CHAPTER 3
OPEN CHANNEL HYDRAULICS

Chapter 3
OPEN CHANNEL HYDRAULICS

Synopsis

A consideration of open channel hydraulics is an integral part of projects in which artificial channels and improvements to natural channels are a primary concern. This chapter emphasizes procedures for performing uniform flow calculations that aid in the selection or evaluation of appropriate channel linings, depths, and grades. Allowable velocities are provided, along with procedures for evaluating channel capacity using Manning's Equation. For most artificial channels, the most desirable lining is vegetation with temporary measures to provide short-term erosion resistance during construction. If vegetation is infeasible, a flexible lining of riprap reinforced with a soil bioengineering technique is generally better than rigid paving for preventing erosion.

3.1 Linings

The three main classifications of open channel linings are vegetative, flexible, and rigid. Vegetative linings include grass with mulch, sod, and lapped sod. Rock riprap is a flexible lining. There are a number of soil bioengineering techniques that are a blend of vegetative and flexible lining techniques. For this discussion they will be presented with the flexible linings. Rigid linings are generally concrete.

3.1.1 Vegetation

Vegetation is the most desirable lining for an artificial channel. It stabilizes the channel body, consolidates the soil mass of the bed, checks erosion on the channel surface, and controls the movement of soil particles along the channel bottom. Conditions under which vegetation may not be acceptable, however, include but are not limited to:

1. Flow conditions in excess of the maximum shear stress for bare soils (see Section 3.2.4)

2. Standing or continuous flowing water

3. Lack of the regular maintenance necessary to prevent domination by taller vegetation

4. Lack of nutrients and inadequate topsoil

5. Excessive shade (under a small bridge or large culvert)

Proper seeding, mulching, and soil preparation are required during construction to assure establishment of a healthy growth of grass. Soil testing should be performed and the results evaluated by an agronomist to determine soil treatment requirements for pH, nitrogen, phosphorus, potassium, and other factors. In many cases, temporary erosion control measures are required to provide time for the seeding to establish a viable vegetative lining.

Sodding should be staggered, to avoid seams in the direction of flow. Lapped or shingle sod should be staggered and overlapped by approximately 25 percent. Staked sod is usually only appropriate for use on steeper slopes to prevent sliding. Additional information on vegitation is presented in Volume 4 Best Management Practices fact sheet PESC-01: Permanent Grasses, Vines and other Vegetation. This fact sheet should be consulted for additional information on applying and design criteria for this practice.

3.1.2 Flexible

Rock riprap including rubble is the most common (while not the most preferred) type of flexible lining. It presents a rough surface that can dissipate energy and mitigate increases in erosive velocity. These linings are usually less expensive than rigid linings and have self-healing qualities that reduce maintenance.

They typically require use of filter fabric and allow the infiltration and exfiltration of water. The use of flexible lining may be restricted where right-of-way is limited, since the higher roughness values create larger cross sections. Additional information on riprap is presented in Volume 4 Best Management Practices fact sheet TCP-20: Riprap.

A "soft" blend of vegetation and flexible lining are applied in geotextiles reinforcement and "bioengineering" techniques. This is presented in more detail in Volume 4 – Best Management Practices fact sheets PESC-01: Permanent Grasses, Vines and other Vegetation, TCP-10 and PESC-02: Geotextiles, TCP-19: Bank Stabilization, and PESC-04: Soil Bioengineering and Bank Stabilization. These fact sheets should be consulted for additional information on applying and design criteria for these practices.

3.1.3 Rigid

Rigid linings are generally constructed of concrete and used where smoothness offers a higher capacity for a given cross-sectional area. They should only be applied when vegetative, flexible and "softer" techniques like soil bioengineering techniques have been thoroughly considered. Higher velocities, however, create the potential for scour at channel lining transitions. A rigid lining can be destroyed by flow undercutting the lining, channel headcutting, or the buildup of hydrostatic pressure behind the rigid surfaces. When properly designed, rigid linings may be appropriate where the channel width is restricted. Filter fabric may be required to prevent soil loss through pavement cracks. Under continuous base conditions when a vegetative lining alone would be inappropriate, a small concrete pilot channel could be used to handle the continuous low flows. Vegetation could then be maintained for handling larger flows. Additional information on channel linings are presented in Volume 4 Best Management Practices fact sheet TCP-21 and PESC-08: Channel Linings. This fact sheet should be consulted for additional information on applying and design criteria for this practice.

3.2 Design Criteria

3.2.1 General

In general, the following criteria shall be considered for open channels:

1. Channels with bottom widths greater than 10 feet shall be designed with a minimum bottom cross slope of 12 to 1.

2. Low flow sections shall be considered in the design of channels with large cross sections.

3. Channels with design flows greater than 100 cfs will be considered to have large cross sections.

4. Channel side slopes shall be stable throughout the entire length and shall consider the channel material. A maximum of 3:1 is allowed for vegetation and 2:1 for riprap, unless otherwise justified by calculations.

5. Superelevation of the water surface at horizontal curves shall be accounted for by increased free-board (see Section 3.3.5).

6. Parabolic cross sections are preferred while an alternative trapezoidal channel may be accepted. Triangular shapes should be avoided.

3.2.2 Channel Transitions

The following criteria shall be considered at channel transitions:

1. Transition to channel sections shall be smooth and gradual.

2. A straight line connecting flow lines at the two ends of the transition shall not make an angle greater than 12.5 degrees with the axis of the main channel.

3. Transition section length shall be considerably greater than the transition width.

4. Energy losses in transitions shall be accounted for as part of water surface profile calculations (see Section 3.4).

3.2.3 Return Period

Minor drainage systems shall be sized to handle a 10-year design storm; major systems shall be sized to handle a 100-year design storm. However, if the 10-year design flow exceeds 100 cfs, then the system shall be capable of passing the 100-year design flow within the drainage easement. Definitions of minor and major systems are provided in Volume 1 along with additional details on design policy.

Sediment transport requirements must be considered for conditions of flow below the design frequency. A low flow channel component within a larger channel can reduce maintenance by improving sediment transport in the channel.

Temporary open channel facilities used during construction should be designed to handle flood flows commensurate with risks. The recommended minimum frequency for temporary facilities and temporary lining of permanent facilities is 20 percent of the standard frequency for permanent facilities.

3.2.4 Velocity Limitations

The final design of artificial open channels should be consistent with the velocity limitations for the selected channel lining. Maximum velocity values for selected lining categories are presented in Table 3-1. Seeding and mulch should only be used when the design value does not exceed the allowable value for bare soil. Velocity limitations for vegetative linings are reported in Table 3-2. Vegetative lining calculations are presented in Section 3.3.5 and Volume 4: PESC-02, and rip-rap procedures are presented in Section 3.3.6 and Volume 4: TCP-20.

3.2.5 Manning's n Values

General

The following general factors should be considered when selecting the value of Manning's n:

1. As a general rule, retardance is increased when conditions tend to induce turbulence and reduced when they tend to minimize turbulence.

2. The physical roughness of the bottom and sides of the channel should be taken into account. Fine particle soils on smooth, uniform surfaces result in relatively low values of n. Coarse materials, such as gravel or boulders, and pronounced surface irregularity cause higher values of n.

3. The value of n will be affected by the height, density, and type of vegetation. Consideration should be given to density and distribution of the vegetation along the reach and the wetted perimeter, the degree to which the vegetation occupies or blocks the cross section of flow at different depths, and the degree to which the vegetation may be bent or "shingled" by flows of different depths. The n value will increase in the spring and summer, as vegetation grows and foliage develops, and diminish in the fall, as the dormant season approaches.

4. Channel shape variations, such as abrupt changes in channel cross sections or alternating small and large cross sections, will require somewhat larger n values than normal. These variations in channel cross section become particularly important if they cause the flow to meander from side to side.

5. A significant increase in the value of n is possible if severe meandering occurs in the alignment of a channel. Meandering becomes particularly important when frequent changes in the direction of curvature occur with relatively small radii of curvature.

6. Active channel erosion or sedimentation will tend to increase the value of n, since these processes may cause variations in the shape of a channel. The potential for future erosion or sedimentation in the channel should also be considered.

7. Obstructions such as log jams or deposits of debris will increase the value of n. The level of this increase will depend on the number, type, and size of obstructions.

8. To be conservative, it is better to use a higher resistance for capacity calculations and a lower resistance for stability calculations.

9. Proper assessment of natural channel n values requires field observations and experience. Special attention is required in the field to identify flood plain vegetation and evaluate possible variations in roughness with depth of flow.

All of these factors should be studied and evaluated with respect to type of channel, degree of maintenance, seasonal requirements, and other considerations as a basis for making a determination of an appropriate design n value. The probable condition of the channel when the design event is anticipated should be considered. Values representative of a freshly constructed channel are rarely appropriate as a basis for design capacity calculations.

Artificial Channels

Recommended Manning's n values for artificial channels with rigid, unlined, temporary, and riprap linings are given in Table 3-3. Recommended values for vegetative linings should be determined using Figure 3-1, which provides a graphical relationship between Manning's n values and the product of velocity and hydraulic radius for several vegetative retardance classifications (see Table 3-4). Figure 3-1 is used iteratively as described in Section 3.3.5.

Natural Channels

For natural stream channels, Manning's n value may be estimated using Cowan's Equation (Cowan, 1956), presented below:

$$n = (n_0 + n_1 + n_2 + n_3 + n_4)m_5 \qquad (3\text{-}1)$$

where:

n = Manning's roughness coefficient for a natural or excavated channel

n_0 = Coefficient associated with channel lining material

n_1 = Coefficient associated with the degree of channel irregularity

n_2 = Coefficient associated with variations of the channel cross section

n_3 = Coefficient associated with the relative effect of channel obstructions

n_4 = Coefficient associated with channel vegetation

m_5 = Coefficient associated with the degree of channel meandering

Coefficients for Equation 3-1 can be determined using information in Table 3-5. Additional information is presented in FHWA-TS-84-204 (USDOT, FHWA, 1984), including procedures for determining Manning's n values for flood plains.

3.2.6 Channel Stability Considerations

A stream channel is a dynamic feature that assumes a particular shape or size in response to the flow forces acting on it and the constraints imposed by geology, vegetation, and other factors, and tends to fluctuate around this equilibrium shape. Channel response to changes may be slow or rapid and may result from natural occurrences (e.g., natural erosion, a fallen tree) or human activity (e.g., a bridge abutment). When a channel is modified locally, the change frequently causes a response and modification of the channel characteristics both up and downstream. Additional detail about channel response to urbanization and river geomorphology is provided in FHWA-HI-90-016 (USDOT, FHWA, 1990) including discussion of channel sinuosity.

Quantification of the effects of human activity on a channel can be difficult. However, a qualitative assessment of possible impacts and appropriate preventive measures is required for all channel modifications and can be aided through use of the following proportionality:

$$QS \; \% \; Q_S \, d_{50} \qquad (3\text{-}2)$$

where:

Q = Typical or dominant discharge, in cfs

S = Energy slope, in feet/foot

Q_s = Sediment transport, in pounds/second

d_{50}= Sediment mean particle diameter, in mm

The two sides of this proportionality will tend to adjust to maintain equilibrium. For example, if a bend in a stream is straightened or a rough channel is paved, the energy slope is steepened. If S increases, either Q_s or d_{50} or both must also increase. The result is generally advancing scour upstream, higher velocities through the reach with associated channel deterioration, scour just below the reach, and eventual deposition downstream. If the channel is paved, excess sediment transport capacity is transferred just downstream, causing greater scour. Volume 3 contains further discussion on this subject.

In general, when the impacts of proposed modifications on channel stability are being assessed, the following considerations and actions apply:

1. A study should be made of the stream to be modified and should include historical information, evidence of other instability (i.e., bank caving, channel movement), results of other developments, and aerial photos showing alignment changes, either natural or manmade. The composition and erodibility of bed and bank material should be determined.

2. Backwater calculations (see Section 3.4) should be performed through the reach for a range of flows, including bankfull flow. Existing worst-case velocities and slopes should be calculated and related to the existing channel configuration to determine maximum velocities and shear stress values for actual conditions.

3. A channel modification scheme that minimizes interference with the channel is preferred. Equation 3-2 can be used to evaluate the likely channel response to proposed changes.

4. Channel modifications should be sized to match existing sizes and shape. A narrow channel will deteriorate and a wide channel may collect silt. Floodways or high flow

channels should be used to carry extreme events rather than over-sizing a channel. Backwater should be recalculated through the modified reach.

5. Protection should be provided where needed, from the downstream through the upstream extent of modification effects (i.e., effects of modification are often felt beyond the project limits). For flow with significant overbank components, a central section velocity must be used instead of the mean flow velocity. Protection should be sized for the design event and design smooth transitions. If velocities are too high, grade control structures or check dams should be considered.

3.3 Uniform Flow Calculations

Manning's Equation, presented in three forms below, is recommended for evaluating uniform flow conditions in open channels:

$$v = \frac{1.49}{n} R^{2/3} S^{1/2} \qquad (3\text{-}3)$$

$$Q = \frac{1.49}{n} A R^{2/3} S^{1/2} \qquad (3\text{-}4)$$

$$S = \left(\frac{Qn}{1.49 A R^{2/3}} \right)^2 \qquad (3\text{-}5)$$

where:

v = Average channel velocity, in feet/second

Q = Discharge rate for design conditions, in cfs

n = Manning's roughness coefficient (see Section 3.2.5)

A = Cross-sectional area, in square feet

R = Hydraulic radius A/P, in feet

P = Wetted perimeter, in feet

S = Slope of the energy grade line, in feet/foot

For prismatic channels, in the absence of backwater conditions, the slope of the energy grade line and channel bottom can be assumed to be the same.

3.3.1 Geometric Relationships

Mathematical expressions for calculating the area, wetted perimeter, hydraulic radius, and channel top width for selected open channel cross sections are presented in Figure 3-2. These cross sections include trapezoidal, rectangular, triangular, parabolic, and circular shapes. Geometric properties of trapezoidal channels also can be evaluated using the chart presented in Figure 3-3.

Irregular channel cross sections (i.e., those with a narrow deep main channel and a wide shallow overbank channel) must be subdivided into segments so that the flow can be computed separately for the main channel and overbank portions. This same process of subdivision may be used when different parts of the channel cross section have different roughness coefficients. When computing the hydraulic radius of the subsections, the water depth common to the two adjacent subsections is not counted as wetted perimeter (USGS, 1976a,b).

3.3.2 Direct Solutions

When the hydraulic radius, cross-sectional area, and roughness coefficient and slope are known, discharge can be calculated directly from Equation 3-4. The slope can be calculated using Equation 3-5 when the discharge, roughness coefficient, area, and hydraulic radius are known. Nomographs for obtaining direct solutions to Manning's Equation are presented in Figures 3-4 and 3-5. Figure 3-4 provides a general solution for the velocity form of Manning's Equation, while Figure 3-5 provides a solution of Manning's Equation for trapezoidal channels.

<u>General</u>

The following steps are used for the general solution nomograph in Figure 3-4:

1. Determine open channel data, including slope in feet/foot, hydraulic radius in feet, and Manning's n value.

2. Connect a line between the Manning's n scale and slope scale and note the point of intersection on the turning line.

3. Connect a line from the hydraulic radius to the point of intersection obtained in Step 2.

4. Extend the line from Step 3 to the velocity scale to obtain the velocity in feet/second.

Trapezoidal

The trapezoidal channel nomograph solution to Manning's Equation in Figure 3-5 can be used to find the depth of flow if the design discharge is known or the design discharge if the depth of flow is known.

1. Determine input data, including slope in feet/foot, Manning's n value, bottom width in feet, and side slope in feet/foot.

2. a. Given the design discharge, find the product of Q times n, connect a line from the slope scale to the Qn scale, and find the point of intersection on the turning line.

   a. Connect a line from the turning point from Step 2a to the b scale and find the intersection with the z = 0 scale.

   b. Project horizontally from the point located in Step 2b to the appropriate z value and find the value of d/b.

   c. Multiply the value of d/b obtained in Step 2c by the bottom width b to find the depth of uniform flow, d.

3. a. Given the depth of flow, find the ratio d divided by b and project a horizontal line from the d/b ratio at the appropriate side slope, z, to the z = 0 scale.

   b. Connect a line from the point located in Step 3a to the b scale and find the intersection with the turning line.

   c. Connect a line from the point located in Step 3b to the slope scale and find the intersection with the Qn scale.

   d. Divide the value of Qn obtained in Step 3c by the n value to find the design discharge, Q.

3.3.3 Trial and Error Solutions

A trial and error procedure for solving Manning's Equation is used to compute the normal depth of flow in a uniform channel when the channel shape, slope, roughness, and design discharge are known. For purposes of the trial and error process. Manning's Equation can be arranged as:

$$AR^{2/3} = \frac{Qn}{1.49S^{1/2}} \qquad (3\text{-}6)$$

where:

A = Cross-sectional area, in square feet

R = Hydraulic radius, in feet

Q = Discharge rate for design conditions, in cfs

n = Manning's roughness coefficient (see Section 3.2.5)

S = Slope of the energy grade line, in feet/foot

To determine the normal depth of flow in a channel by the trial and error process, trial values of depth are used to determine A, P, and R for the given channel cross section. Trial values of $AR^{2/3}$ are computed until the equality of Equation 3-6 is satisfied such that the design flow is conveyed for the slope and selected channel cross section.

Graphical procedures for simplifying trial and error solutions are presented in Figure 3-6 for trapezoidal channels, which is described below.

1. Determine input data, including design discharge, Q, Manning's n value, channel bottom width, b, channel slope, S, and channel side slope, z.

2. Calculate the trapezoidal conveyance factor using the equation:

$$K_T = \frac{Qn}{b^{8/3} S^{1/2}} \qquad (3\text{-}7)$$

where:

K_T = Trapezoidal open channel conveyance factor

Q = Discharge rate for design conditions, in cfs

n = Manning's roughness coefficient (see Section 3.2.5)

b = Bottom width, in feet

S = Slope of the energy grade line, in feet/foot

3. Enter the x-axis of Figure 3-6 with the value of K_T calculated in Step 2 and draw a line vertically to the curve corresponding to the appropriate z value from Step 1.

4. From the point of intersection obtained in Step 3, draw a horizontal line to the y-axis and read the value of the normal depth of flow over the bottom width, d/b.

5. Multiply the d/b value from Step 4 by b to obtain the normal depth of flow.

3.3.4 Critical Flow Calculations

Critical depth depends only on the discharge rate and channel geometry. The general equation for determining critical depth is expressed as:

$$\frac{Q^2}{g} = \frac{A^3}{T} \qquad (3\text{-}8)$$

where:

Q = Discharge rate for design conditions, in cfs

g = Acceleration due to gravity, 32.2 feet/second^2

A = Cross-sectional area, in square feet

T = Top width of water surface, in feet

A trial and error procedure is needed to solve Equation 3-8. Semi-empirical equations (as presented in Table 3-6) or section factors (as presented in Figure 3-2) can be used to simplify trial and error critical depth calculations. The following equation from Chow (1959) is used to determine critical depth with the critical flow section factor, Z:

$$Z = Q / \sqrt{g} \qquad (3\text{-}9)$$

where:

Z = Critical flow section factor

Q = Discharge rate for design conditions, in cfs

g = Acceleration due to gravity, 32.2 feet/second^2

The following guidelines are presented for evaluating critical flow conditions of open channel flow:

1. A normal depth of uniform flow within about 10 percent of critical depth is unstable and should be avoided in design, if possible.

2. If the velocity head is less than one-half the mean depth of flow, the flow is subcritical.

3. If the velocity head is equal to one-half the mean depth of flow, the flow is critical.

4. If the velocity head is greater than one-half the mean depth of flow, the flow is supercritical.

5. If an unstable critical depth cannot be avoided in design, the least favorable type of flow should be assumed for the design.

The Froude number, Fr, calculated by the following equation, is useful for evaluating the type of flow conditions in an open channel:

$$Fr = v/ (gA/T)^{0.5} \qquad (3\text{-}10)$$

where:

Fr = Froude number, dimensionless

v = Velocity of flow, in feet/second

g = Acceleration due to gravity, 32.2 feet/second^2

A = Cross-sectional area of flow, in square feet

T = Top width of flow, in feet

If Fr is greater than 1.0, flow is supercritical; if it is under 1.0, flow is subcritical. Fr is 1.0 for critical flow conditions.

3.3.5 Vegetative Design

A two-part procedure, adapted from Chow (1959) and presented below, is recommended for final design of temporary and vegetative channel linings. Part 1, the design stability component, involves determining channel dimensions for low vegetative retardance conditions, using Class D as defined in Table 3-4. Part 2, the design capacity component, involves determining the depth increase necessary to maintain capacity for higher vegetative retardance conditions, using Class C as defined in Table 3-4. If temporary lining is to be used during construction, vegetative retardance Class E should be used for the design stability calculations.

Temple et al. (1987) present an alternative procedure for designing grass-lined channels that is acceptable but not duplicated in the manual.

If the channel slope exceeds 10 percent, or a combination of channel linings will be used, additional procedures not presented below are required. References include HEC-15 (USDOT, FHWA, 1986) and HEC-14 (USDOT, FHWA, 1983).

Design Stability

1. Determine appropriate design variables, including discharge, Q, bottom slope, S, cross section parameters, and vegetation type.

2. Use Table 3-2 to assign a maximum velocity, v_m, based on vegetation type and slope range.

3. Assume a value of n and determine the corresponding value of vR from the n versus vR curves in Figure 3-1. Use retardance Class D for permanent vegetation and E for temporary construction.

4. Calculate the hydraulic radius using the equation:

$$R = \frac{(vR)}{V_m} \qquad (3\text{-}11)$$

where:

R = Hydraulic radius of flow, in feet

*v*R = Value obtained from Figure 3-1 in Step 2

V_m = Maximum velocity from Step 2

5. Use the following form of Manning's Equation to calculate the value of vR:

$$vR = \frac{1.49 R^{5/3} S^{1/2}}{n} \qquad (3\text{-}12)$$

where:

vR = Calculated value of vR product

R = Hydraulic radius value from Step 4, in feet

S = Channel bottom slope, in feet/foot

n = Manning's n value assumed in Step 3

6. Compare the vR product value obtained in Step 5 to the value obtained from Figure 3-1 for the assumed n value in Step 3. If the values are not reasonably close, return to Step 3 and repeat the calculations using a new assumed n value.

7. For trapezoidal channels, find the flow depth using Figures 3-5 or 3-6, as described in Section 3.3.3. The depth of flow for other channel shapes can be evaluated using the trial and error procedure in Section 3.3.3.

8. If bends are considered, calculate the length of downstream protection, L_p, for the bend using Figure 3-7. Provide additional protection, such as gravel or riprap in the bend and extending downstream for length, L_p.

Design Capacity

1. Assume a depth of flow greater than the value from Step 7 above and compute the waterway area and hydraulic radius (see Figure 3-2 for equations).

2. Divide the design flow rate, obtained using appropriate procedures from Chapter 2, by the waterway area from Step 1 to find the velocity.

3. Multiply the velocity from Step 2 by the hydraulic radius from Step 1 to find the value of vR.

4. Use Figure 3-1 to find a Manning's n value for retardance Class C based on the vR value from Step 3.

5. Use Manning's Equation (Equation 3-3) or Figure 3-4 to find the velocity using the hydraulic radius from Step 1, Manning's n value from Step 4, and appropriate bottom slope.

6. Compare the velocity values from Steps 2 and 5. If the values are not reasonably close, return to Step 1 and repeat the calculations.

7. Add an appropriate freeboard to the final depth from Step 6. Generally, 20 percent is adequate.

8. If bends are considered, calculate superelevation of the water surface profile at the bend using the equation:

$$\Delta d = \frac{v^2 T}{g R_c} \qquad (3\text{-}13)$$

where:

Δd = Superelevation of the water surface profile due to the bend, in feet

v = Average velocity from Step 6, in feet/second

T = Top width of flow, in feet

g = Acceleration due to gravity, 32.2 feet/second^2

R_c = Mean radius of the bend, in feet

Add freeboard consistent with the calculated Δd.

3.3.6 Riprap Design

Riprap should only be utilized when "green", "soft", geotextiles, and soil bioengineering techniques have been explored and thoroughly considered. Riprap may be used provided calculations are presented to MWS that illustrate that soil bioengineering or other techniques are not cost effective for the site or are not feasible.

The following procedure is based on results and analysis of laboratory and field data (Maynord, 1987; Reese, 1984; Reese, 1988). This procedure applies to riprap placement in both natural and prismatic channels and has the following assumptions and limitations:

1. Minimum riprap thickness equal to d_{100}.

2. The value of d_{85}/d_{15} less than 4.6.

3. Froude number less than 1.2.

4. Side slopes up to 2:1.

5. A safety factor of 1.2.

6. Maximum velocity less than 18 feet per second.

If significant turbulence is caused by boundary irregularities, such as installations near obstructions or structures, this procedure is not applicable.

1. Determine the average velocity in the main channel for the design condition. Use the higher value of velocity calculated both with and without riprap in place (this may require

iteration using procedures in Section 3.3.3). Manning's n values for riprap can be calculated from the equation:

$$n = 0.0395\,(d_{50})^{1/6} \qquad (3\text{-}14)$$

where:

n = Manning's roughness coefficient for stone riprap

d_{50} = Diameter of stone for which 50 percent, by weight, of the gradation is finer, in feet

2. If rock is to be placed at the outside of a bend, multiply the velocity determined in Step 1 by the bend correction coefficient, C_b, given in Figure 3-8 for either a natural or prismatic channel. This requires determining the channel top width, T, just upstream from the bend and the centerline bend radius, R_b.

3. If the specific weight of the stone varies from 165 pounds per cubic foot, multiply the velocity from Step 1 or 2 (as appropriate) by the specific weight correction coefficient, C_g, from Figure 3-9.

4. Determine the required minimum d_{30} value from Figure 3-10, which is based on the equation:

$$d_{30}/D = 0.193\,Fr^{2.5} \qquad (3\text{-}15)$$

where:

d_{30} = Diameter of stone for which 30 percent, by weight, of the gradation is finer, in feet

D = Depth of flow above stone, in feet

Fr = Froude number (see Equation 3-10), dimensionless

v = Mean velocity above the stone, in feet/second

g = Acceleration of gravity, 32.2 feet/second^2

5. Determine available riprap gradations. A well graded riprap is preferable to uniform size or gap graded. The diameter of the largest stone, d_{100}, should not be more than 1.5 times the d_{50} size. Blanket thickness should be greater than or equal to d_{100} except as noted below. Sufficient fines (below d_{15}) should be available to fill the voids in the larger rock sizes. The stone weight for a selected stone size can be calculated from the equation:

$$W = 0.5236\ \gamma_s\ d^3 \qquad (3\text{-}16)$$

where:

W = Stone weight, in pounds

d = Selected stone diameter, in feet

γ_s = Specific weight of stone, in pounds/cubic foot

Filter fabric or a filter stone layer should be used to prevent turbulence or groundwater seepage from removing bank material through the stone or to serve as a foundation for unconsolidated material. Layer thickness should be increased by 50 percent for underwater placement.

6. If d_{85}/d_{15} is between 2.0 and 2.3 and a smaller d_{30} size is desired, a thickness greater than d_{100} can be used to offset the smaller d_{30} size. Figure 3-11 can be used to make an approximate adjustment using the ratio of d_{30} sizes. Enter the y-axis with the ratio of the desired d_{30} size to the standard d_{30} size and find the thickness ratio increase on the x-axis. Other minor gradation deficiencies may be compensated for by increasing the stone blanket thickness.

7. Perform preliminary design, ensuring that adequate transition is provided to natural materials both up and downstream to avoid flanking and that toe protection is provided to avoid riprap undermining.

3.3.7 Approximate Flood Limits

For small streams and tributaries not included in the basin studies, analysis may be required to identify the 100-year flood elevation and to evaluate flood plain encroachment as required by Volume 1. For such cases, when the design engineer can demonstrate that a complete backwater analysis is unwarranted, approximate methods may be used.

A generally accepted method for approximating the 100-year flood elevation is outlined as follows:

1. Divide the stream or tributary into reaches that may be approximated using average slopes, cross sections, and roughness coefficients for each reach.

2. Estimate the 100-year peak discharge for each reach using an appropriate hydrologic method from Chapter 2.

3. Compute normal depth for uniform flow in each reach using Manning's Equation for the reach characteristics from Step 1 and peak discharge from Step 2.

4. Use the normal depths computed in Step 3 to approximate the 100-year flood elevation in each reach. The 100-year flood elevation is then used to delineate the flood plain.

This approximate method is based on several assumptions, including, but not limited to, the following:

1. A channel reach is accurately approximated by average characteristics throughout its length.

2. The cross-sectional geometry, including area, wetted perimeter, and hydraulic radius, of a reach may be approximated using typical geometric properties that can be used in Manning's Equation to solve for normal depth.

3. Uniform flow can be established and backwater effects are negligible between reaches.

4. Expansion and contraction effects are negligible.

As indicated, the approximate method is based on a number of restrictive assumptions that may limit the accuracy of the approximation and applicability of the method. The engineer is responsible for appropriate application of this method.

After the 100-year flood elevation and flood plain are established, floodway setback limits may be approximated by requiring conveyance of the encroached section, including any allowable flood elevation increases specified in Volume 1, to equal the conveyance of the non-encroached section. From Manning's Equation, the conveyance is given as follows:

$$K = \frac{1.49}{n} AR^{2/3} \qquad (3\text{-}17)$$

$$Q = CS^{1/2} \qquad (3\text{-}18)$$

where:

K = Channel conveyance

A = Cross-sectional area, in square feet

R = Hydraulic radius A/P, in feet

P = Wetted perimeter, in feet

Q = Discharge, in cfs

S = Slope of the energy grade line, in feet/foot

The following procedure may be used to approximate setback limits for a stream:

1. Divide the stream cross section into segments for which the geometric properties may be easily solved and estimate an n value for each segment.

2. Compute the area, hydraulic radius, and conveyance (Equation 3-17) of each segment for both the encroached and non-encroached segment. Include the allowable flood elevation increase from Volume 1 in the computations for the encroached segments.

3. Sum the conveyance for each cross-sectional segment to obtain the total conveyance for both the encroached and non-encroached conditions.

4. Set the total conveyance of the encroached cross section equal to the total conveyance of the non-encroached section and solve for the allowable encroachment by trial and error.

This method for approximating the allowable encroachment is based on the assumptions that the 100-year flood elevation has been established or can be approximated and that the energy grade line of the encroached and non-encroached sections remains unchanged. The accuracy of results obtained using this method may be highly subject to the accuracy of the flood elevation used. In addition, since the method assumes no change in the energy grade line, the method should not be used near bridges or similar contraction-expansion areas.

For typical natural channel cross sections, the procedure may result in an equality that is very difficult to solve for the allowable encroachment dimensions. Morris (1984) provides a series of dimensionless graphs that are solved for the allowable encroachment as a percentage of the non-encroached overbank width. These graphs are based on an allowable flood elevation increase of 1 foot and assume a symmetrical cross section with triangular overbanks and equal encroachment on both overbanks. The limitations listed above for the general procedure also apply.

Because of the simplifying assumptions required, this approximate method will have limited applicability. Generally, only very small streams will satisfy the assumptions and the engineer should use extreme caution to avoid misapplication.

3.3.8 Example Problems

Example 3-1. Direct Solution of Manning's Equation

Use Manning's Equation to find the velocity, v, for an open channel with a hydraulic radius value of 0.6 foot, an n value of 0.020, and slope of 0.003 foot/foot.

Solve using Figure 3-4:

1. Connect a line between the slope scale at 0.003 and the roughness scale at 0.020 and note the intersection point on the turning line.

2. Connect a line between that intersection point and the hydraulic radius scale at 0.6 feet and read the velocity of 2.9 feet/second from the velocity scale.

Example 3-2. Grassed Channel Design Stability

A trapezoidal channel is required to carry 50 cfs at a bottom slope of 0.015 foot/foot. Find the channel dimensions required to comply with design stability criteria (retardance Class D) for a grass mixture.

1. From Table 3-2, the maximum velocity, v_m, for a grass mixture with a bottom slope less than 5 percent is 4 feet/second.

2. Assume an n value of 0.035 and find the value of vR from Figure 3-1.

   vR = 5.4

3. Use Equation 3-11 to calculate the value of R:

$$R = \frac{5.4}{4} = 1.35\,feet$$

4. Use Equation 3-12 to calculate the value of vR:

5. Since the vR value calculated in Step 4 is higher than the value obtained from Step 2, a

$$vR = \frac{1.49(1.35)^{5/3}(0.015)^{1/2}}{(0.035)} = 8.60$$

   higher n value is required and calculations are repeated. The results from each trial of calculations are presented below:

Assumed n Value	vR (Figure 3-1)	R (Equation 3-11)	vR (Equation 3-12)
0.035	5.4	1.35	8.60
0.038	3.8	0.95	4.41
0.039	3.4	0.85	3.57
0.040	3.2	0.80	3.15

Select n = 0.040 for stability criteria.

6. Use Figure 3-5 to select channel dimensions for a trapezoidal shape with 3:1 side slopes.

   Qn = (50) (0.040) = 2.0

   S = 0.015

   For b = 10 feet, d = (10) (0.098) = 0.98 feet
   b = 8 feet, d = (8) (0.14) = 1.12 feet

   Select: b = 10 feet, such that R is approximately 0.80 feet
   z = 3
   d = 1 foot
   v = 3.9 feet/second (Equation 3-3)
   Fr = 0.76 (Equation 3-10)
   Flow is subcritical

   Design capacity calculations for this channel are presented in Example 3-3.

Example 3-3. Grassed Channel Design Capacity

Use a 10-foot bottom width for the trapezoidal channel sized in Example 3-2 and find the depth of flow for retardance Class C.

1. Assume a depth of 1.0 foot and calculate the following (see Figure 3-2):

   A = (b + zd) d

   A = [10+ (3) (1)] (1)

   A = 13.0 square feet

$$R = \frac{[b + zd]d}{b + 2d\sqrt{1 + z^2}}$$

R = 0.796 feet

$$R = \frac{[10 + (3)(1)](1)}{10 + (2)(1)\sqrt{1 + 3^2}}$$

2. Find the velocity.

v = 50/13.0

v = 3.85 feet/second

3. Find the value of vR.

vR = (3.85) (0.796) = 3.06

4. Using the vR product from Step 3, find Manning's n from Figure 3-1 for retardance Class C.

n = 0.047

5. Use Figure 3-4 or Equation 3-3 to find the velocity for S = 0.015, R = 0.796, and n = 0.047.

v = 3.34 feet/second

6. Since 3.34 feet/second is less than 3.85 feet/second, a higher depth is required and calculations are repeated. Results from each trial of calculations are presented below:

Assumed Depth (ft)	Area $(ft)^2$	R (ft)	Velocity Q/A (ft/sec)	vR	Manning's n (Fig. 3-1)	Velocity (Eq. 3-3)
1.0	13.00	0.796	3.85	3.06	0.047	3.34
1.05	13.81	0.830	3.62	3.00	0.0475	3.39
1.1	14.63	0.863	3.42	2.95	0.048	3.45
1.2	16.32	0.928	3.06	2.84	0.049	3.54

7. Select a depth of 1.1 with an n value of 0.048 for design capacity requirements. Add at least 0.2 feet for freeboard to give a design depth of 1.3 feet. Design data for the trapezoidal channel are summarized as follows:

Vegetation lining = grass mixture, v_m = 4 feet/second

Q = 50 cfs

b = 10 feet, d = 1.3 feet, z = 3, S = 0.015

Top width = (10) + (2) (3) (1.3) = 17.8 feet

n (stability) = 0.040, d = 1.0 foot, v = 3.9 feet/second, Froude number = 0.76 (Equation 3-10)

n (capacity) = 0.048, d = 1.1 feet, v = 3.45 feet/second, Froude number = 0.64 (Equation 3-10)

Example 3-4. Riprap Design

A natural channel has an average bankfull channel velocity of 8 feet per second with a top width of 20 feet and a bend radius of 50 feet. The depth over the toe of the outer bank is 5 feet. Available stone weight is 170 pounds per cubic foot. Stone placement is on a side slope of 2:1 (horizontal: vertical).

1. Use 8 feet per second as the design velocity, because the reach is short and the bend is not protected.

2. Determine the bend correction coefficient for the ratio of R_b/T = 50/20 = 2.5. From Figure 3-8, C_b = 1.55. The adjusted effective velocity is (8) (1.55) = 12.4 feet/second.

3. Determine the correction coefficient for the specific weight of 170 pounds from Figure 3-9 as 0.98. The adjusted effective velocity is (12.4) (0.98) = 12.15 feet/second.

4. Determine minimum d_{30} from Figure 3-10 or Equation 3-15 as about 10 inches.

5. An available gradation has a minimum d_{30} size of 12 inches and is acceptable. It has enough fines that a filter course will not be required.

6. (Optional) Another gradation is available with a d_{30} of 8 inches. The ratio of desired to standard stone size is 8/10 or 0.8. From Figure 3-11, this gradation would be acceptable if the blanket thickness were increased from the original d_{100} thickness by 35 percent (a ratio of 1.35 on the horizontal axis).

7. Perform preliminary design. Make sure that the stone is carried up and downstream far enough to ensure stability of the channel and that the toe will not be undermined. The downstream length of protection for channel bends can be determined using Figure 3-7.

3.4 Gradually Varied Flow

The most common occurrence of gradually varied flow in storm drainage is the backwater created by culverts, storm sewer inlets, or channel constrictions. For these conditions, the flow depth will be greater than normal depth in the channel and the water surface profile should be computed using backwater techniques.

Many computer programs are available for computation of backwater curves. The most general and widely used program, HEC-RAS, was developed by the U.S. Army Corps of Engineers (1998) and SWMM-Extran block was developed by the U.S. Environmental Protection Agency (Huber et al 1992, Roesner et al 1994) are recommended for floodwater profile computations. This program can be used to compute water surface profiles for both natural and artificial channels.

For prismatic channels, the backwater calculation can be computed manually using the direct step method, as presented by Chow (1959). For an irregular nonuniform channel, the standard step method is recommended, although it is a more tedious and iterative process. The use of HEC-RAS or SWMM Extran are recommended as an alternative for manual standard step calculations.

Cross sections for water surface profile calculations should be normal to the direction of flood flow. The number of sections required will depend on the irregularity of the stream and flood plain. In general, a cross section should be obtained at each location where there are significant changes in stream width, shape, or vegetal patterns. Sections should usually be no more than 4 to 5 channel widths apart or 100 feet apart for ditches or streams and 500 feet apart for flood plains, unless the channel is very regular. Additional details regarding survey requirements are provided in Chapter 11.

3.4.1 Direct Step Method

The direct step method is limited to prismatic channels. A form for recording the calculations described below is presented in **Table 3-7** (Chow, 1959).

1. Record the following parameters across the top of Table 3-7:

   Q = Design flow, in cfs

   n = Manning's n value (see Section 3.2.5)

   S_o = Channel bottom slope, in feet/foot

   α = Energy coefficient

y_c = Critical depth, in feet

y_n = Normal depth, in feet

2. Using the desired range of flow depths, y, recorded in column 1, compute the cross-sectional area, A, the hydraulic radius, R, and average velocity, v, and record results in columns 2, 3, and 4, respectively.

3. Compute the velocity head, $\alpha\, v^2/2g$, in feet, and record the result in column 5.

4. Compute specific energy, E, in feet, by summing the velocity head in column 5 and the depth of flow in column 1. Record the result in column 6.

5. Compute the change in specific energy, ΔE, between the current and previous flow depths and record the result in column 7 (not applicable for row 1).

6. Compute the friction slope using the equation:

$$S_f = \frac{n^2 v^2}{2.22 R^{4/3}} \qquad (3\text{-}19)$$

where:

S_f = Friction slope, in feet/foot

n = Manning's n value

v = Average velocity in feet/second

R = Hydraulic radius, in feet

Record the result in column 8.

7. Determine the average, of the friction slope between this depth and the previous depth (not applicable for row 1). Record the result in column 9.

8. Determine the difference between the bottom slope, S_o, and the average friction slope, $\overline{S}_f$, from column 9 (not applicable for row 1). Record the result in column 10.

9. Compute the length of channel between consecutive rows or depths of flow using the equation:

$$\Delta x = \frac{\Delta E}{S_o - S_f} = \frac{column\ 7}{column\ 10}$$

(3-20)

where:

Δx = Length of channel between consecutive depths of flow, in feet

ΔE = Change in specific energy, in feet

S_o = Bottom slope, in feet/foot

S_f = Friction slope, in feet/foot

Record the result in column 11.

10. Sum the distances from the starting point to give cumulative distances, x, for each depth in column 1 and record the result in column 12.

3.4.2 Standard Step Method

The standard step method is a trial and error procedure applicable to both natural and prismatic channels. The step computations are arranged in tabular form, as shown in Table 3-8 and described below (Chow, 1959):

1. Record the following parameters across the top of Table 3-8:

   Q = Design flow, in cfs

   n = Manning's n value (see Section 3.2.5)

   S_o = Channel bottom slope, in feet/foot

   α = Energy coefficient

   k_e = Eddy head loss coefficient, in feet

   y_c = Critical depth, in feet

   y_n = Normal depth, in feet

2. Record the location of measured channel cross sections and the trial water surface elevation, z, for each section in columns 1 and 2. The trial elevation will be verified or rejected based on computations of the step method.

3. Determine the depth of flow, y, based on trial elevation and channel section data. Record the result in column 3.

4. Using the depth from Step 3 and section data, compute the cross-sectional area. A, in feet, and hydraulic radius, R, in feet. Record the results in columns 4 and 5.

5. Divide the design discharge by the cross-sectional area from Step 4 to compute the average velocity, v, in feet/second. Record the result in column 6.

6. Compute the velocity head, $\alpha v^2/2g$, in feet, and record the result in column 7.

7. Compute the total head, H, in feet, by summing the water surface elevation, z, in column 2 and the velocity head in column 7. Record the result in column 8.

8. Compute the friction slope, S_f, using Equation 3-19 and record the result in column 9.

9. Determine the average friction slope, $\bar{S}_f$, between the sections in each step (not applicable for row 1). Record the result in column 10.

10. Determine the distance between sections, Δx, and record the result in column 11.

11. Multiply the average friction slope, $\bar{S}_f$ (column 10), by the reach length, Δx (column 11), to give the friction loss in the reach, h_f. Record the result in column 12.

12. Compute the eddy loss using the equation:

$$h_e = k_e \frac{v^2}{2g} \qquad (3\text{-}21)$$

where:

h_e = Eddy head loss, in feet

k_e = Eddy head loss coefficient, in feet (for prismatic and regular channels, $k_e = 0$; for gradually converging and diverging channels, k_e = 0 to 0.1 or 0.2; for abrupt expansions and contractions, $k_e = 0.5$)

v = Average velocity, in feet/second (column 6)

g = Acceleration due to gravity, 32.2 feet/second^2

12. Compute the elevation of the total head, H, by adding the values of h_f and h_e (columns 12 and 13) to the elevation at the lower end of the reach, which is found in column 14 of the previous reach or row. Record the result in column 14.

13. If the value of H computed above does not agree closely with that entered in column 8, a new trial value of the water surface elevation is used in column 2 and calculations are repeated until agreement is obtained. The computation may then proceed to the next step or section reported in column 1.

3.4.3 Example Problems

Example 3-5. Direct Step Method

Use the direct step method (Section 3.4.1) to compute a water surface profile for a trapezoidal channel using the following data:

Q = 400 cfs

B = 20 feet

z = 2

S = 0.0016 foot/foot

n = 0.025

α = 1.10

A dam backs up water to a depth of 5 feet immediately behind the dam. The upstream end of the profile is assumed to have a depth 1 percent greater than normal depth.

Results of calculations, as obtained from Chow (1959), are reported in Table 3-9. Values in each column of the table are briefly explained below.

1. Depth of flow, in feet, arbitrarily assigned values ranging from 5 to 3.4 feet.

2. Water area in square feet, corresponding to the depth, y, in column 1.

3. Hydraulic radius, in feet, corresponding to y in column 1.

4. Mean velocity, in feet/second, obtained by dividing 400 cfs by the water area in column 2.

5. Velocity head, in feet, calculated using the mean velocity from column 4 and an α value of 1.1.

6. Specific energy, E, in feet, obtained by adding the velocity head in column 5 to the depth of flow in column 1.

7. Change of specific energy, ΔE, in feet, equal to the difference between the E value in column 6 and that of the previous step.

8. Friction slope, S_f, computed by Equation 3-19, with n = 0.025, v as given in column 4, and R as given in column 3.

9. Average friction slope between the steps, $\overline{S}_f$, equal to the arithmetic mean of the friction slope computed in column 8 and that of the previous step.

10. Difference between the bottom slope, S_o, 0.0016 and the average friction slope, $\overline{S}_f$, in column 9.

11. Length of the reach. Δx, in feet, between the consecutive steps, computed by Equation 3-20 or by dividing the value of ΔE in column 7 by the value of $S_o - S_f$ in column 10.

12. Distance from the section under consideration to the dam site. This is equal to the cumulative sum of the values in column 11 computed for previous steps.

Example 3-6. Standard Step Method

Use the standard step method (see Section 3.4.2) to compute a water surface profile for the channel data and stations considered in Example 3-5. Assume the elevation at the dam site is 600 feet.

Results of the calculations, as obtained from Chow (1959), are reported in Table 3-10. Values in each column of the table are briefly explained below:

1. Section identified by station number such as "station 1 + 55." The locations of the stations are fixed at the distances determined in Example 3-5 to compare the procedure with that of the direct step method.

2. Water surface elevation, z, at the station. A trial value is first entered in this column; this will be verified or rejected on the basis of the computations made in the remaining columns of the table. For the first step, this elevation must be given or assumed. Since the elevation of the dam site is 600 feet and the height of the dam is 5 feet, the first entry is 605.00 feet. When the trial value in the second step has been verified, it becomes the basis for the verification of the trial value in the next step, and the process continues.

3. Depth of flow, y, in feet, corresponding to the water surface elevation in column 2. For instance, the depth of flow at station 1+55 is equal to the water surface elevation minus the elevation at the dam site minus the distance from the dam site times bed slope.

   605.048 - 600.00 - (155) (0.0016) = 4.80 feet

4. Water area, A, in square feet, corresponding to y in column 3.

5. Hydraulic radius, R, in feet, corresponding to y in column 3.

6. Mean velocity, v, equal to the given discharge 400 cfs divided by the water area in column 4.

7. Velocity head, in feet, corresponding to the velocity in column 6 and an % value of 1.1.

8. Total head, H, equal to the sum of z in column 2 and the velocity head in column 7.

9. Friction slope, S_f, computed by Equation 3-19, with n = 0.025, v from column 6, and R from column 5.

10. Average friction slope through the reach, S_f, between the sections in each step, approximately equal to the arithmetic mean of the friction slope just computed in column 9 and that of the previous step.

11. Length of the reach between the sections, Δx, equal to the difference in station numbers between the stations.

12. Friction loss in the reach, h_f, equal to the product of the values in columns 10 and 11.

13. Eddy loss in the reach, h_e equal to zero.

14. Elevation of the total head, H, in feet, computed by adding the values of h_f and h_e in columns 12 and 13 to the elevation at the lower end of the reach, which is found in column 14 of the previous reach. If the value obtained does not agree closely with that entered in column 8, a new trial value of the water surface elevation is assumed until agreement is obtained. The value that leads to agreement is the correct water surface elevation. The computation may then proceed to the next step.

3.5 Rapidly Varied Flow

Rapidly varied flow common to storm drainage systems occurs at flow control structures, hydraulic jumps, and bridges. Sharp-crested weir flow equations for no end contractions, two end contractions, and submerged discharge conditions, and equations for broad-crested weirs, v-notch weirs, and orifices are presented in Chapter 8. Bridges are detailed in Chapter 9. The hydraulic jump is presented below.

3.6 Hydraulic Jump

A hydraulic jump can occur when flow passes rapidly from supercritical to subcritical depth. The evaluation of a hydraulic jump should consider the high energy loss and erosive forces that are associated with the jump. For rigid-lined facilities such as pipes or concrete channels, the forces and the change in energy can affect the structural stability or the hydraulic capacity. For grass-lined channels, unless the erosive forces are controlled, serious damage can result. Control of jump location is usually obtained by check dams or grade control structures that confine the erosive forces to a protected area. Flexible material such as riprap, rock, or rubble usually affords the most effective protection.

The analysis of the hydraulic jump inside storm sewers must be approximate, because of the lack of data for circular, elliptical, or arch sections. The jump can be approximately located by intersecting the energy grade line of the super-critical and subcritical flow reaches. The primary concerns are whether the pipe can withstand the forces, which may separate the joint or damage the pipe wall, and whether the jump will affect the hydraulic characteristics. The effect on pipe capacity can be determined by evaluating the energy grade line, taking into account the energy lost by the jump. In general, for Froude numbers less than 2.0, the loss of energy is less than 10 percent. French (1985) provides semi-empirical procedures to evaluate the hydraulic jump in circular and other non-rectangular channel sections.

For long box culverts with a concrete bottom, the concerns about jump are the same as for storm sewers. However, the jump can be adequately defined for box culverts/drains and for spillways using the jump characteristics of rectangular sections.

The relationship between variables for a hydraulic jump in rectangular sections can be expressed as:

$$d_2 = -\frac{d_1}{2} + \left(\frac{d_1^{\,2}}{4} + \frac{2v_1^{\,2} d_1}{g} \right)^{1/2} \qquad (3\text{-}22)$$

where:

d_2 = Depth below jump, in feet

d_1 = Depth above jump, in feet

v_1 = Velocity above jump, in feet/second

g = Acceleration due to gravity, 32.2 feet/second^2

A nomograph for solving Equation 3-22 is presented in Figure 3-12. Additional details on evaluating hydraulic jumps can be found in publications by USDOT, FHWA (HEC-14, 1983), Chow (1959), Peterska (1978), and French (1985).

3.7 Construction and Maintenance Considerations

An important step in the design process involves identifying whether special provisions are warranted to properly construct or maintain proposed facilities.

Open channels rapidly lose hydraulic capacity without adequate maintenance. Maintenance may include repairing erosion damage, mowing grass, cutting brush, and removing sediment or debris. Brush, sediment, or debris can reduce design capacity and can harm or kill vegetative linings, thus creating the potential for erosion damage during large storm events. Maintenance of vegetation should include the repeated application of fertilizer, irrigation during dry periods, and reseeding or resodding to restore the viability of damaged areas. Additional information is available in Volume 4: TCP-05, 10, 19, 20, 21; PESC-01,02, 04 and 08.

Table 3-1
Maximum Velocities for Comparing Lining Materials

Material	Maximum Velocity [a] (feet/second)
Bare soil	
Silt or fine sand	1.50
Sandy loam	1.75
Silt loam	2.00
Stiff clay	3.75
Sod	4.0
Lapped sod	5.5
Vegetation	Use Table 3-2
Rigid [a]	10

[a] Higher velocities may be acceptable for rigid linings if appropriate protection is provided (see Chapters 9 and 10).

Table 3-2
Maximum Velocities For Vegetative Channel Linings

Vegetation Type	Slope Range (%)	Maximum Velocity [a] (feet per second)
Bermudagrass	0-5	6
	5-10	5
Kentucky bluegrass Buffalo grass	0-5	5
	5-10	4
Grass Mixture	0-5	4
	5-10	3
Lespedeza Sericea Kudzu, alfalfa	0-5	2.5
Annuals	0-5	2.5

[a] Based on erosive soils.

Reference: USDA, TP-61 (1947).

Table 3-3
Recommended Manning's n Values For Artificial Channels

Lining Category [a]	Lining Type	n Value Depth Ranges 0 – 0.5 ft.	0.5 – 2.0 ft.	>2.0 ft.
Rigid	Concrete (Broom or Float Finish)	0.015	0.013	0.013
	Gunite	0.022	0.020	0.020
	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	0.016	0.015	0.015
	Jute Net	0.028	0.022	0.019
	Fiberglass Roving	0.028	0.021	0.019
	Straw with Net	0.065	0.033	0.025
	Curled Wood Mat	0.066	0.035	0.028
	Synthetic Mat	0.036	0.025	0.021
Gravel Riprap	1-inch (2.5-cm) d_{50}	0.044	0.033	0.030
	2-inch (5-cm) d_{50}	0.066	0.041	0.034
Rock Riprap [b]	N/A	$n = 0.0395\,(d_{50})^{1/6}$ d_{50} = Diameter of stone for which 50 percent, by weight, of the gradation is finer, in feet		

[a] n values for vegetative linings should be determined using Figure 3-1.

[b] See Section 3.3.6.

Reference: USDOT, FHWA, HEC-15 (1986).

Table 3-4
Classification of Vegetative Covers as to Degree of Retardance

Retardance Class	Cover	Condition
A	Weeping lovegrass	Excellent stand, tall (average 30") (76 cm)
	Yellow bluestem	
	Ischaemum	Excellent stand, tall (average 36") (91 cm)
B	Kudzu	Very dense growth, uncut
	Bermudagrass	Good stand, tall (average 12") (30 cm)
	Native grass mixture	
	(little bluestem, bluestem, blue gamma, and other long and short midwest grasses)	Good stand, unmowed
	Weeping lovegrass	Good stand, tall (average 24") (61 cm)
	Lespedeza sericea	Good stand, not woody, tall (average 19") (48 cm)
	Alfalfa	Good stand, uncut (average 11") (28 cm)
	Weeping lovegrass	Good stand, unmowed (average 13") (33 cm)
	Kudzu	Dense growth, uncut
	Blue gamma	Good stand, uncut (average 13") (33 cm)
C	Crabgrass	Fair stand, uncut (10 to 48") (25 to 122 cm)
	Bermudagrass	Good stand, mowed (average 6") (15 cm)
	Common lespedeza	Good stand, uncut (average 11") (28 cm)
	Grass-legume mixture--	
	Summer (orchard grass, redtop, Italian ryegrass, and common lespedeza)	Good stand, uncut (6 to 8 inches) (15 to 20 cm)
	Centipedegrass	Very dense cover (average 6 inches) (15 cm)
	Kentucky bluegrass	Good stand, headed (6 to 12 inches) (15 to 30 cm)
D	Bermudagrass	Good stand, cut to 2.5-inch height (6 cm)
	Common lespedeza	Excellent stand, uncut (average 4.5") (11 cm)
	Buffalograss	Good stand, uncut (3 to 6 inches) (8 to 15 cm)
	Grass-legume mixture--	
	fall, spring (orchard grass, redtop, Italian ryegrass, and common lespedeza)	Good stand, uncut (4 to 5 inches) (10 to 13 cm)
	Lespedeza sericea	After cutting to 2-inch height (5 cm) Very good stand before cutting
E	Bermudagrass	Good stand, cut to 1.5-inch height (4 cm)
	Bermudagrass	Burned stubble

Note: Covers classified have been tested in experimental channels. Covers were green and generally uniform.

Reference: USDA, TP-61 (1947).

Table 3-5
Coefficients For Computing Manning's n Values
For Natural Or Excavated Channels Using Cowan's Equation [a]

Channel Conditions			Values [b]
Material Involved	Earth	n^0	0.020
	Rock Cut		0.025
	Fine Gravel		0.024
	Coarse Gravel		0.028
Degree of Irregularity	Smooth	n^1	0.000
	Minor		0.005
	Moderate		0.010
	Severe		0.020
Variations of Channel Cross Section	Gradual	n^2	0.000
	Alternating Occasionally		0.005
	Alternating Frequently		0.010-0.015
Relative Effect of Obstructions	Negligible	n^3	0.000
	Minor		0.010-0.015
	Appreciable		0.020-0.030
	Severe		0.040-0.060
Vegetation	Low	n^4	0.005-0.010
	Medium		0.010-0.025
	High		0.025-0.050
	Very High		0.050-0.100
Degree of Meandering	Minor	m^5	1.000
	Appreciable		1.150
	Severe		1.300

[a] Cowan's Equation is presented as Equation 3-1.

[b] From Chow (1959), Table 5-5, page 109.

Table 3-6
Critical Depth Equations For Uniform Flow In Selected Channel Cross Sections

Channel Type [a]	Semi-Empirical Equation [b] For Estimating Critical Depth	Range of Applicability
1. Rectangular [c]	$d_c = \left(\frac{Q^2}{gb^2} \right)^{1/3}$	**N/A**
2. Trapezoidal [c]	$d_c = 0.81 \left(\frac{Q^2}{gz^{0.75}\ b^{1.25}} \right)^{0.27} - \frac{b}{30z}$	$0.1 < 0.5522 \frac{Q}{b^{2.5}} < 0.4$ For $0.5522 \frac{Q}{b^{2.5}} < 0.1$, use rectangular channel equation
3. Triangular [c]	$d_c = \left(\frac{2Q^2}{gz^2} \right)^{1/5}$	**N/A**
4. Circular [d]	$d_c = 0.325 \left(\frac{Q}{D} \right)^{2/3} + 0.083D$	$0.3 < \frac{d_c}{D} < 0.9$
5. General [e]	$\frac{A^3}{T} = \frac{Q^2}{g}$	N/A

Where:
- d_c = Critical depth, in feet
- Q = Design discharge, in cfs
- g = Acceleration due to gravity, 32.2 feet/second ^{2}
- b = Bottom width of channel, in feet
- z = Side slopes of a channel (horizontal to vertical)
- D = Diameter of circular conduit, in feet
- A = Cross-sectional area of flow, in square feet
- T = Top width of water surface, in feet

[a] See Figure 3-2 for channel sketches.
[b] Assumes uniform flow with the kinetic energy coefficient equal to 1.0.
[c] Reference: French (1985).
[d] Reference: USDOT, FHWA, HDS-4 (1965).
[e] Reference: Brater and King (1976).

TABLE 3-7
Water Surface Profile Computation Form for the Direct Step Method

Location ______________________________

Q = __________ | n = __________ | S_o = __________ | α = __________ | y_c = __________ | y_n = __________

y (1)	A (2)	R (3)	v (4)	$\alpha v^2/2g$ (5)	E (6)	ΔE (7)	S_f (8)	$\bar{S}_f$ (9)	$S_o - \bar{S}_f$ (10)	Δx (11)	x (12)
1.											
2.											
3.											
4.											
5.											
6.											
7.											
8.											
9.											
10.											
11.											
12.											
13.											
14.											
15.											
16.											
17.											
18.											
19.											
20.											

(8) $S_f = \dfrac{n^2 v^2}{2.22 R^{4/3}}$ (11) $\Delta x = \dfrac{\Delta E}{S_o - \bar{S}_f}$

TABLE 3-8

Water Surface Profile Computation Form for the Standard Step Method

Location ______________________________

Q = ________		n = ________		S_o = ________		α = ________		k_e = ________		y_c = ________		y_n = ________	
Station (1)	z (2)	y (3)	A (4)	R (5)	v (6)	$\alpha v^2/2g$ (7)	H (8)	$\overline{S}_f$ (9)	$\overline{S}_f$ (10)	Δx (11)	h_f (12)	h_e (13)	H (14)
1.													
2.													
3.													
4.													
5.													
6.													
7.													
8.													
9.													
10.													
11.													
12.													
13.													
14.													
15.													
16.													
17.													
18.													
19.													
20.													

(9) $S_f = \dfrac{n^2 v^2}{2.22R^{4/3}}$ (12) $h_f = \Delta x \ \overline{S}_f$ (13) $h_e = k_e \dfrac{v^2}{2_g}$.

Table 3-9
DIRECT STEP METHOD RESULTS FOR EXAMPLE 3-5

y (1)	A (2)	R (3)	v (4)	$\alpha v^2/2g$ (5)	E (6)	ΔE (7)	S_f (8)	$\overline{S}_f$ (9)	$S_o - \overline{S}_f$ (10)	Δx (11)	x (12)
5.00	150.00	3.54	2.667	0.1217	5.1217	--	0.000370	--	--	--	--
4.80	142.08	3.43	2.819	0.1356	4.9356	0.1861	0.000433	0.000402	0.001198	155	155
4.60	134.32	3.31	2.979	0.1517	4.7517	0.1839	0.000507	0.000470	0.001130	163	318
4.40	126.72	3.19	3.156	0.1706	4.5706	0.1811	0.000598	0.000553	0.001047	173	491
4.20	119.28	3.08	3.354	0.1925	4.3925	0.1781	0.000705	0.000652	0.000948	188	679
4.00	112.00	2.96	3.572	0.2184	4.2184	0.1741	0.000850	0.000778	0.000822	212	891
3.80	104.88	2.84	3.814	0.2490	4.0490	0.1694	0.001020	0.000935	0.000665	255	1,146
3.70	101.38	2.77	3.948	0.2664	3.9664	0.0826	0.001132	0.001076	0.000524	158	1,304
3.60	97.92	2.71	4.085	0.2856	3.8856	0.0808	0.001244	0.001188	0.000412	196	1,500
3.55	96.21	2.68	4.158	0.2958	3.8458	0.0398	0.001310	0.001277	0.000323	123	1,623
3.50	94.50	2.65	4.233	0.3067	3.8067	0.0391	0.001382	0.001346	0.000254	154	1,777
3.47	93.48	2.63	4.278	0.3131	3.7831	0.0236	0.001427	0.001405	0.000195	121	1,898
3.44	92.45	2.61	4.326	0.3202	3.7602	0.0229	0.001471	0.001449	0.000151	152	2,050
3.42	91.80	2.60	4.357	0.3246	3.7446	0.0156	0.001500	0.001486	0.000114	137	2,187
3.40	91.12	2.59	4.388	0.3292	3.7292	0.0154	0.001535	0.001518	0.000082	188	2,375

Note: Q = 400 cfs   n = 0.025 $S_o = 0.0016$ $\alpha = 1.10$ $y_c = 2.22$ ft. $y_n = 3.36$ ft.

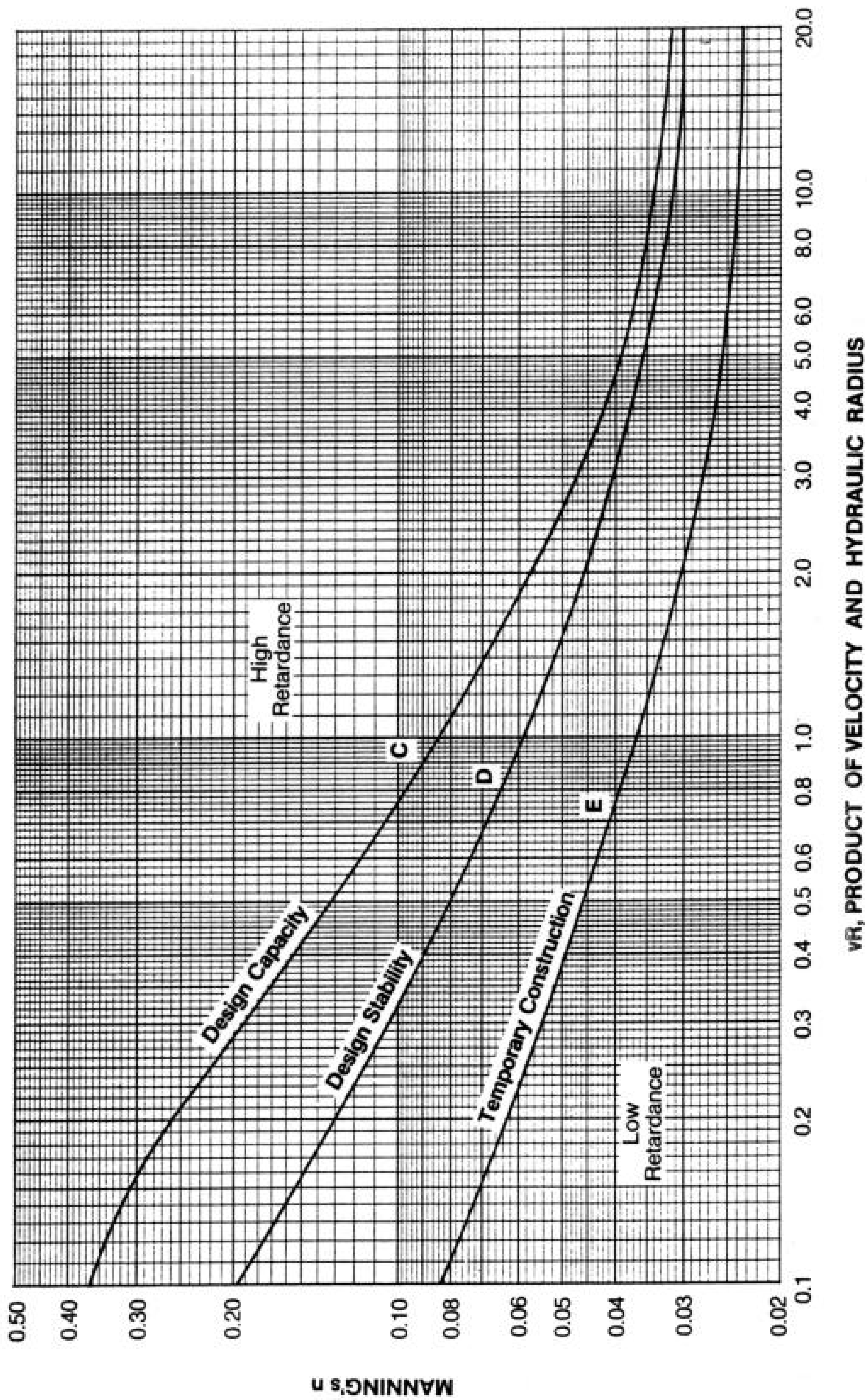
Reference: Chow (1959).

Table 3-10
STANDARD STEP METHOD RESULTS FOR EXAMPLE 3-6

Station (1)	z (2)	y (3)	A (4)	R (5)	v (6)	$\alpha v^2/2g$ (7)	H (8)	S_f (9)	$\bar{S}_f$ (10)	Δx (11)	h_f (12)	h_e (13)	H (14)
0 + 00	605.000	5.00	150.00	3.54	2.667	0.1217	605.122	0.000370	--	--	--	--	605.122
1 + 55	605.048	4.80	142.08	3.43	2.819	0.1356	605.184	0.000433	0.000402	155	0.062	0	605.184
3 + 18	605.109	4.60	134.32	3.31	2.979	0.1517	605.261	0.000507	0.000470	163	0.077	0	605.261
4 + 91	605.186	4.40	126.72	3.19	3.156	0.1706	605.357	0.000598	0.000553	173	0.096	0	605.357
6 + 79	605.286	4.20	119.28	3.08	3.354	0.1925	605.479	0.000705	0.000652	188	0.122	0	605.479
8 + 91	605.426	4.00	112.00	2.96	3.572	0.2184	605.644	0.000850	0.000778	212	0.165	0	605.644
11 + 46	605.633	3.80	104.88	2.84	3.814	0.2490	605.882	0.001020	0.000935	255	0.238	0	605.882
13 + 04	605.786	3.70	101.38	2.77	3.948	0.2664	606.052	0.001132	0.001076	158	0.170	0	606.052
15 + 00	605.999	3.60	97.92	2.71	4.085	0.2856	606.285	0.001244	0.001188	196	0.233	0	606.285
16 + 23	606.146	3.55	96.21	2.68	4.158	0.2958	606.442	0.001310	0.001277	123	0.157	0	606.442
17 + 77	606.343	3.50	94.50	2.65	4.233	0.3067	606.650	0.001382	0.001346	154	0.208	0	606.650
18 + 98	606.507	3.47	93.48	2.63	4.278	0.3131	606.820	0.001427	0.001405	121	0.170	0	606.820
20 + 50	606.720	3.44	92.45	2.61	4.326	0.3202	607.040	0.001471	0.001449	152	0.220	0	607.040
21 + 87	606.919	3.42	91.80	2.60	4.357	0.3246	607.244	0.001500	0.001486	137	0.204	0	607.244
23 + 75	607.201	3.40	91.12	2.59	4.388	0.3292	607.530	0.001535	0.001518	188	0.286	0	607.530

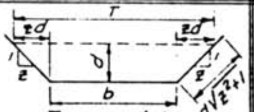
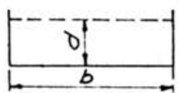
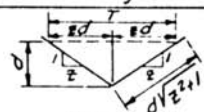
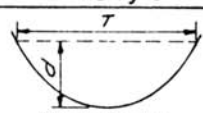
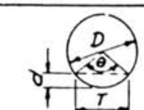
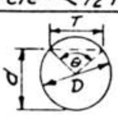
Note: Q = 400 cfs n = 0.025 $S_o = 0.0016$ $\alpha = 1.10$ $h_e = 0$ $y_c = 2.22$ ft. $y_n = 3.36$ ft.

Reference: Chow (1959).



Reference: USDA, TP-61 (1947).

Figure 3-1
Manning's n Value for Vegetated Channels

Section	Area A	Wetted Perimeter P	Hydraulic Radius R	Top Width T	Critical Depth Factor, Z
 Trapezoid	$bd+zd^2$	$b+2d\sqrt{z^2+1}$	$\frac{bd+zd^2}{b+2d\sqrt{z^2+1}}$	$b+2zd$	$\frac{[(b+zd)d]^{1.5}}{\sqrt{b+2zd}}$
 Rectangle	bd	$b+2d$	$\frac{bd}{b+2d}$	b	$bd^{1.5}$
 Triangle	zd^2	$2d\sqrt{z^2+1}$	$\frac{zd}{2\sqrt{z^2+1}}$	$2zd$	$\frac{\sqrt{2}}{2}zd^{2.5}$
 Parabola	$\frac{2}{3}dT$	$T+\frac{8d^2}{3T}$ [1]	$\frac{2dT^2}{3T^2+8d^2}$ [1]	$\frac{3a}{2d}$	$\frac{2}{9}\sqrt{6}\,Td^{1.5}$
 Circle - < ½ full [2]	$\frac{D^2}{8}\left(\frac{\pi\theta}{180}-\sin\theta\right)$	$\frac{\pi D\theta}{360}$	$\frac{45D}{\pi\theta}\left(\frac{\pi\theta}{180}-\sin\theta\right)$	$D\sin\frac{\theta}{2}$ or $2\sqrt{d(D-d)}$	$a\sqrt{\frac{a}{D\sin\frac{\theta}{2}}}$
 Circle - > ½ full [3]	$\frac{D^2}{8}\left(2\pi-\frac{\pi\theta}{180}+\sin\theta\right)$	$\frac{\pi D(360-\theta)}{360}$	$\frac{45D}{\pi(360-\theta)}\left(2\pi-\frac{\pi\theta}{180}+\sin\theta\right)$	$D\sin\frac{\theta}{2}$ or $2\sqrt{d(D-d)}$	$a\sqrt{\frac{a}{D\sin\frac{\theta}{2}}}$

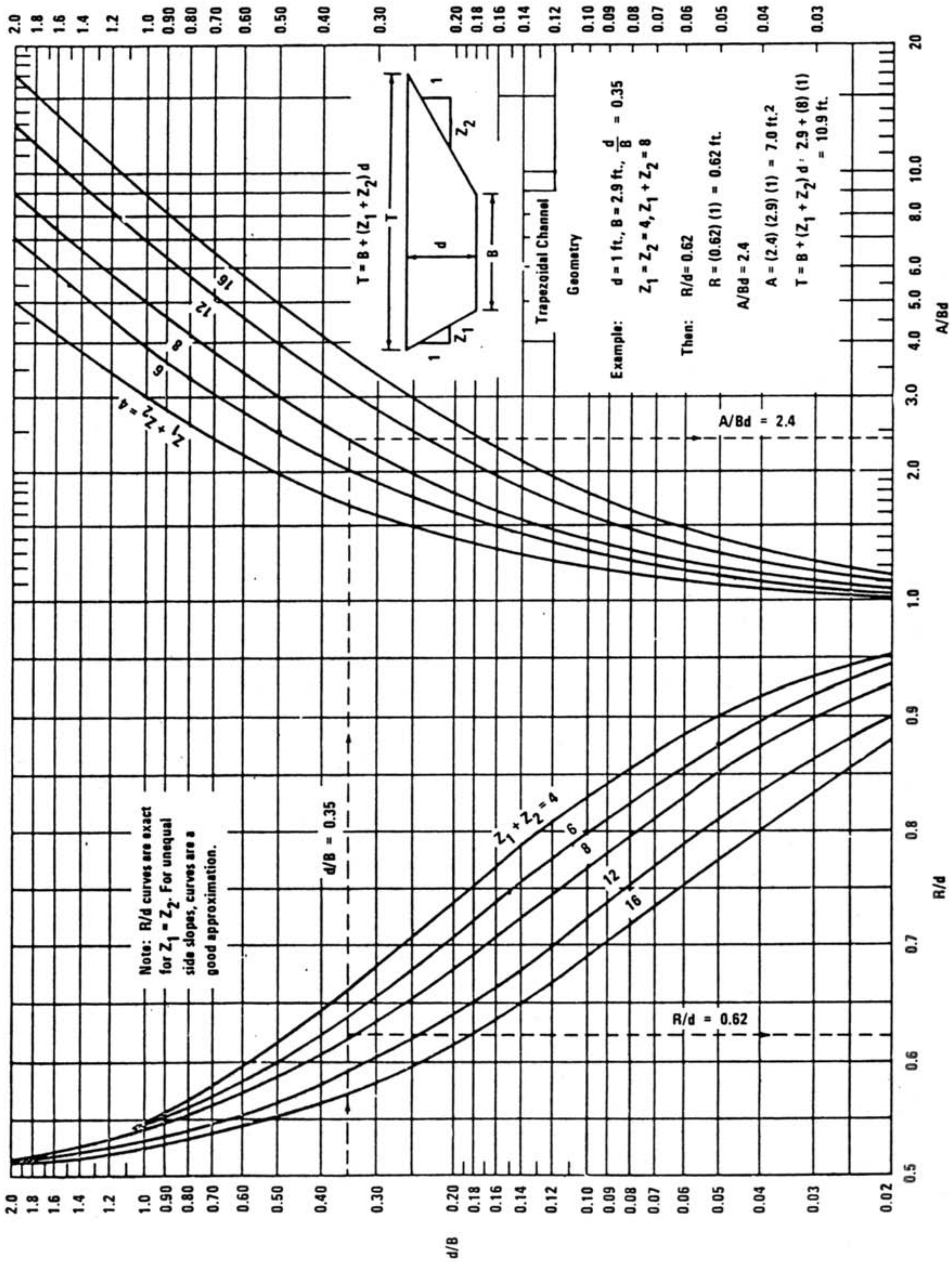
[1] Satisfactory approximation for the interval $0 < \frac{d}{T} \leq 0.25$
When $d/T > 0.25$, use $P = \frac{1}{2}\sqrt{16d^2+T^2} + \frac{T^2}{8d}\sinh^{-1}\frac{4d}{T}$

[2] $\theta = 4\sin^{-1}\sqrt{d/D}$
[3] $\theta = 4\cos^{-1}\sqrt{d/D}$
Insert θ in degrees in above equations

Note: Small z = Side Slope Horizontal Distance
Large Z = Critical Depth Section Factor

Reference: USDA, SCS, NEH-5 (1956).

Figure 3-2
Open Channel Geometric Relationships for Various Cross Sections



Reference: USDOT, FHWA, HEC-15 (1986).

Figure 3-3
Trapezoidal Channel Geometry

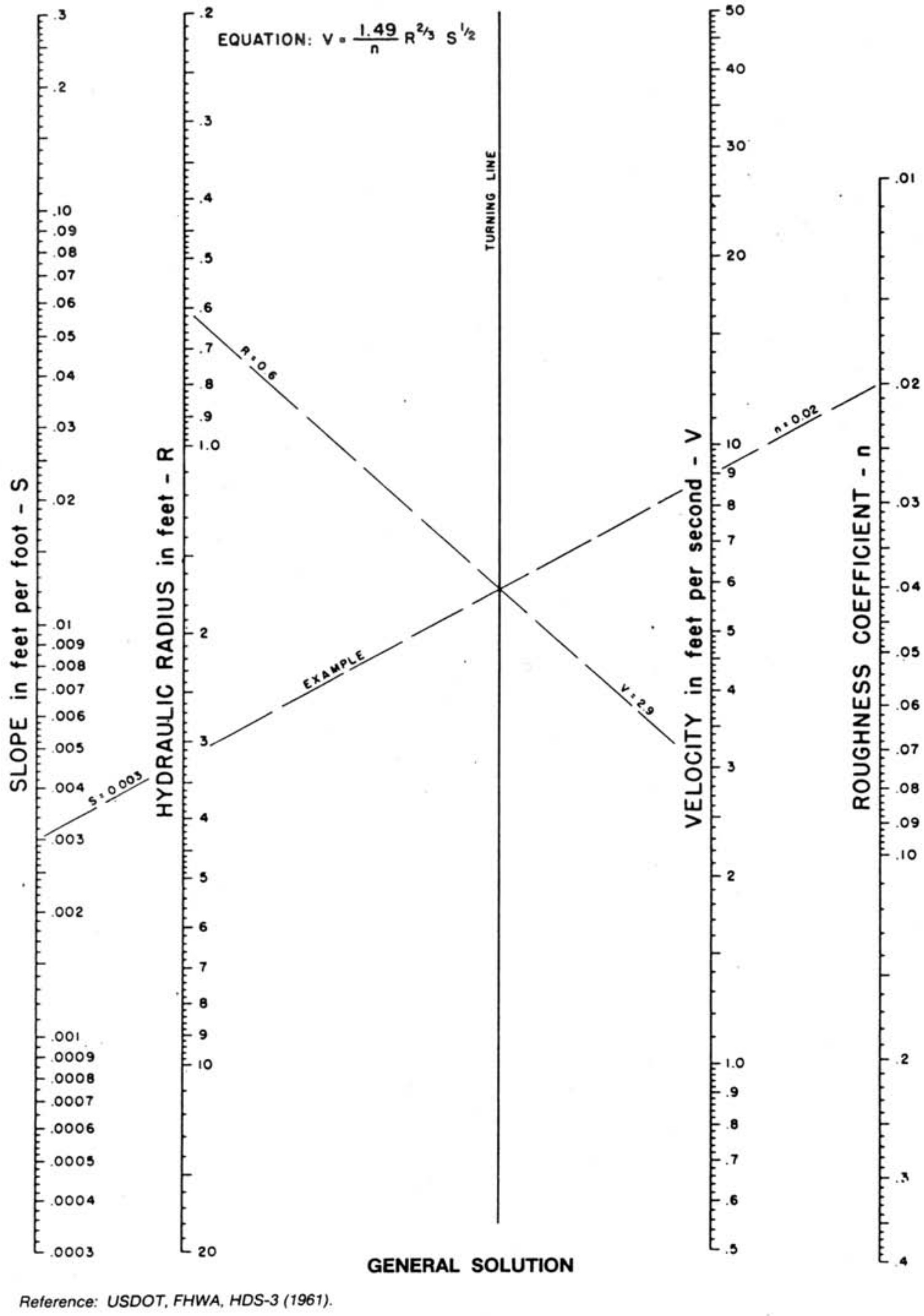
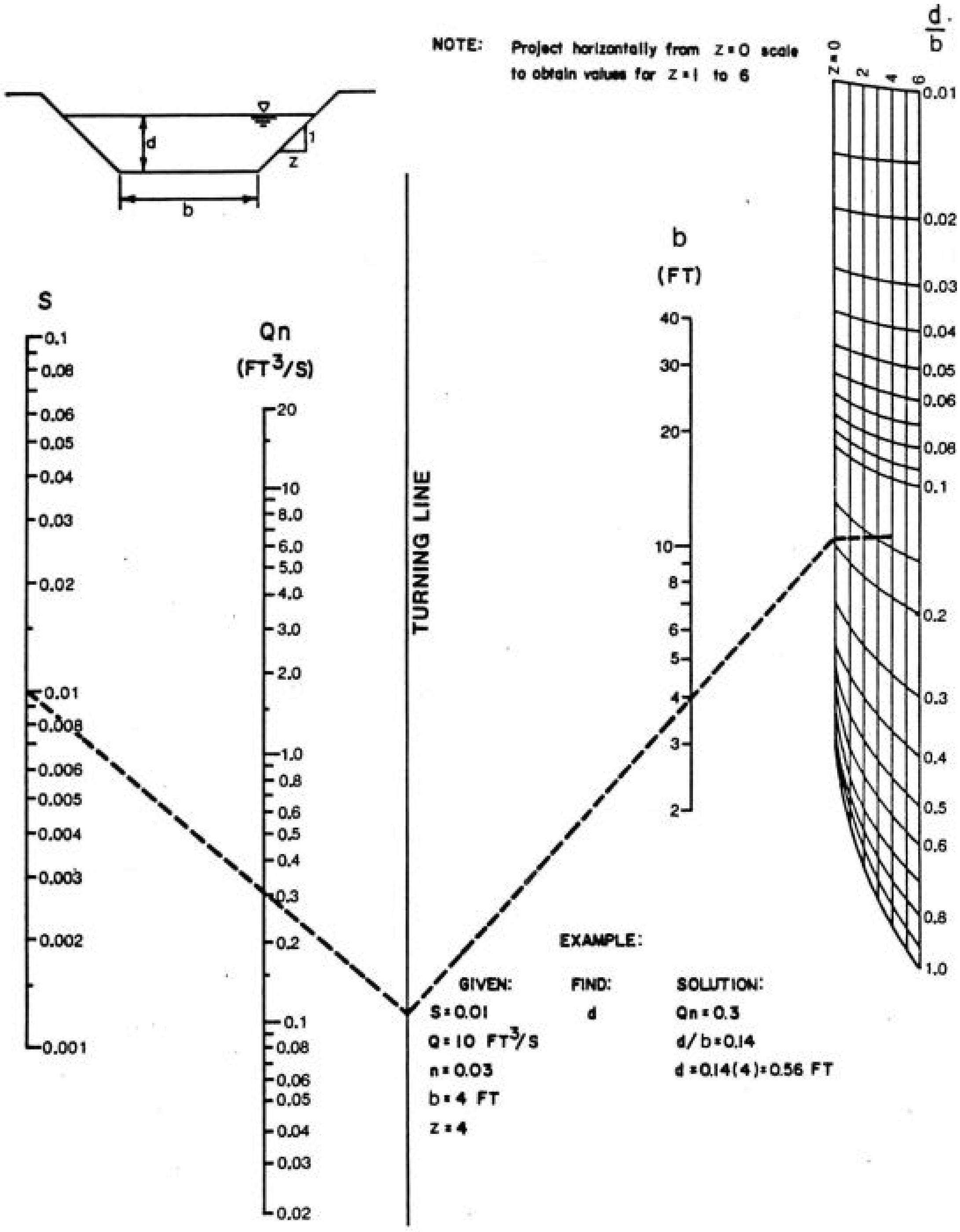


Figure 3-4
Nomograph for the Solution of Manning's Equation



Reference: USDOT, FHWA, HEC-15 (1986).

Figure 3-5
Solution of Manning's Equation for Trapezoidal Channels

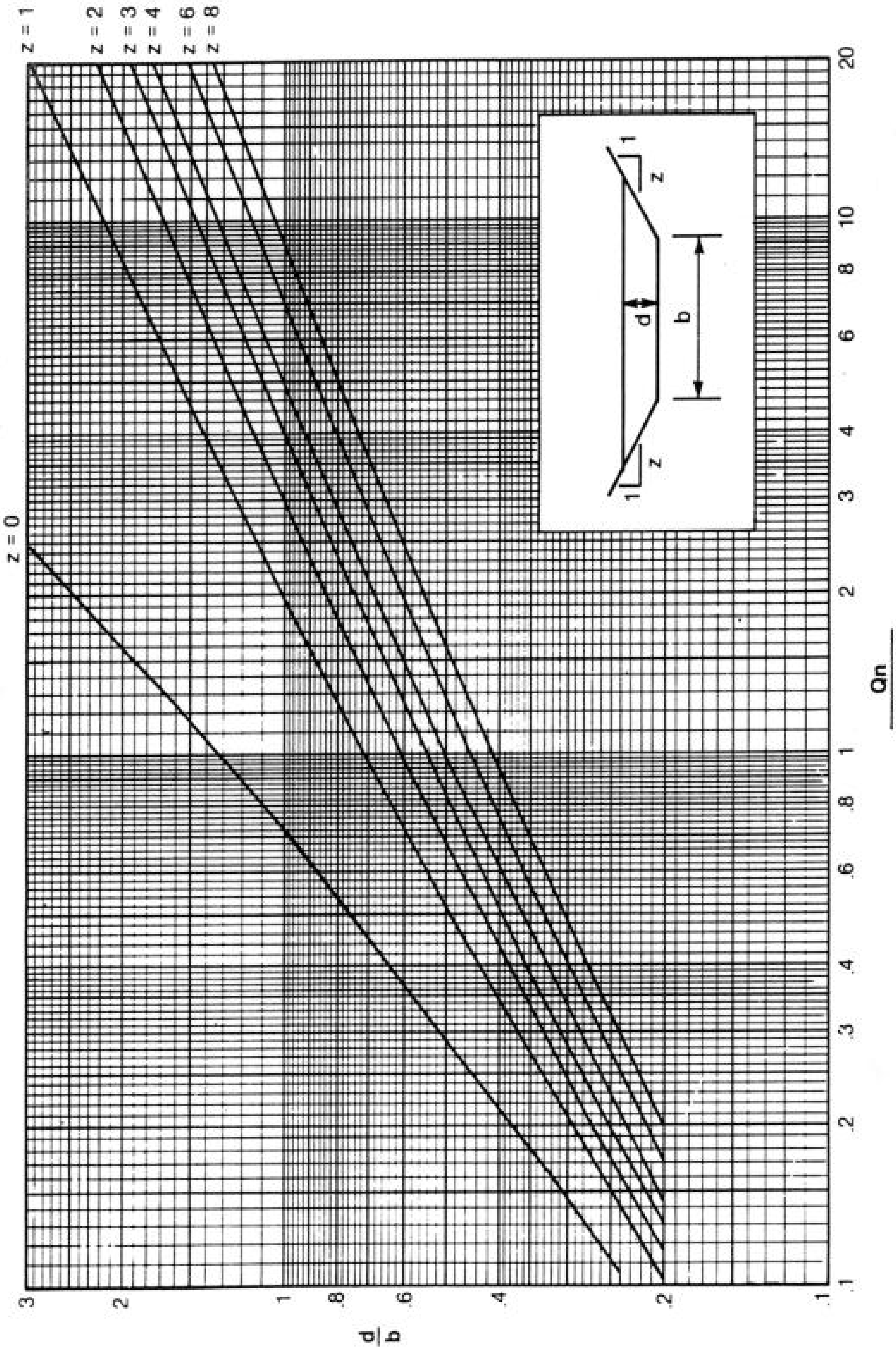
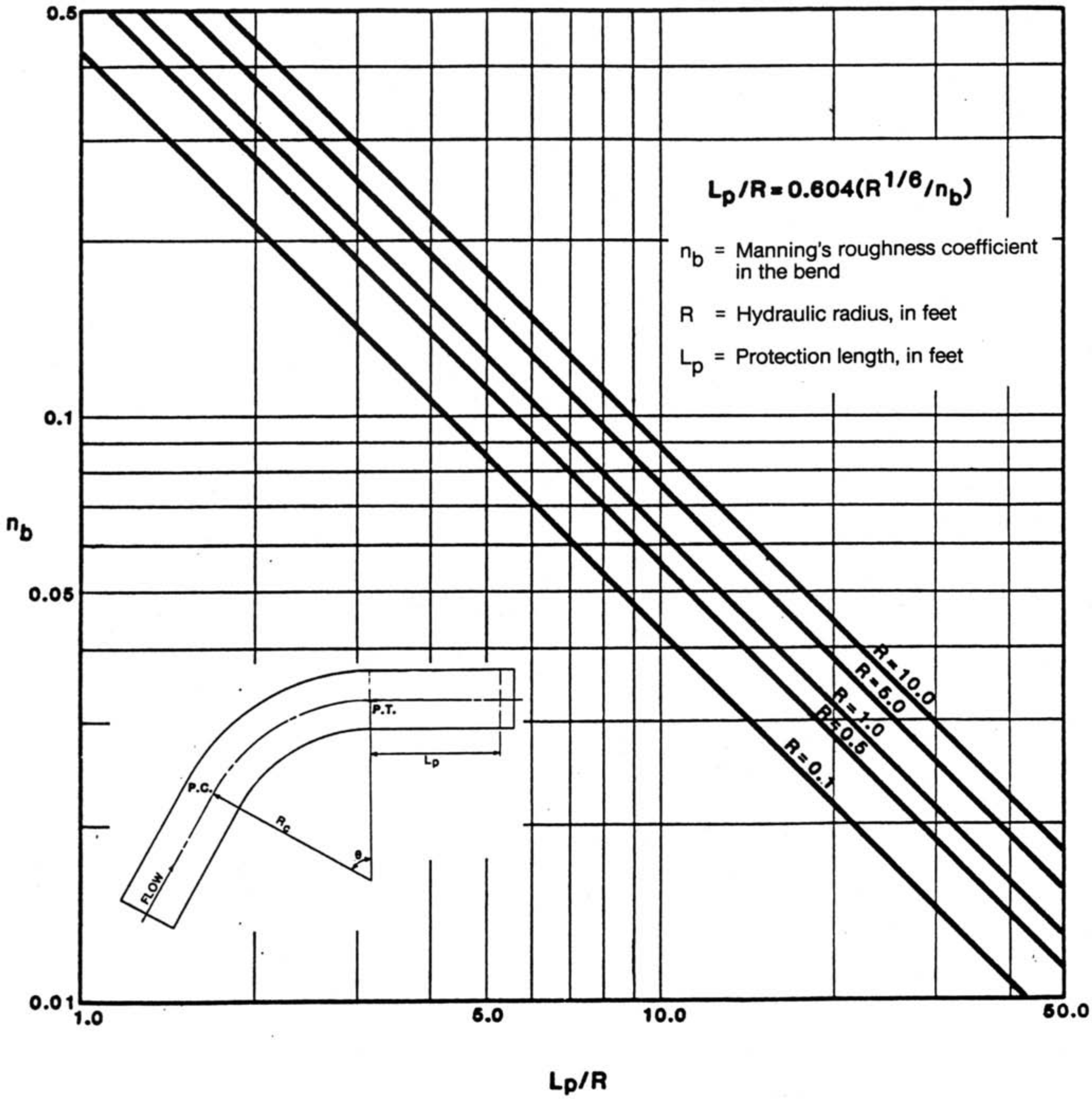
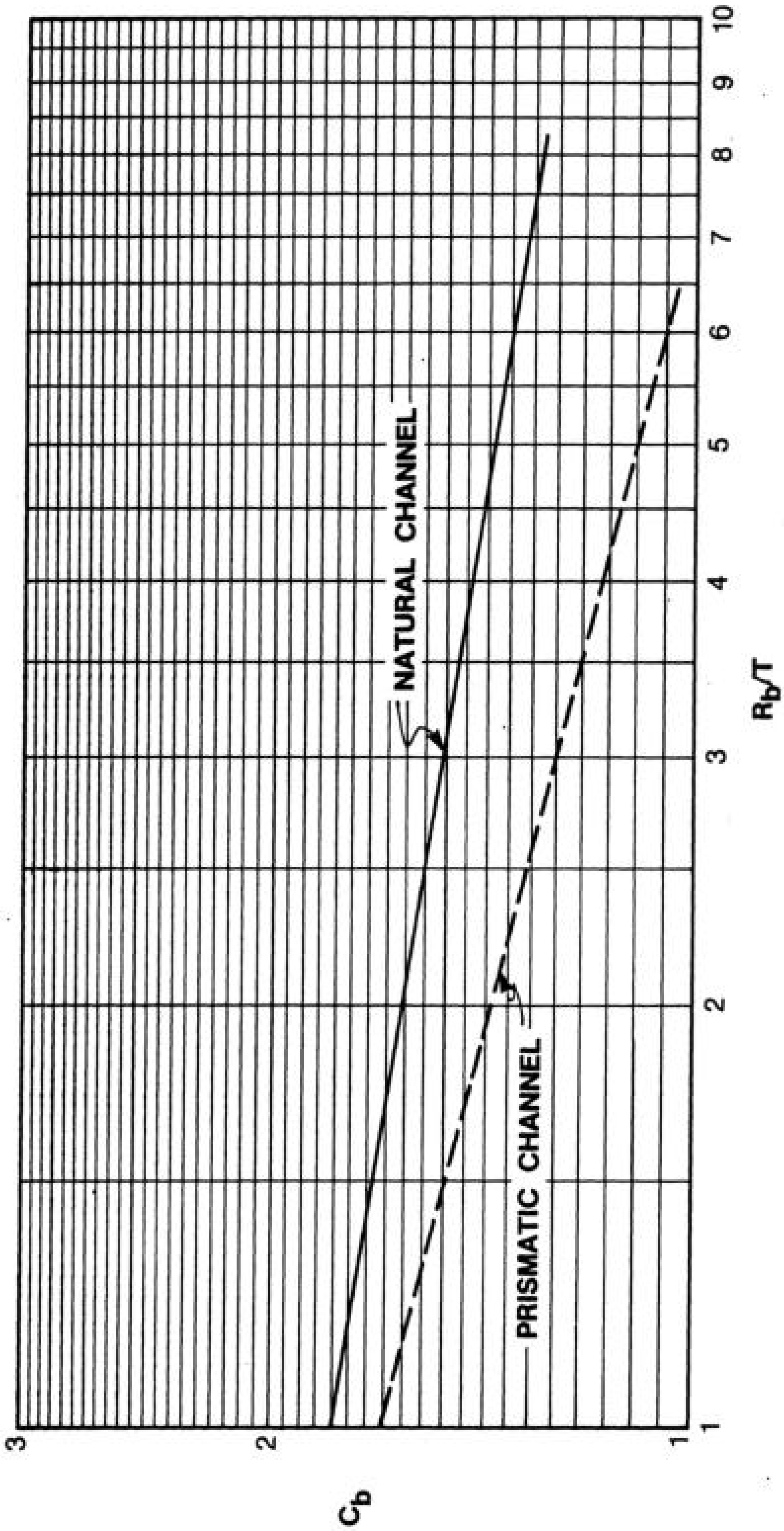


Figure 3-6
Trapezoidal Channel Capacity Chart



Reference: USDOT, FHWA, HEC-15 (1986).

Figure 3-7
Protection Length, L_p, Downstream of Channel Bend



To obtain effective velocity, multiply known velocity by C_b.

T = Channel Top Width
R_b = Centerline Bend Radius
C_b = Correction Coefficient

Reference: Maynord (1987).

Figure 3-8
Riprap Lining Bend Correction Coefficient

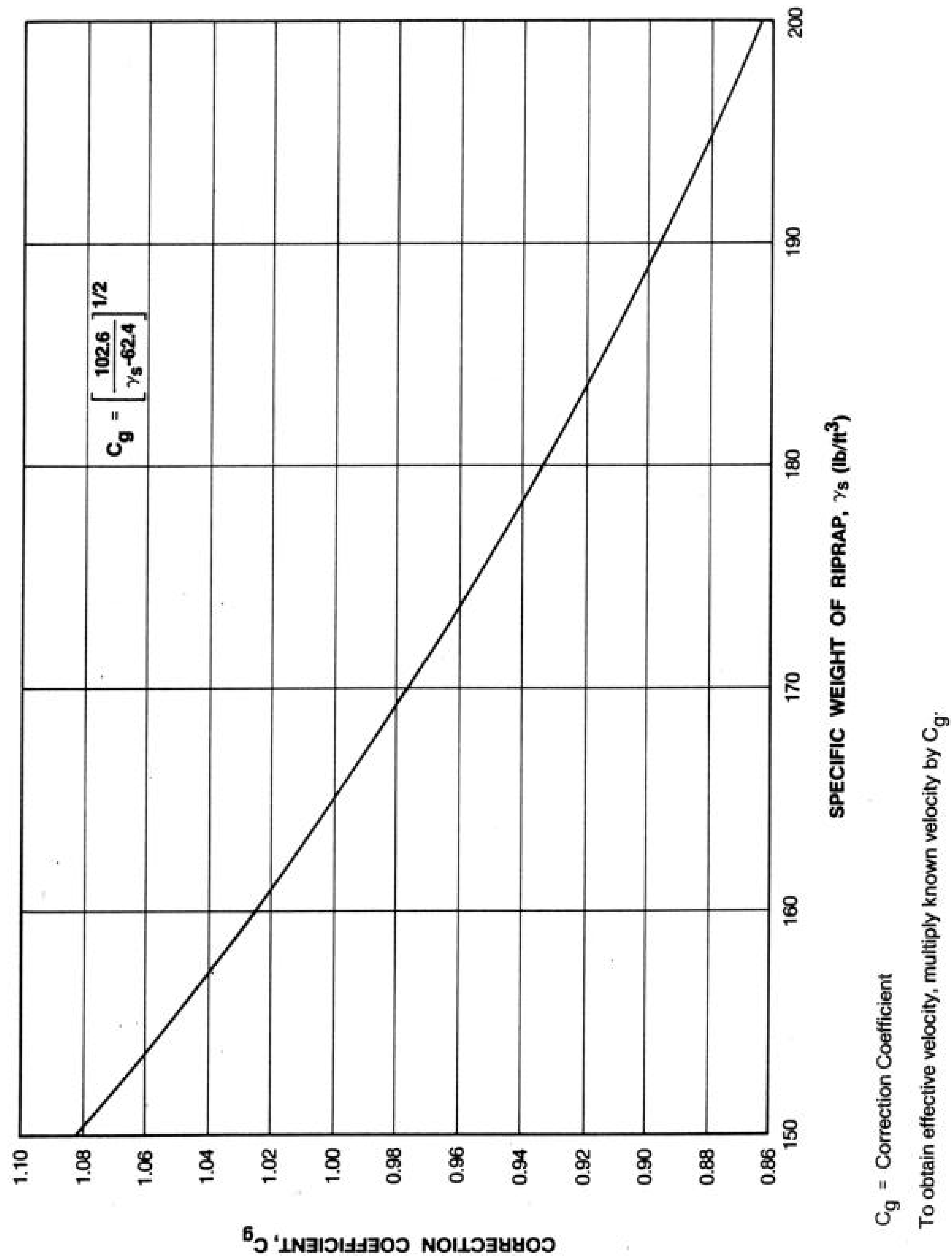
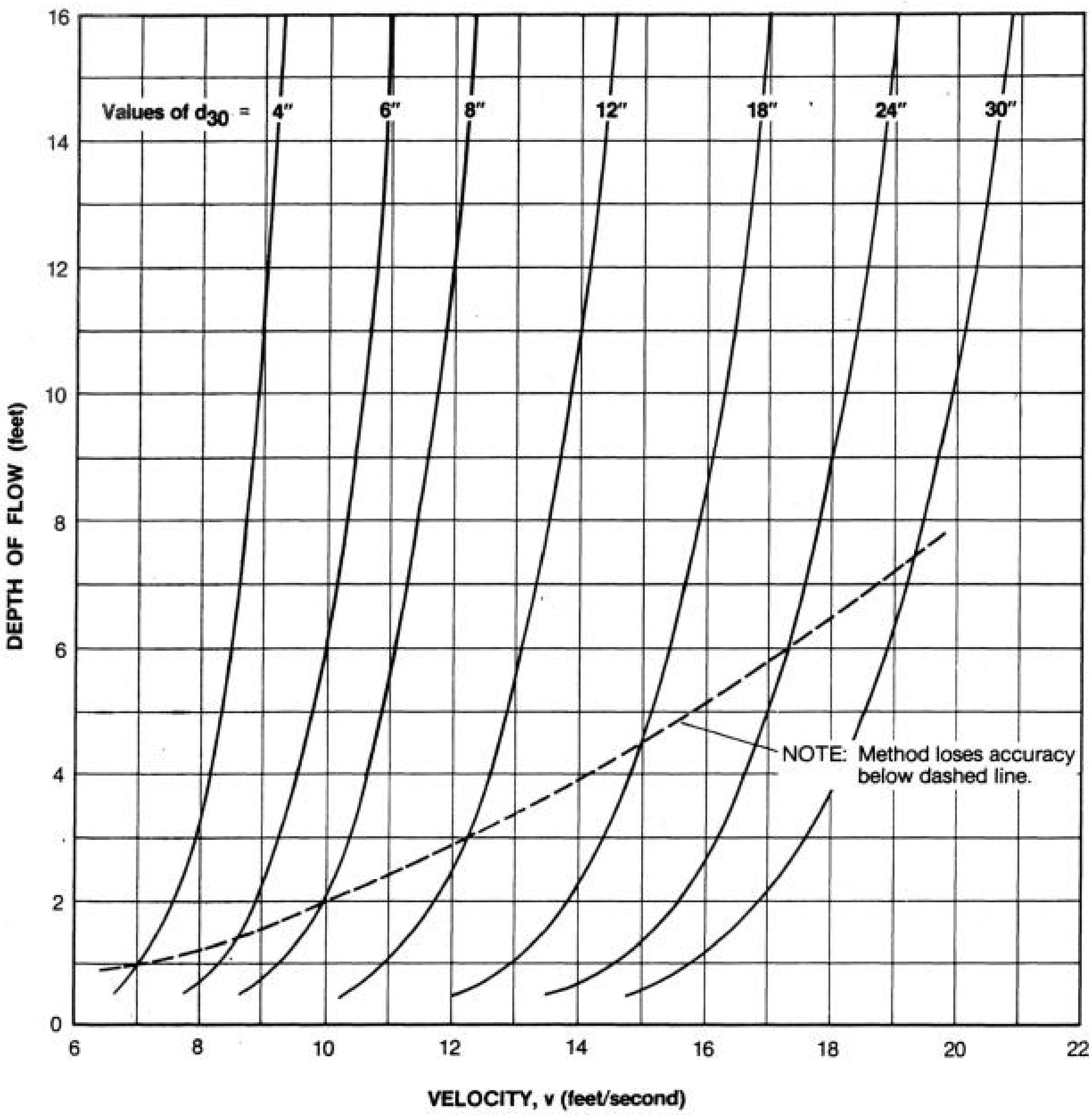
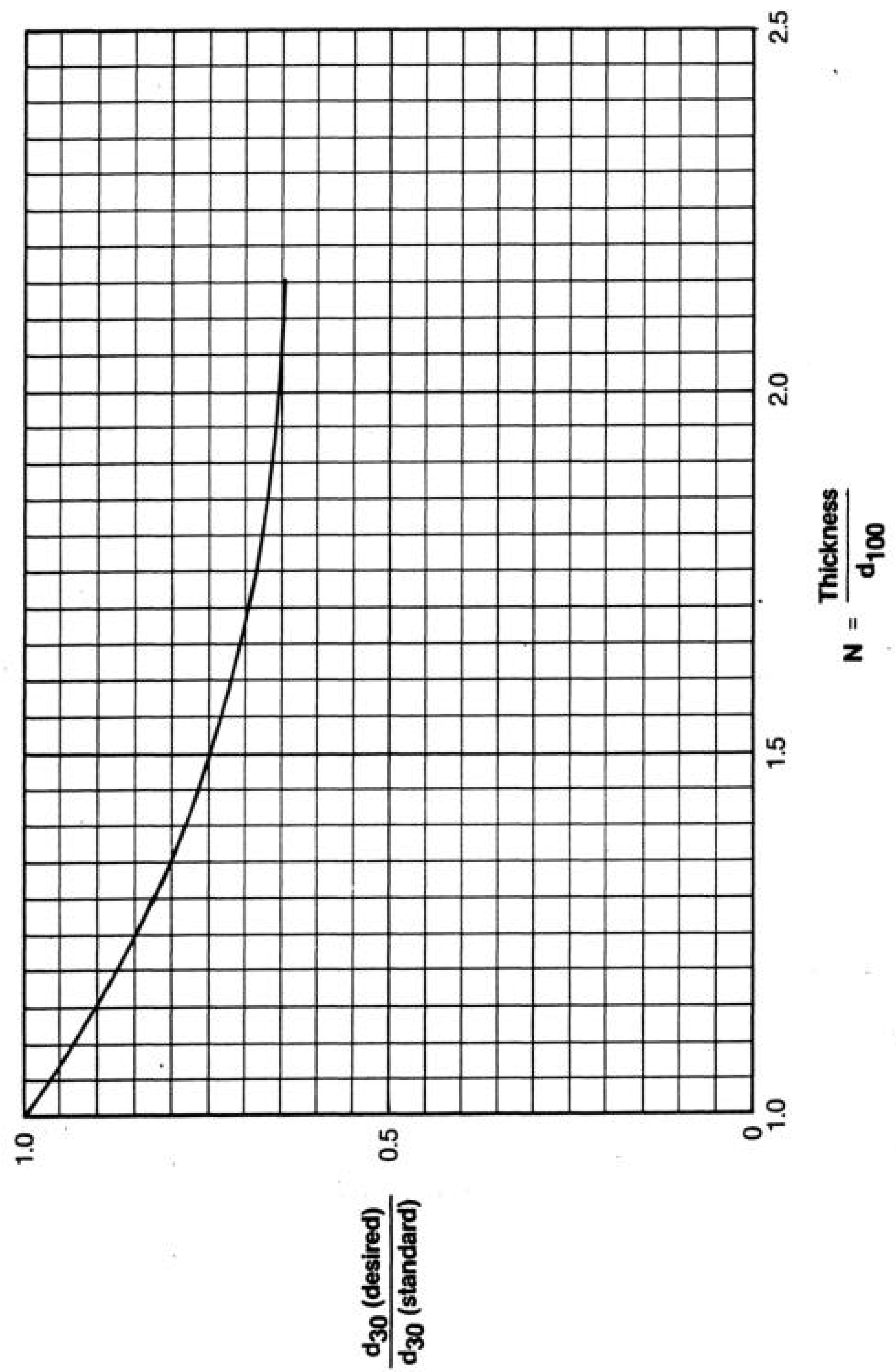


Figure 3-9
Riprap Lining Specific Weight Correction Coefficient



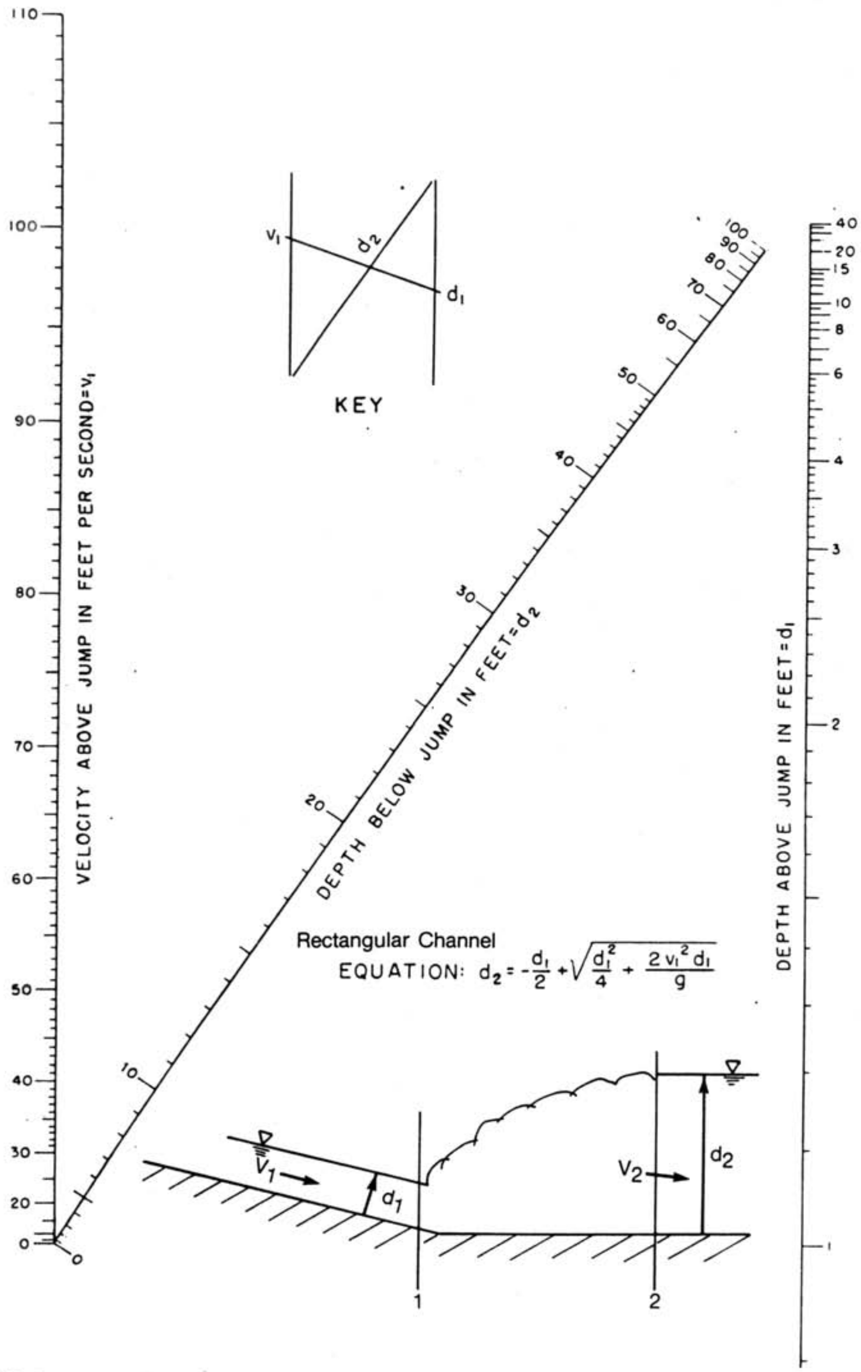
Reference: Reese (1988).

Figure 3-10
Riprap Lining d_{30} Stone Size as a Function
of Mean Velocity and Depth



Reference: Maynord (1987).

Figure 3-11
Riprap Lining Thickness Adjustment for $d_{85}/d_{15} = 2.0$ to 2.3



Reference: U.S. Department of the Interior (1973).

Figure 3-12
Nomograph for Solving the Rectangular Channel Hydraulic Jump Equation