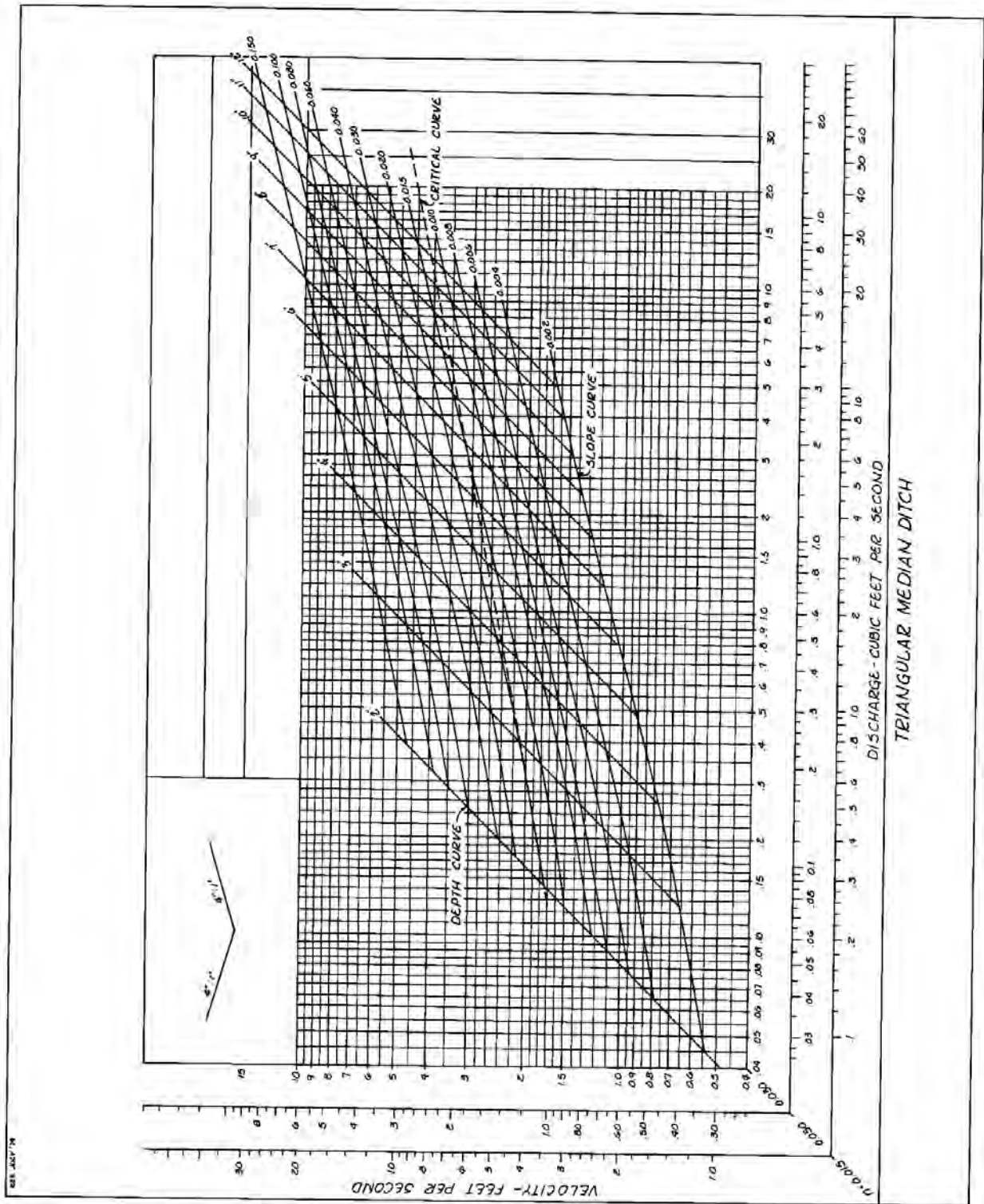
Side Ditch Flow Chart  
(Side Slopes = 3:1, 1.5:1)



Source: VDOT

Appendix 7C-9

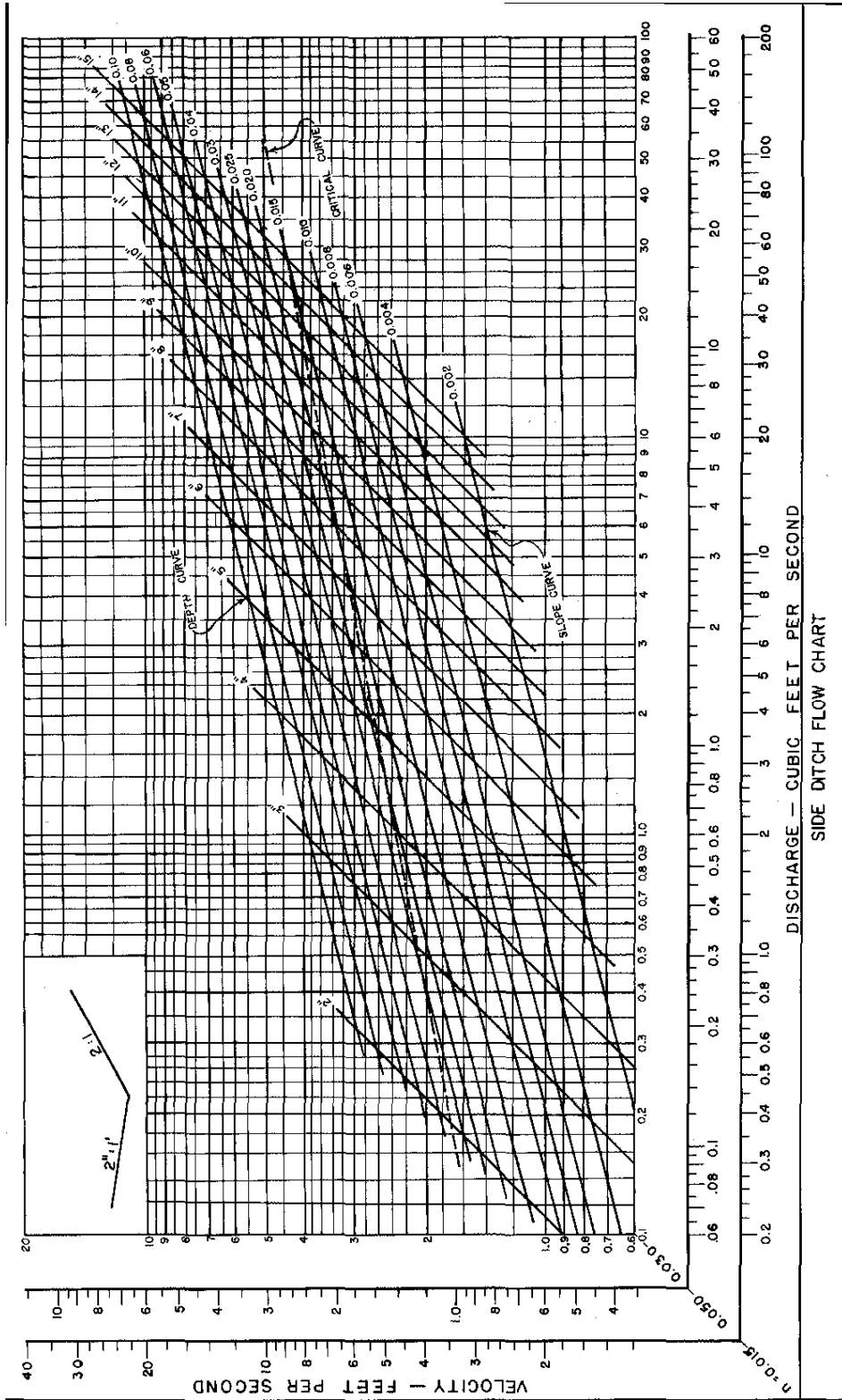
Side Ditch Flow Chart  
(Side Slopes = 3:1, 3:1)



Source: VDOT

Appendix 7C-10

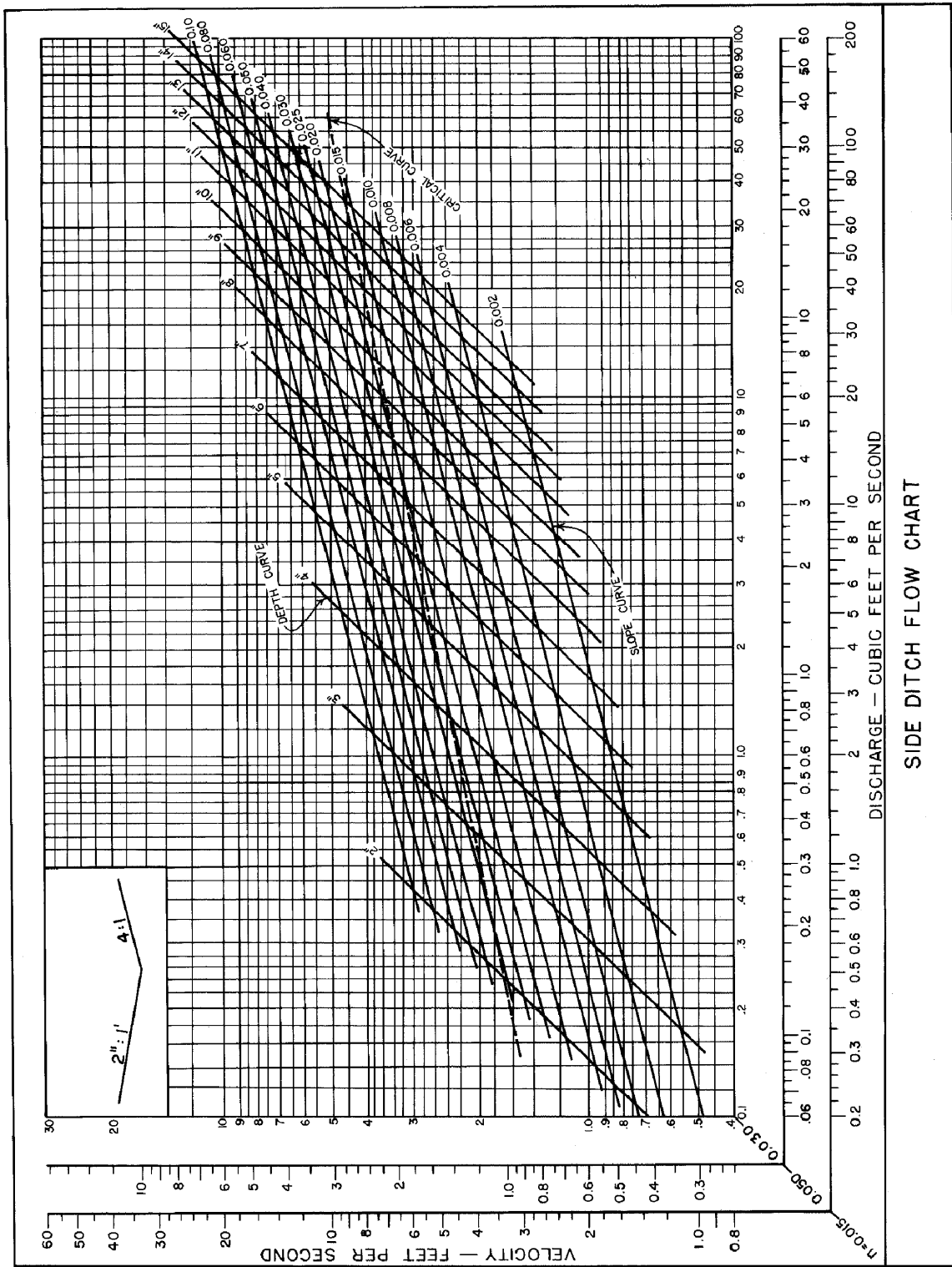
Side Ditch Flow chart  
(Side Slopes = 6:1, 2:1)



Source: VDOT

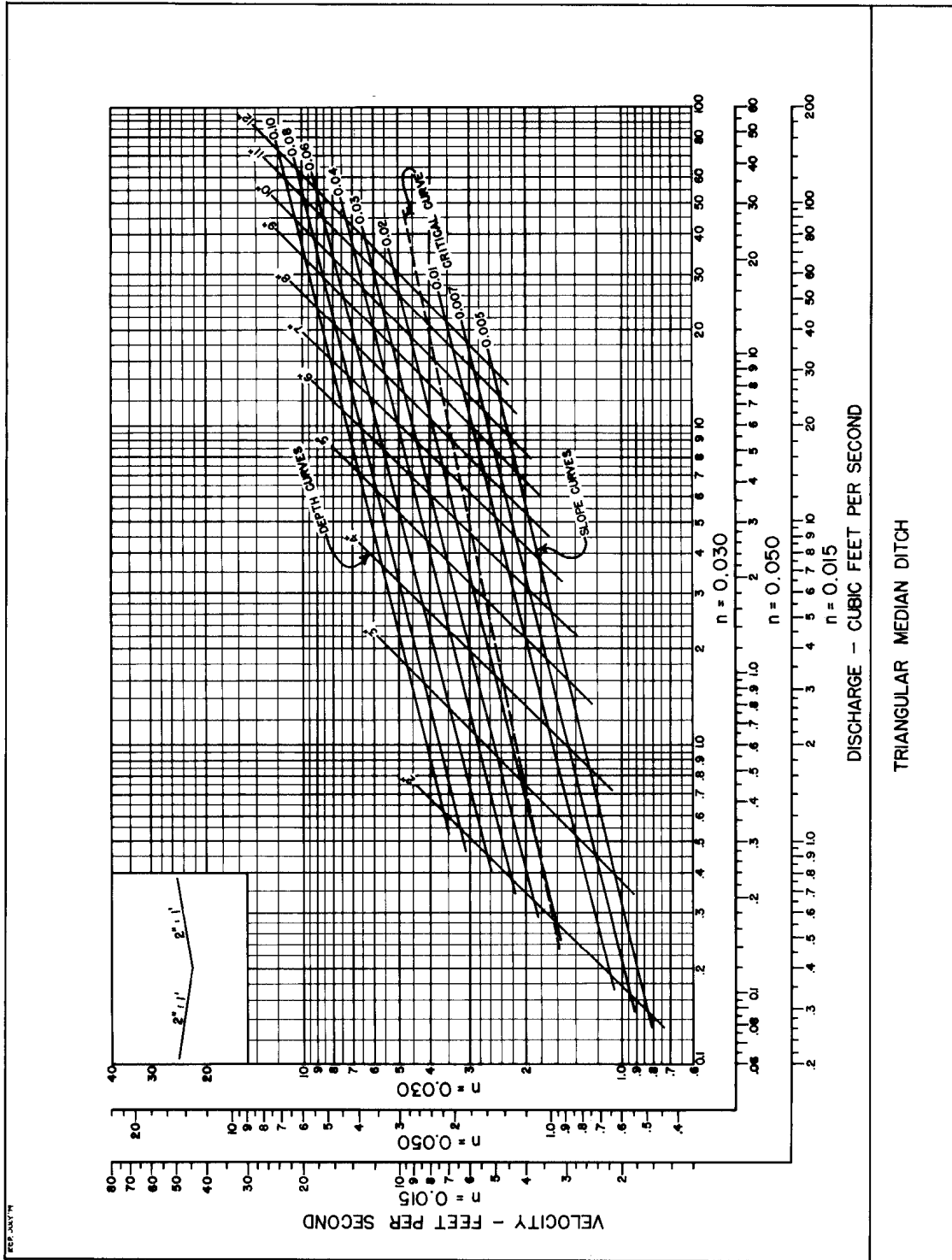
Appendix 7C-11

Side Ditch Flow Chart  
(Side Slopes = 6:1, 4:1)



Source: VDOT

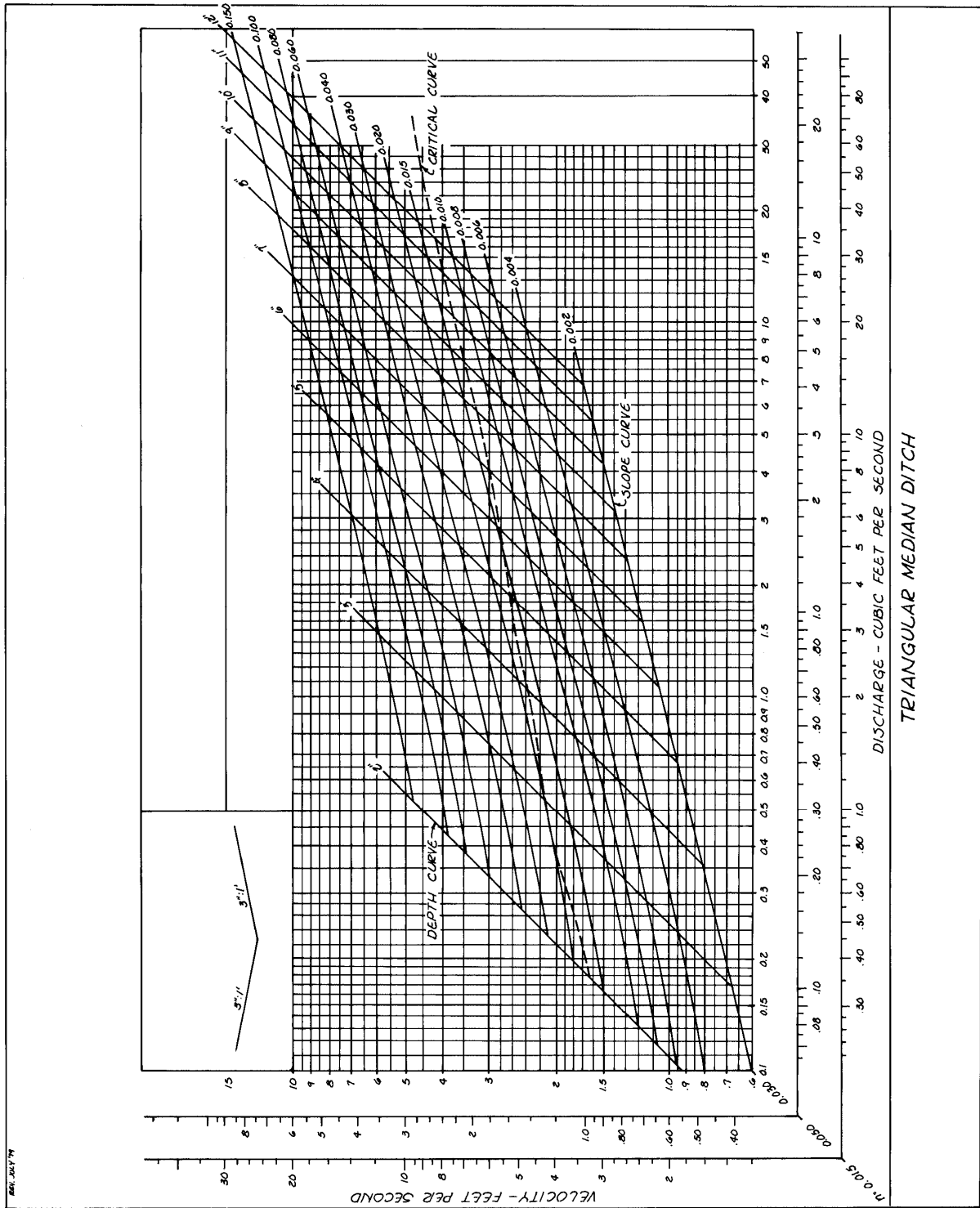
Appendix 7C-12 Triangular Median Ditch Flow Chart  
(Side Slopes = 6:1, 6:1)



Source: VDOT

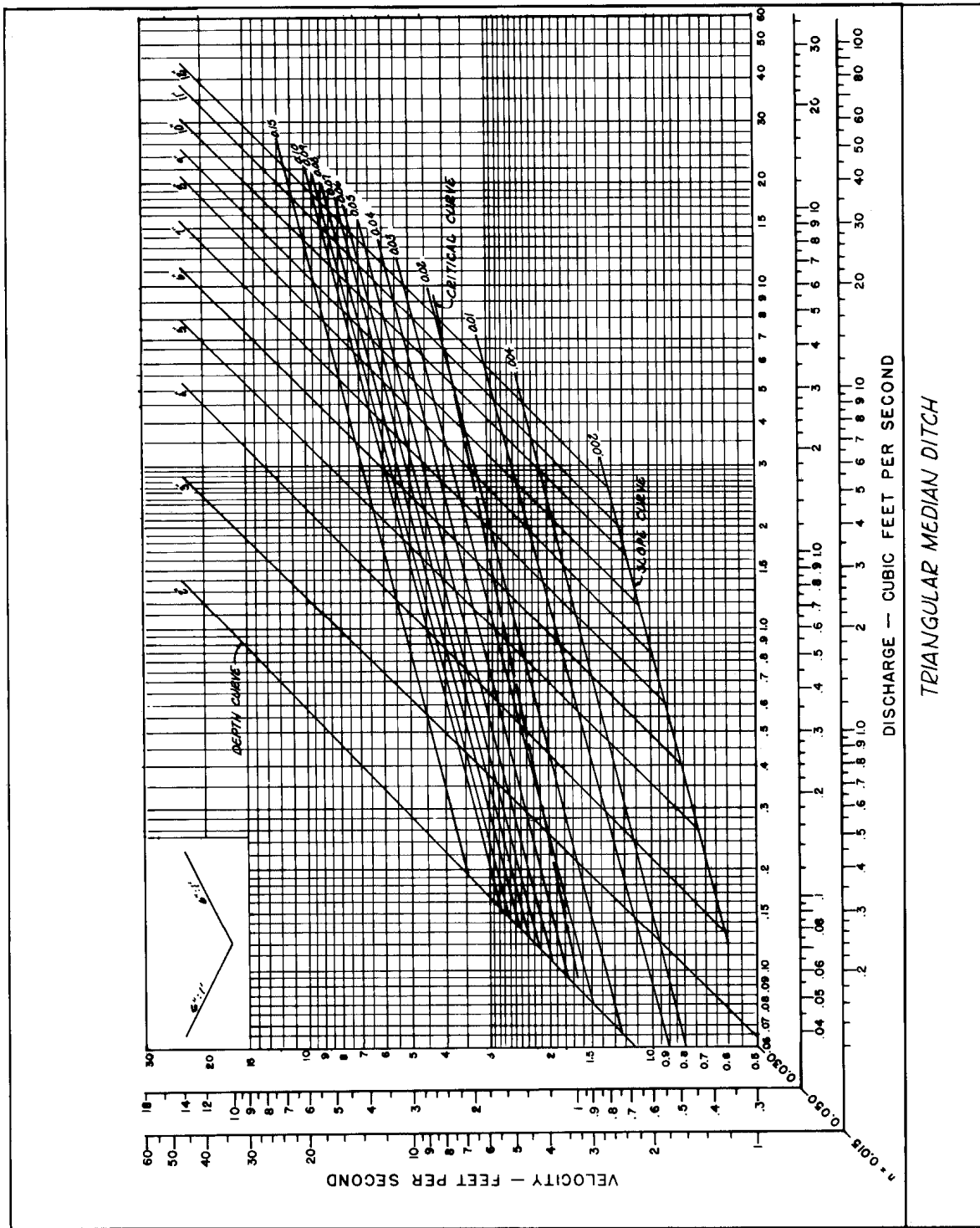


Appendix 7C-13 Triangular Median Ditch Flow Chart  
(Side Slopes = 4:1, 4:1)



Source: VDOT

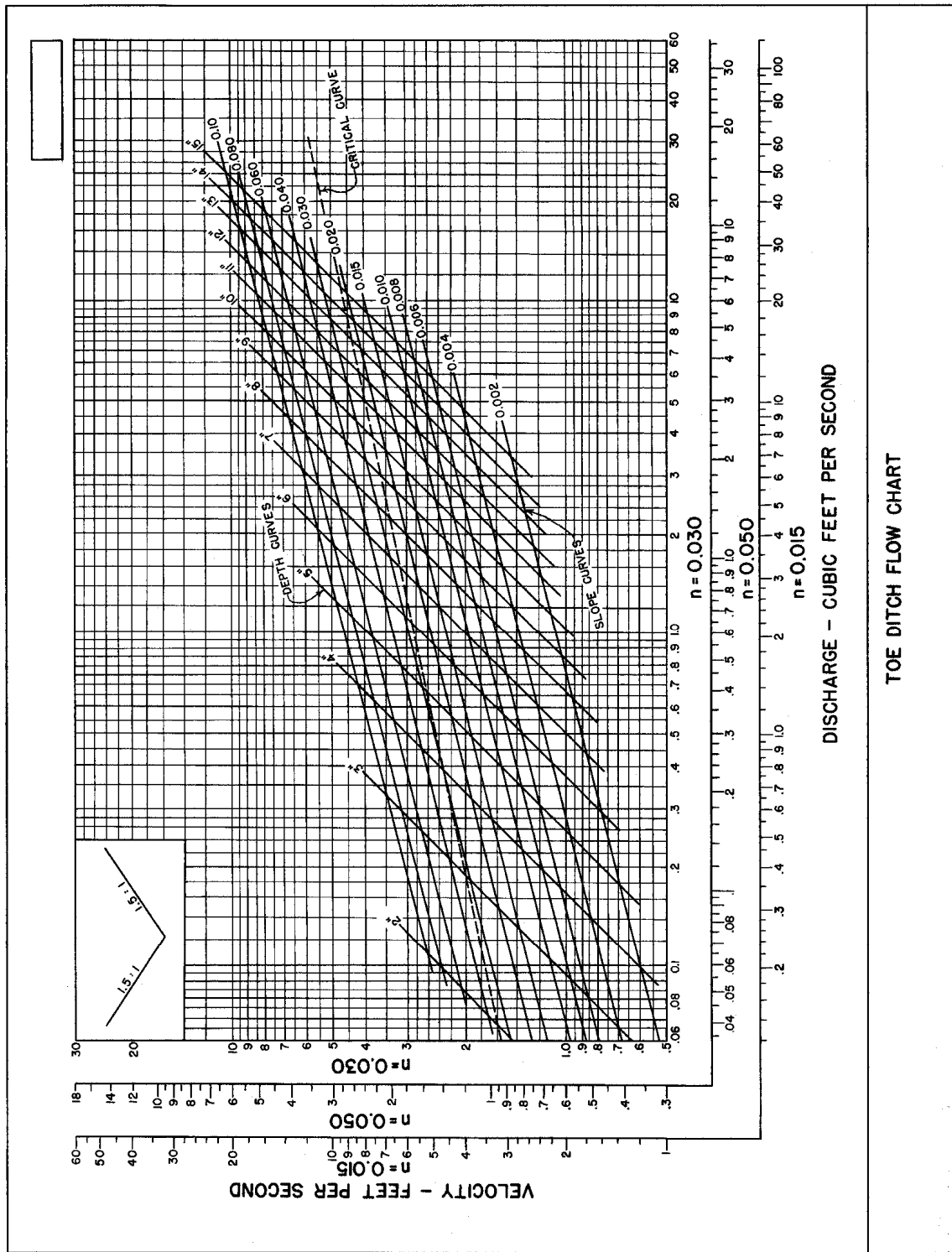
Appendix 7C-14 Triangular Median Ditch Flow Chart  
(Side Slopes = 2:1, 2:1)



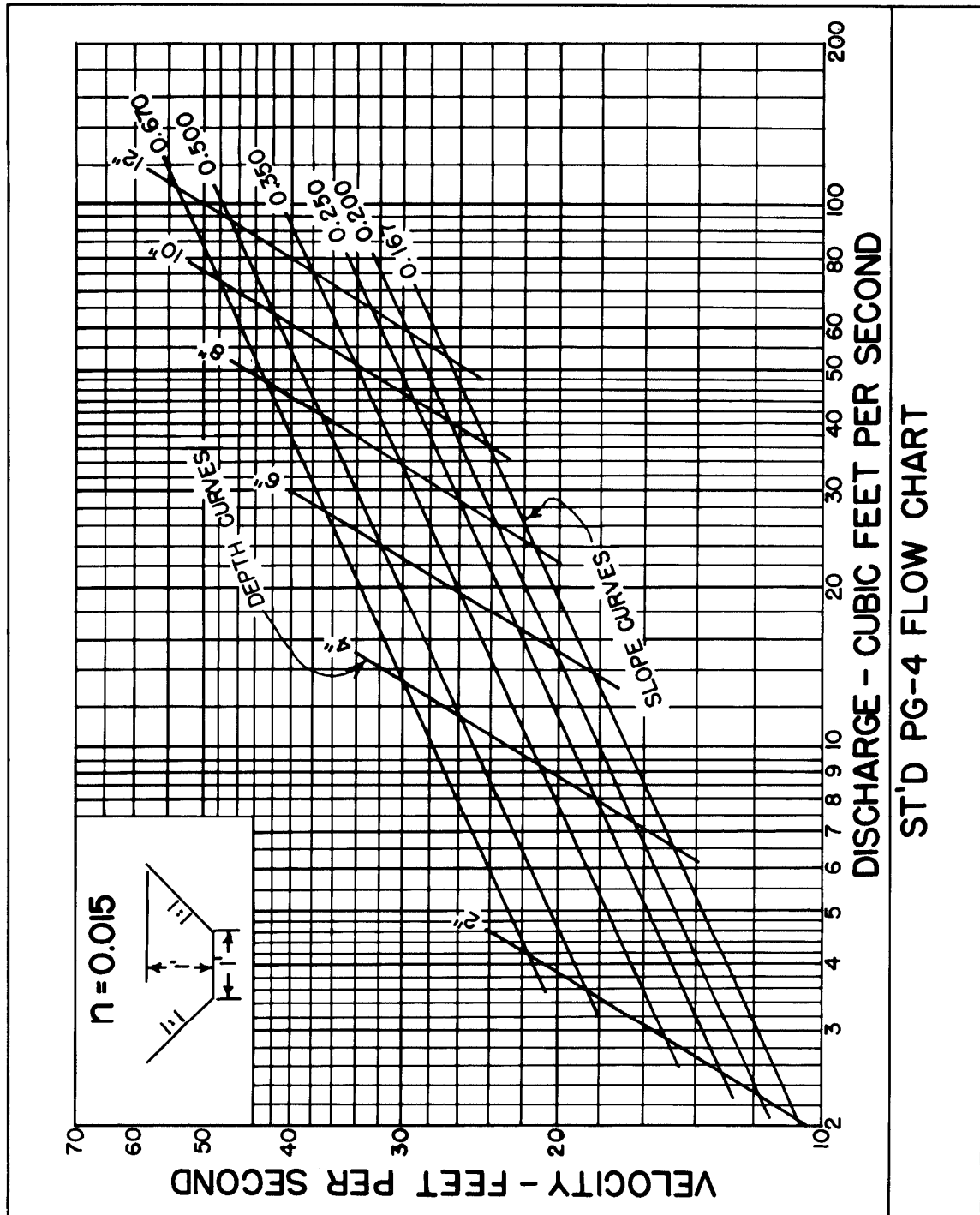
Source: VDOT

Appendix 7C-15

Toe Ditch Flow Chart  
(Side Slopes = 1.5:1, 1.5:1)

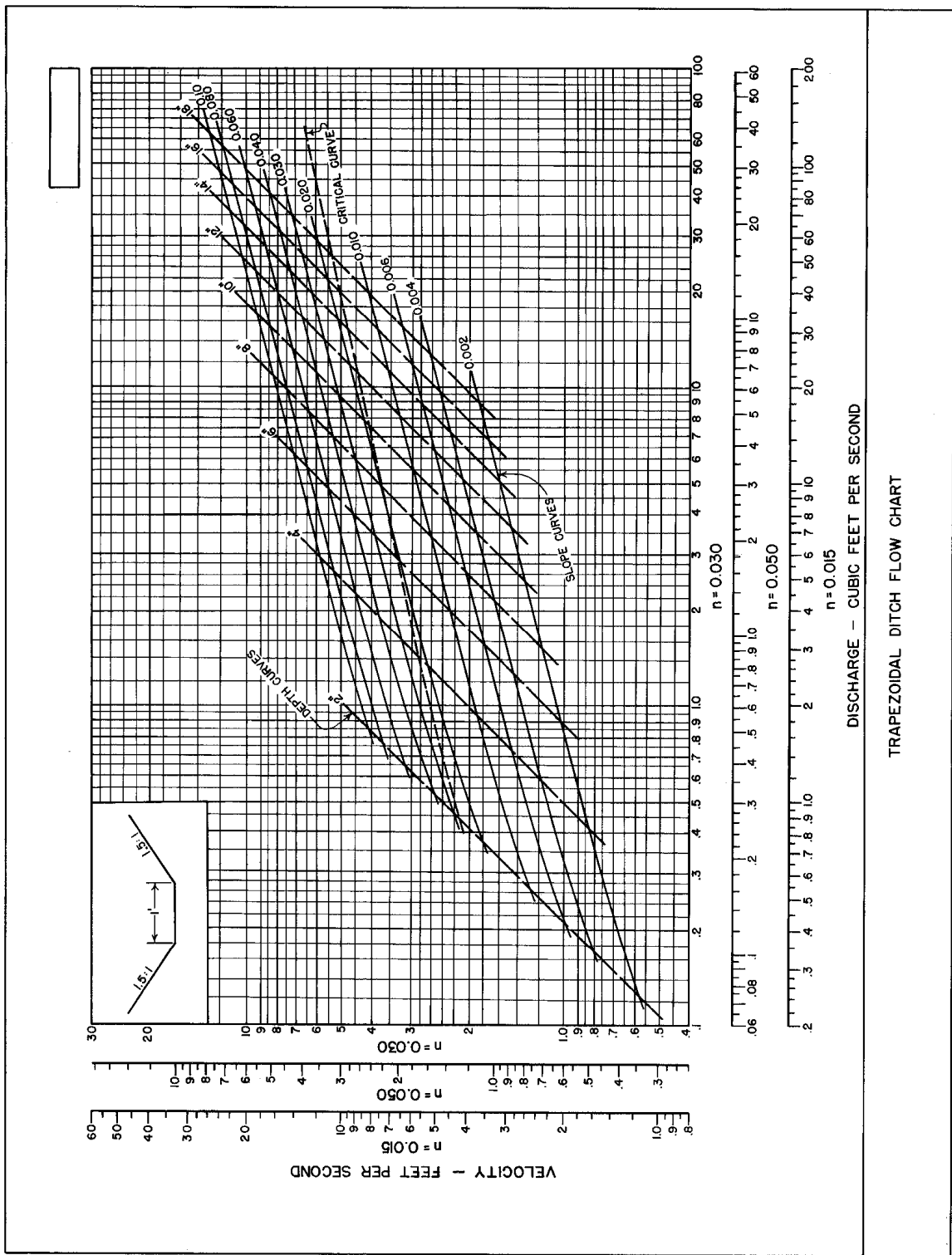


Source: VDOT



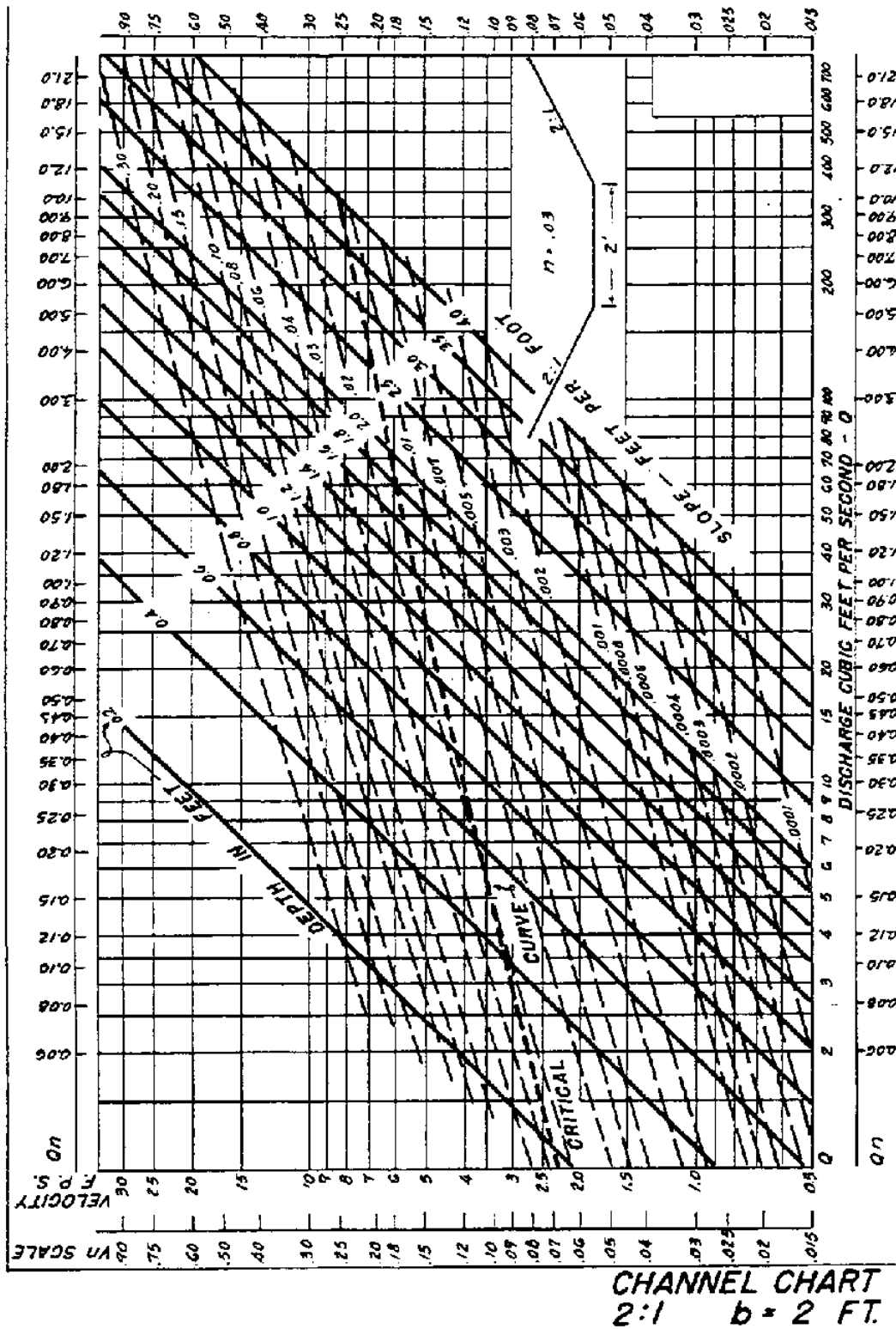
Source: VDOT

Appendix 7C-17 Trapezoidal Ditch Flow Chart  
(B=1', Side Slopes = 1.5:1)



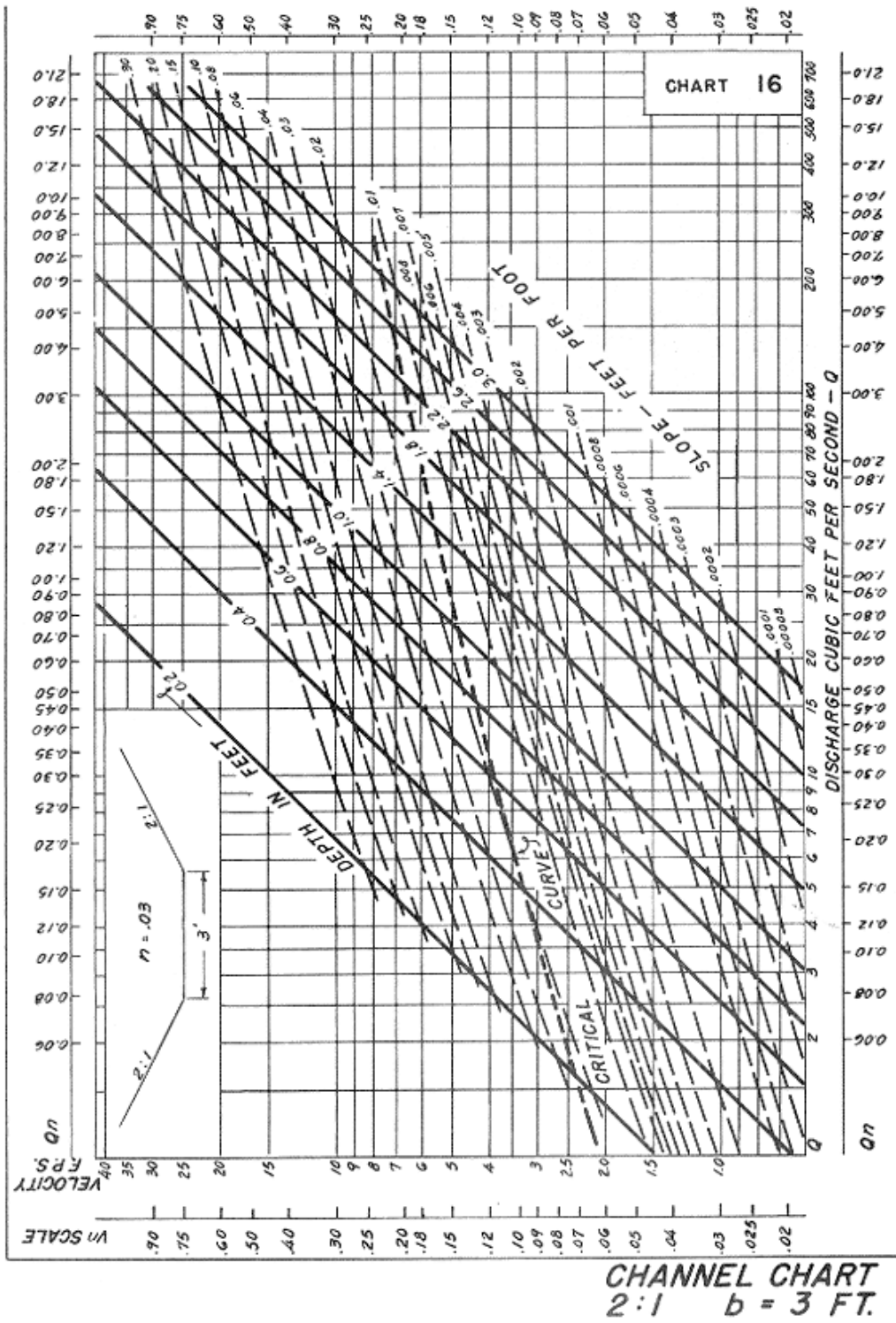
Source: HDS-3

Appendix 7C-18 Trapezoidal Channel Flow Chart  
(B=2', Side Slopes = 2:1)



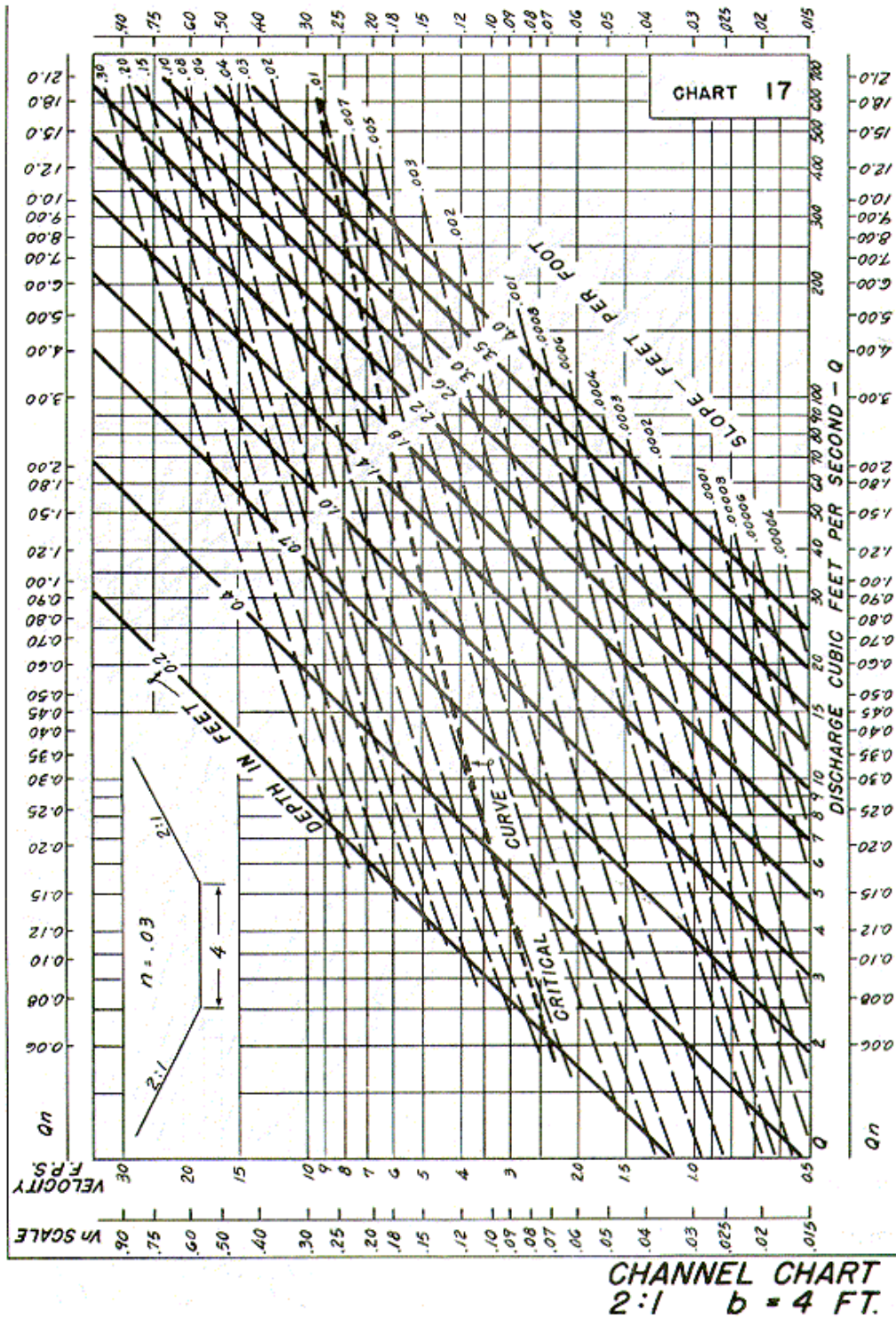
Source: HDS-3

Appendix 7C-19 Trapezoidal Channel Flow Chart  
(B=3', Side Slopes = 2:1)



Source: HDS-3

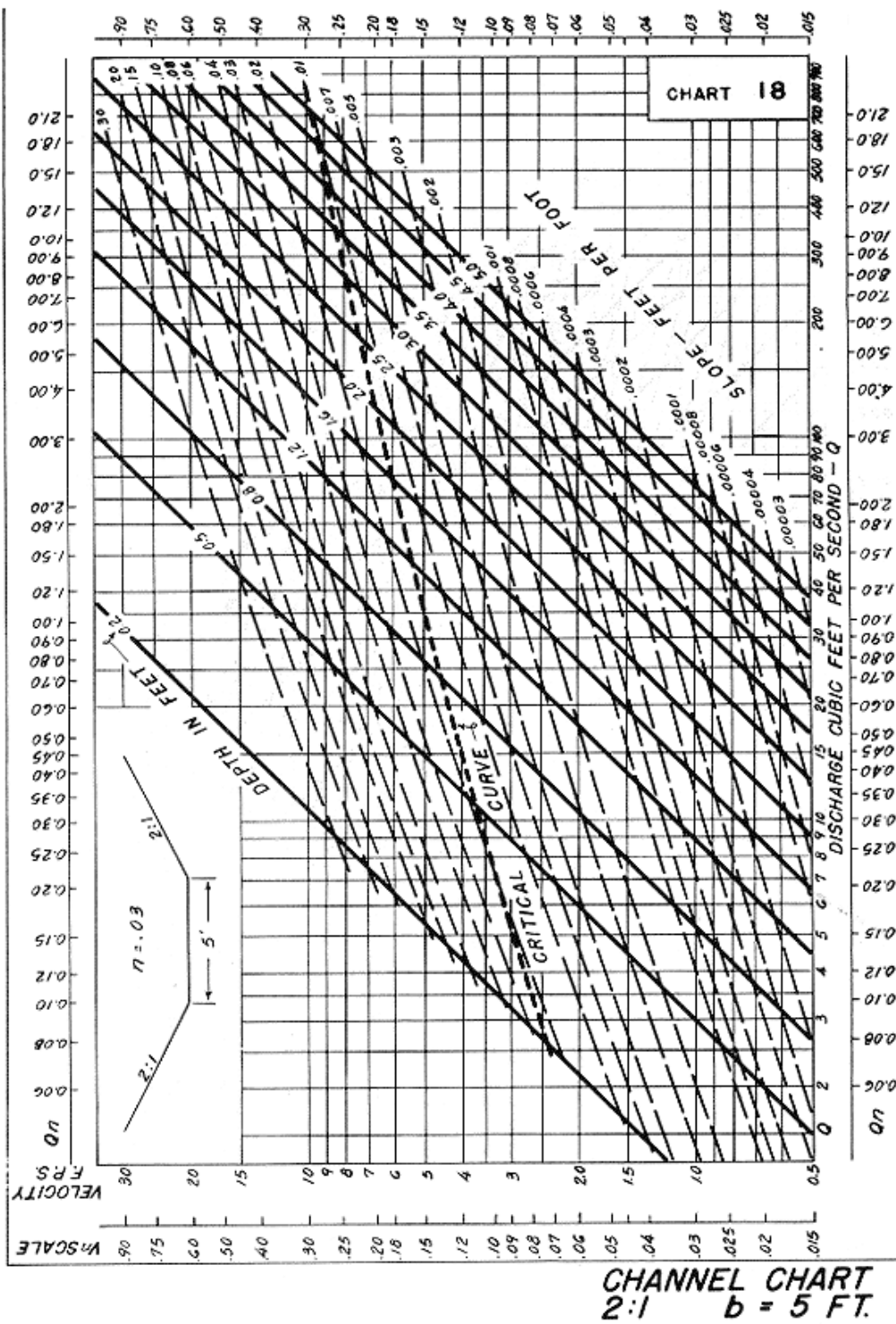
Appendix 7C-20 Trapezoidal Channel Flow Chart  
(B=4', Side Slopes = 2:1)



Source: HDS-3

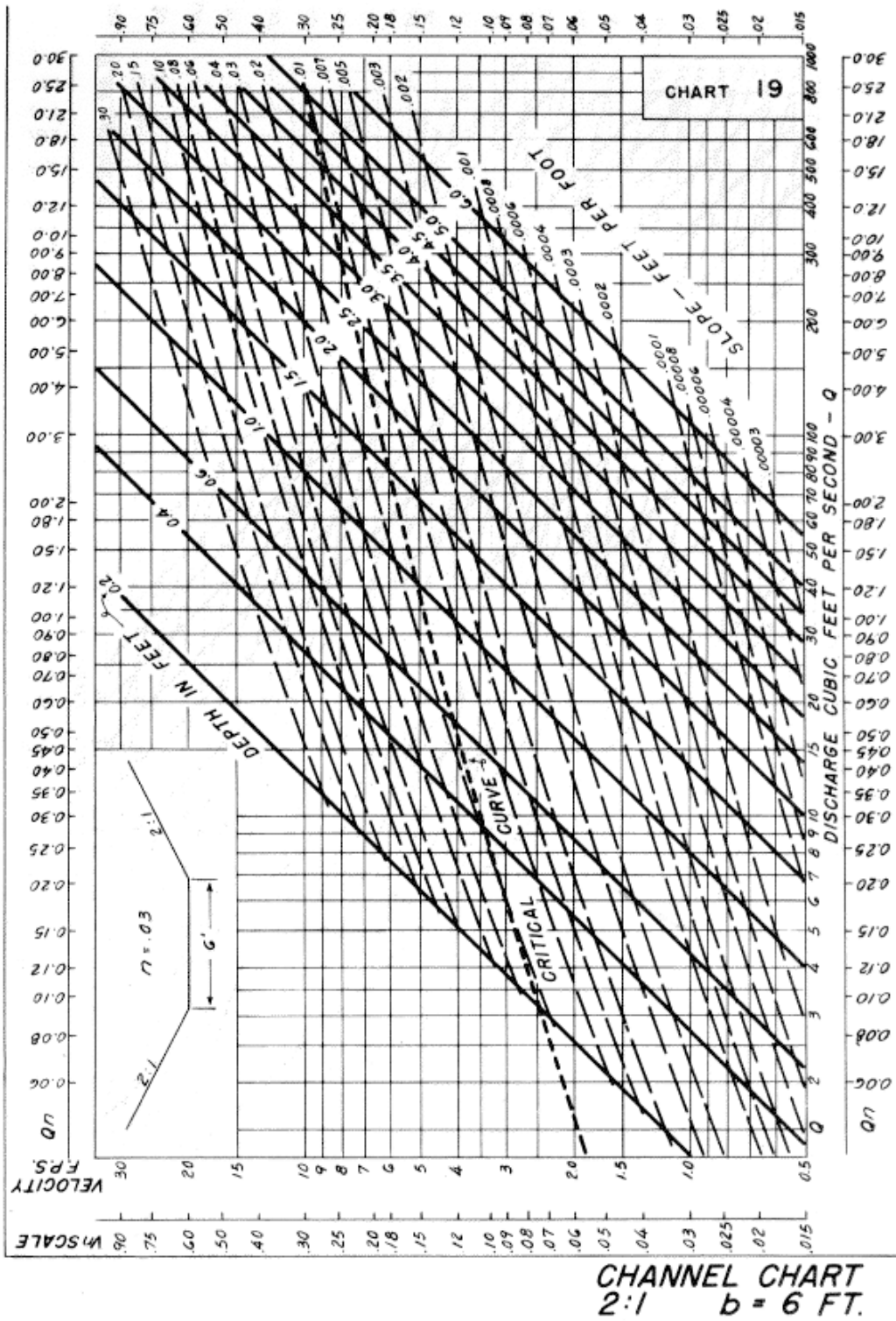


Appendix 7C-21 Trapezoidal Channel Flow Chart  
(B = 5', Side Slopes = 2:1)



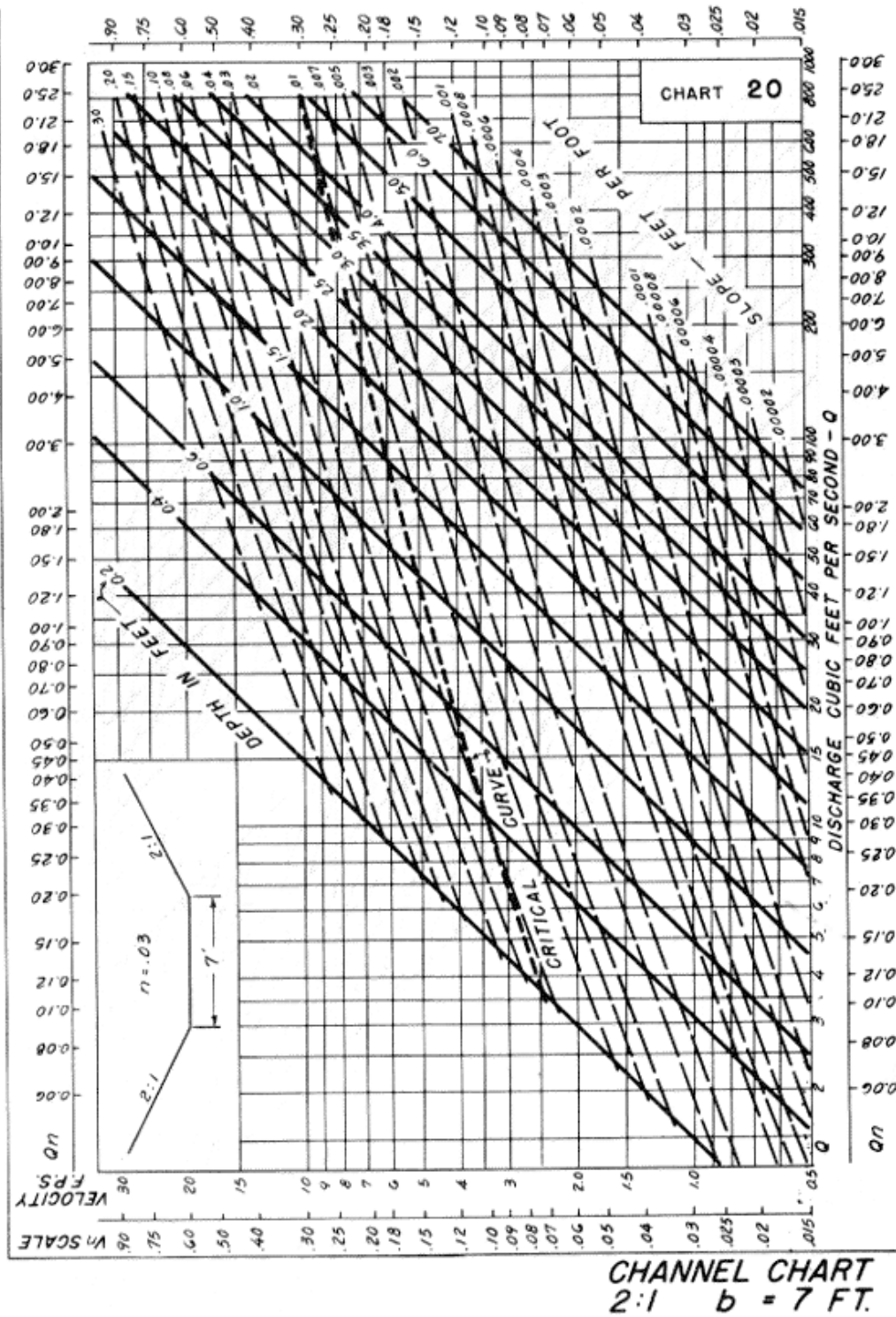
Source: HDS-3

Appendix 7C-22 Trapezoidal Channel Flow Chart  
(B = 6', Side Slopes 2:1)



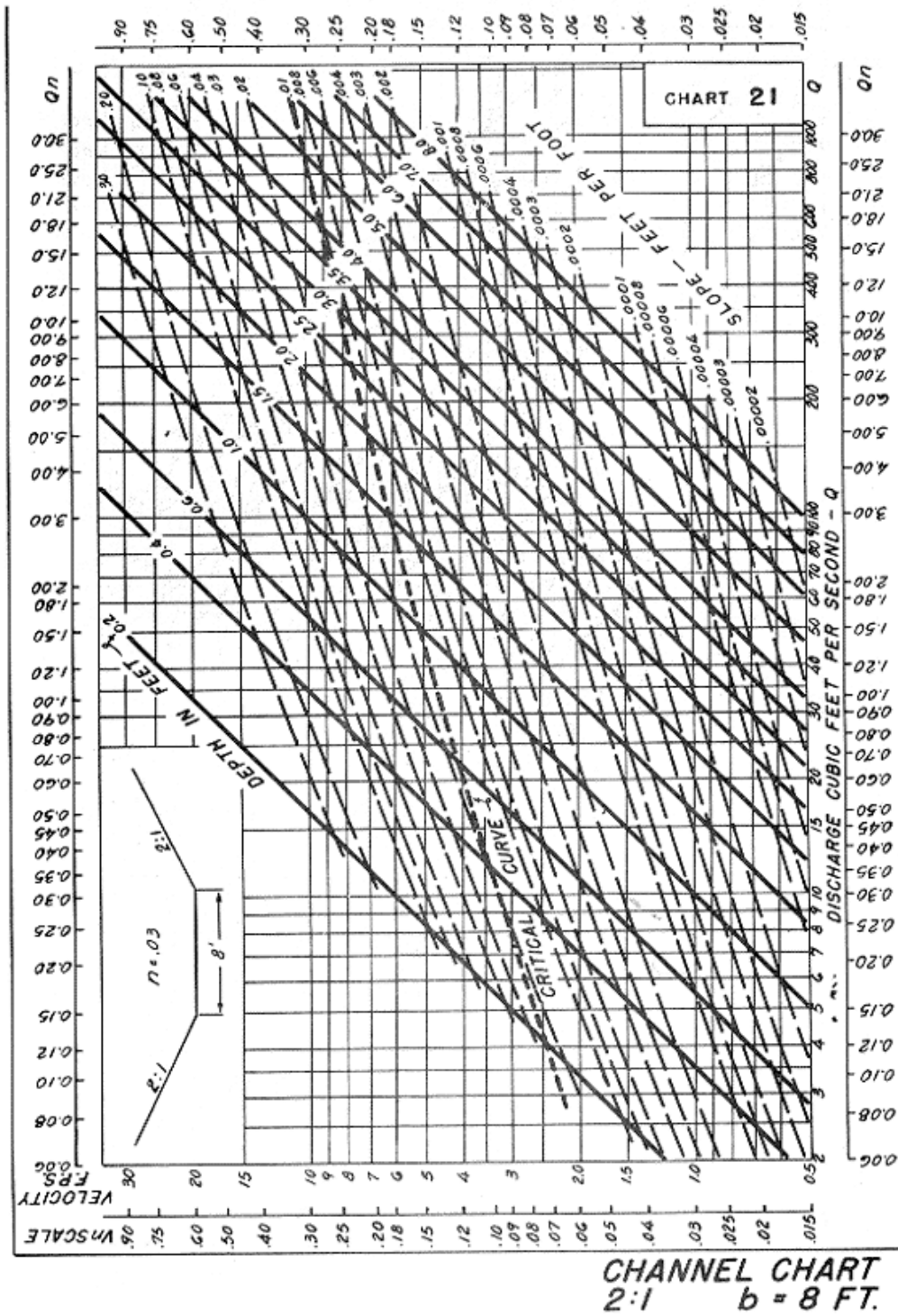
Source: HDS-3

Appendix 7C-23 Trapezoidal Channel Flow Chart  
(B = 7', Side Slopes = 2:1)



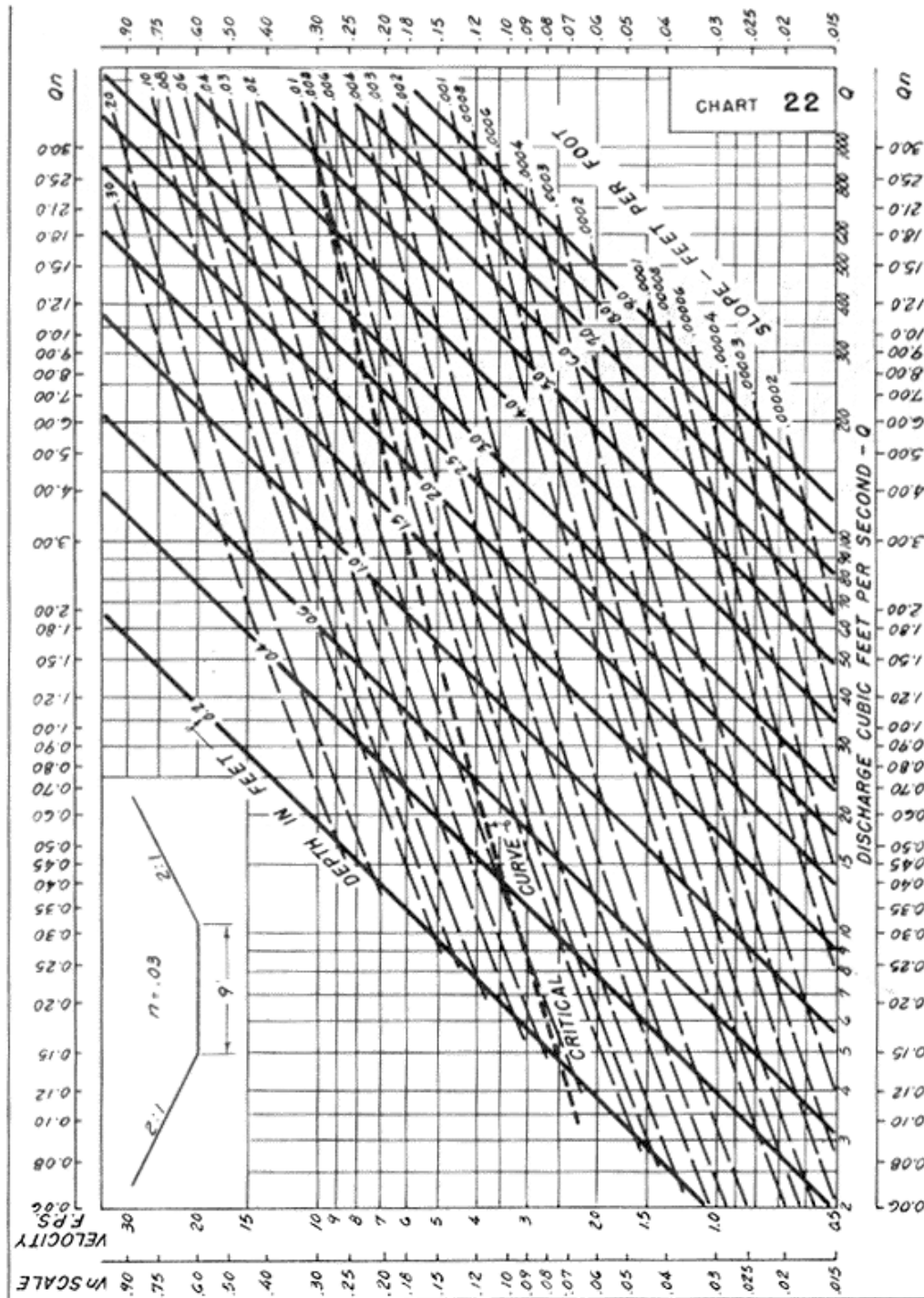
Source: HDS-3

Appendix 7C-24 Trapezoidal Channel Flow Chart  
(B = 8', Side Slopes = 2:1)



Source: HDS-3

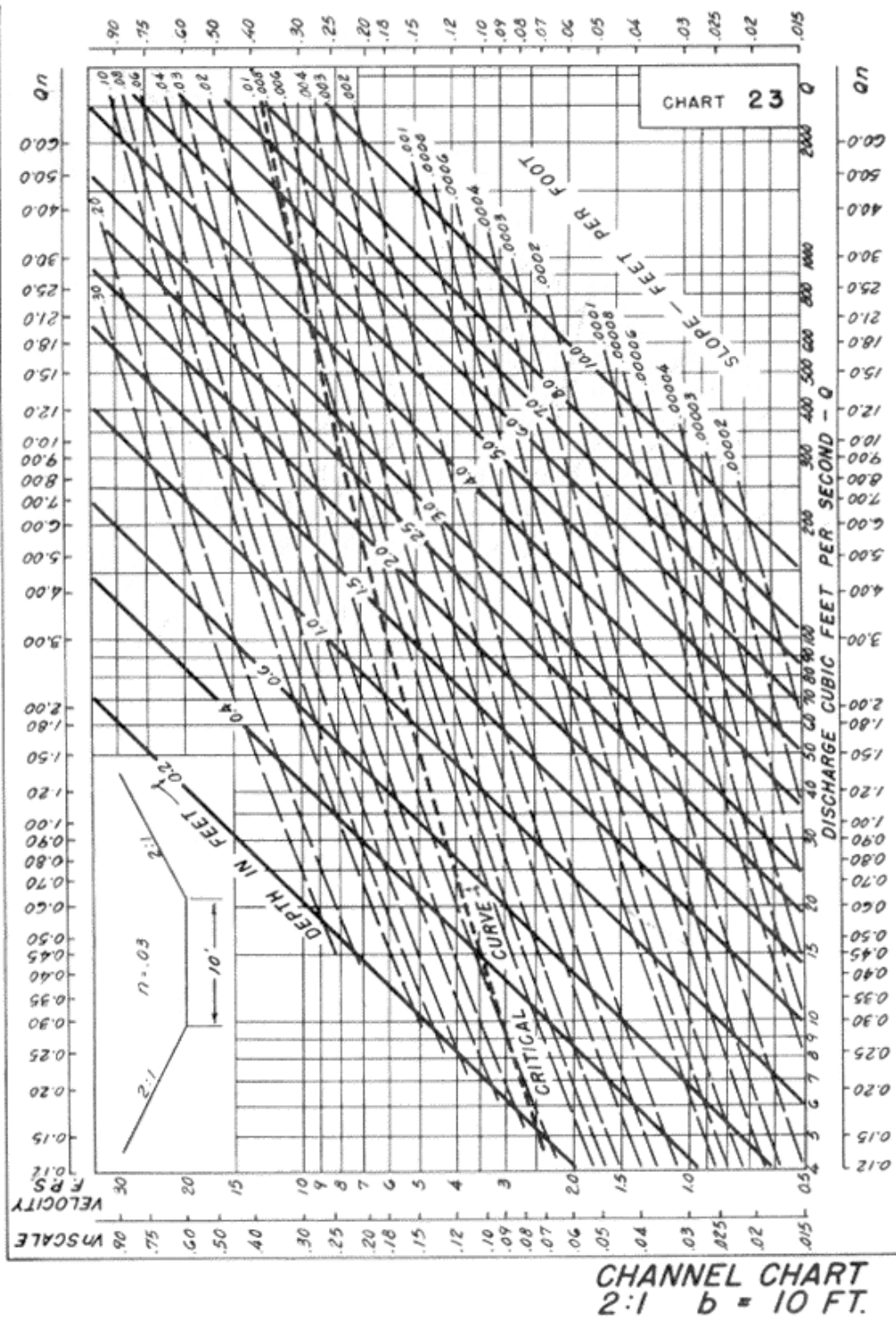
Appendix 7C-25 Trapezoidal Channel Flow Chart  
(B = 9', Side Slopes = 2:1)



CHANNEL CHART  
2:1 b = 9 FT.

Source: HDS-3

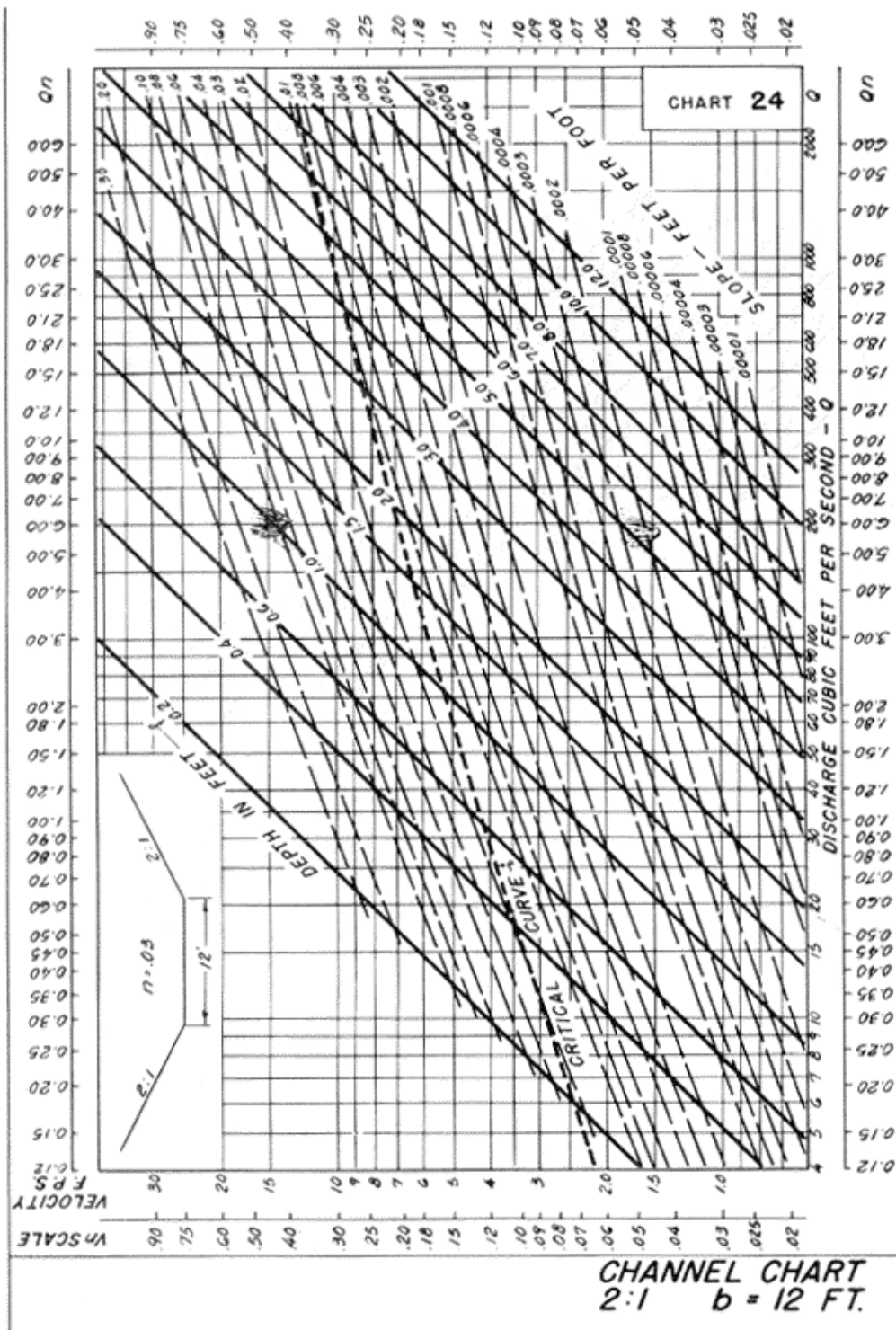
Appendix 7C-26 Trapezoidal Channel Flow Chart  
(B = 10', Side Slopes = 2:1)



Source: HDS-3

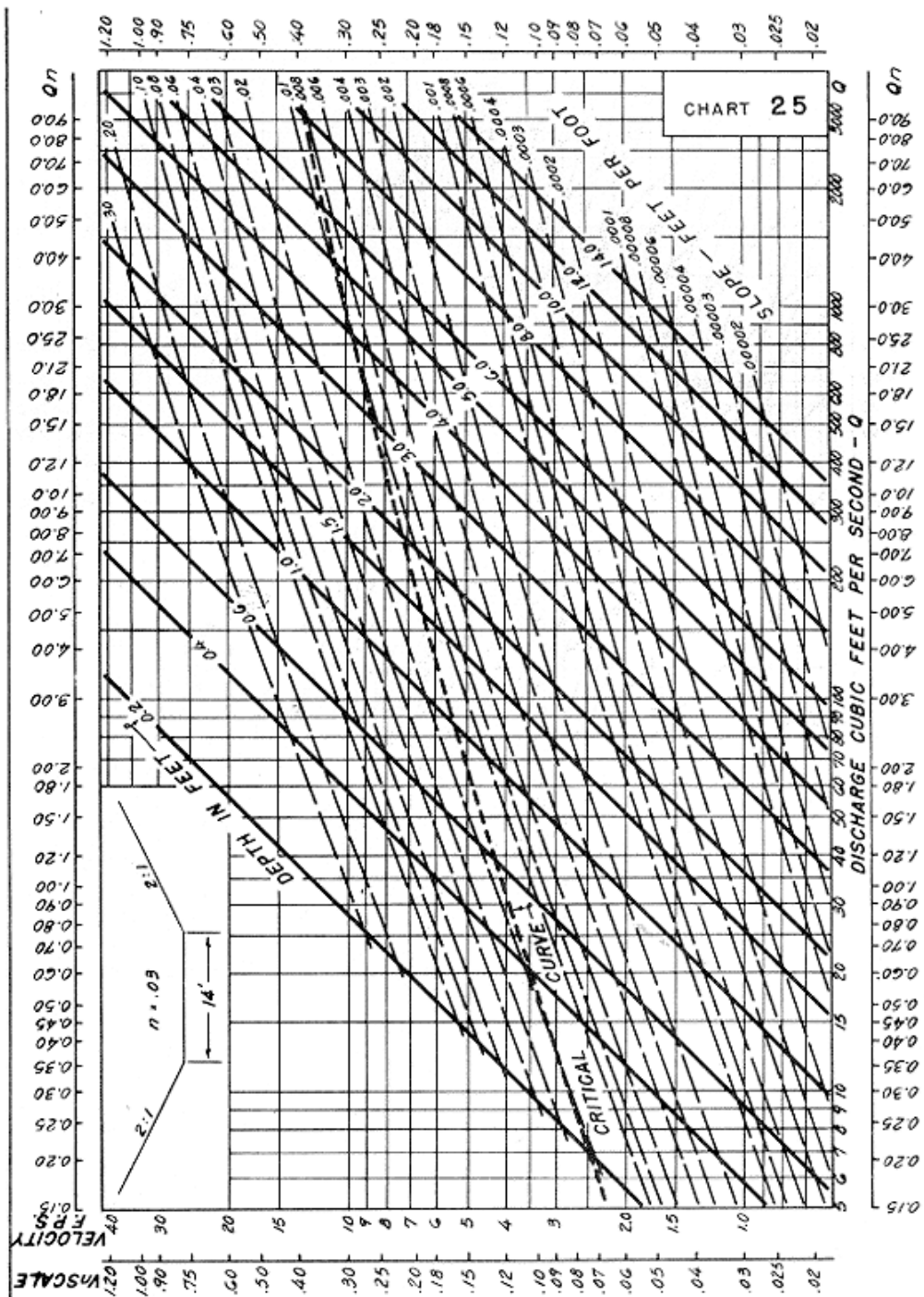


Appendix 7C-27 Trapezoidal Channel Flow Chart  
(B = 12', Side Slopes = 2:1)



Source: HDS-3

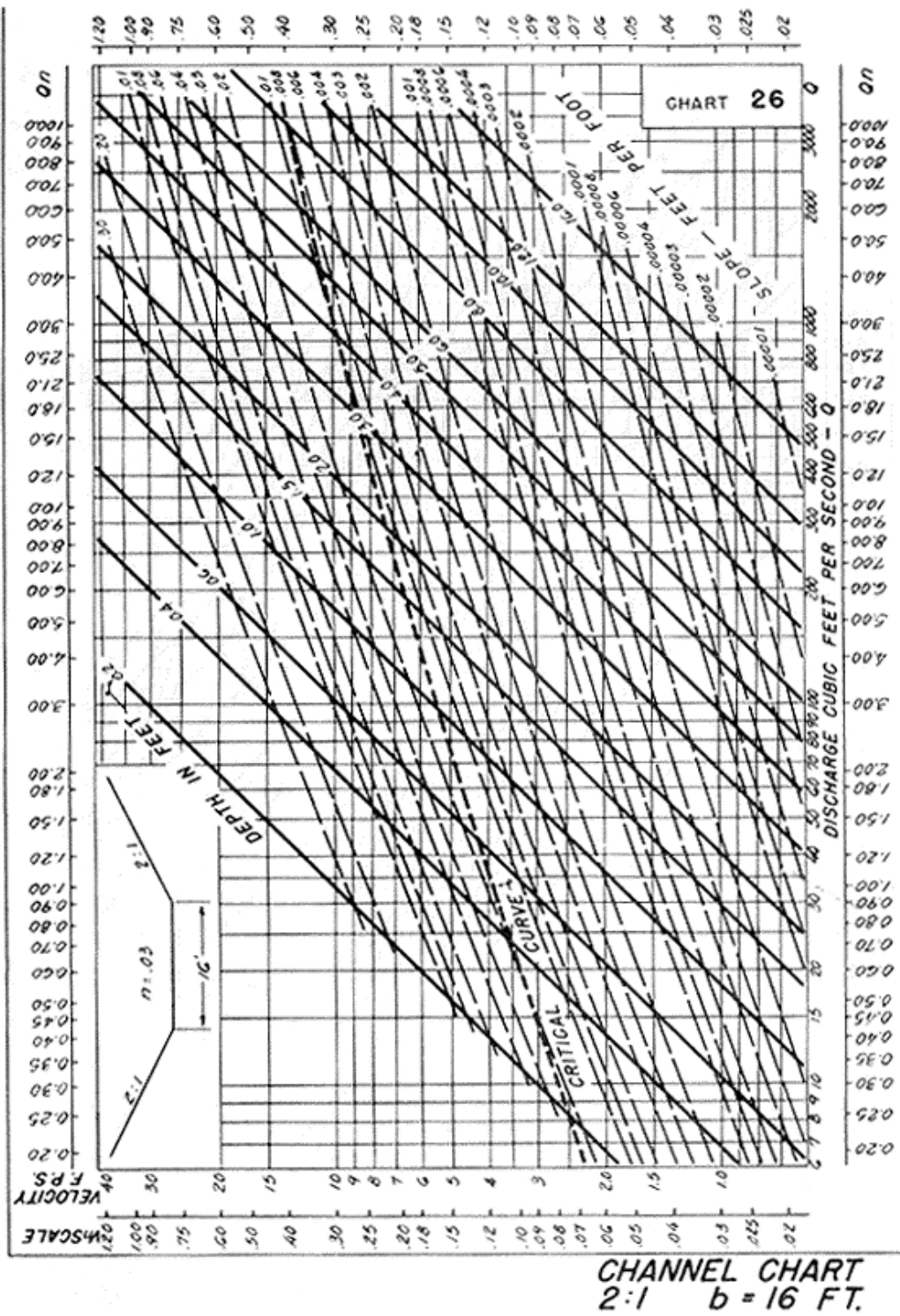
Appendix 7C-28 Trapezoidal Channel Flow Chart  
(B = 14', Side Slopes = 2:1)



Source: HDS-3

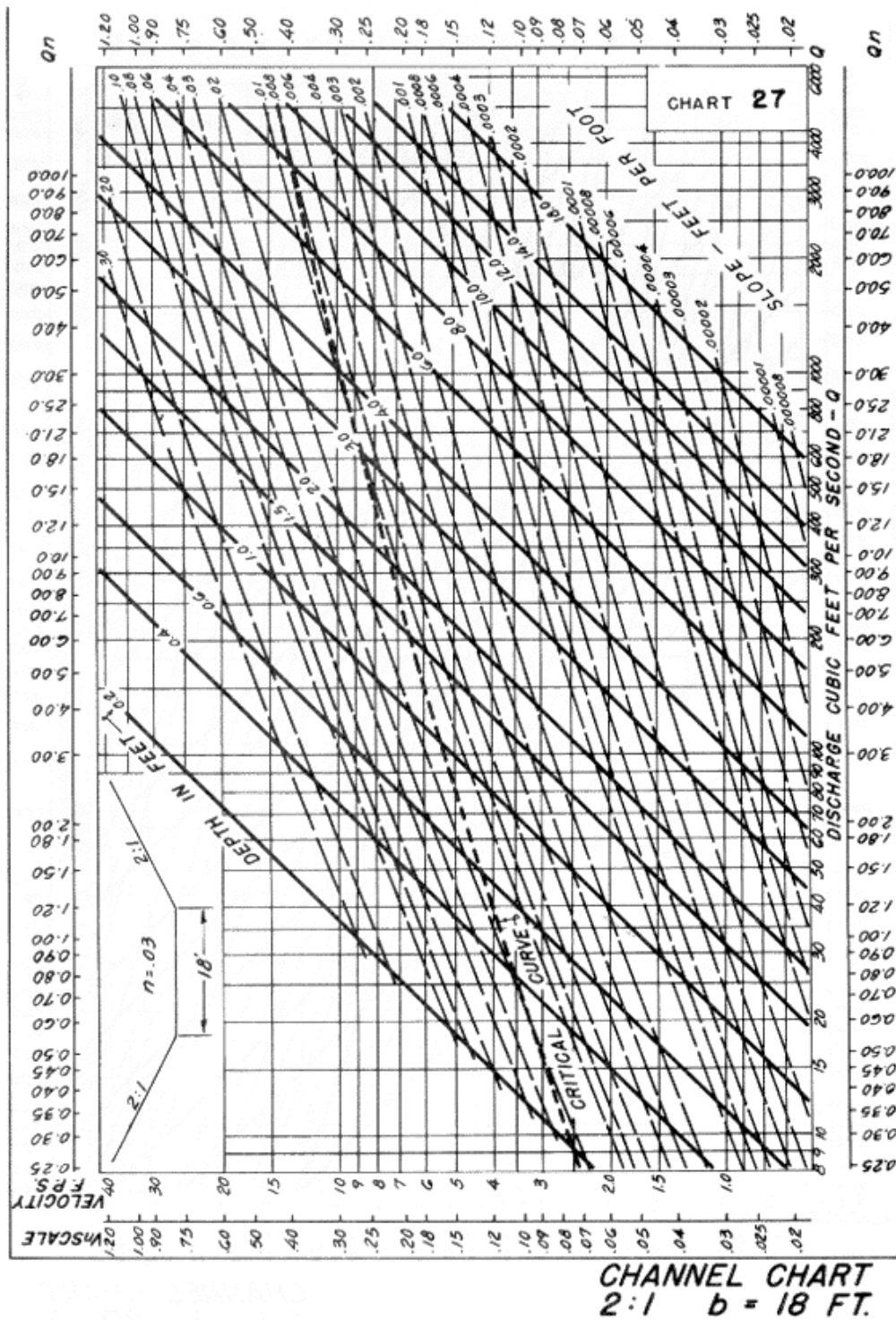


Appendix 7C-29 Trapezoidal Channel Flow Chart  
(B = 16', Side Slopes = 2:1)



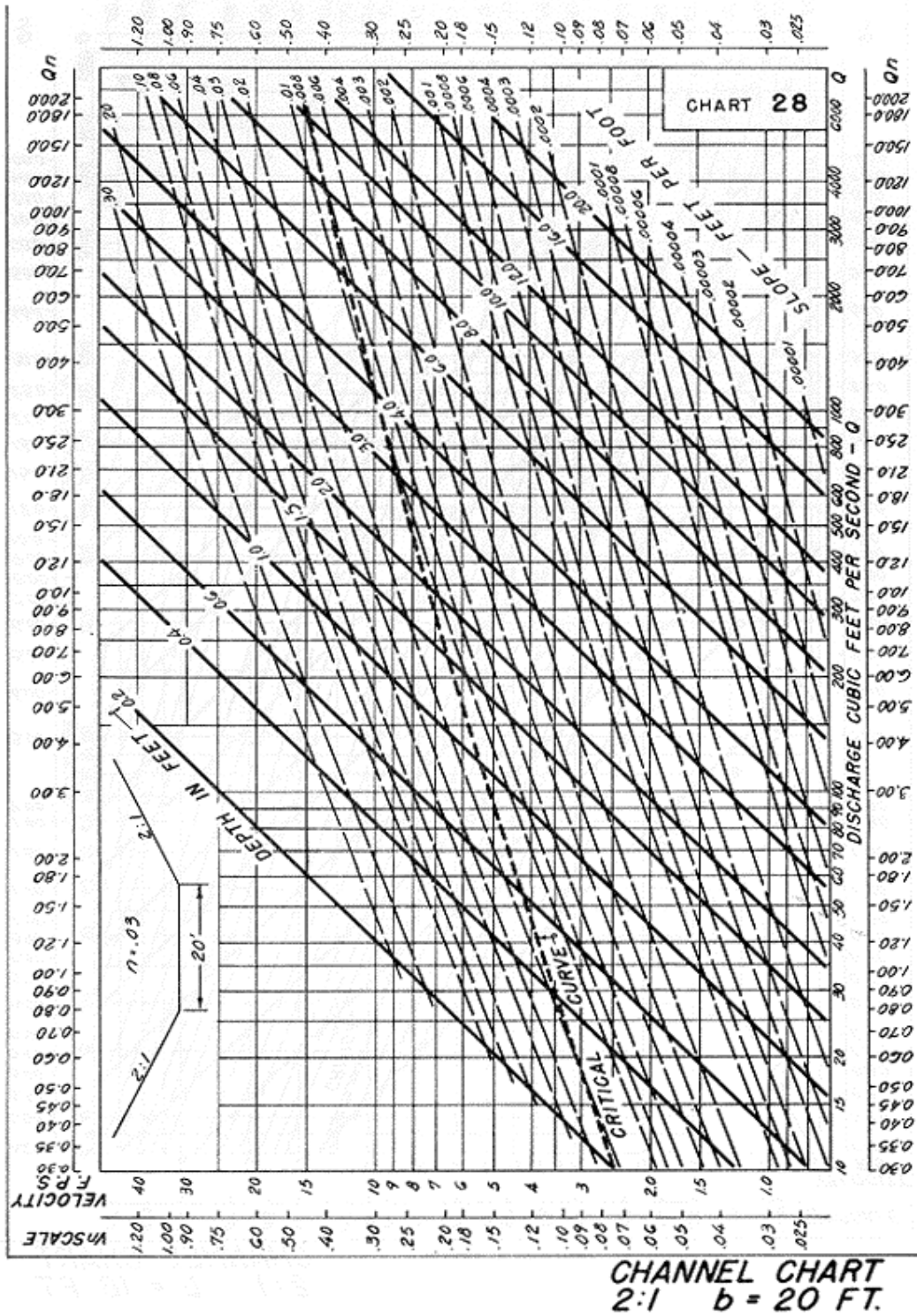
Source: HDS-3

Appendix 7C-30 Trapezoidal Channel Flow Chart  
(B = 18', Side Slopes = 2:1)



Source: HDS-3

Appendix 7C- 31 Trapezoidal Channel Flow Chart  
(B = 20', Side Slopes = 2:1)



Source: HDS-3

**Appendix 7D-1**

**Values of Roughness  
Coefficient n (Uniform Flow)**

Type of Channel and Description	Minimum	Normal	Maximum
<b>LINED CHANNELS (Selected linings)</b>			
a. Concrete			
1. Trowel finish	0.011	0.013	0.015
2. Float finish	0.013	0.015	0.016
3. Gunite, good section	0.016	0.019	0.023
b. Asphalt			
1. Smooth	0.013	0.013	-
2. Rough	0.016	0.016	-
c. Riprap (st'd VDOT sizes)			
1. Class 1A	0.033	0.038	-
2. Class 1	0.035	0.040	-
3. Class 2	0.037	0.042	-
4. Class 3	0.039	0.045	-
5. Type I	0.041	0.047	-
6. Type II	0.044	0.050	-
<b>EXCAVATED OR DREDGED</b>			
a. Earth, straight and uniform			
1. Clean, recently completed	0.016	0.018	0.020
2. Clean, after weathering	0.018	0.022	0.025
3. Gravel, uniform section, clean	0.022	0.025	0.030
4. With short grass, few weeds	0.022	0.027	0.033
b. Earth, winding and sluggish			
1. No vegetation	0.023	0.025	0.030
2. Grass, some weeds	0.025	0.030	0.033
3. Dense weeds or aquatic plants in deep channels	0.030	0.035	0.040
4. Earth bottom and rubble sides	0.025	0.030	0.035
5. Stony bottom and weedy sides	0.025	0.035	0.045
6. Cobble bottom and clean sides	0.030	0.040	0.050
c. Dragline excavated or dredged			
1. No vegetation	0.025	0.028	0.033
2. Light brush on banks	0.035	0.050	0.060
d. Rock cuts			
1. Smooth and uniform	0.025	0.035	0.040
2. Jagged and irregular	0.035	0.040	0.050
e. Channels not maintained, weeds and brush uncut			
1. Dense weeds, high as flow depth	0.050	0.080	0.120
2. Clean bottom, brush on sides	0.040	0.050	0.080
3. Same, highest stage of flow	0.045	0.070	0.110
4. Dense brush, high stage	0.080	0.100	0.140
<b>NATURAL STREAMS</b>			
1. Minor streams (top width at flood stage <100 ft)			
a. Streams on Plain			
1. Clean, straight, full stage, no rifts or deep pools	0.025	0.030	0.033
2. Same as above, but more stones/weeds	0.030	0.035	0.040
3. Clean, winding, some pools/shoals	0.033	0.040	0.045
4. Same as above, but some weeds/stones	0.035	0.045	0.050
5. Same as above, lower stages, more ineffective slopes and sections	0.040	0.048	0.055
6. Same as 4, but more stones	0.045	0.050	0.060
7. Sluggish reaches, weedy, deep pools	0.050	0.070	0.080

\* Rev 7/09

**Appendix 7D-1 Values of Roughness Coefficient n (Uniform Flow)**

Type of Channel and Description	Minimum	Normal	Maximum
8. Very weedy reaches, deep pools, or floodways with heavy stand of timber and underbrush	0.075	0.100	0.150
b. Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stages			
1. Bottom: gravels, cobbles and few boulders	0.030	0.040	0.050
2. Bottom: cobbles with large boulders	0.040	0.050	0.070
2. Floodplains			
a. Pasture, no brush			
1. Short grass	0.025	0.030	0.035
2. High grass	0.030	0.035	0.050
b. Cultivated area			
1. No crop	0.020	0.030	0.040
2. Mature row crops	0.025	0.035	0.045
3. Mature field crops	0.030	0.040	0.050
c. Brush			
1. Scattered brush, heavy weeds	0.035	0.050	0.070
2. Light brush and trees, in winter	0.035	0.050	0.060
3. Light brush and trees, in summer	0.040	0.060	0.080
4. Medium to dense brush, in winter	0.045	0.070	0.110
5. Medium to dense brush, in summer	0.070	0.100	0.160
d. Trees			
1. Dense Willows, summer, straight	0.110	0.150	0.200
2. Cleared land with tree stumps, no sprouts	0.030	0.040	0.050
3. Same as above, but with heavy growth of sprouts	0.050	0.060	0.080
4. Heavy stand of timber, a few down trees, little undergrowth, flood stage below branches	0.080	0.100	0.120
5. Same as above, but with flood stage reaching branches	0.100	0.120	0.160
3. Major Streams (top width at flood stage > 100 ft)			
The n-value is less than that for minor streams of similar description, because banks offer less effective resistance.			
a. Regular section with no boulders or brush	0.025	-	0.060
b. Irregular and rough section	0.035	-	0.100

Source: Chow, V.T., FHWA's HDS-6 publication

\* For bare earth linings when the soil classifications in accordance with either AASHTO or USCS designations are known, use the Manning's "n" values recommended in the appropriate table from Appendix 7D-2

\* Rev 7/09

**Appendix 7D-2 Recommended Maximum Water Velocities and Manning's n as a Function of Soil Type and Flow Depth**

ASSHTO Classification	ASSHTO Soil Description	Fortier and Scobey Soil Description	Maximum Water Velocity (ft/s)	Manning's n -Flow Depth 0.5-2.0 ft
	<b>BROKEN ROCK and COBBLES</b>	Cobbles and Shingles	5.5	0.030
A-1-a	Stone fragments or <b>GRAVEL</b> , with or without well-graded <sup>1</sup> binder <sup>2</sup>	Coarse gravel, non-colloidal	4.5	0.025
same	same	Fine gravel	3.5	0.020
A-1-b	Coarse <b>SAND</b> , with or without well-graded <sup>1</sup> binder <sup>2</sup>	Graded loam to cobbles when non-colloidal	4.0	0.030
A-2 (A-2-4, A-2-5, A-2-6, A-2-7)	Mixture of <b>GRAVEL</b> and <b>SAND</b> , with silty or clay fines <sup>3</sup> , or nonplastic silt fines	Graded silts to cobbles when colloidal	4.5	0.030
same	same	Sandy loam, non-colloidal	2.0	0.020
A-3	Fine <b>SAND</b> , without silty clay fines; e.g. beach sand or stream-deposited fine sand	Fine Sand, non-colloidal	1.5	0.020
same	same	Silt loam, non-colloidal	2.3	0.020
A-4	Non- to moderately plastic <sup>4</sup> <b>SILT</b> ; mixtures of silt, sand, and/or gravel, with a minimum silt content of 36%	Alluvial silts, non-colloidal	2.3	0.020
A-5	Moderately to highly plastic <sup>4</sup> <b>SILT</b> . Soil; mixtures of silt, sand, and/or gravel, with a minimum fines <sup>3</sup> content of 36%	Ordinary firm loam	2.5	0.020
A-6	Plastic <sup>4</sup> <b>CLAY</b> soil; mixtures of clay, sand, and/or gravel, with a minimum fines <sup>3</sup> content of 36%	Alluvial silts, colloidal	3.5	0.025
A-7	Moderately to highly plastic, <b>CLAY</b> ; mixtures of clay, sand, and/or gravel, with a minimum clay content of 36%	Stiff clay, very colloidal	4.0	0.025

- 1) Well-graded-containing a broad range of particle sizes with no intermediate sizes missing.
  - 1) Binder - soil particles consisting of fine sand, silt, and clay.
  - 2) Fines - particle sizes finer than 0.074 mm (e.g., silt and clay particles).
  - 3) Plasticity - ability of a soil mass to deform at constant volume without cracking or crumbling.
- + Relationship between AASHTO classification and Fortier and Scobey description is loosely correlated.

**Chapter 7 – Ditches and Channels**

<b>USCS Classification</b>	<b>USCS Soil Description</b>	<b>Fortier and Scobey Soil Description</b>	<b>Maximum Water Velocity (ft/s)</b>	<b>Manning's n -Flow Depth 0.5-2.0 ft</b>
	<b>BROKEN ROCK and COBBLES</b>	Cobbles and Shingles	5.5	0.030
GP, GW, SW, SP	Poorly graded gravel, well graded gravel, well graded sand, poorly graded sand	Coarse gravel, non-colloidal	4.5	0.025
		Fine gravel	3.5	0.020
SW	Well graded sand	Graded loam to cobbles when non-colloidal	4.0	0.030
GC, SC	Clayey gravel, clayey sand	Graded silts to cobbles when colloidal	4.5	0.030
SM	Silty sand	Sandy loam, non-colloidal	2.0	0.020
SP, SW	Poorly graded sand, well graded sand	Fine Sand, non-colloidal	1.5	0.020
ML	Silt	Silt loam, non-colloidal	2.3	0.020
CL	Lean clay	Alluvial silts, non-colloidal	2.3	0.020
ML, CL	Silt, lean clay	Ordinary firm loam	2.5	0.020
CL	Lean clay	Alluvial silts, colloidal	3.5	0.025
CH	Fat clay	Stiff clay, very colloidal	4.0	0.025

Note: Relationship between Unified Soil Classification System (USCS) classification and Fortier and Scobey description is loosely correlated.

**Appendix 7D-3 Standard VDOT Riprap Classifications,  
Weights, and Blanket Thickness**

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<b>Classification</b>	<b>D<sub>50</sub> (ft)</b>	<b>W<sub>50</sub> (lbs)</b>	<b>T (in)</b>
Class A1	0.8	50	20
Class I	1.1	100	26
Class II	1.6	300	38
Class III	2.2	1000	53
Type I	2.8	2000	60
Type II	4.5	8000	97



**Appendix 7D-4      Approximate Rock Dimensions and  
Equivalent Weights for Riprap**

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WEIGHT	MEAN SPHERICAL DIAMETER	RECTANGULAR SHAPE	
		LENGTH	HT./WIDTH
25 lbs.	0.7'	1.1'	0.4'
50 lbs.	0.8'	1.4'	0.5'
75 lbs.	1.0'	1.6'	0.5'
100 lbs.	1.1'	1.75'	0.6'
150 lbs.	1.3'	2.0'	0.67'
300 lbs.	1.6'	2.6'	0.9'
500 lbs.	1.9'	3.0'	1.0'
1000 lbs.	2.2'	3.7'	1.25'
1500 lbs.	2.6'	4.7'	1.5'
2000 lbs.	2.75'	5.4'	1.8'
2 tons	3.6'	6.0'	2.0'
3 tons	4.0'	6.9'	2.3'
4 tons	4.5'	7.6'	2.5'
10 tons	6.1'	10.0'	3.3'

**Appendix 7D-5 Selection of Stability Factors**

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<b>CONDITION</b>	<b>STABILITY FACTOR RANGE</b>
Uniform flow; straight or mildly curving reach (curve radius/channel width >30); impact from wave action and floating debris is minimal; little or no uncertainty in design parameters.	1.0 - 1.2
Gradually varying flow; moderate bend curvature ( $30 > \text{curve radius/channel width} > 10$ ); impact from waves or floating debris is moderate.	1.3 - 1.6
Approaching rapidly varying flow; sharp bend curvature ( $30 > \text{curve radius/channel width} > 10$ ); significant impact potential from floating debris and/or ice; significant wind and/or bore generated waves (1-2 ft); high flow turbulence; mixing flow at bridge abutments; significant uncertainty in design parameters.	1.6 - 2.0
Channel bends when ratio of curve radius to channel width ( $R/W$ ) > 30.	1.2
Channel bends when $30 > R/W > 10$ .	1.3 - 1.6
Channel bends when $R/W < 10$ .	1.7

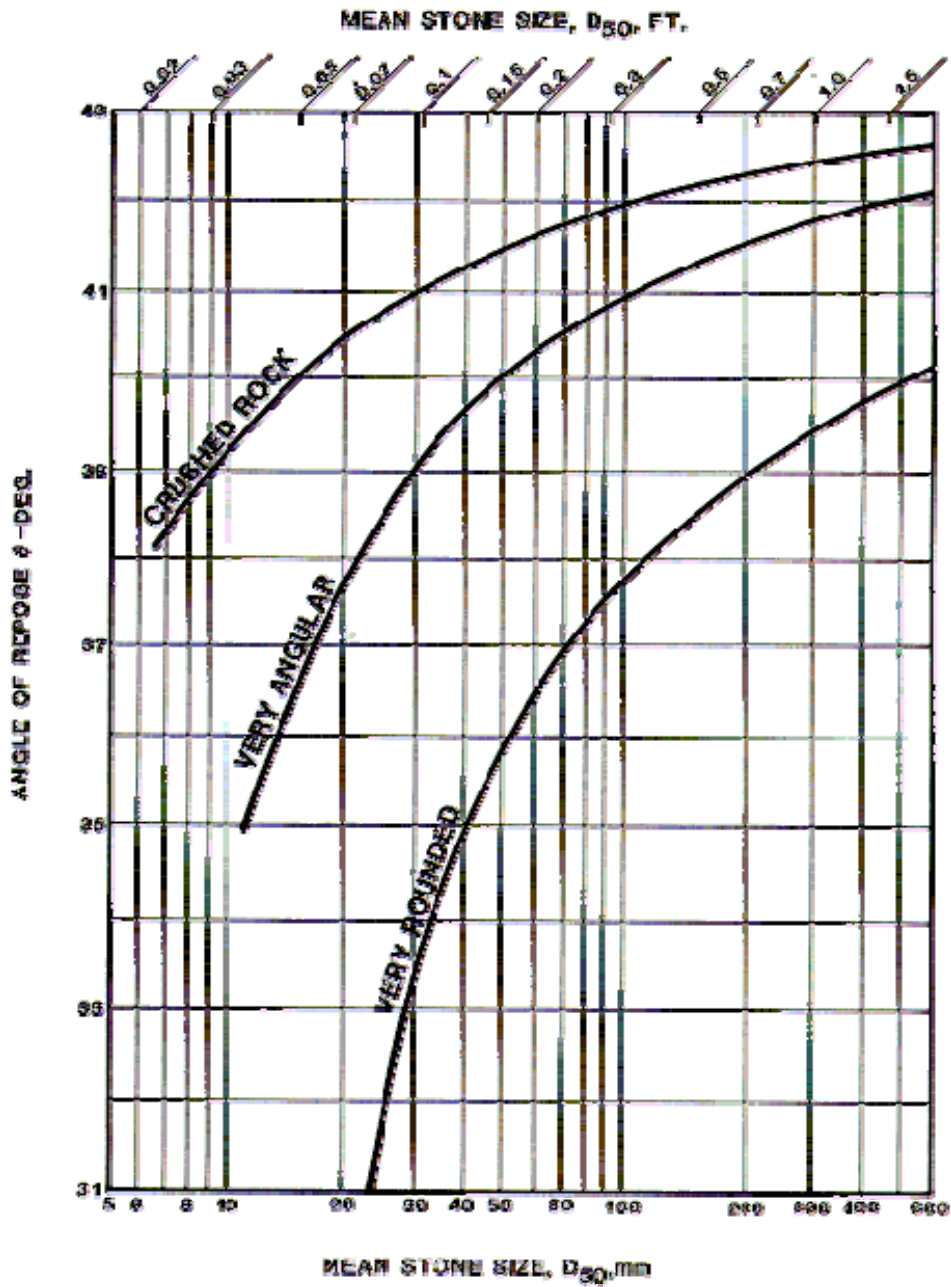
**Appendix 7D-6 Permissible Velocities  
for Erodible Linings**

Permissible velocities for channels with erodible linings, based on uniform flow in continuously wet, aged channels<sup>1</sup>:

Soil type or lining (earth; no vegetation)	Maximum permissible velocities for		
	Clear water	Water carrying fine silts	Water carrying sand and gravel
	<i>F.p.s.</i>	<i>F.p.s.</i>	<i>F.p.s.</i>
Fine sand (noncolloidal)	1.5	2.5	1.5
Sandy loam (noncolloidal)	1.7	2.5	2.0
Silt loam (noncolloidal)	2.0	3.0	2.0
Ordinary firm loam	2.5	3.5	2.2
Volcanic ash	2.5	3.5	2.7
Fine gravel	2.5	5.0	3.7
Stiff clay (very colloidal)	3.7	5.0	3.0
Graded, loam to cobbles (noncolloidal)	3.7	5.0	5.0
Graded, silt to cobbles (colloidal)	4.0	5.5	5.0
Alluvial silts (noncolloidal)	2.0	3.5	2.0
Alluvial silts (colloidal)	3.7	5.0	3.0
Coarse gravel (noncolloidal)	4.0	6.0	6.5
Cobbles and shingles	5.0	5.5	6.5
Shales and hard pans	6.0	6.0	5.0

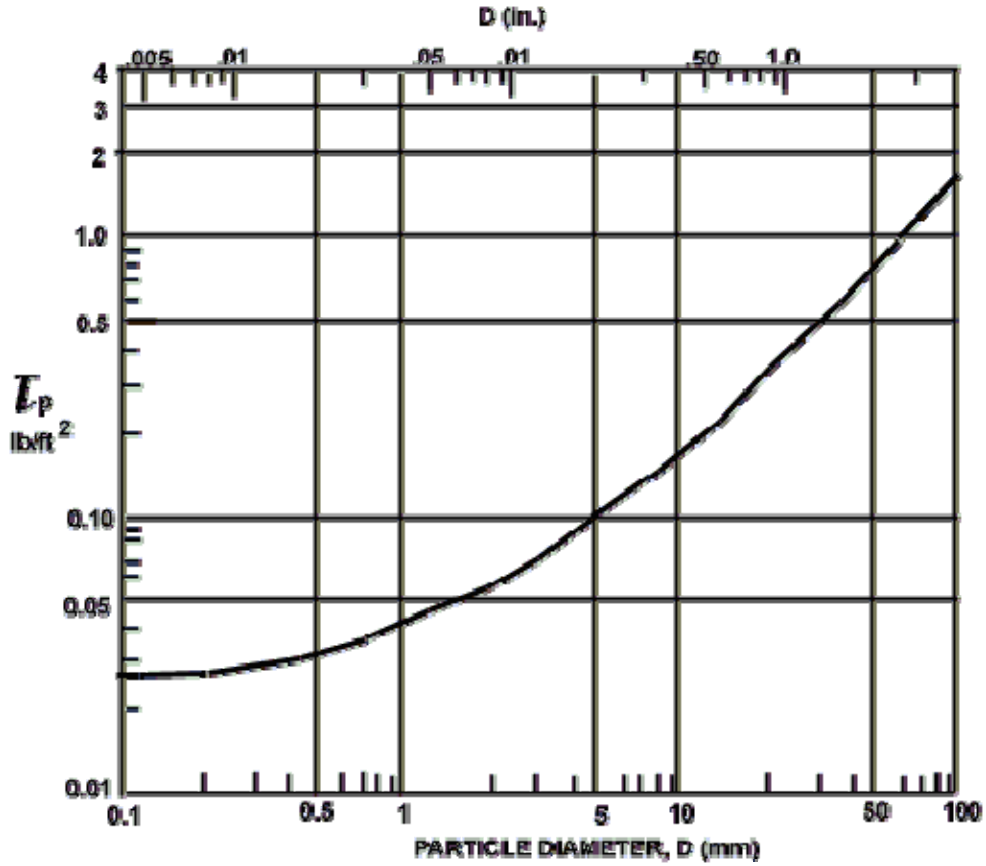
Source: <sup>1</sup>As recommended by Special Committee on Irrigation Research, American Society of Civil Engineers, 1926.

Appendix 7E-1 Angle of Repose of Riprap in Terms of Mean Size and Shape of Stone



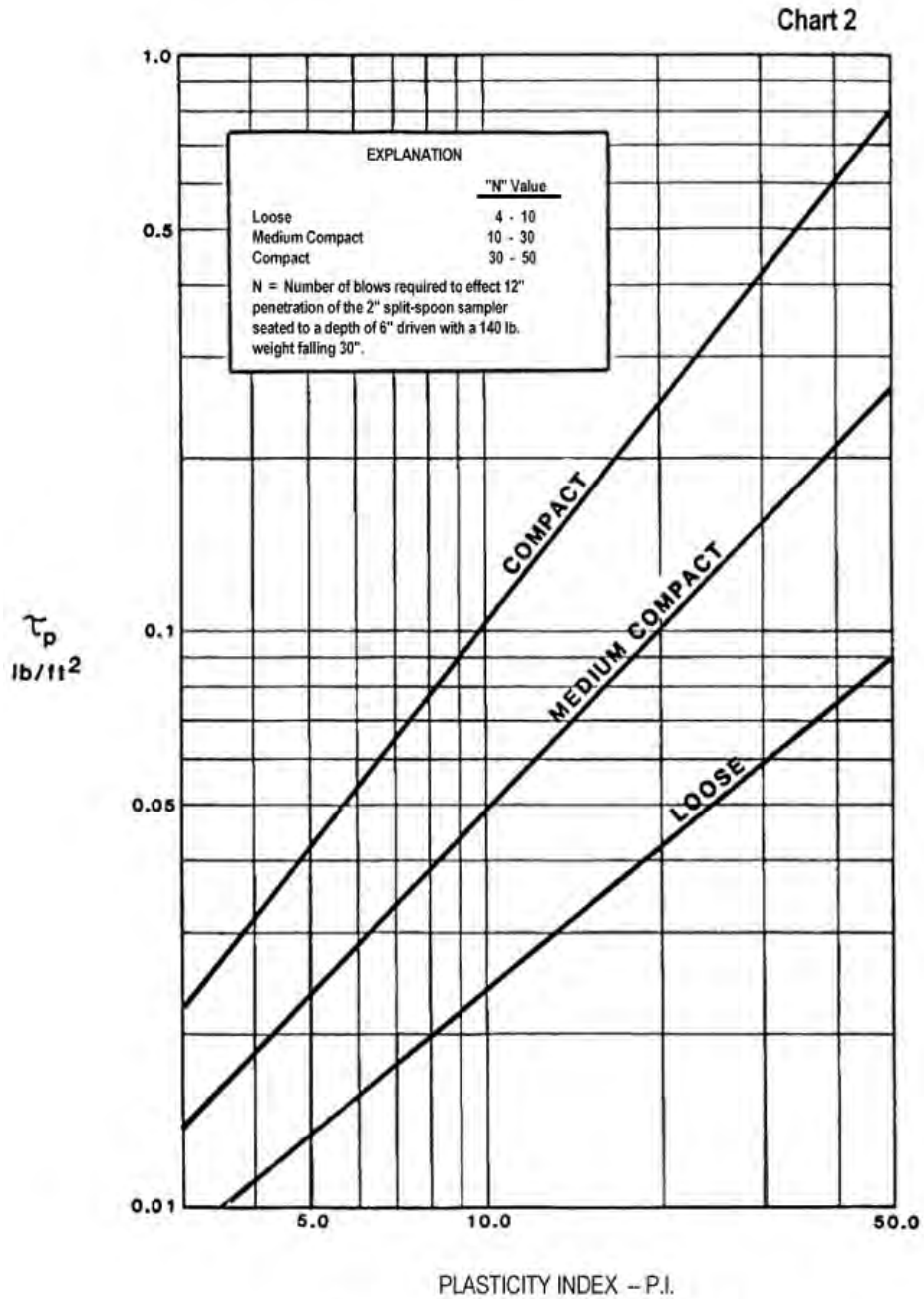
Source: HEC-15 (Archived 1988)

Appendix 7E-2 Permissible Shear Stress for Non-Cohesive Soils



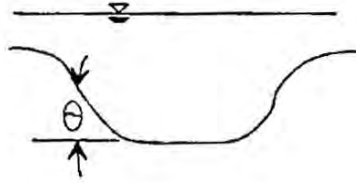
Source: HEC-15 (Archived) 1988

Appendix 7E-3 Permissible Shear Stress for Cohesive Soils



Source: HEC-15 (Archived) 1988

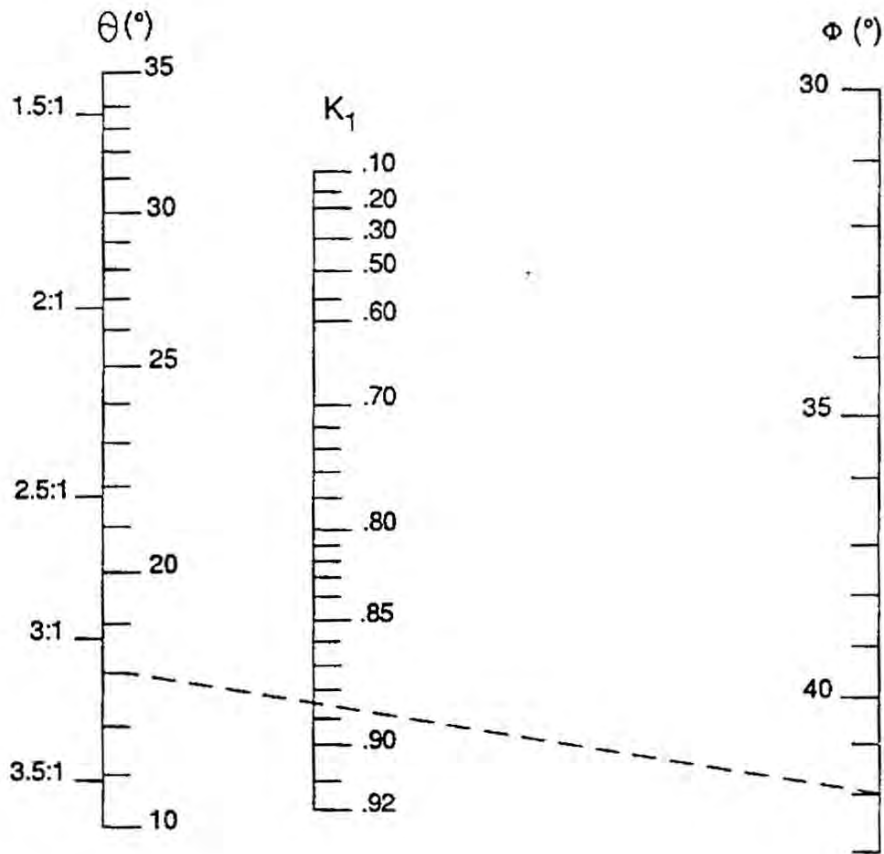
Appendix 7E-4 Bank Angle Correction Factor ( $K_1$ ) Nomograph



$$K_1 = \left[ 1 - \frac{\sin^2 \theta}{\sin^2 \phi} \right]^{0.5}$$

$\theta$  = Bank angle with horizontal

$\phi$  = Material angle of repose  
(See chart 4)



Example

Given:  
 $\theta = 18^\circ$   
Very Angular  
 $D_{50} = 1.5$  ft.

Find:  
 $K_1$

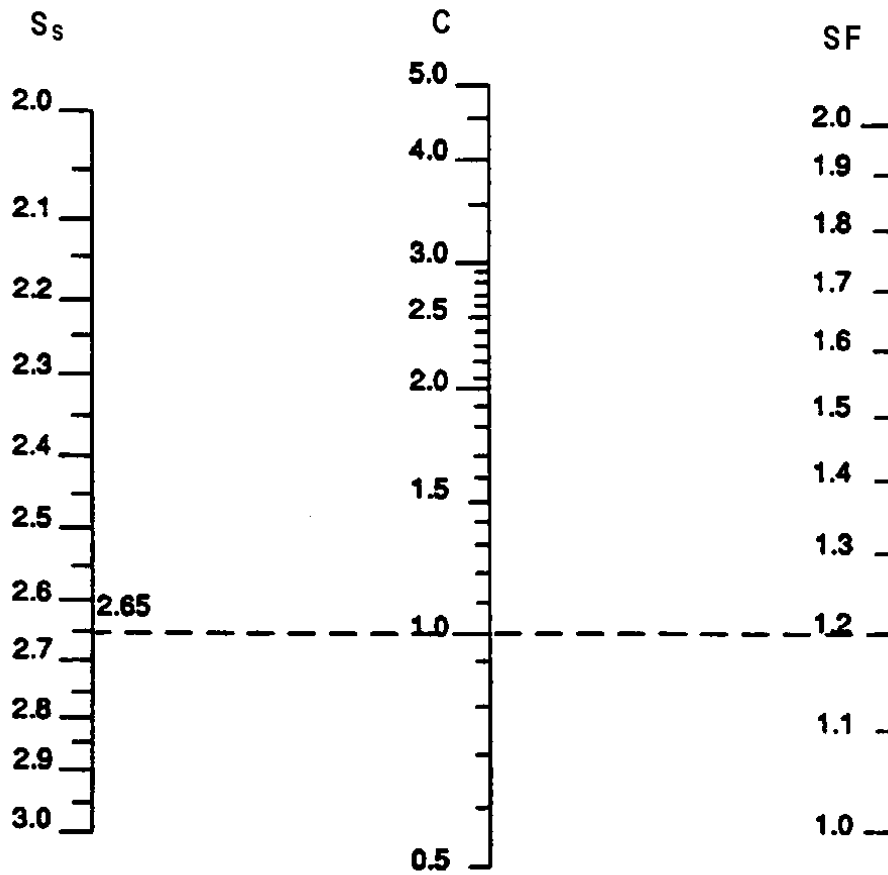
Solution:  
 $\phi = 42^\circ$   
 $K_1 = 0.885$

Source: HEC-11

Appendix 7E-5 Correction Factor for Riprap Size

$$C = 1.61 SF^{1.5} / (S_s - 1)^{1.5}$$

C = D<sub>50</sub> CORRECTION FACTOR  
 SF = STABILITY FACTOR  
 S<sub>s</sub> = SPECIFIC GRAVITY OF ROCK



Example:

Given:  
 S<sub>s</sub> = 2.65  
 SF = 1.2

Solution:  
 C = 1.0

Source: HEC-11  
 Comment: S<sub>s</sub>=S<sub>g</sub> (text)



Appendix 7E-6 Riprap Size Relationship

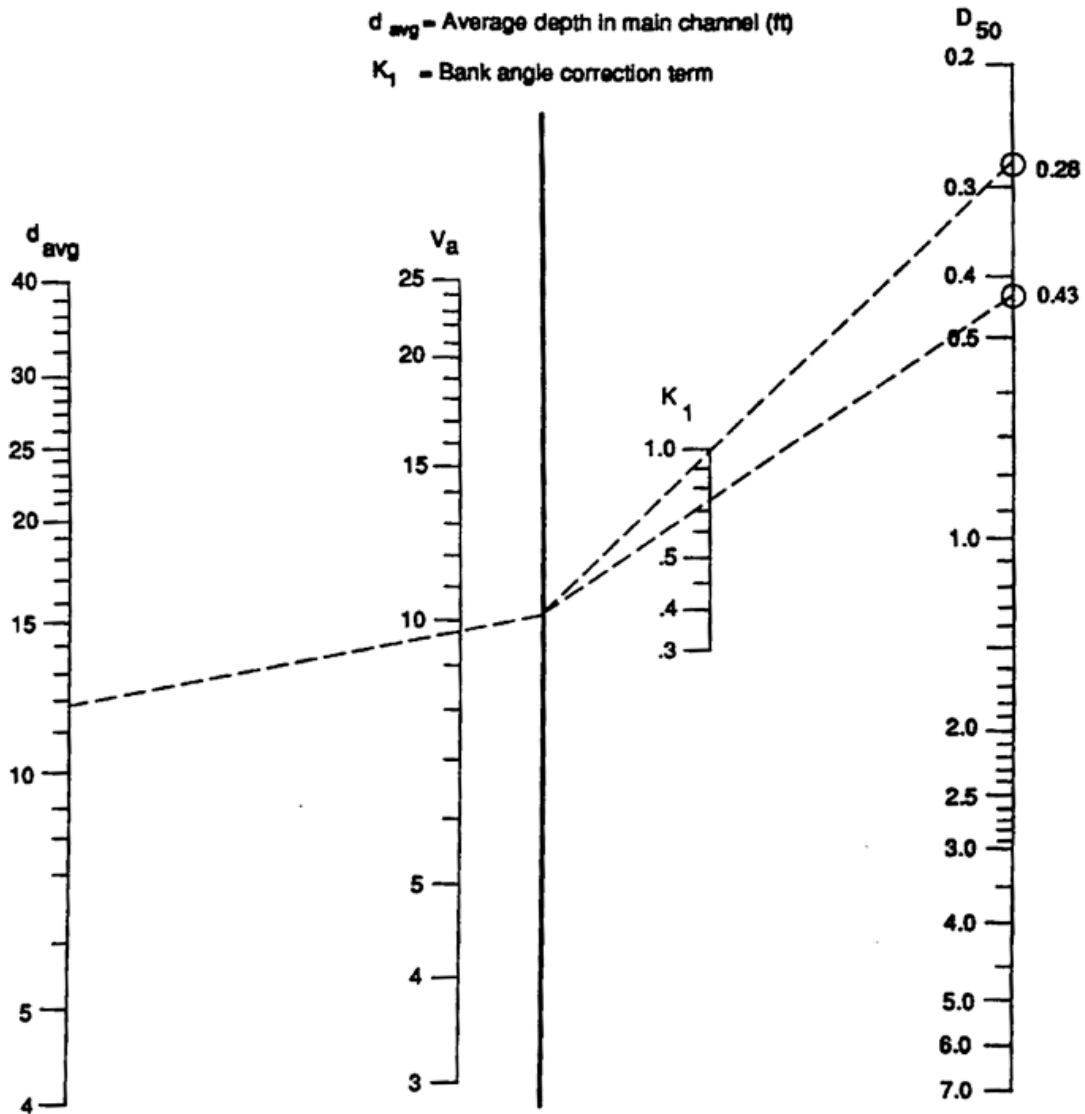
$$D_{50} = 0.001 V_a^3 / (d_{avg}^{1/2} K_1^{3/2})$$

$D_{50}$  = Median Riprap Size (ft.)

$V_a$  = Average velocity in main channel (ft/sec)

$d_{avg}$  = Average depth in main channel (ft)

$K_1$  = Bank angle correction term



**Example**

Given:

$V_a = 9.7$

$d_{avg} = 11.8$  ft.

$K_1 = 0.73$

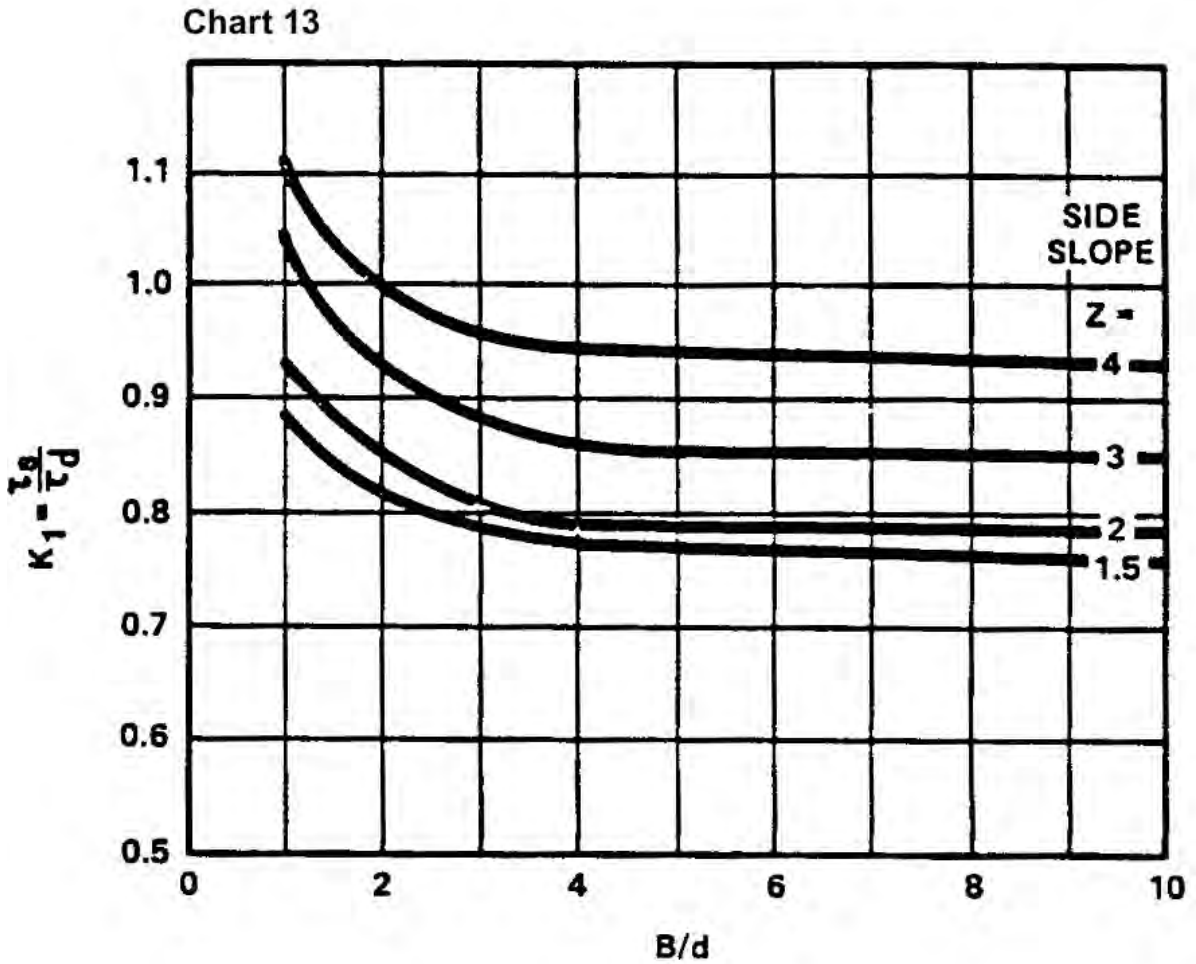
Find:

$D_{50}$

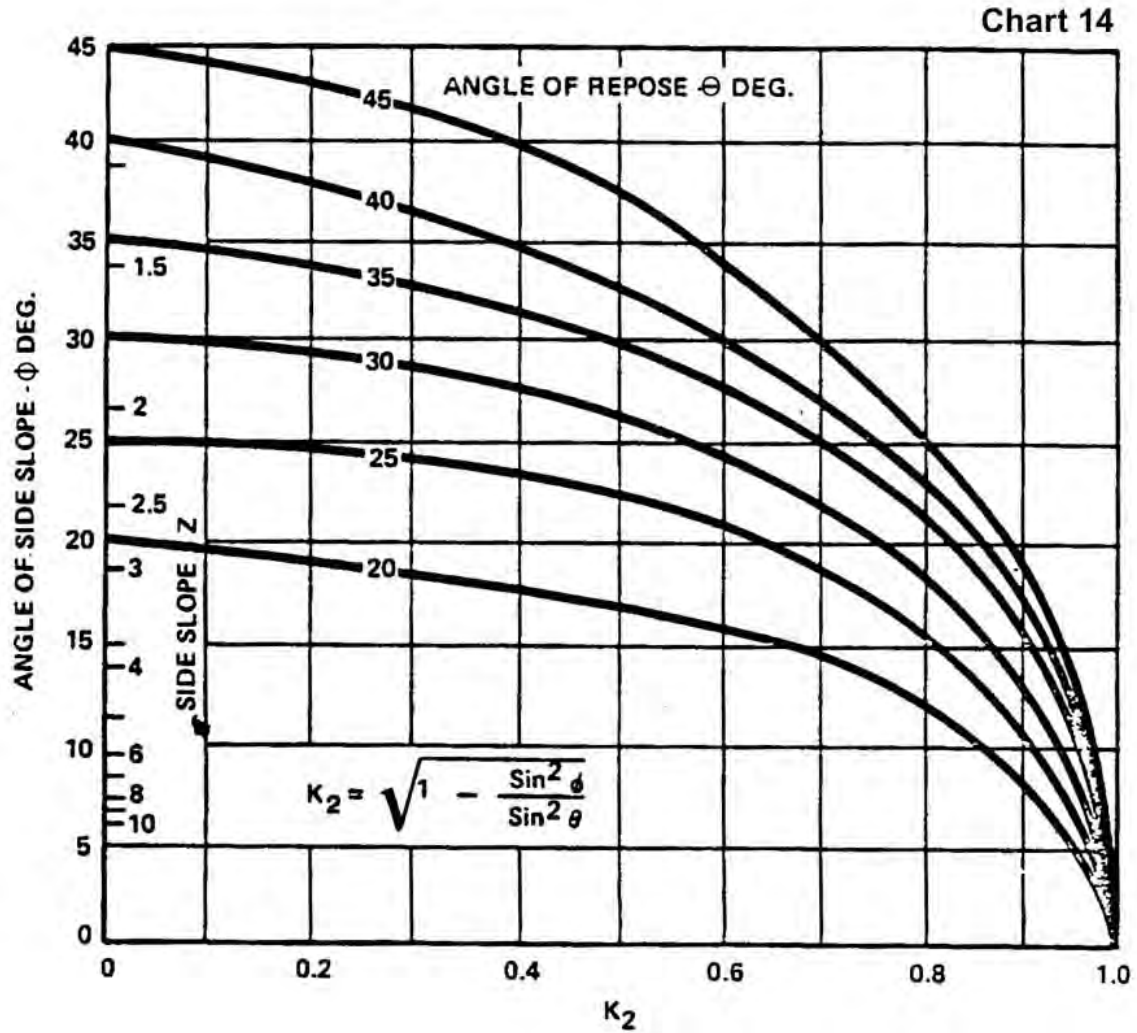
Solution:

$D_{50} = 0.43$

Source: HEC-11



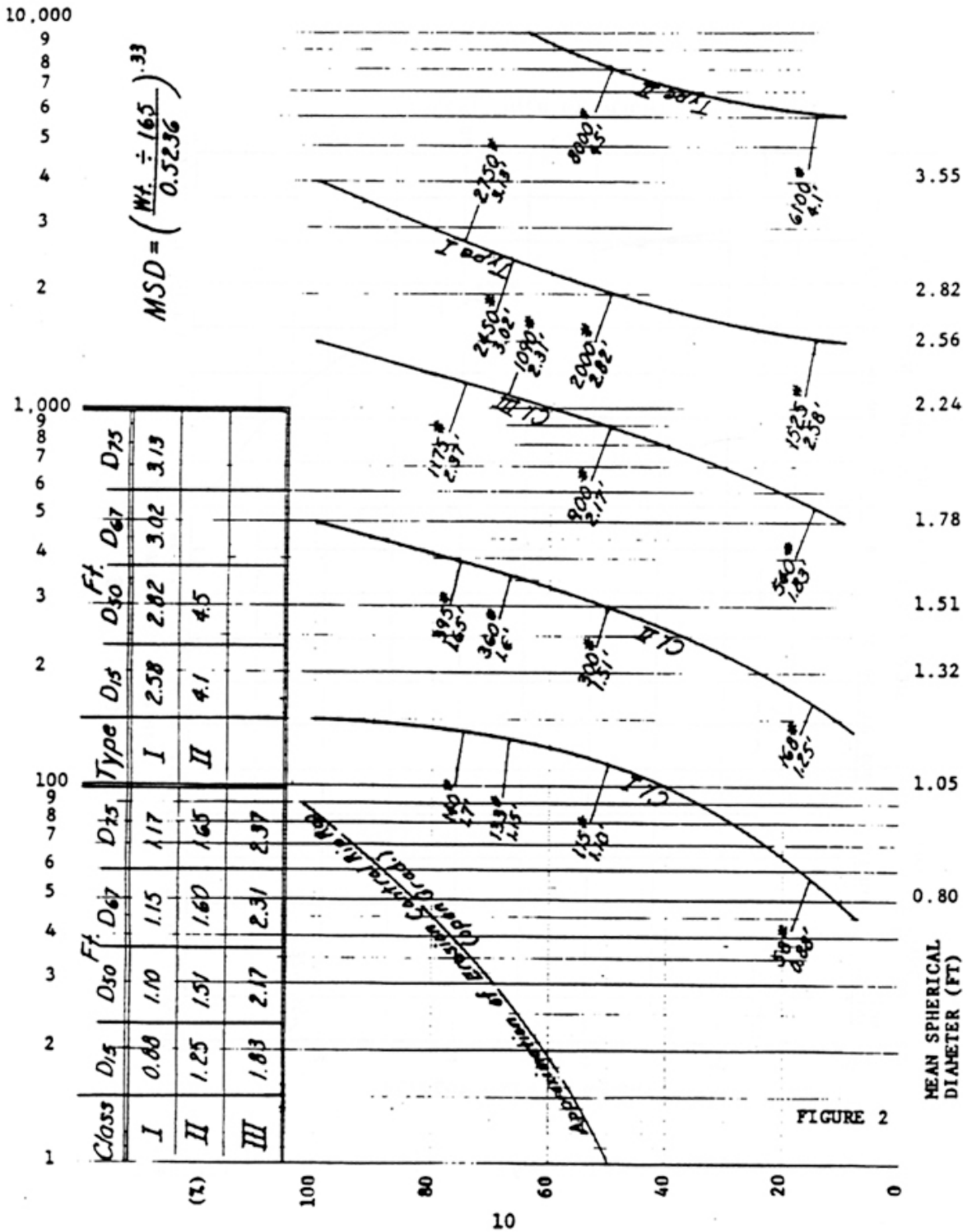
Source: HEC-15 (Archived 1988)



Source: HEC-15 (Archived 1988)

Comment: The symbols of  $\phi$  and  $\theta$  are reversed from Appendix 7E-4.

Appendix 7E-9 Determination of Mean Spherical Diameter



Source: VDOT  
 Comment: Use this chart to obtain D<sub>75</sub> information for the Channel Stability Worksheet (Appendix 7B-3).