Chapter 4 CHANNELS

4.1 INTRODUCTION

Hydraulic design associated with natural channels and roadway ditches is a process which selects and evaluates alternatives according to established criteria. These criteria are the standards established by Mn/DOT to insure that a highway facility meets its intended purpose without endangering the structural integrity of the facility itself and without undue adverse effects on the environment or the public welfare. The purpose of this chapter is to establish Mn/DOT policy, specify design criteria, review design philosophy, and outline channel design procedures.

Channel analysis is necessary for the design of transportation drainage systems in order to assess:

- potential flooding caused by changes in water surface profiles,
- disturbance of the river system upstream or downstream of the highway right-of-way,
- changes in lateral flow distributions,
- changes in velocity or direction of flow,
- need for conveyance and disposal of excess runoff, and
- need for channel lining to prevent erosion.

4.1.1 Definition

Open channels are a natural or man-made conveyance for water in which:

- the water surface is exposed to the atmosphere, and
- the gravity force component in the direction of motion is the driving force.

There are various types of open channels encountered by the designer of transportation facilities: stream channel, roadside channel or ditch, irrigation channel, and drainage ditch. The principles of open channel flow hydraulics are applicable to all drainage facilities including culverts and storm drains. While the principles of open channel flow are the same regardless of the channel type, stream channels and artificial channels (primarily roadside channels) will be treated separately in this chapter as needed.

Stream channels are:

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

Artificial channels include roadside channels, irrigation channels and drainage ditches which are:

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

4.1.2 Concept Definitions

Critical Flow

Critical flow occurs when the specific energy is a minimum. Plotting depth against specific energy at a constant discharge results in a curve where the minimum specific energy occurs at a depth called critical depth, where the Froude number has a value of one.

The general expression for flow at critical depth is:

$$\frac{Q^2}{g} = \frac{A^3}{T} \tag{4.1}$$

Where: Q = discharge (cfs)
g = acceleration due to gravity (32.2 ft/s^2)
A = flow area (ft^2)
T = channel top width at the water surface (ft)

Critical Flow (continued)

Critical depth is also the depth of maximum discharge when the specific energy is held constant. These relationships are illustrated in Figure 4.1. During critical flow the velocity head is equal to half the hydraulic depth. When flow is at critical depth, Equation 4.1 must be satisfied, no matter what the shape of the channel.

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. For rectangular channels the hydraulic depth is equal to the flow depth. This expression for Froude number applies to any single section channel and is defined by:

$$F_r = \frac{V}{\left(\frac{gd}{\alpha}\right)^{0.5}} \quad (4.2)$$

Where:
- V = mean velocity (ft/s); V = Q/A,
- g = acceleration due to gravity (32.2 ft/s^2)
- d = hydraulic depth (ft); d = A/T
- A = cross-sectional area of flow(ft^2)
- Q = total discharge (cfs)
- T = channel top width at the water surface (ft)
- α = velocity distribution coefficient

Gradually Varied Flow

A nonuniform flow in which the depth and velocity change gradually enough in the flow direction that vertical accelerations can be neglected, is referred to as a gradually-varied flow; otherwise, it is considered to be rapidly-varied flow.

Hydraulic Jump

A hydraulic jump occurs as an abrupt transition from supercritical to subcritical flow in the flow direction. There are significant changes in depth and velocity in the jump, and energy is dissipated. For this reason, the hydraulic jump is often employed to dissipate energy and control erosion at highway drainage structures.

Specific Energy

Specific energy, E, is defined as the energy head relative to the channel bottom. If the channel is not too steep (slope 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:

$$E = y + \alpha\frac{V^2}{2g} \quad (4.3)$$

Where:
- y = depth (ft)
- α = kinetic energy correction coefficient
  The velocity distribution coefficient is taken to have a value of one for turbulent flow in prismatic channels, but may be significantly different that one in natural channels.
- V = mean velocity (ft/s)
- g = acceleration due to gravity (32.2 ft/s^2)

Steady and Unsteady Flow

A steady flow is one in which the discharge passing a given cross-section is constant with respect to time. The maintenance of steady flow in any reach requires that the rates of inflow and outflow be constant and equal. When the discharge varies with time, the flow is unsteady.

Subcritical Flow

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

Supercritical Flow Depths less than critical depth occur in supercritical flow where 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.

Thalweg Profile The line extending along a channel profile that follows the lowest elevation of the channel bed.

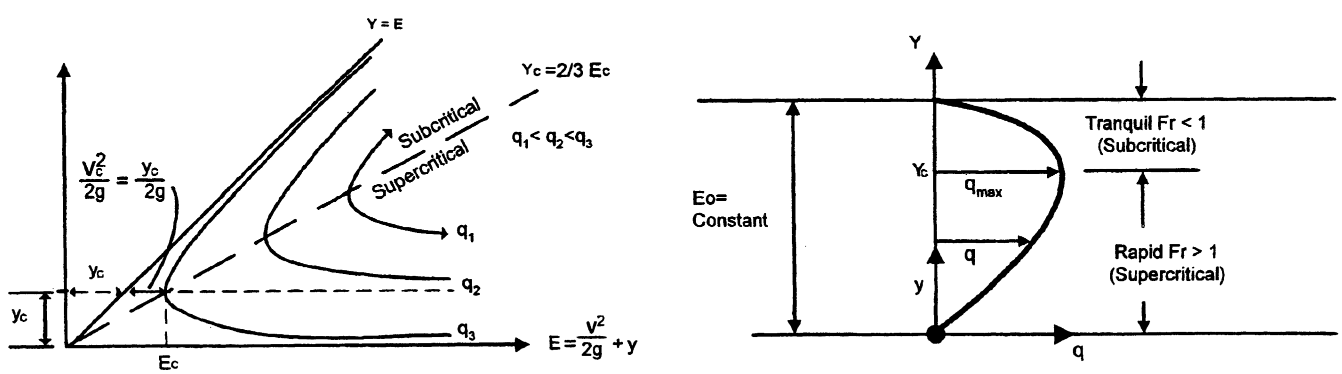
Total Energy Head The total energy head is the specific energy head plus the elevation of the channel bottom with respect to some datum. The locus of the energy head from one cross section to the next defines the energy grade line. See Figure 4.1 for a plot of the specific energy diagram.

Uniform and Nonuniform Flow A nonuniform flow is one in which the velocity and depth vary in the direction of motion, while they remain constant in uniform flow. Uniform flow can only occur in a prismatic channel, which is a channel of constant cross section, roughness, and slope in the flow direction; however, nonuniform flow can occur either in a prismatic channel or in a natural channel with variable properties.

Velocity Distribution Coefficient Due to the presence of a free surface and also due to friction along the channel boundary, the velocities in a channel are not uniformly distributed in the channel section. As a result of nonuniform 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, α, defined as:

$$\alpha = \frac{\sum_{i=1}^{n} (K_i^3/A_i^2)}{(K_t^3/A_t^2)} \qquad (4.4)$$

Where: K_i = conveyance in subsection (see Equation 4.8)
K_t = total conveyance in section (see Equation 4.8)
A_i = cross-sectional area of subsection (ft²)
A_t = total cross-sectional area of section (ft²)
n = number of subsections



(a) Specific Energy Diagram (b) Discharge Diagram

Figure 4.1 Specific Energy And Discharge Diagram For Rectangular Channels
Source: adopted From Highways In The River Environment (FHWA, 1990)

4.2 DESIGN CRITERIA

Design criteria establishes the standards by which a policy is placed into action. They form the basis for the selection of the final design configuration. Listed below are the design criteria which shall be considered for channel design.

4.2.1 Policy

Policy is a set of goals that establish a definite course or method of action and are selected to guide and determine present and future decisions. Policy is implemented through design criteria established as standards for making decisions. The following policies are specific to channels:

- Channel designs and/or designs of highway facilities that impact channels shall satisfy the policies of the Federal Highway Administration applicable to floodplain management if Federal funding is involved.
- Federal Emergency Management Agency floodway regulations and U.S. Army Corps of Engineers wetland restrictions for permits shall be satisfied.
- Coordination with other Federal, State, and local agencies concerned with water resources planning shall have high priority in the planning of highway facilities.
- Safety of the general public shall be an important consideration in the selection of cross-sectional geometry of artificial drainage channels.
- The design of artificial drainage channels or other facilities shall consider the frequency and type of maintenance expected and make allowance for access of maintenance equipment.
- A stable channel is the goal for all channels that are located on highway right-of-way or that impact highway facilities.
- Environmental impacts of channel modifications, including disturbance of fish habitat, wetlands, and channel stability shall be assessed.
- The range of design channel discharges shall be selected and approved by the designer based on class of roadway, consequences of traffic interruption, flood hazard risks, economics, and local site conditions.

4.2.2 Stream Channels

The following criteria applies to natural channels:

- The hydraulic effects of flood plain encroachments shall normally be evaluated over a full range of frequency-based peak discharges from the mean annual or bank full flood through the 500-year flood on any major highway facility as deemed necessary by the designer.
- If relocation of a stream channel is unavoidable, the cross-sectional shape, meander pattern, roughness, sediment transport, and slope shall conform to the existing conditions insofar as practicable. Some means of energy dissipation may be necessary when existing conditions cannot be duplicated.
- Streambank stabilization shall be provided, when appropriate, to prevent bank erosion resulting from stream disturbances caused by highway projects and shall include both upstream and downstream banks as well as the local site.
- Provision should be incorporated into the design and construction for access by maintenance personnel and equipment to maintain features such as dikes and levees.

4.2.3 Roadside Channels

The following criteria applies to roadside channels:

- The design discharge for permanent roadside ditch linings shall have a 10-year frequency while temporary linings shall be designed for the 2-year frequency flow. Engineering judgement shall be utilized in selecting a design frequency for evaluating flood damage potential; however, a 100 year frequency is generally recommended.
- Channel side slopes shall not exceed the angle of repose of the soil and/or lining and shall be 1V:3H or flatter.
- Channel side slopes should meet requirements for clear zones as specified in the Mn/DOT Road Design Manual.
- Ditch depths are typically 4′ to 6′ in order to provide adequate drainage for the base of the road.
- Flexible linings shall be designed according to the method of allowable tractive force.
- Channel freeboard shall be the larger of one foot or two velocity heads.

4.3 OPEN CHANNEL FLOW

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 exist. Manning's equation can be used to estimate highwater elevations for bridges that do not constrict the flow. 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.

The single-section method will generally yield less reliable results because it requires more judgment and assumptions than the step-backwater method. In many situations, however, the single-section method is all that is justified for instance when analyzing a standard roadway ditch, culvert, or storm drain outfall.

Design analysis of both natural and artificial channels proceeds according to the basic principles of open channel flow (see Chow, 1970; 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 artificial channels.

4.3.1 Flow Classification

The classification of open channel flow can be summarized as follows:

Steady Flow
- Uniform Flow
- Nonuniform Flow
  - Gradually Varied Flow
  - Rapidly Varied Flow

Unsteady Flow
- Unsteady Uniform Flow (rare)
- Unsteady Nonuniform Flow
  - Gradually Varied Unsteady Flow
  - Rapidly Varied Unsteady Flow

The steady uniform flow case and the steady nonuniform flow case are the most fundamental types of flow treated in highway engineering hydraulics.

4.3.2 Equations

The following equations are those most commonly used to analyze open channel flow.

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 where the subscripts 1 and 2 refer to successive cross sections along the flow path:

$$Q = A_1 V_1 = A_2 V_2 \tag{4.5}$$

Where: Q = discharge (cfs)
A = cross-sectional area of flow (ft^2)
V = mean cross-sectional velocity (ft/s); Where V is perpendicular to cross-section

Manning's Equation

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

$$V = 1.49\frac{R^{2/3}S^{1/2}}{n} \tag{4.6}$$

Where: V = mean velocity (ft/s)
n = Manning's roughness coefficient
R = hydraulic radius (ft); R = area/wetted perimeter
A = cross-sectional area of flow (ft^2)
P = wetted perimeter (ft)
S = slope of the energy gradeline (ft/ft);
For steady uniform flow, S ≈ channel slope (ft/ft)

The selection of Manning's n is generally based on observation; however, considerable experience is essential in selecting appropriate n values. The range of n values for various types of channels and floodplains is given in Table 4.1. The continuity equation can be combined with Manning's equation to obtain the steady, uniform flow discharge as:

$$Q = VA = 1.49\frac{AR^{2/3}S^{1/2}}{n} \tag{4.7}$$

For a given channel geometry, slope, and roughness, and a specified value of discharge Q, a unique value of depth occurs in steady, uniform flow. It is called normal depth and is computed from equation 4.7 by expressing the area and hydraulic radius in terms of depth. The resulting equation may require a trial and error solution. If the normal depth is greater than critical depth, the slope is classified as a mild slope, while on a steep slope, the normal depth is less than critical depth. Thus, uniform flow is subcritical on a mild slope and supercritical on a steep slope.

Conveyance

In channel analysis, it is often convenient to group the channel properties in a single term called the channel conveyance, K. 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, α (see Equation 4.4).

$$K = 1.49\frac{AR^{2/3}}{n} \tag{4.8}$$

and then Equation 4.7 can be written as:

$$Q = KS^{1/2} \tag{4.9}$$

Energy Equation

The energy equation expresses conservation of energy in open channel flow expressed as energy per unit weight of fluid which has dimensions of length and is therefore 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 which give the total energy head at any cross section when added. Written between an upstream open channel cross section designated 1 and a downstream cross section designated 2, the energy equation is:

$$h_1 + \alpha_1 \frac{V_1^2}{2g} = h_2 + \alpha_2 \frac{V_2^2}{2g} + h_L \quad (4.10)$$

Where: h_1 = upstream stage(ft)
h_2 = downstream stage (ft)
α = velocity distribution coefficient
V = mean velocity (ft/s)
h_L = head loss due to local cross-sectional changes (minor loss) as well as boundary resistance (ft)

The stage, h is the sum of the elevation head, z at the channel bottom and the pressure head, or depth of flow, y (h=z+y). The terms in the energy equation are illustrated graphically in Figure 4.2. 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 the intervening energy head loss. The energy equation can only be applied between two cross sections at which the streamlines are nearly straight and parallel so that vertical accelerations can be neglected.

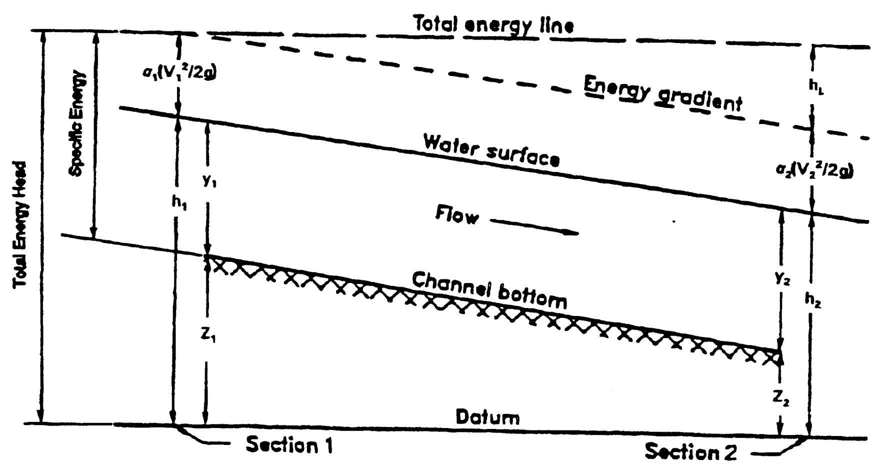


Figure 4.2 Terms In The Energy Equation
Source: FHWA, 1990

Shear Stress
The shear stress is the hydrodynamic force of water flowing in a channel. Stable channel design is based on the rule that the flow induced shear stress should not exceed the permissible shear stress for the lining material. Shear stress is calculated:

$$\tau = \gamma R S \quad (4.11)$$

If the width is very large in relation to the depth substitute d = R, to compute the maximum shear stress:

$$\tau_d = \gamma d S \quad (4.12)$$

Where: τ = shear stress (lb/ft^2)
τ_d = maximum shear stress at normal depth (lb/ft^2)
τ_p = permissible shear stress (lb/ft^2)
γ = unit weight of water (62.4 lb/ft^3)
R = hydraulic radius (feet)
d = maximum depth of flow (feet)
S = average bed slope or energy slope (ft/ft)

Determine the permissible shear stress, τ_p in lbs/ft^2 for the lining material. If $\tau_d < \tau_p$ then lining is acceptable.

Table 4.1 Values of Roughness Coefficient for Uniform Flow

MANNINGS 'N' FOR UNIFORM FLOW				
Channel Type	Channel Description	Minimum	Normal	Maximum
Excavated or Dredged	Earth, straight and uniform	0.016	0.018	0.020
	1. Clean, recently completed	0.018	0.022	0.025
	2. Clean, after weathering	0.022	0.025	0.030
	3. Gravel, uniform section, clean	0.022	0.027	0.033
	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
	Dragline-excavated or dredged			
	1. No vegetation	0.025	0.028	0.033
	2. Light brush on banks	0.035	0.050	0.060
	Rock cuts			
	1. Smooth and uniform	0.025	0.035	0.040
	2. Jagged and irregular	0.035	0.040	0.050
	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
Natural Streams	Streams on Plain			
	1. Clean, straight, full stage, no rifts or deep pools	0.025	0.030	0.033
Minor streams (top width	2. Same as 1, but more stones and weeds	0.030	0.035	0.040
at flood stage < 100 ft)	3. Clean, winding, some pools and shoals	0.033	0.040	0.045
	4. Same as 3, but some weeds and some stones	0.035	0.045	0.050
	5. Same as 4, lower stages, more ineffective slopes and sections	0.040	0.048	0.055
	6. Same as 4, but more stones			
	7. Sluggish reaches, weedy, deep pools	0.045	0.050	0.060
	8. Very weedy reaches, deep pools, or floodways with heavy	0.050	0.070	0.080
	stand of timber and underbrush	0.075	0.100	0.150
	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
Natural Streams	Pasture, no brush			
	1. Short grass	0.025	0.030	0.035
Flood Plains	2. High grass	0.030	0.035	0.050
	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
	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	0.116
	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 spouts	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
Natural Major Streams	Regular section with no boulders or brush	0.025	----	0.060
(top width at flood stage > 100 ft). [1]	Irregular and rough section	0.035	----	0.100

[1] The n value is less than that for minor streams of similar description, because banks offer less effective resistance.

Source: Chow, 1970.

4.3.3 Cross Sections

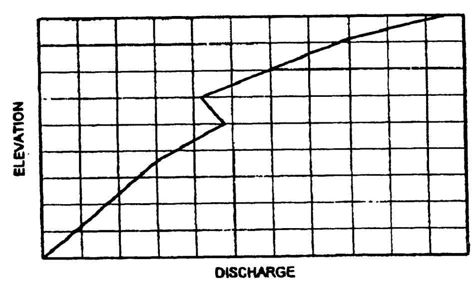
Cross sectional geometry of streams is defined by coordinates of lateral distance and ground elevation which locate individual ground points. The cross section is taken normal to the flow direction along a single straight line where possible, but in wide floodplains or bends it may be necessary to use a section along intersecting straight lines, 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 subreaches 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 change 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 α, 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).

Manning's n is affected by many factors and its selection in natural channels depends heavily on engineering experience. Pictures of channels and flood plains for which the 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 in *Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Flood Plains* (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 high water marks and/or gaged streamflow data. Manning's n values for artificial channels are more easily defined than for natural stream channels. See Table 4.1 for typical n values of both artificial channels and natural stream channels.

The equations should be calibrated with historical high water marks and/or gaged streamflow data to ensure that they accurately represent local channel conditions. The following parameters, in order of preference, should be used for calibrations: Manning's n, slope, discharge, and cross section. Proper calibration is essential if accurate results are to be obtained.



SWITCHBACK

If the cross section is improperly subdivided, the mathematics of the 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 it had 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 in order to avoid the switchback. This phenomenon can occur in any type of conveyance computation, including the step-backwater method. Computer logic can be seriously confused if a switchback were to occur in any cross-section being used in a step backwater program. For this reason, the cross section should always be subdivided with respect to both vegetation and geometric changes. Note that the actual n-value, itself, may be the same in adjacent subsections.

4.3.4 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 cross-section properties including geometry, slope, and roughness. It implicitly assumes the existence of steady, uniform flow; however, uniform flow rarely exists in either artificial or stream channels. Nevertheless, the single-section method is often used to design artificial channels for uniform flow as a first approximation, and to develop a stage-discharge rating curve in a stream channel for tailwater determination at a culvert or storm drain outlet.

A stage-discharge curve is a graphical relationship of streamflow depth or elevation to discharge at a specific point on a stream. This relationship should cover a range of discharges up to at least the base (100-year) flood. The stage-discharge curve can be determined as follows:

- Select the typical cross section at or near the location where the stage-discharge curve is needed.

- Subdivide cross section and assign n-values to subsections as described in Section 4.3.3
- Estimate water surface slope. Since uniform flow is assumed, the average slope of the streambed can usually be used.
- Apply a range of incremental water surface elevations to the cross section.
- Calculate the discharge using Manning's equation for 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.
- 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 elevation corresponding to the design discharge or other discharge of interest.

In stream 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 in highway fills, for example. The best method of establishing transverse velocity variations is by current meter measurements. If this is not possible, the single-section 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, $V = Q/A$.

Alluvial channels present a more difficult problem in establishing stage-discharge relations by the single-section method because the bed itself is deformable and may generate bed forms such as ripples and dunes in lower regime flows. These bed forms are highly variable with the addition of form resistance, and selection of a value of Manning's n is not straightforward. Instead, several methods outlined in (Vanoni, 1977) have been developed for this case (Einstein-Barbarossa; Kennedy-Alam-Lovera; and Engelund) and should be followed unless it is possible to obtain a measured stage-discharge relation.

There may be locations where a stage-discharge relationship has already been measured in a channel. These usually exist at gaging stations on streams monitored by the USGS. Measured stage-discharge curves will generally yield more accurate estimates of water surface elevation and should take precedence over the analytical methods described above.

4.3.5 Step-Backwater Analysis

Step-backwater analysis is useful for determining unrestricted water surface profiles where a highway crossing is planned, and for analyzing how far upstream the water surface elevations are affected by a culvert or bridge. Because the calculations involved in this analysis are tedious and repetitive, it is recommended that a computer program such as the FHWA/USGS program WSPRO, Corps of Engineers HEC-2 or HEC-RAS be used.

The WSPRO program has been designed to provide a water surface profile for six major types of open channel flow situations:

- unconstricted flow,
- single opening bridge,
- bridge opening(s) with spur dikes,
- single opening embankment overflow,
- multiple alternatives for a single site, and
- multiple openings.

The HEC-2 or HEC-RAS program developed by the Corps of Engineers is widely used for calculating water surface profiles for steady gradually varied flow in a natural or constructed channels. Both subcritical and supercritical flow profiles can be calculated. The effects of bridges, culverts, weirs, and structures in the floodplain may also be considered in the computations. This program is designed for application in floodplain management and flood insurance studies.

The computation of water surface profiles by WSPRO, HEC-2 or HEC-RAS is based on the standard step method in which the stream reach is divided into a number of subreaches by cross sections spaced such that the flow is gradually-varied in each subreach. The energy equation is then solved in a step-wise fashion for the stage at one cross section based on the stage at the previous cross section.

Water surface profile computation requires a beginning value of elevation or depth (boundary condition) and proceeds upstream for subcritical flow and downstream for supercritical flow. In the case of supercritical flow, critical depth is often the boundary condition at the control section, but in subcritical flow, uniform flow and normal depth may be the boundary condition. The starting depth in this case can either be found by the single-section method (slope-area method) or by computing the water surface profile upstream to the desired location for several starting depths and the same discharge. These profiles should converge toward the desired normal depth at the control section to establish one point on the stage-discharge relation. If the several profiles do not converge, then the stream reach may need to be extended downstream, or a shorter cross-section interval should be used, or the range of starting water-surface elevations should be adjusted. In any case, a plot of the convergence profiles can be a very useful tool in such an analysis (see Figure 4.3).

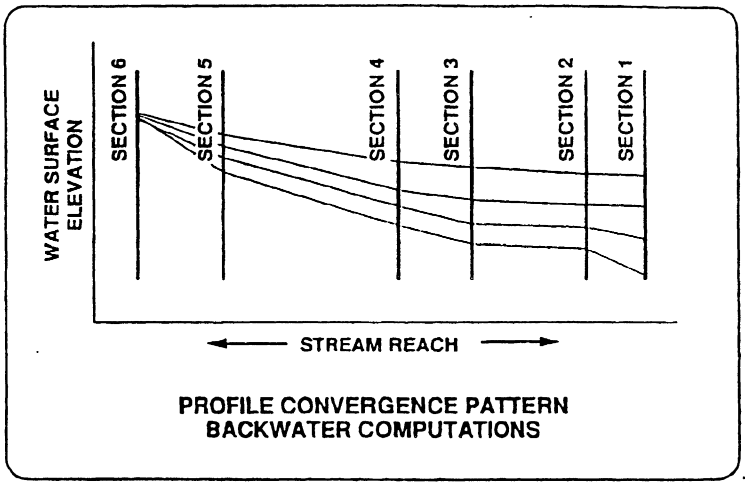


Figure 4.3 Profile Convergence Pattern Backwater Computation

Given a long enough stream reach, the water surface profile computed by step-backwater will converge to normal depth at some point upstream for subcritical flow. Establishment of the upstream and downstream boundaries of the stream reach is required to define limits of data collection and subsequent analysis. Calculations must begin sufficiently far downstream to assure accurate results at the structure site, and continued a sufficient distance upstream to accurately determine the impact of the structure on upstream water surface profiles (see Figure 4.4). The Corps of Engineers (USACE, 1986) developed equations for determining upstream and downstream reach lengths as follows:

$$L_{dn} = 8{,}000\frac{HD^{0.8}}{S} \qquad \textbf{(4.13)}$$

$$L_{u} = 10{,}000\frac{HD^{0.6}\ HL^{0.5}}{S} \qquad \textbf{(4.14)}$$

Where: L_{dn} = downstream study length (along main channel), (ft) for normal depth starting conditions

L_u = estimated upstream study length (along main channel), (ft) required for convergence of the modified profile to within 0.1 feet of the base profile

HD = average hydraulic depth (1-percent chance event flow area divided by the top width), (ft)

S = average reach slope (ft/mile)

HL = headloss ranging between 0.5 and 5.0 feet at the channel crossing structure for the 1-percent chance flood (ft)

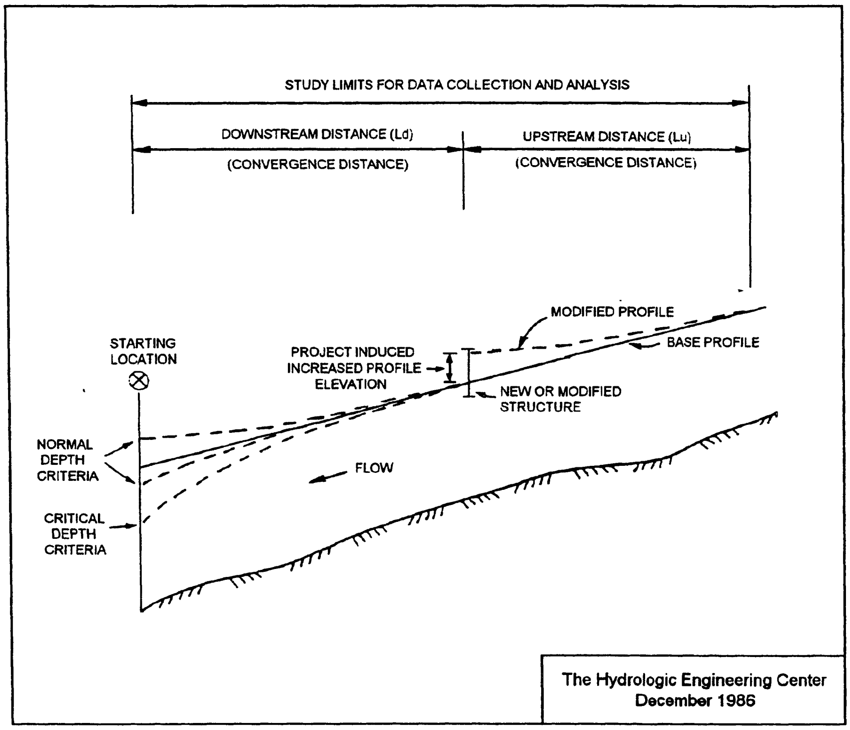


Figure 4.4 Profile Study Limits
Source: USACE, 1986

References (Davidian, 1984; USACE, 1986) are very valuable sources of additional guidance on the practical application of the step-backwater method to highway drainage problems involving open-channels. These references contain more specific guidance on cross-section determination, location, spacing and stream reach determination. 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. A sample procedure is available in *Hydrologic Engineering Methods For Water Resources Development - Volume 6, Water Surface Profiles,* The Hydrologic Engineering Center, Corps of Engineers, U.S. Army, Davis, California.

4.3.6 Water and Sediment Routing

The BRI-STARS (Bridge Stream Tube Model for Sediment Routing Alluvial River Simulation) Model was developed by the National Cooperative Highway Research Program and FHWA (Molinas, 2000). It is based on utilizing the stream tube method of calculation which allows the lateral and longitudinal variation of hydraulic conditions as well as sediment activity at various cross sections along the study reach. Both energy and momentum functions are used in the BRI-STARS model so the water surface profile computation can be carried out through combinations of subcritical and supercritical flows without interruption. The stream tube concept is used for hydraulic computations in a semi-two-dimensional way. Once the hydraulic parameters in each stream tube are computed, the scour or deposition in each stream tube determined by sediment routing will give the variation of channel geometry in the vertical direction.

The BRI-STARS model contains a rule-based expert system program for classifying streams by size, bed and bank material stability, planform geometry, and other hydrologic and morphological features. Due to the complexities of a single classification system that utilizes all parameters, no universally acceptable stream classification method presently exists. Consequently this model does not contain a single methodology for classifying all streams. Instead, methodologies were first classified according to the channel sediment sizes they were derived for, then within each size group, one or more classification schemes have been included to cover a wider range of environments. The stream classification information can be used to assist in the selection of model parameters and algorithms. Applications of the BRI-STARS can be summarized as follows:

- Fixed bed model to compute water surface profiles for subcritical, supercritical, or both flow conditions involving hydraulic jumps.
- Movable bed model to route water and sediment through alluvial channels.
- Use of stream tubes to allow the model to compute the variation of hydraulic conditions and sediment activity in the longitudinal as well as the lateral direction.
- The armoring option allows simulation of longer term riverbed changes.
- The minimization procedure option allows the model to simulate channel widening and narrowing processes.
- The local bridge scour option allows the computation of pier and abutment scour.
- The bridge routines for fixed geometry mode from WSPRO are available as an option in the program.

4.4 DESIGN PROCEDURE

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

4.4.1 Stream Channels

The analysis of a stream channel in most cases is in conjunction with the design of a highway hydraulic structure such as a culvert or bridge. In general, the objective is to convey the water along or under the highway in such a manner that will not cause damage to the highway, stream, or adjacent property. An assessment of the existing channel is usually necessary to determine the potential for problems that might result from a proposed action. The extent of the study should be commensurate with the risk associated with the action and with the environmental sensitivity of the stream and adjoining flood plain. Although the following step-by-step procedure may not be appropriate for all possible applications, it does outline a generalized process which will usually apply.

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

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 (See Section 4.2)

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
- Qualitative assessment
- Single-section analysis
- Step-backwater analysis
- 2-D modeling

C. Determine additional survey information
- Extent of streambed profiles
- Elevations of flood-prone property
- Details of existing structures
- Properties of bed and bank materials

Step 3 **Evaluate hydrologic variables.**

Compute discharges for selected frequencies.
Consult Hydrology Chapter

Step 4 **Perform hydraulic analysis.**

A. Single-section analysis
- Select representative cross section
- Select appropriate n values
- Compute stage-discharge relationship

B. Step-backwater analysis

C. Calibrate with known high water

Step 5 **Perform stability analysis.**

A. Geomorphic factors

B. Hydraulic factors

C. Stream response to change

D. For additional information
- HEC-20 Stream Stability
- Highways in the River Environment (HIRE)

Step 6 **Design countermeasures.**

A. Criteria for selection
- Erosion mechanism
- Stream characteristics
- Construction and maintenance requirements
- Vandalism considerations
- 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-23 Bridge Scour and Stream Instability Countermeasures
- Review References, Section 4.6

Step 7 **Documentation.**

Prepare report and file with background information

4.4.2 Roadside Channels

A roadside channel is defined as an open channel, usually paralleling the highway embankment and within the limits of the highway right-of-way. It is normally trapezoidal in cross section and lined with grass or a special protective lining. The primary function of roadside channels is to collect surface runoff from the highway and areas which drain to the right-of-way and convey the accumulated runoff to acceptable outlet points. A secondary function of a roadside channel 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 channels is usually constrained to a large extent by the geometric and safety standards applicable to the project. These channels should accommodate the design runoff in a manner which assures the safety of motorists and minimizes future maintenance, damage to adjacent properties, and adverse environmental or aesthetic effects. Proper shaping such as slope and ditch rounding, cut to fill blending, built in gully elimination, and feathering cuts are effective means of erosion control. Each project is unique, but the following six basic design steps are normally applicable:

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.
C. Determine and plot on the plan the locations of natural basin divides and roadside channel outlets.
D. Perform the layout of the proposed roadside channels to minimize diversion flow lengths.

Step 2 **Obtain or establish cross section data.**

A. Provide channel depth adequate to drain the subbase and minimize freeze-thaw effects.
B. Choose channel side slopes based on geometric design criteria including safety, clear zone requirements, economics, soil, aesthetics, and access.
C. Establish bottom width of trapezoidal channel. A desirable width is 8′.
D. Identify features which may restrict cross section design:
   - right-of-way limits,
   - trees or environmentally-sensitive areas,
   - utilities, and
   - existing drainage facilities.

Step 3 **Determine initial channel grades.**

A. Plot initial grades on plan-profile layout. (Slopes in roadside ditch in cuts are usually controlled by highway grades.)
B. Provide minimum grade of 0.3%, if possible, to minimize ponding and sediment accumulation.
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 channel segment (see Hydrology chapter).
B. Set preliminary values of channel size, roughness coefficient, and slope.
C. Determine maximum allowable depth of channel including freeboard.
D. Check flow capacity using Manning's equation and single-section analysis.
E. If capacity is inadequate, possible adjustments are as follows:
   - 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,
   - provide smooth transitions at changes in channel cross sections, or
   - provide extra channel storage where needed to replace floodplain storage and/or to reduce peak discharge.

Step 5 **Determine channel lining/protection needed.**

A. Select a lining and determine the permissible shear stress τ_p in lbs/ft^2 from Table 4.3.
B. Estimate the flow depth and choose an initial Manning's n value from Table 4.4 or from Figures 4.5 through 4.10. To use Figures 4.5 through 4.10 determine the vegetation classification using Table 4.2. For a know channel geometry and therefore known hydraulic radius (R) and slope (S) use the figure to find Manning's n on the Y axis.
C. Calculate normal flow depth y_o (ft) at design discharge using Manning's equation and compare with the estimated depth. If they do not agree, repeat steps 5B and 5C.
D. Compute maximum shear stress at normal depth using Equation 4.11 or 4.12.
   If $\tau_d < \tau_p$ then lining is acceptable. Otherwise consider the following options:
   - choose a more resistant lining,
   - decrease channel slope,
   - decrease slope in combination with drop structures, and/or
   - increase channel width and/or flatten side slopes

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, or
- diversion of flow from another watershed.

B. Mitigate any adverse impacts identified in 6A. Possibilities include:
- enlarge outlet channel and/or install control structures to provide detention of increased runoff in channel,
- install velocity control structures (energy dissipators),
- increase capacity and/or improve lining of downstream channel,
- install sedimentation/infiltration basins,
- install sophisticated weirs or other outlet devices to redistribute concentrated channel flow,
- eliminate diversions which result in downstream damage and which cannot be mitigated in a less expensive fashion.

In order to obtain the optimum roadside channel system design, it may be necessary to make several trials of the previous procedure before a final design is achieved. More details on channel lining design may be found in HEC-15 (FHWA, 1988) including consideration of channel bends, steep slopes, and composite linings. The riprap design procedures covered in HEC-15 are for channels having a design discharge of 50 cfs or less. When the design discharge exceeds 50 cfs, the designer should refer to other sources such as Design of Riprap Revetment, HEC-11, (FHWA, 1989).

Table 4.2 Classification of Vegetal Covers as to Degrees of Retardancy

Retardance	Cover	Condition
Class A	Weeping Lovegrass	Excellent stand, tall (average 30")
	Yellow Bluestem, Ischaemum	Excellent stand, tall (average 36")
Class B	Kudzu	Dense growth, uncut
	Bermuda grass	Good stand, tall (average 12")
	Native grass mixture: little bluestem, bluestem, blue gamma, other short and long stem midwest grasses	Good stand, unmowed
	Weeping lovegrass	Good Stand, tall (average 24")
	Laspedeza sericea	Good stand, not woody, tall (average 19")
	Alfalfa	Good stand, uncut (average 11")
	Weeping lovegrass	Good stand, unmowed (average 13")
Class C	Crabgrass	Fair stand, uncut (10 - 48")
	Bermuda grass	Good stand, mowed (average 6")
	Common lespedeza	Good stand, uncut (average 11")
	Grass-legume mixture: summer orchard grass redtop, Italian ryegrass, and common lespedeza	Good stand, uncut (6-8")
	Centipedegrass	Very dense cover (average 6")
	Kentucky bluegrass	Good stand, headed (6 - 12")
Class D	Bermuda grass	Good stand, cut to 2.5"
	Common lespedeza	Excellent stand, uncut (average 4.5")
	Buffalo grass	Good stand, uncut (3-6")
	Grass-legume mixture: fall, spring (orchard grass, redtop, Italian ryegrass, and common lespedeza	Good Stand, uncut (4-5")
	Lespedeza serices	After cutting to 2" (very good before cutting)
Class E	Bermuda grass	Good stand, cut to 1.5"
	Bermuda grass	Burned stubble

Note: Table is used to determine the class of retardance used in Table 4.3 and Figures 4.6 - 4.10.
Covers classified have been tested in experimental channel. Covers were green and generally uniform. Source of table is HEC-15 (FHWA, 1988).

Table 4.3 Summary Of Permissible Shear Stress For Various Protection Measures

Protective Cover	Underlying Soil Type	τ_p (lb/ft^2)	Protective Cover	Underlying Soil Type	τ_p (lb/ft^2)
Vegetation Class	Erosion		Structural	Type I	
A	Resistant or	3.70	6 in Gabions		35
B	Erodible	2.10	4 in Geoweb		10
C		1.00	Soil Cement (8% Cement)		>45
D		0.60			
E		0.35			
Temporary			Dycel w/o Grass		>7.0
Woven Paper		0.15	Petraflex w/o Grass		>32
Jute Net		0.45	Armorflex w/o Grass		12-20
Single Fiberglass		0.60	Enkamat w/3 in Asphalt		13-16
Double Fiberglass		0.85	Enkamat w/1 in Asphalt		<5
Straw w/Net		1.45			
Curled Wood Mat		1.55			
Synthetic Mat		2.00			
Gravel Riprap			Armorflex Class 30 with longitudinal and lateral cables, no grass		>34
D_{50} = 1 in		0.40			
D_{50} = 2 in		0.80			
Rock Riprap			Dycell 100, longitudinal cables, cells filled with mortar		<12
D_{50} = 6 in		2.50			
D_{50} = 12 in		5.00			
Concrete construction Blocks, granular filter underlayer	Type I	>20	Wedge-shaped blocks with drainage slot		>25

Type I soil is a silty clay to silty sand (SC-SM) with AASHTO classification A-4(0).
Source: FHWA-RD-89-199 (Clopper, 1989)

Table 4.4 Manning's Roughness Coefficients - Channel Lining Roughness Element Height, k_s

Lining Category	Lining Type	k_s (ft)	n - value for Depth Ranges		
			0 - 0.5 ft	0.5 - 2.0 ft	> 2.0 ft
Rigid	Concrete		0.015	0.013	0.013
	Grouted Riprap		0.040	0.030	0.028
	Stone Masonry		0.042	0.032	0.030
	Soil Cement		0.025	0.022	0.020
	Asphalt		0.018	0.016	0.016
Unlined	Bare Soil		0.023	0.020	0.020
	Rock Cut		0.045	0.035	0.025
Temporary*	Woven Paper Net	0.004	0.016	0.015	0.015
	Jute Net	0.038	0.028	0.022	0.019
	Fiberglass Roving	0.035	0.028	0.022	0.019
	Straw with Net	0.120	0.065	0.033	0.025
	Curled Wood Mat	0.110	0.066	0.035	0.028
	Synthetic Mat	0.065	0.036	0.025	0.021
Gravel Riprap	1-inch D_{50}	0.083	0.044	0.033	0.030
	2-inch D_{50}	0.167	0.066	0.041	0.034
Rock Riprap	6-inch D_{50}	0.500	0.104	0.069	0.035
	12-inch D_{50}	1.000	–	0.078	0.040

Values listed are representative values for the respective depth ranges.
Manning's roughness coefficients, n, vary with the flow depth.
Source: HEC-15 (FHWA, 1988)

For riprap $k_s = D_{50}$.
* Some "temporary" linings become permanent when buried.

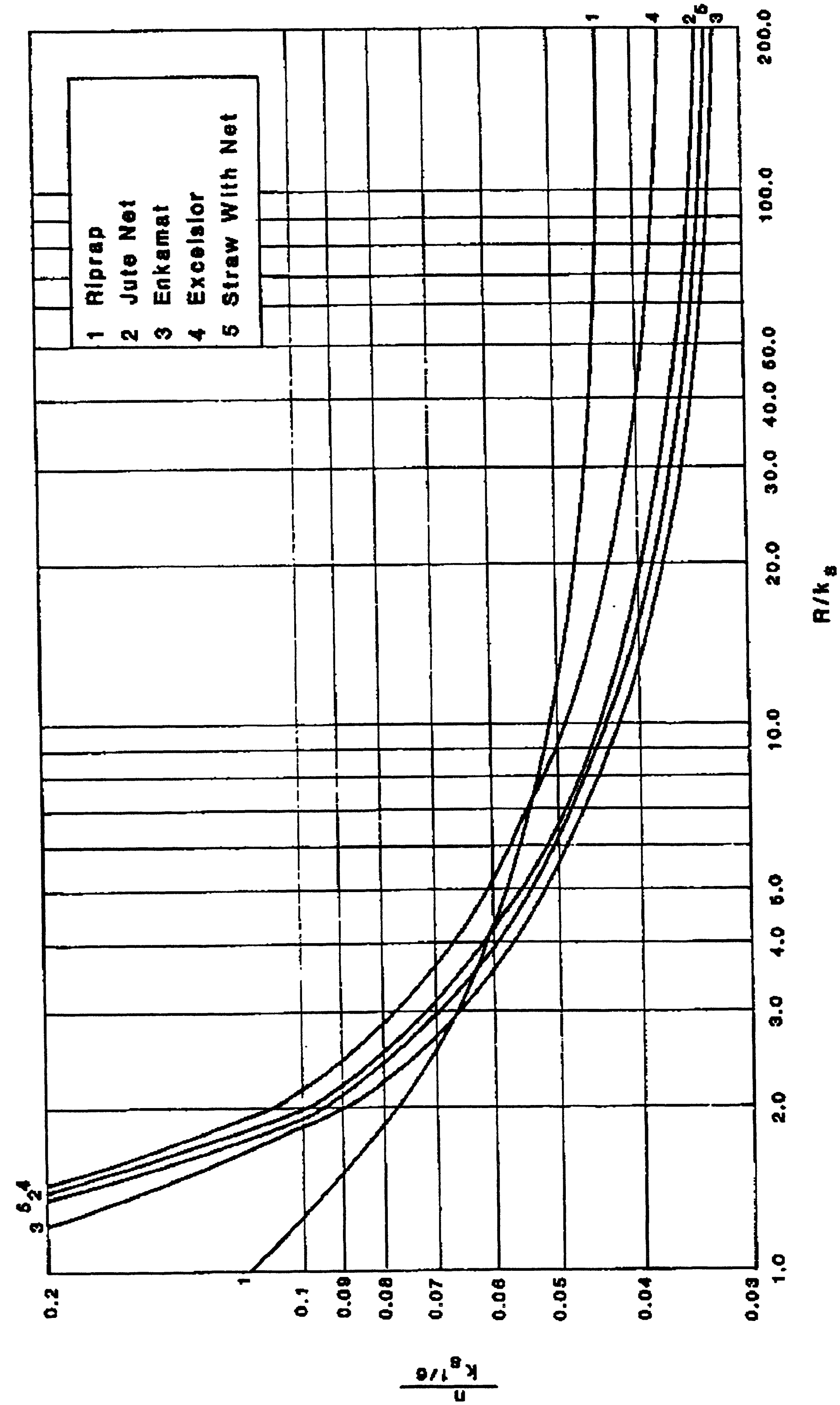


Figure 4.5 Manning's n Versus Relative Roughness for Selected Lining Types
Source: HEC-15 (FHWA, 1988)

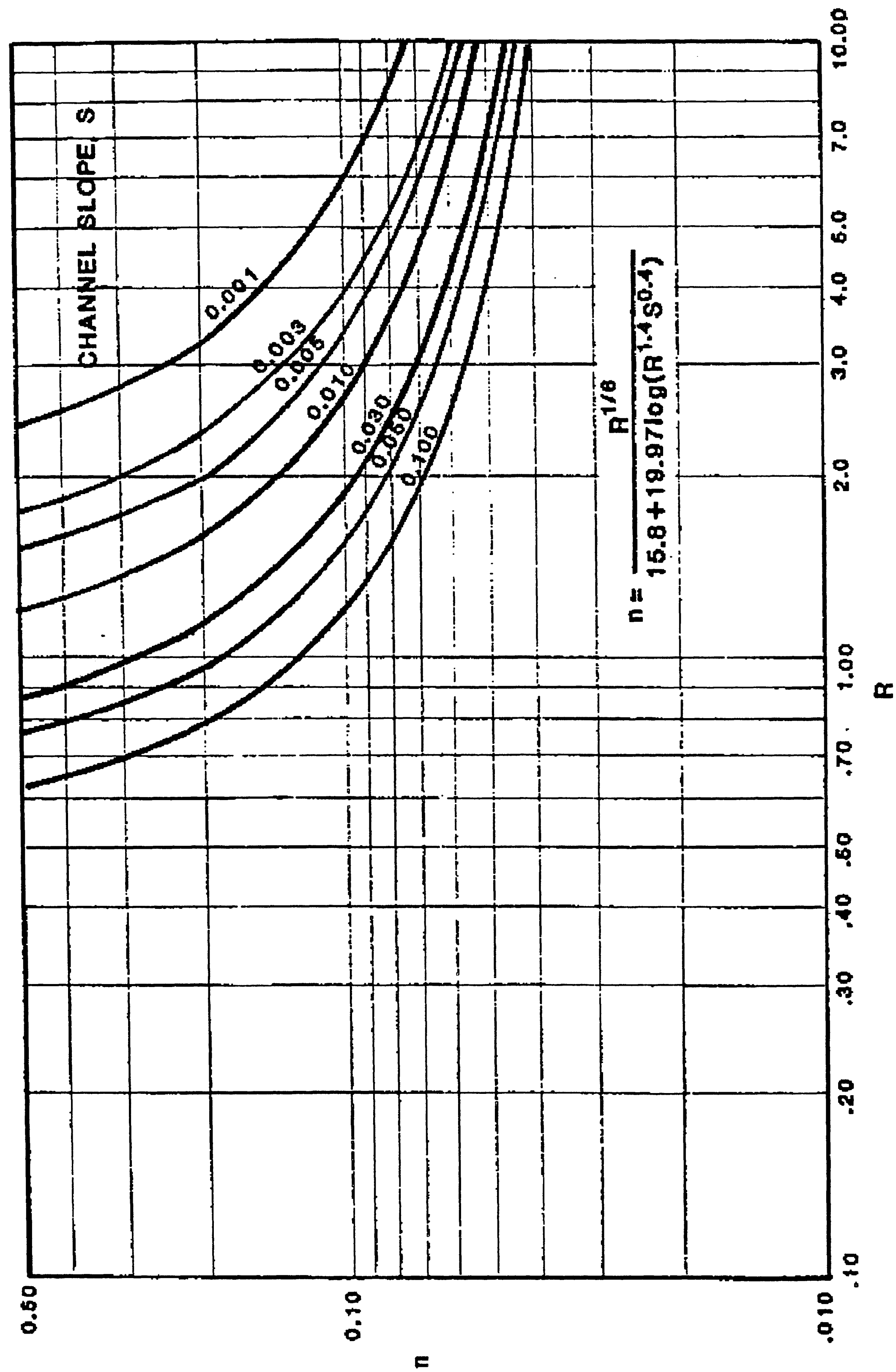


Figure 4.6 Manning's n Versus Hydraulic Radius, R, For Class A Vegetation
Source: HEC-15 (FHWA, 1988)

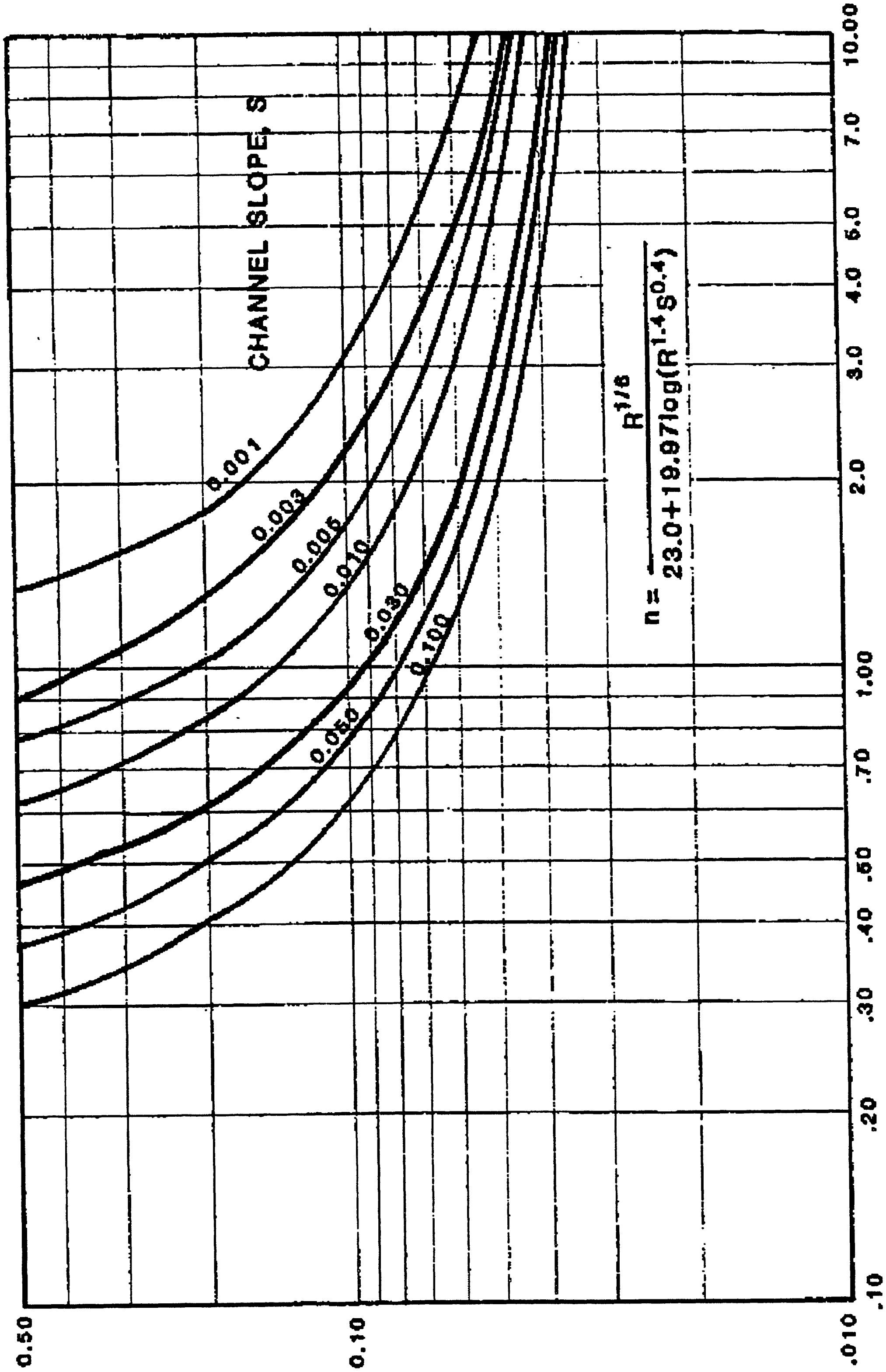


Figure 4.7 Manning's n Versus Hydraulic Radius, R for Class B Vegetation
Source: HEC-15 (FHWA, 1988)

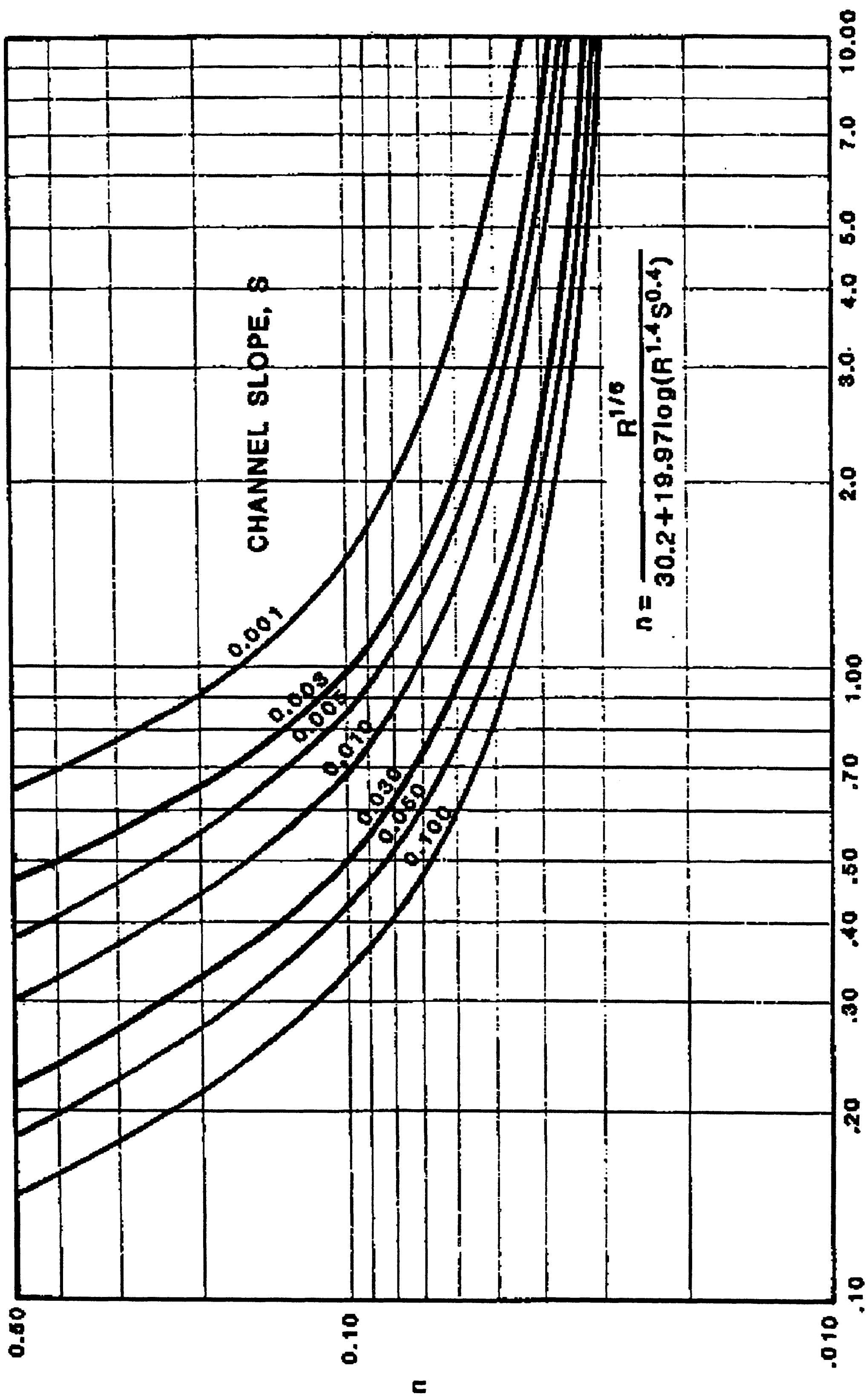


Figure 4.8 Manning's n Versus Radius, R, For Class C Vegetation
Sources: HEC-15 (FHWA, 1998)

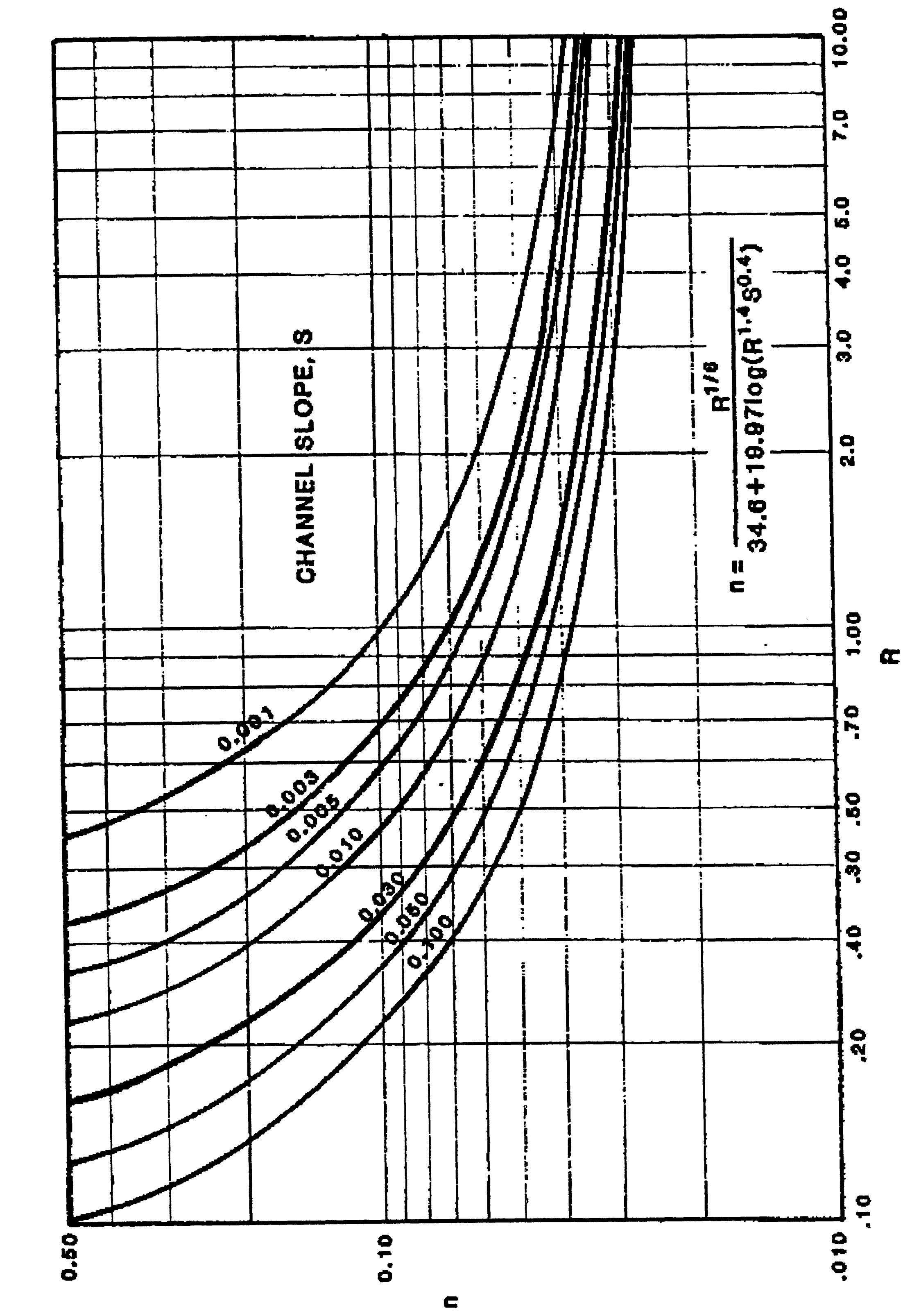


Figure 4.9 Manning's n Versus Hydraulic Radius, R, For Class D Vegetation
Source: HEC-15 (FHWA, 1988)

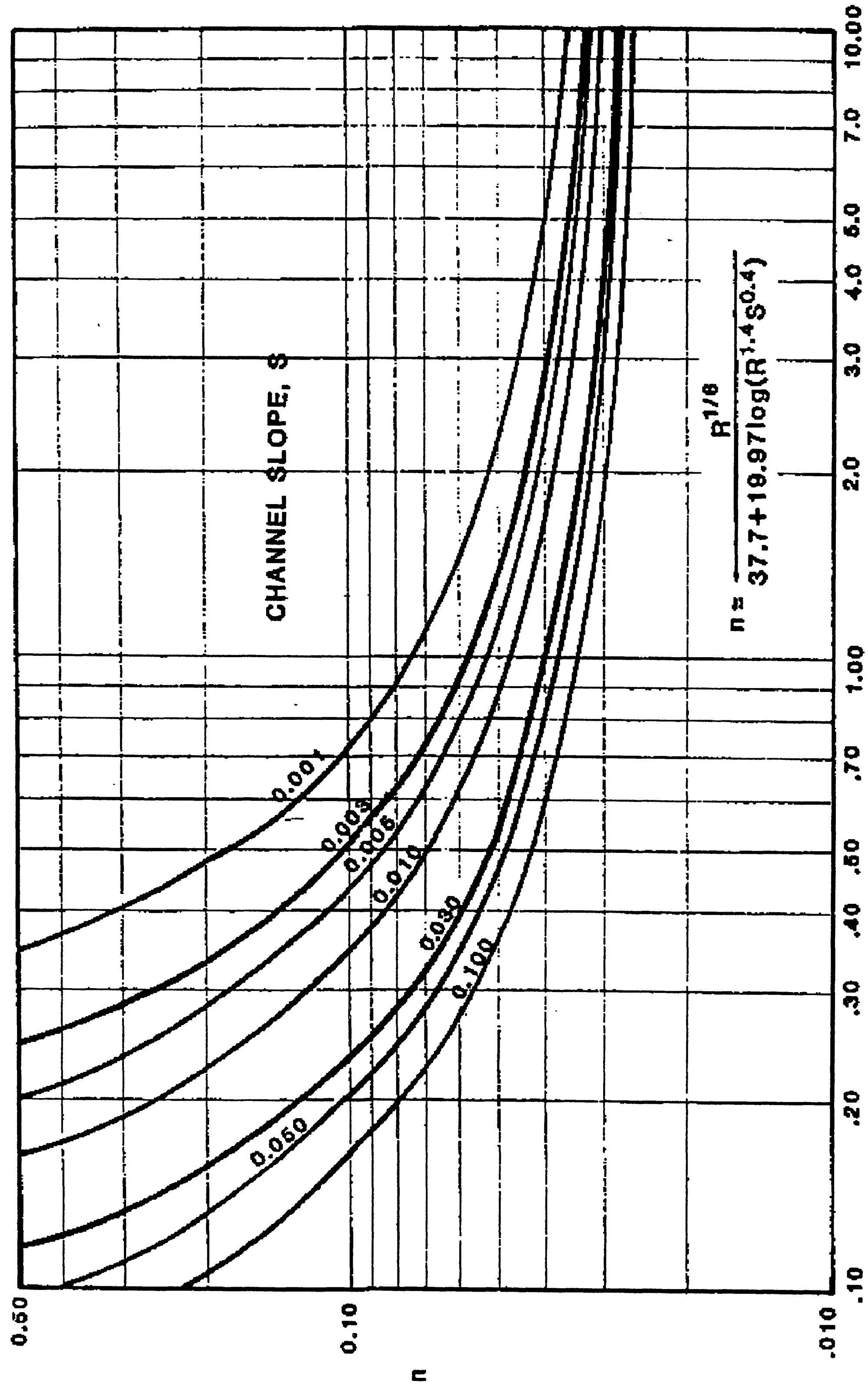


Figure 4.10 Manning's n Versus Hydraulic Radius, R, For Class E Vegetation
Source: HEC-15 (FHWA, 1988)

4.5 STREAM MORPHOLOGY

Most streams in Minnesota that highways cross or encroach upon are alluvial; that is, the streams are formed in materials that have been and can be transported by the stream. In alluvial stream systems, it is the rule rather than the exception that banks will erode; sediments will be deposited; and floodplains, islands and side channels will undergo modification with time. Alluvial channels continually change position and shape as a consequence of hydraulic forces exerted on the bed and banks. These changes may be gradual or rapid and may be the result of natural causes or human activities.

A study of the plan and profile of a stream is very useful in understanding stream morphology. Plan view appearance of streams are varied and result from many interacting variables. The planform 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. Small changes in a variable can change the plan view and profile of a stream, adversely affecting a highway crossing or encroachment. This is particularly true for alluvial streams. Conversely, a highway crossing or encroachment can inadvertently change a variable which can adversely affect the stream.

The natural stream channel will assume a geomorphological form which will be compatible with the sediment load and discharge history which it has experienced over time. To the extent that a highway structure disturbs this delicate balance by encroaching on the natural channel, the consequences of flooding, erosion, and deposition can be significant and widespread. The hydraulic analysis of a proposed highway structure should include a consideration of the extent of these consequences.

4.5.1 Levels of Assessment

The analysis and design of a stream channel will usually require an assessment of the existing channel and the potential for problems as a result of the proposed action. The detail of studies necessary should be commensurate with the risk associated with the action and with the environmental sensitivity of the stream. Observation is the best means of identifying potential locations for channel bank erosion and subsequent channel stabilization. Analytical methods for the evaluation of channel stability can be classified as either hydraulic or geomorphic, and it is important to recognize that these analytical tools should only be used to substantiate the erosion potential indicated through observation. A brief description of the three levels of assessment are as follows:

Level 1 Qualitative assessment involving the application of geomorphic concepts to identify potential problems and alternative solutions. Data needed may include historic information, current site conditions, aerial photographs, old maps and survey notes, bridge design files, maintenance records, and interviews with long-time residents.

Level 2 Quantitative analysis combined with a more detailed qualitative assessment of geomorphic factors. Generally includes water surface profile and scour calculations. This level of analysis will be adequate for most locations if the problems are resolved and relationships between different factors affecting stability are adequately explained. Data needed will include Level 1 data in addition to the information needed to establish the hydrology and hydraulics of the stream.

Level 3 Complex quantitative analysis based on detailed mathematical modeling and possibly physical hydraulic modeling. Necessary only for high risk locations, extraordinarily complex problems, and possibly after the fact analysis where losses and liability costs are high. This level of analysis may require professionals experienced with mathematical modeling techniques for sediment routing and/or physical modeling. Data needed will require Level 1 and 2 data as well as field data on bed load and suspended load transport rates and properties of bed and bank materials such as size, shape, gradation, fall velocity, cohesion, density, and angle of repose.

4.5.2 Factors That Affect Stream Stability

Factors that affect stream stability and, potentially, bridge and highway stability at stream crossings, can be classified as geomorphic factors and hydraulic factors.

Hydraulic Factors

- magnitude, frequency and duration of floods
- bed configuration
- resistance to flow
- water surface profiles

Geomorphic Factors

Figure 4.12 depicts examples of the various geomorphic factors.

- stream size
- valley setting
- natural levees
- sinuosity
- width variability
- bar development
- flow variability
- floodplains
- apparent incision
- channel boundaries
- degree of braiding
- degree of anabranching

Figure 4.11 depicts the changes in channel classification and relative stability to hydraulic factors. Rapid and unexpected changes may occur in streams in response to human activities in the watershed such as alteration of vegetative cover. Changes in perviousness can alter the hydrology of a stream, sediment yield and channel geometry. Channelization, stream channel straightening, stream levees and dikes, bridges and culverts, reservoirs and changes in land use can have major effects on stream flow, sediment transport and channel geometry and location. Knowing that these activities can influence stream stability can help the designer anticipate some of the problems that can occur.

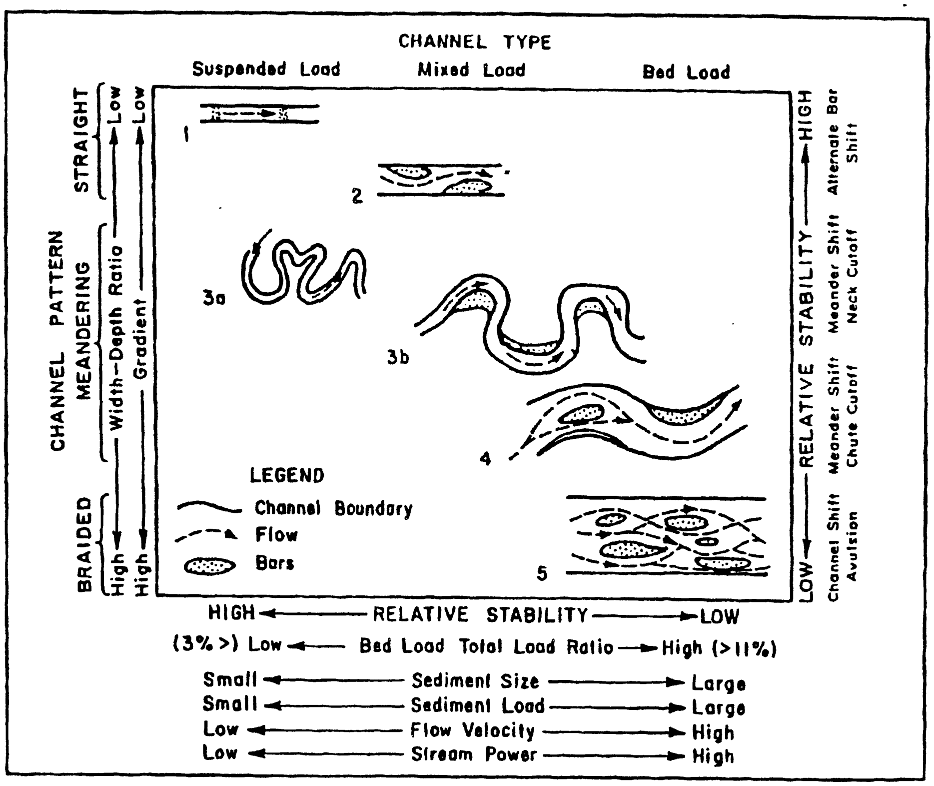


Figure 4.11 Channel Classification And Relative Stability As Hydraulic Factors Are Varied
Source: after Shen et. al., 1981

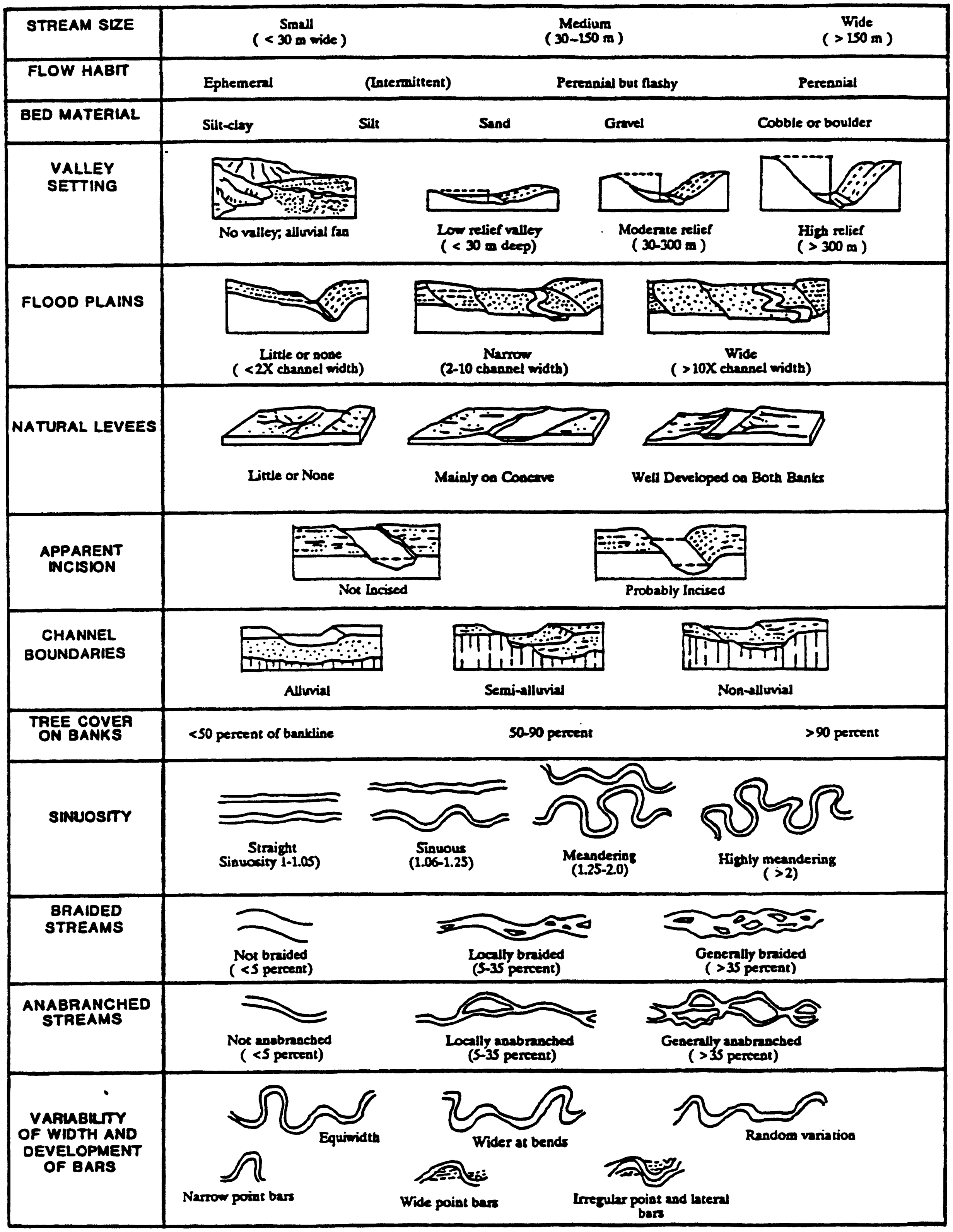


Figure 4.12 Geomorphic Factors That Affect Stream Stability
Source: adapted from Brice and Blodgett, 1978

Natural disturbances such as floods, drought, earthquakes, landslides, volcanoes and forest fires can also cause large changes in sediment load and major changes in the stream channel. Although difficult to plan for such disturbances, it is important to recognize that when natural disturbances do occur, it is likely that changes will also occur to the stream channel.

4.5.3 Stream Response to Change

The major complicating factors in river mechanics are: the large number of interrelated variables that can simultaneously respond to natural or imposed changes in a stream system; and the continual evolution of stream channel patterns, channel geometry, bars, and forms of bed roughness with changing water and sediment discharge. In order to better understand the responses of a stream to the actions of human's and nature, a few simple hydraulic and geomorphic concepts are presented herein.

The dependence of stream form on slope, which may be imposed independently of other stream characteristics, is illustrated schematically in Figure 4.13. Any natural or artificial change which alters channel slope can result in modifications to the existing stream pattern. For example, a cutoff of a meander loop decreases channel sinuosity and increases channel slope. Referring to

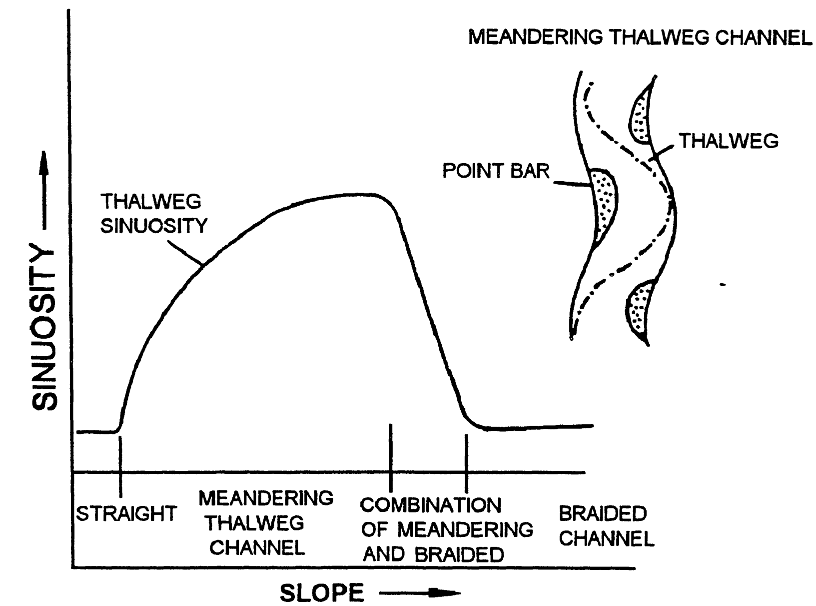


Figure 4.13 Sinuosity Vs Slope With Constant Discharge
Source: after Richardson et. al., 1990

Figure 4.13, this shift in the plotting position to the right could result in a shift from a relatively tranquil, meandering pattern toward a braided pattern that varies rapidly with time, has high velocities, is subdivided by sandbars, and carries relatively large quantities of sediment. Conversely, it is possible that a slight decrease in slope could change an unstable braided stream into a meandering one.

The different channel dimensions, shapes, and patterns associated with different quantities of discharge and amounts of sediment load indicate that as these independent variables change, major adjustments of channel morphology can be anticipated. Further, a change in hydrology may cause changes in stream sinuosity, meander wave length, and channel width and depth. A long period of channel instability with considerable bank erosion and lateral shifting of the channel may be required for the stream to compensate for the hydrologic change.

Figure 4.14 illustrates the dependence of river form on channel slope and discharge. It shows that when $SQ^{1/4} \leq .0017$ in a sandbed channel, the stream will meander. Similarly, when $SQ^{1/4} \geq .010$, the stream is braided. In these equations, S is the channel slope in feet per foot and Q is the mean discharge in cfs. The zone between the lines defining braided streams and meandering streams in Figure 4.14 is the transitional range, i.e., the range in which a stream can change readily from one stream form to the other.

Many United States rivers plot in this zone between the limiting curves defining meandering and braided streams. If a stream is meandering but its discharge and slope border on a boundary of the transitional zone, a relatively small increase in channel slope may cause it to change, in time, to a transitional or braided stream.

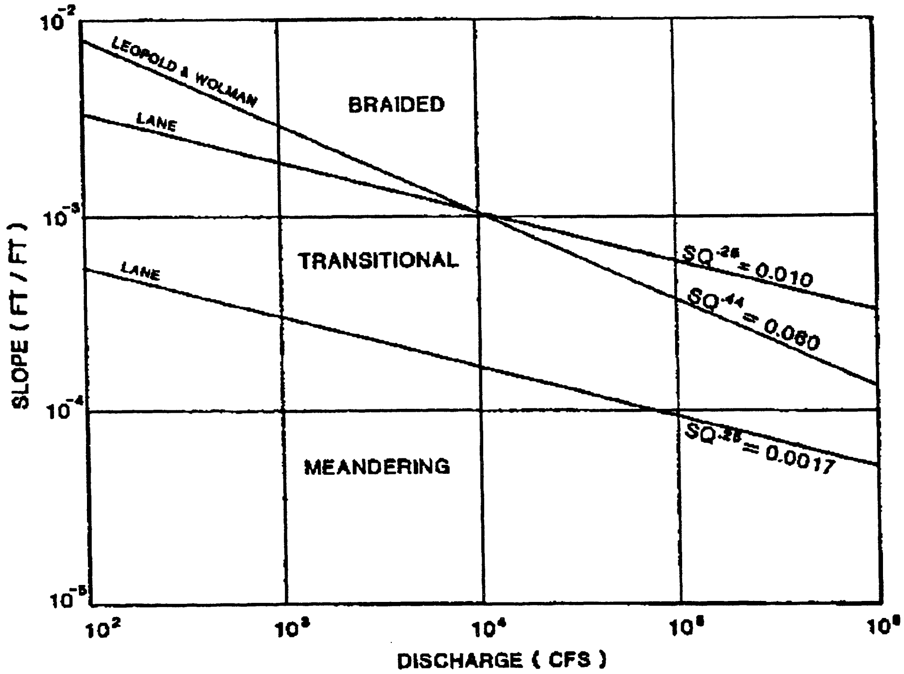


Figure 4.14 Slope-Discharge For Braiding or Meandering Bed Streams
Source: After, lane, 1957

4.5.4 Countermeasures

A countermeasure is defined as a measure incorporated into a highway crossing of a stream to control, inhibit, change, delay, or minimize stream and bridge stability problems. They may be installed at the time of highway construction or retrofitted to resolve stability problems at existing crossings. Retrofitting is good economics and good engineering practice in many locations because the magnitude, location, and nature of potential stability problems are not always discernible at the design stage, and indeed, may take a period of several years to develop. The selection of an appropriate countermeasure for a specific bank erosion problem is dependent on factors such as the erosion mechanism, stream characteristics, construction and maintenance requirements, potential for vandalism, and costs. Below is a brief discussion of possible countermeasures for some common river stability problems. The reader is encouraged to consult with the references listed at the end of this chapter for detailed information on the design and construction of the countermeasures.

<u>Meander Migration</u>
The best countermeasure against meander migration is a crossing location on a relatively straight reach of stream between bends. Other countermeasures include the protection of an existing bank line, the establishment of a new flowline or alignment, and the control and constriction of channel flow. Countermeasures identified for bank stabilization and bend control are bank revetments, spurs, retardance structures, longitudinal dikes, vane dikes, bulkheads, and channel relocations. Countermeasures may be used individually or a combination of two or more countermeasures may be used to combat meander migration at a site. (Highways in the River Environment, FHWA, 1990; and HEC-20, FHWA, 1991).

<u>Channel Braiding</u>
Countermeasures used at braided streams are usually intended to confine the multiple channels to one channel. This tends to increase sediment transport capacity in the principal channel and encourage deposition in secondary channels. The measures usually consist of dikes constructed from the limits of the multiple channels to the channel over which the bridge is constructed. Spur dikes at bridge ends used in combination with revetment on highway fill slopes, riprap on highway fill slopes only, and spurs arranged in the stream channels to constrict flow to one channel have also been used successfully.

<u>Degradation</u>
Degradation in streams can cause the loss of bridge piers in stream channels, and piers and abutments in caving banks. A check dam, which is a low dam or weir constructed across a channel, is one of the most successful techniques for halting degradation on small to medium streams. Longitudinal stone dikes placed at the toe of channel banks can be effective counter measures for bank caving in degrading streams. Precautions to prevent outflanking, such as tiebacks to the banks, may be necessary where installations are limited to the vicinity of the highway stream crossing. In general, channel lining alone is not a successful countermeasure against degradation problems (HEC-20, FHWA, 1991).

<u>Aggradation</u>
Current measures in use to alleviate aggradation problems at highways include channelization, bridge modification, continued maintenance, or combinations of these. Channelization may include excavating and cleaning channels, constructing cutoffs to increase the local slope, constructing flow control structures to reduce and control the local channel width, and constructing relief channels to improve flow capacity at the crossing. Except for relief channels, these measures are intended to increase the sediment transport capacity of the channel, thus reducing or eliminating problems with aggradation. Another technique which shows promise is the submerged vane technique developed by the University of Iowa. The studies suggest that the submerged vane structure may be an effective, economic, low-maintenance, and environmentally acceptable sediment-control structure with a wide range of applications (HEC-20, FHWA, 1991; Odgaard, et. al, 1986).

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