CHAPTER 4

OPEN CHANNEL HYDRAULICS

Chapter Table of Contents

4.1 Overview

Introduction
4.1.1

This chapter emphasizes procedures for performing uniform calculations that aid in the selection or evaluation of appropriate channel linings, depths, and grades for natural or man-made channels. Allowable velocities are provided, along with procedures for evaluating channel capacity using Manning's equation.

Channel Linings
4.1.2

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, while rigid linings are generally concrete.

Vegetative
4.1.2.1

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
2. Standing or continuous flowing water
3. Lack of regular maintenance necessary to prevent domination by taller vegetation
4. Lack of nutrients and inadequate topsoil
5. Excessive shade
6. Velocities

Proper seeding, mulching, and soil preparation are required during construction to assure establishment of a healthy growth of grass. Soil testing may 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 necessary for use on steeper slopes to prevent sliding.

Flexible
4.1.2.2

Rock riprap including rubble is the most common 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 growth of grass and weeds through the lining may present maintenance problems. The use of flexible lining may be restricted where space is limited, since the higher roughness values create larger cross sections.

Rigid
4.1.2.3

Rigid linings are generally constructed of concrete and used where smoothness offers a higher capacity for a given cross-sectional area. 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 appropriate, a small concrete pilot channel could be used to handle the continuous low flows. Vegetation could then be maintained for handling larger flows.

4.2 Symbols and Definitions

Symbol Table

To provide consistency within this chapter as well as throughout this manual the following symbols will be used. These symbols were selected because of their wide use in open channel publications.

Table 4-1

SYMBOLS AND DEFINITIONS

Symbol	Definition	Units
α	Energy Coefficient	-
A	Cross-sectional area	ft^2
b	Bottom width	ft
C_g	Specific weight correction factor	-
D or d	Depth of flow	ft
d	Stone diameter	ft
Δ d	Superelevation of the water surface profile due to a bend	ft
d_x	Diameter of stone for which x percent, by weight, of the gradation is finer	ft
E	Specific energy	ft
Fr	Froude Number	-
g	Acceleration due to gravity	ft/s^2
h_e	Eddy head loss	ft
h_L	Head loss	ft
K	Channel Conveyance	-
k_e	Eddy head loss coefficient	ft
K_T	Trapezoidal open channel conveyance factor	-
L	Length of channel	ft
L_p	Length of downstream protection	ft
M	Side slope, M to 1	-
n	Manning's roughness coefficient	-
P	Wetted perimeter	ft
Q	Discharge rate	cfs
R	Hydraulic radius of flow	ft
R_c	Mean radius of the bend	ft
S	Slope	ft/ft
S_f	Friction slope	ft/ft
S_o	Channel bottom slope	ft/ft
SW_s	Specific weight of stone	lbs/ft^3
T	Top width of water surface	ft
V or v	Velocity of flow	ft/s
W	Channel top width	ft
w	Stone weight	lbs
y	Depth of flow	ft
y_c	Critical depth	ft
y_n	Normal depth	ft
z	Critical flow section factor	-
Z	Vertical distance from datum	ft

4.3 Design Criteria

General Criteria 4.3.1

The following criteria shall be used for open channel design:

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

2. Channel side slopes shall be stable throughout the entire length and slope shall be a maximum of 2:1.

3. Superelevation of the water surface at horizontal curves shall be accounted for by increased freeboard.

4. Transition to channel sections shall be smooth and gradual, with a minimum of 5:1 taper.

5. Low flow sections shall be considered in the design of channels with large cross sections (Q > 100cfs). Some channel designs will be required to have increased freeboard for superelevation of water surface at horizontal curves (see NC Erosion and Sediment Control Planning and Design Manual, Section 8.05.21).

Return Period 4.3.2

Open channel drainage systems shall be designed to handle a 10-year design storm. The 100-year design storm shall be routed through the channel system to determine if a 100+1 flood study is required as described in Section 4.3.3.

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.

100 + 1 Flood Analysis 4.3.3

All streams in the City of Charlotte and Mecklenburg County which drain more than one square mile (640 acres) are regulated by Floodway Ordinances, which restrict development in those flood plains. However, streams that drain less than one square mile will also flood and require regulation as well. This regulation is known as the 100 + 1 flood analysis.

The following criteria will be used to determine how and when the 100 + 1 flood analysis will be used.

1. The 100 + 1 analysis will be required for all portions of the drainage system which are expected to carry 50 cfs or more for the 100-year storm.

2. For drainage systems which are expected to carry 150 cfs or more for the 100-year storm, the 100-year + 1 elevation and flood limits shall be shown on the recorded map of the area for residential property as further described in the Subdivision Ordinance.

3. For portions of the drainage system which are expected to carry between 50 and 150 cfs, the City/County Engineering Department can require that the 100-year + 1 elevation be shown on the recorded map if the engineering analysis indicates that one of the following conditions are present:

- The 100-year + 1 foot line would exceed the set-back limits.
- The estimated runoff would create a hazard for adjacent properties or residents.
- The flood limits would be of such magnitude that adjacent residents should be informed of these limits.

Velocity Limitations 4.3.4

The final design of artificial open channels should be consistent with the velocity limitations for the selected channel lining. For design information on channel linings, refer to the "North Carolina Erosion and Sediment Control Planning and Design Manual". Additional sources are listed in the bibliography for analysis and design criteria for the channel stabilization.

4.4 Hydraulic Terms

Energy of Flow 4.4.1

Flowing water contains energy in two forms -- potential and kinetic. The potential energy at a particular point is represented by the depth of the water plus the elevation of the channel bottom above a convenient datum plane. The kinetic energy (in feet) is represented by the velocity head, $V^2/2g$. Figure 4-1 illustrates open channel energy concepts and equation 4.1 is the energy equation.

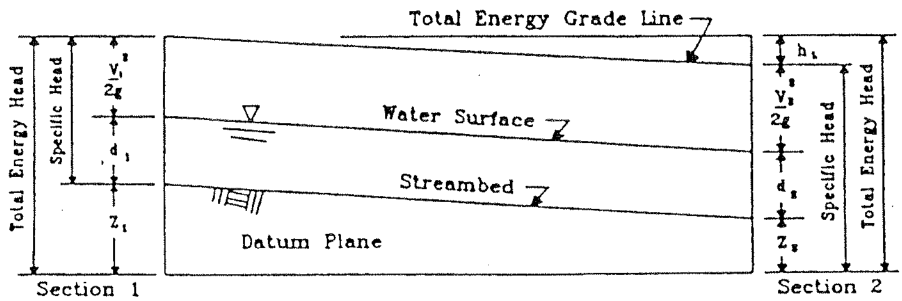


Figure 4-1 Energy In Open Channel Flow

$$d_1 + V_1^2/2g + Z_1 = d_2 + V_2^2/2g + Z_2 + h_L \qquad (4.1)$$

Where: d = depth of flow above streambed (ft)
V = mean velocity of flow (ft/s)
Z = vertical distance from datum (ft)
g = acceleration due to gravity (32.2 ft/s^2)
h_L = head loss (ft)

The slope (gradient) of the total energy grade line is a measure of the friction slope or rate of energy head loss due to friction. The total head loss for a length of channel is the product of the length and friction slope (h_L = S x L). Under uniform flow, the energy line is parallel to the water surface and streambed.

Steady & Unsteady 4.4.2

Flow in open channels is classified as either steady or unsteady flow. Steady flow occurs when discharge or rate of flow at any cross section is constant with time. In unsteady flow, the discharge or rate of flow varies from one cross section to another with time.

Uniform and Non-Uniform Flow 4.4.3

Uniform flow exists when the channel cross section, roughness, and slope are constant; and non-uniform or varied flow exists when the channel properties vary from section to section.

Froude Number 4.4.4

The Froude number is the ratio of the inertial force to that of gravitational force, expressed by the following equation:

$$Fr = v / (gD)^{1/2} \qquad (4.2)$$

Where: v = mean velocity of flow (ft/s)
g = acceleration due to gravity (32.2 ft/s^2)
D = hydraulic depth (ft) - defined as the cross sectional area of water normal to the direction of channel flow divided by free surface width.

Critical Flow 4.4.5

Critical flow is defined as the condition for which the Froude Number is equal to one. At that state of flow, the specific energy is a minimum for a constant discharge. By plotting specific energy head against depth of flow for a constant discharge, a specific energy diagram can be drawn as illustrated in Figure 4-2. Also, by plotting discharge against specific energy head we can illustrate not only minimum specific energy for a given discharge per unit width, but also maximum discharge per unit for a given specific energy.

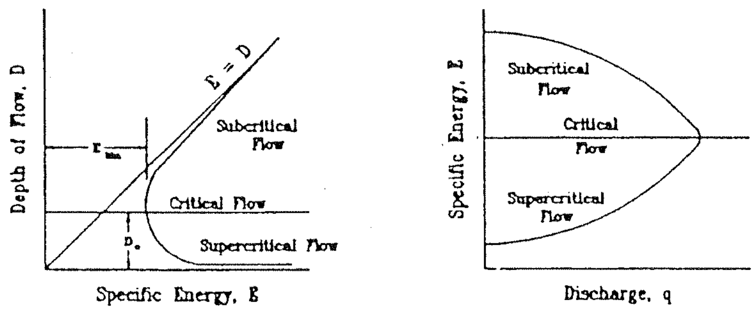


Figure 4-2 Definition Sketch of Specific Energy

Subcritical Flow 4.4.6

When the Froude Number is smaller than 1, the state of flow is defined as subcritical or tranquil flow, and surface waves propagate upstream as well as downstream. Control of subcritical flow depth is always downstream.

Supercritical Flow 4.4.7

When the Froude Number is larger than 1, the state of flow is defined as supercritical or rapid flow, and surface disturbance can propagate only in the downstream direction. Control of supercritical flow depth is always at the upstream end of the critical flow region.

4.5 Manning's n Values

General Considerations 4.5.1

The Manning's n value is an important variable in open channel flow computations. Variation in this variable can significantly affect discharge, depth, and velocity estimates. Since Manning's n values depend on many different physical characteristics of natural and man-made channels, care and good engineering judgement must be exercised in the selection process.

Selection 4.5.2

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

1. The physical roughness of the bottom and sides of the channel. 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.

2. The value of n depends on the height, density, type of vegetation, and how the vegetation affects the flow through the channel reach.

3. Channel shape variations, such as abrupt changes in channel cross sections or alternating small and large 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.

4. 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.

5. 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 must also be considered.

6. 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.

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

8. 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 evaluated with respect to type of channel, degree of maintenance, seasonal requirements, and other considerations as a basis for determining appropriate design n values. 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 calculations.

Manning's n Values 4.5.3

Recommended Manning's n values for artificial and natural channels are given in Table 4-2 shown below.

Table 4-2 Recommended Manning's n Values

Type of channel and description	Minimum	Normal	Maximum
B. LINED OR BUILT-UP CHANNELS			
B-1. Metal			
a. Smooth steel surface			
1. Unpainted	0.011	0.012	0.014
2. Painted	0.012	0.013	0.017
b. Corrugated	0.021	0.025	0.030
B-2. Nonmetal			
a. Cement			
1. Neat, surface	0.010	0.011	0.013
2. Mortar	0.011	0.013	0.015
b. Wood			
1. Planed, untreated	0.010	0.012	0.014
2. Planed, creosoted	0.011	0.012	0.015
3. Unplaned	0.011	0.013	0.015
4. Plank with battens	0.012	0.015	0.018
5. Lined with roofing paper	0.010	0.014	0.017
c. Concrete			
1. Trowel finish	0.011	0.013	0.015
2. Float finish	0.013	0.015	0.016
3. Finished, with gravel on bottom	0.015	0.017	0.020
4. Unfinished	0.014	0.017	0.020
5. Gunite, good section	0.016	0.019	0.023
6. Gunite, wavy section	0.018	0.022	0.025
7. On good excavated rock	0.017	0.020	
8. On irregular excavated rock	0.022	0.027	
d. Concrete bottom float finished with sides of			
1. Dressed stone in mortar	0.015	0.017	0.020
2. Random stone in mortar	0.017	0.020	0.024
3. Cement rubble masonry, plastered	0.016	0.020	0.024
4. Cement rubble masonry	0.020	0.025	0.030
5. Dry rubble or riprap	0.020	0.030	0.035
e. Gravel bottom with sides of			
1. Formed concrete	0.017	0.020	0.025
2. Random stone in mortar	0.020	0.023	0.026
3. Dry rubble or riprap	0.023	0.033	0.036
f. Brick			
1. Glazed	0.011	0.013	0.015
2. In cement mortar	0.012	0.015	0.018
g. Masonry			
1. Cemented rubble	0.017	0.025	0.030
2. Dry rubble	0.023	0.032	0.035
h. Dressed ashlar	0.013	0.015	0.017
i. Asphalt			
1. Smooth	0.013	0.013	
2. Rough	0.016	0.016	
j. Vegetal lining	0.030	.....	0.500

Table 4-2 (continued)

Type of channel and description	Minimum	Normal	Maximum
C. EXCAVATED OR DREDGED			
a. Earth, straight and uniform			
1. Clean, recently completed	0.016	0.018	0.020
2. Clean, after weathering	0.018	0.022	0.025
3. Gravel, uniform section, clean	0.022	0.025	0.030
4. With short grass, few weeds	0.022	0.027	0.033
b. Earth, winding and sluggish			
1. No vegetation	0.023	0.025	0.030
2. Grass, some weeds	0.025	0.030	0.033
3. Dense weeds or aquatic plants in deep channels	0.030	0.035	0.040
4. Earth bottom and rubble sides	0.028	0.030	0.035
5. Stony bottom and weedy banks	0.025	0.035	0.040
6. Cobble bottom and clean sides	0.030	0.040	0.050
c. Dragline-excavated or dredged			
1. No vegetation	0.025	0.028	0.033
2. Light brush on banks	0.035	0.050	0.060
d. Rock cuts			
1. Smooth and uniform	0.025	0.035	0.040
2. Jagged and irregular	0.035	0.040	0.050
e. Channels not maintained, weeds and brush uncut			
1. Dense weeds, high as flow depth	0.050	0.080	0.120
2. Clean bottom, brush on sides	0.040	0.050	0.080
3. Same, highest stage of flow	0.045	0.070	0.110
4. Dense brush, high stage	0.080	0.100	0.140
D. NATURAL STREAMS			
D-1. Minor streams (top width at flood stage <100 ft)			
a. Streams on plain			
1. Clean, straight, full stage, no rifts or deep pools	0.025	0.030	0.033
2. Same as above, but more stones and weeds	0.030	0.035	0.040
3. Clean, winding, some pools and shoals	0.033	0.040	0.045
4. Same as above, but some weeds and stones	0.035	0.045	0.050
5. Same as above, lower stages, more ineffective slopes and sections	0.040	0.048	0.055
6. Same as 4, but more stones	0.045	0.050	0.060
7. Sluggish reaches, weedy, deep pools	0.050	0.070	0.080
8. Very weedy reaches, deep pools, or floodways with heavy stand of timber and underbrush	0.075	0.100	0.150

Table 4-2 (continued)

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

Reference: Chow, V.T., ed. 1959, Open-Channel Hydraulics

4.6 Best Hydraulic Section

Introduction
4.6.1

For a given discharge, slope, and channel roughness, maximum velocity implies minimum cross sectional area. From Manning's equation, if velocity is maximized and area is minimized, wetted perimeter will also be minimized. The best hydraulic section therefore, simultaneously minimizes area and wetted perimeter. A minimum freeboard of 6" must be provided.

For ease of construction, most channels are built with trapezoidal cross-sections. Therefore, this chapter deals with computing the best hydraulic section for trapezoidal section channels.

Equations
4.6.2

Given that the desired side slope, M to one, has been selected for a given channel, the minimum wetted perimeter (P) exists when:

$$P = 4y\,(1 + M^2)^{1/2} - 2My \qquad (4.3)$$

(Figure 4-3 below shows a definition of variables.)

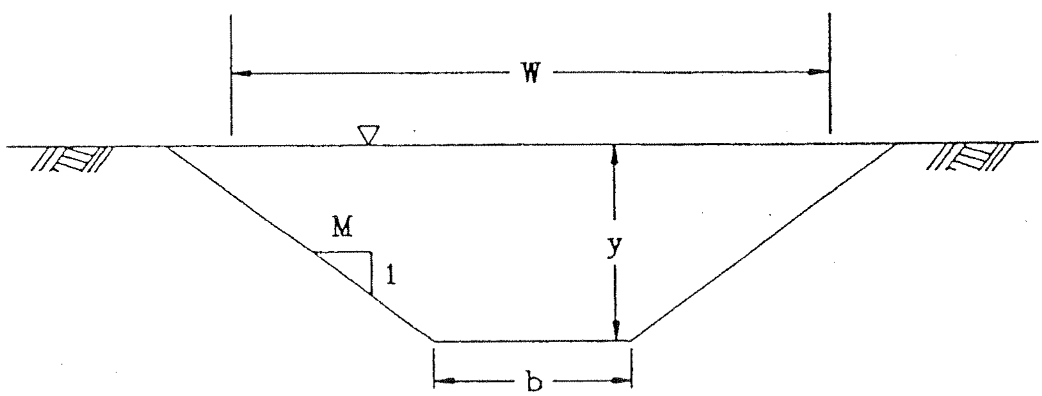


Figure 4-3 - Trapezoidal Channel - Definition of Variables

Equations (continued)

From the geometry of the channel cross-section and the Manning equation, design equations can be developed for determining the dimensions of the best hydraulic section for a trapezoidal channel.

The depth of the best hydraulic section is defined by:

$$y = C_M \left(Qn/(S^{1/2})\right)^{3/8} \qquad (4.4)$$

Where: $$C_M = \left[\frac{\{k + 2(M^2 + 1)^{1/2}\}^{2/3}}{1.49\ (k + M)^{5/3}}\right]^{3/8} \qquad (4.5)$$

The associated bottom width is:

$$b = ky \qquad (4.6)$$

The cross-sectional area of the resulting channel is:

$$A = by + My^2 \qquad (4.7)$$

Table 4-3 lists values of C_M and k for various values of M.

Table 4-3

Values of C_M and k for determining bottom width and depth of best hydraulic section for a trapezoidal channel.

M	C_M	k
0/1	0.790	2.00
0.5/1	0.833	1.236
0.577/1	0.833	1.155
1.0/1	0.817	0.828
1.5/1	0.775	0.606
2.0/1	0.729	0.472
2.5/1	0.688	0.385
3.0/1	0.653	0.325
3.5/1	0.622	0.280
4.0/1	0.595	0.246
5.0/1	0.522	0.198
6.0/1	0.518	0.166
8.0/1	0.467	0.125
10.0/1	0.430	0.100
12.0/1	0.402	0.083

4.7 Open Channel Flow Calculations

Design Charts
4.7.1

Following is a discussion of the equations that can be used for the design and analysis of open channel flow. The Federal Highway Administration has prepared numerous design charts to aid in the design of rectangular, triangular, and trapezoidal open channel cross sections. In addition, design charts for grass lined channels have been developed. For a complete discussion of these charts and their use in open channel design refer to the publication Design Charts For Open Channel Flow, Federal Highway Administration, Hydraulic Design Series No. 3, 1973.

Manning's
Evaluation
4.7.2

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

$$v = (1.49/n)R^{2/3}\,S^{1/2} \qquad (4.8)$$

$$Q = (1.49/n)A\,R^{2/3}\,S^{1/2} \qquad (4.9)$$

$$S = [(Qn/(1.49\,A\,R^{2/3})]^2 \qquad (4.10)$$

Where: v = average channel velocity (ft/s)

Q = discharge rate for design conditions (cfs)

n = Manning's roughness coefficient

A = cross-sectional area (ft^2)

R = hydraulic radius (ft)

P = wetted perimeter (ft)

S = slope of the energy grade line (ft/ft)

For prismatic channels, in the absence of backwater conditions, the slope of the energy grade line and channel bottom are equal.

Geometric
Relationships
4.7.3

Area, wetted perimeter, hydraulic radius, and channel top width for standard channel cross-sections can be calculated from their geometric dimensions. 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.

Direct Solutions 4.7.4

Nomographs for obtaining direct solutions to Manning's Equation are presented in Figures 4-4 and 4-5. Figure 4-4 provides a general solution for the velocity form of Manning's Equation, while Figure 4-5 provides a solution of Manning's Equation for trapezoidal channels.

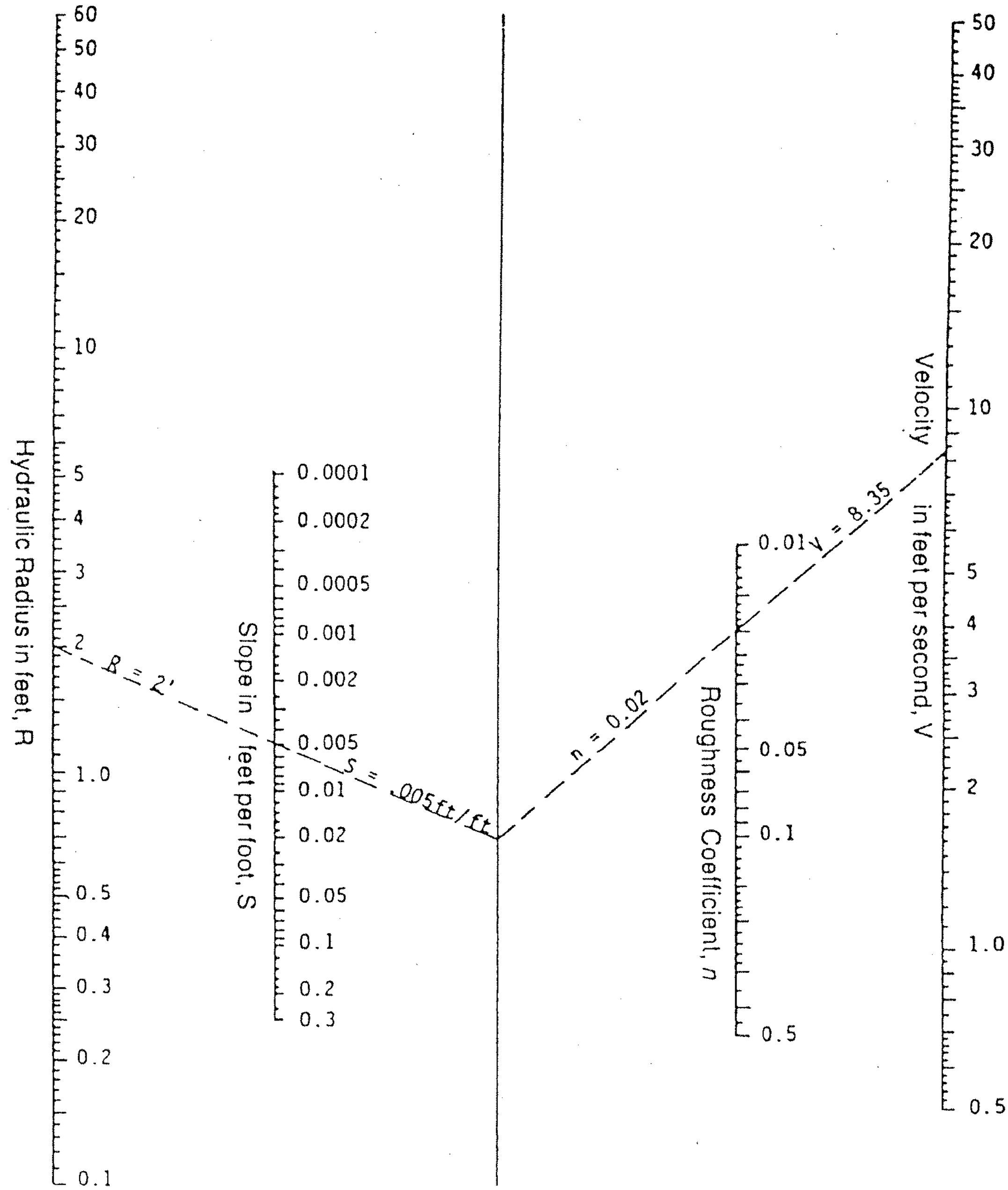


Figure 4-4
Nomograph for the Solution of Manning's Equation

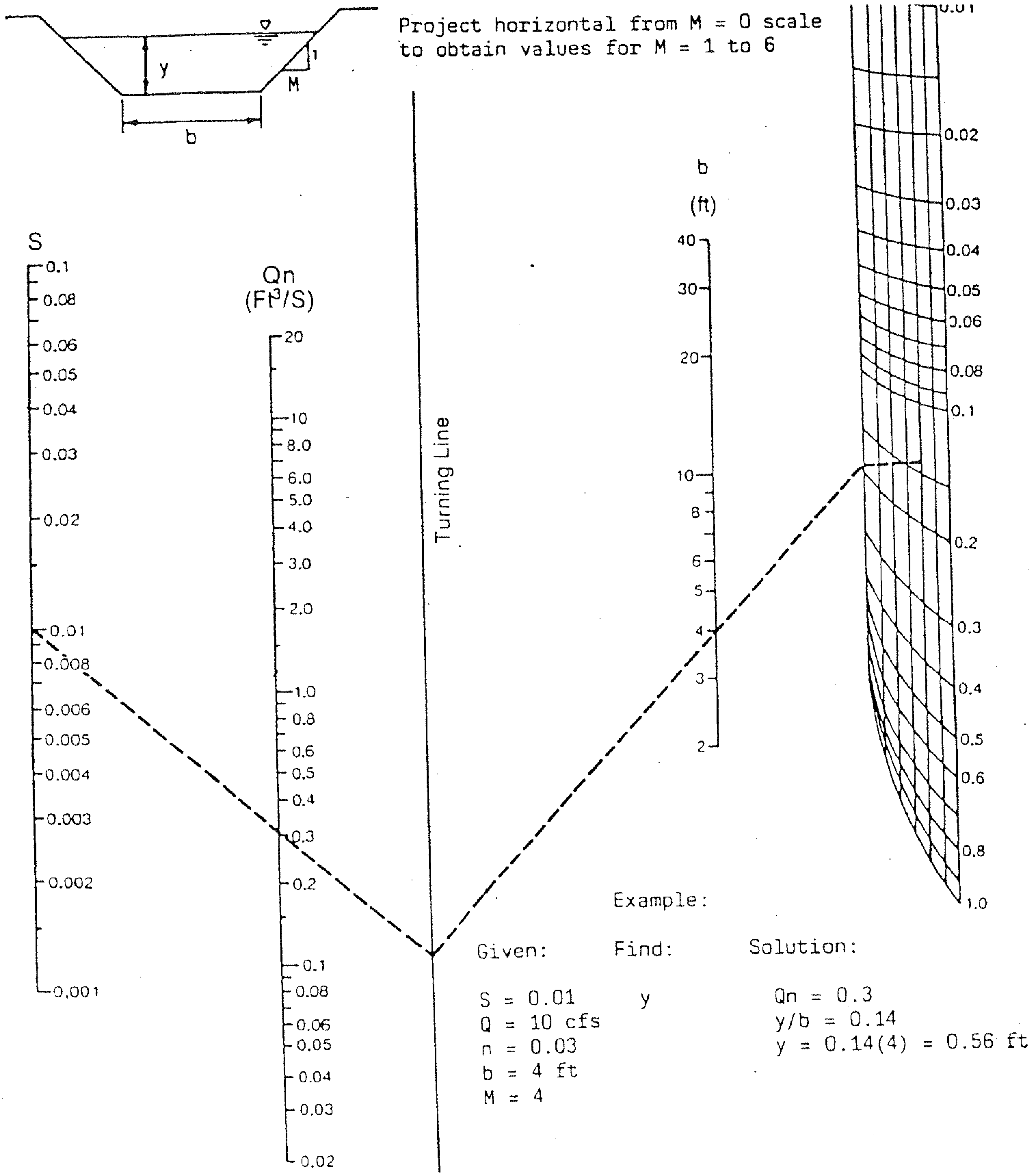


Figure 4-5

Solution of Manning's Equation for Trapezoidal Channels

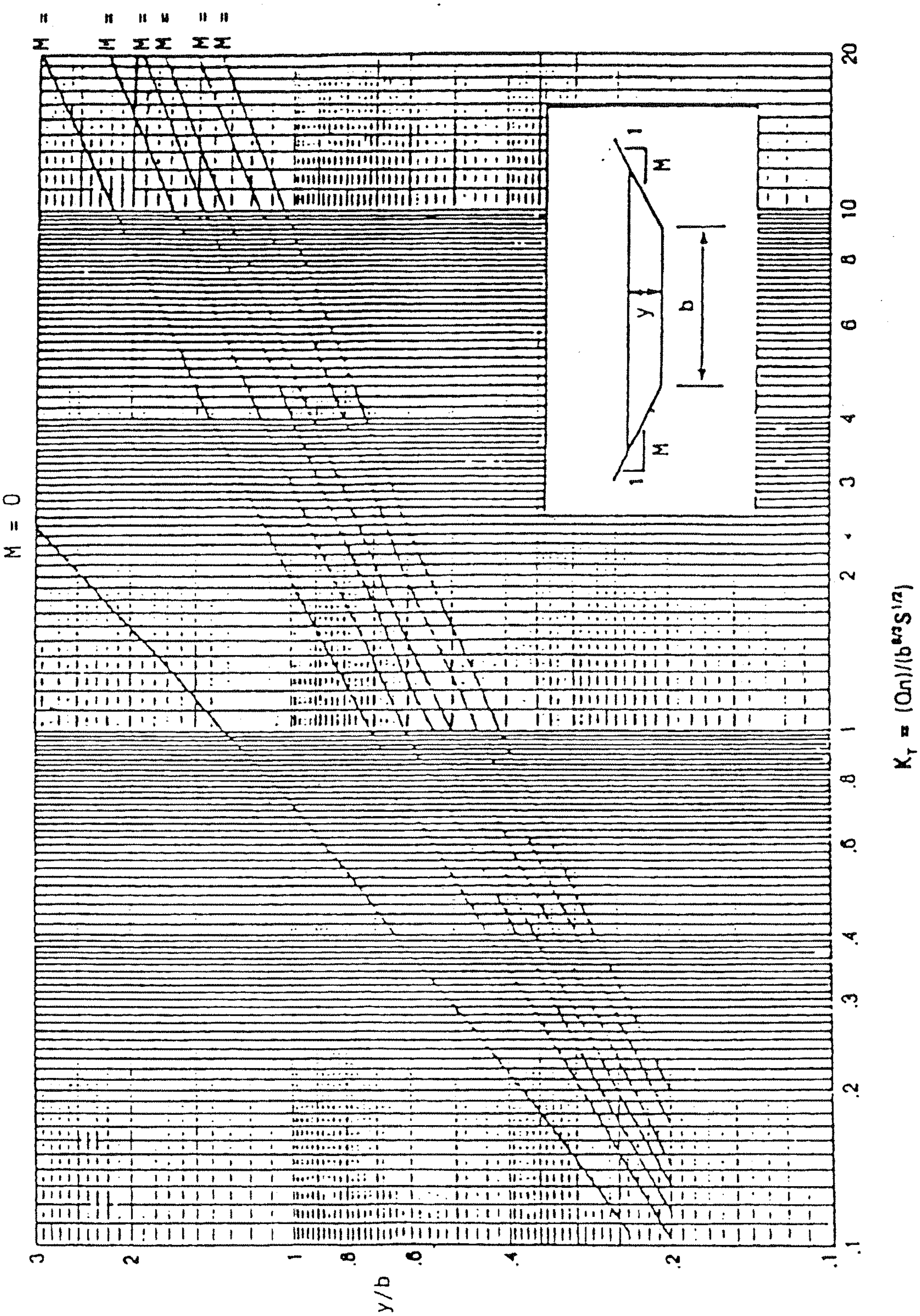


Figure 4-6

Trapezoidal Channel Capacity Chart

Normal Depth Solutions 4.7.5

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:

$$AR^{2/3} = (Qn)/(1.49\ S^{1/2}) \qquad (4.11)$$

Where: A = cross-sectional area (ft)

R = Hydraulic radius (ft)

Q = discharge rate for design conditions (cfs)

n = Manning's roughness coefficient

S = slope of the energy grade line (ft/ft)

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 4.11 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 4-6 for trapezoidal channels, which is described below.

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

2. Calculate the trapezoidal conveyance factor using the equation:

$$K_T = (Qn)/(b^{8/3}S^{1/2}) \qquad (4.12)$$

Where: K_T = Trapezoidal open channel conveyance factor

Q = Discharge rate for design conditions (cfs)

n = manning's roughness coefficient

b = bottom width (ft)

S = slope of the energy grade line (ft/ft)

3. Enter the x-axis of Figure 4-6 with the value of K_T calculated in Step 2 and draw a line vertically to the curve corresponding to the appropriate m value from Step 1.

Normal Depth Solutions (continued)

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, y/b.

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

4.8 Critical Flow Calculations

Background 4.8.1

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

$$Q^2/g = A^3/T \quad (4.13)$$

Where: Q = discharge rate for design conditions (cfs)

g = acceleration due to gravity (32.2 ft/sec^2)

A = cross-sectional area (ft^2)

T = top width of water surface (ft)

A trial and error procedure is needed to solve equation 4-13.

Semi-Empirical Equations 4.8.2

Semi-empirical equations (as presented in Table 4-4) or section factors (as presented in Figure 4-7) 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:

$$Z = Q/(g^{1/2}) \quad (4.14)$$

Where: Z = critical flow section factor

Q = discharge rate for design conditions (cfs)

g = acceleration due to gravity (32.2 ft/sec^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 p~rcent 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 type of flow should be assumed for the design.

Table 4-4 Critical Depth Equations for Uniform Flow In Channel Cross Sections

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

Where:
y_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
M = 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

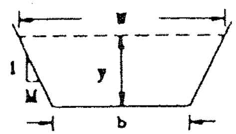
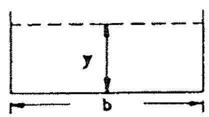
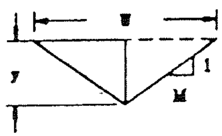
[a]See Figure 4-7 for channel sketches

[b]Assumes uniform flow with the kinetic energy coefficient eoual to 1.0.

[c]Reference: French (1985)

[d]Reference: USDOT, FHWA, HDS-4 (1965)

[e]Reference: Brater and King (1976)

Section	Area A	Wetted Perimeter, P	Hydraulic Radius, R	Top Width, W	Critical Depth Factor, Z
 Trapezoid	$by + My^2$	$b + 2y\sqrt{M^2 + 1}$	$\dfrac{by + My^2}{\sqrt{b + 2y\, M^2 + 1}}$	$b + 2My$	$\dfrac{[(b + My)y]^{1.5}}{\sqrt{b + 2My}}$
 Rectangle	by	$b + 2y$	$\dfrac{by}{b + 2y}$	b	$by^{1.5}$
 Triangle	My^2	$2y\sqrt{M^2 + 1}$	$\dfrac{My}{2\sqrt{M^2 + 1}}$	$2My$	$\dfrac{\sqrt{2}}{2} My^{2.5}$

Open Channel Geometric Relationships for Various Cross Sections

Figure 4–7

Froude Number 4.8.3

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

$$Fr = v / (gA/T)^{1/2} \qquad (4.15)$$

Where: Fr = Froude number (dimensionless)

v = velocity of flow (ft/s)

g = acceleration due to gravity (32.2 ft/sec^2)

A = cross-sectional area of flow (ft^2)

T = top width of flow (ft)

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

4.9 Open Channel Design

Procedures for designing open channels, both vegetative and riprap are found in Section 8.05 of the North Carolina Erosion and Sediment Control Planning and Design Manual.

4.10 Riprap Design

Assumptions 4.10.1

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. Maximum side slope is 2:1
2. Maximum allowable velocity is 14 feet per second

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

Procedure 4.10.2

Following are the steps in the procedure for riprap design.

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 4.7.5). Manning's n values for riprap can be calculated from the equation:

   $$n = 0.0395\ (d_{50})^{1/6} \qquad (4.17)$$

   Where: n = manning's roughness coefficient for stone riprap

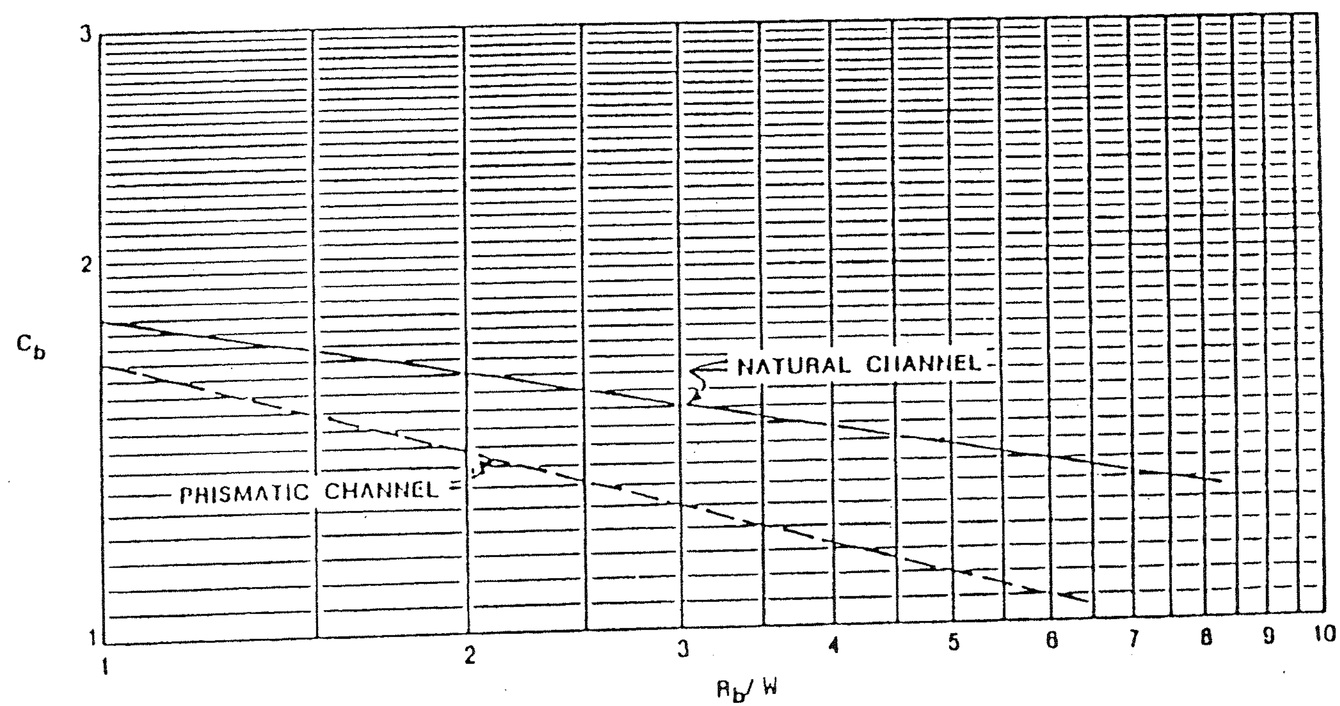
   d_{50} = diameter of stone for which 50 percent, by weight, of the gradation is finer (ft)

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 4-8 for either a natural or prismatic channel. This requires determining the channel top width, W, 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 4-9.

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

   $$d_{30}/D = 0.193\ Fr^{2.5} \qquad (4.18)$$



To obtain effective velocity, multiply known velocity by C .

W = Channel Top Width

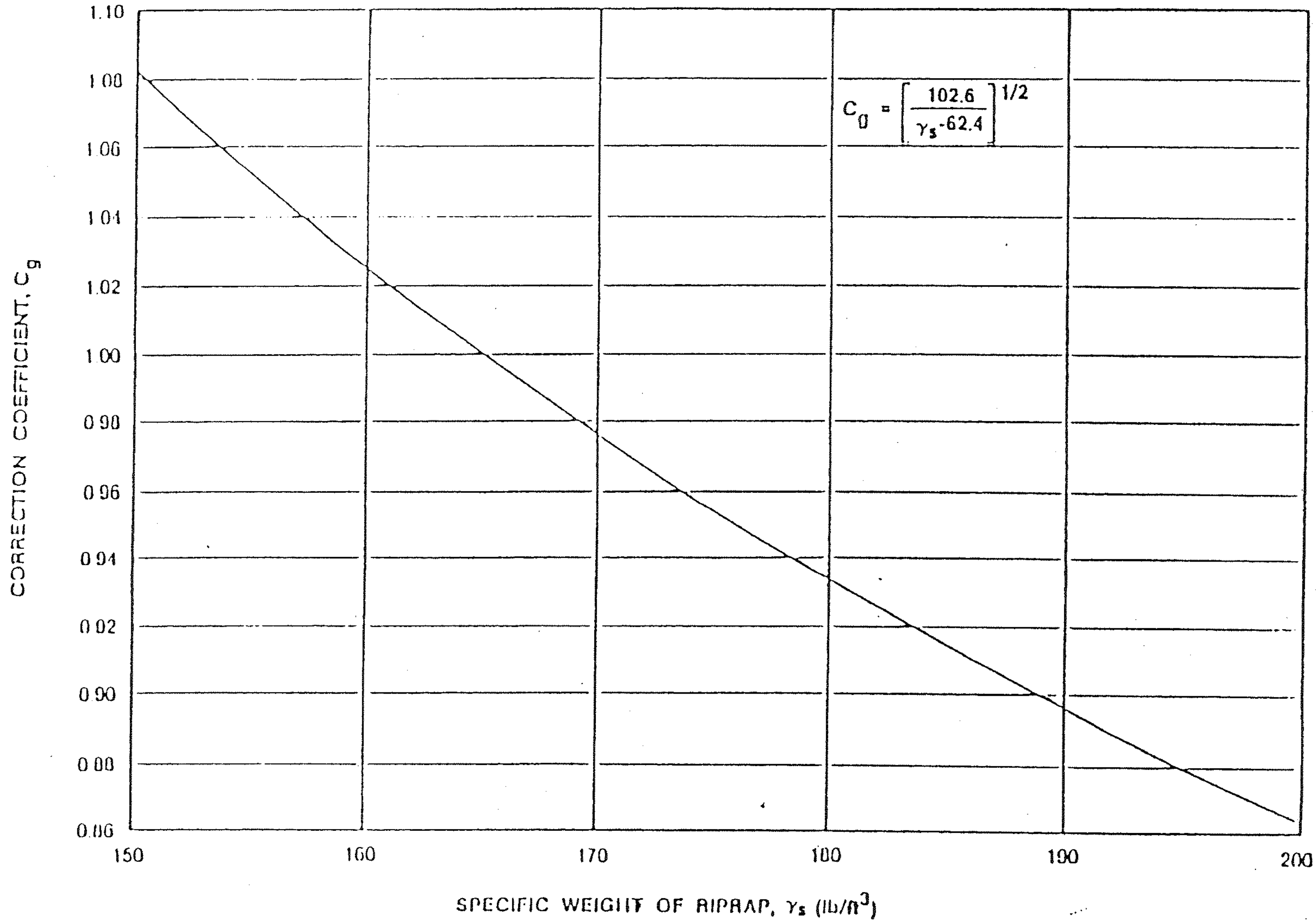
R_b= Centerline Bend Radius

C_b= Correction Coefficient

Reference: Maynord (1987)

Figure 4-8

Riprap Lining Bend Correction Coeffic



C_g = Correction Coefficient

To obtain effective velocity, multiply known velocity by C_g.

Figure 4–9

Riprap Lining Specific Weight Correction Coefficie

Procedure (continued)

Where: d_{30} = diameter of stone for which 30 percent, by weight, of the gradation is finer (ft)

D = depth of flow above stone (ft)

Fr = Froude number (see equation 4.15), dimensionless

v = mean velocity above the stone (ft/s)

g = acceleration due to gravity (32.2 ft/sec^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_{60} 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 equations:

$$w = 0.5236\ SW_s\ d^3 \tag{4.19}$$

Where: w = stone weight (lbs)

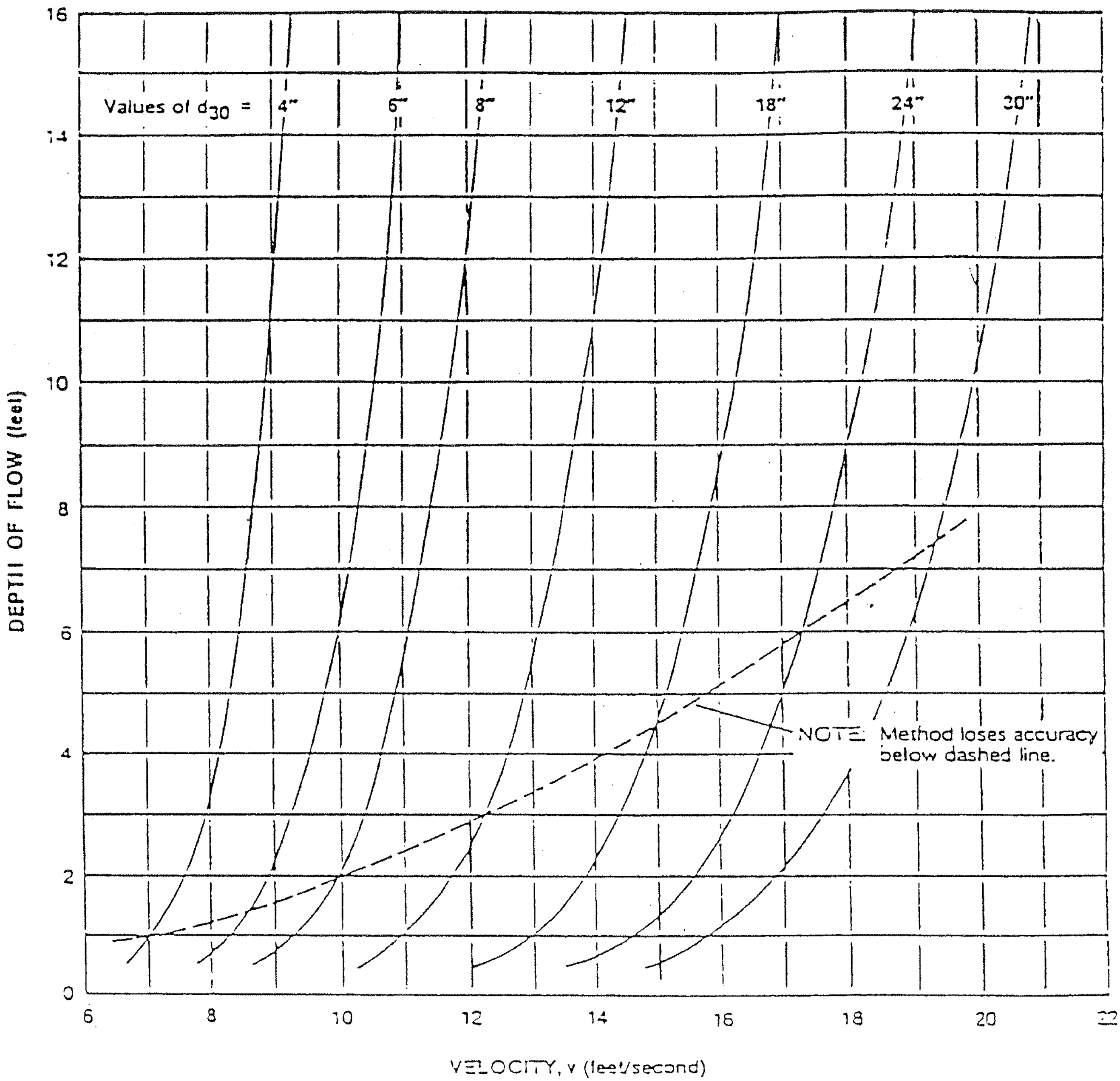
d = selected stone diameter (ft)

SW_s = specific weight of stone (lbs/ft^3)

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 4-13 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.

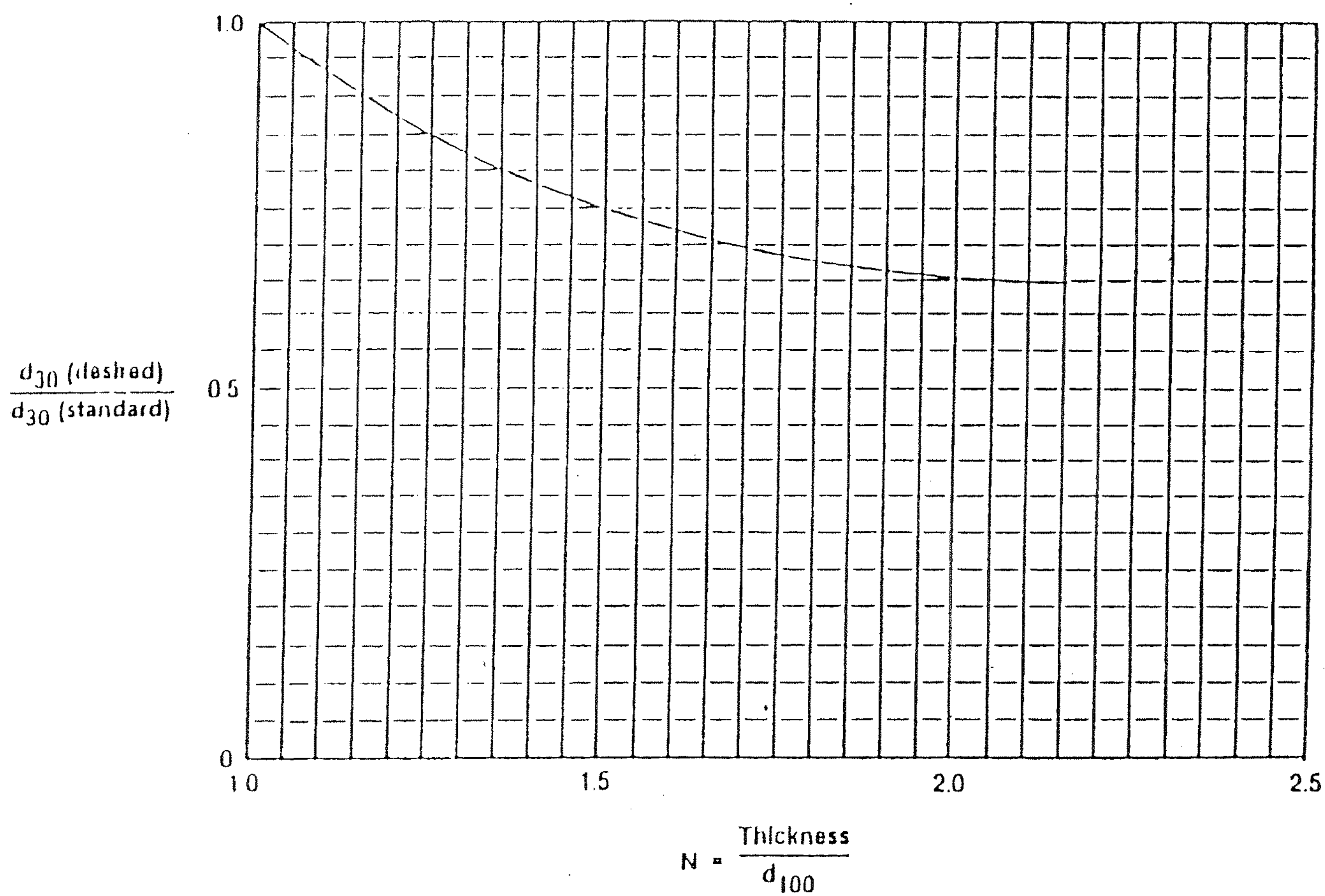
7. Perform preliminary design, ensuring that adequate transition is provided to natural materials both up and downstream to avoid flanking and that toe protection is provided to avoid riprap undermining.



Reference: Reese (1988).

Figure 4–10

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



Reference: Maynord (1987)

Figure 4-11

Riprap Lining Thickness Adjustment for d_{85}/d_{15} = 2.0 to 2.

4.11 Gradually Varied Flow

Introduction
4.11.1

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 programs are, HEC-2, developed by the U.S. Army Corps of Engineers (1982) and Bridge Waterways Analysis Model (WSPRO) developed for the Federal Highway Administration. These programs 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 more tedious and iterative process.The use of HEC-2 or WSPRO is recommended for standard step calculations.

Energy losses in transitions, junctions, and bends shall be accounted for as part of water surface profile 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 ft apart for ditches or streams and 500 ft apart for flood plains, unless the channel is very regular.

Direct Step Method
4.11.2

The direct step method is limited to prismatic channels. A form for recording the calculations described below is presented in Table 4-5 (Chow 1959).

1. Record the following parameters across the top of Table 4-5:

   Q = design flow (cfs)

   n = manning's n value

   S_o = channel bottom slope (ft/ft)

   α = energy coefficient

   y_c = critical depth (ft)

   y_n = normal depth (ft)

Location ______________

$Q =$ ________ $n =$ ________ $S_o =$ ________ $\alpha =$ ________ $y_c =$ ________ $y_n =$ ________

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.											
21.											
22.											

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

Water Surface Profile Computation Form for the Direct Step Method

Table 4–5

Direct Step Method (continued)

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 the results in columns 2, 3, and 4, respectively.

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

4. Compute specific energy, ΔE, in ft, by summing the Method 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:

$$S_f = (n^2 v^2)/(2.22 R^{4/3}) \tag{4.24}$$

   Where: S_f = friction slope (ft/ft)

   n = manning's n value

   v = average velocity (ft/s)

   R = hydraulic radius (ft)

   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, 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:

$$\Delta x = \Delta E/(S_o - S_f) = \text{Column 7/Column 10} \tag{4.25}$$

   Where: Δx = length of channel between consecutive depths of flow (ft)

   ΔE = change in specific energy (ft)

   S_o = bottom slope (ft/ft)

   S_f = friction slope (ft/ft)

   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.

Standard Step Method 4.11.3

The standard step method is a trial and error procedure applicable to both the natural and prismatic channels. The step computations are arranged in tabular form, as shown in Table 4-6 and described below (Chow, 1959):

1. Record the following parameters across the top of Table 4-6:

   Q = design flow (cfs)

   n = Manning's n value

   S_o = Channel bottom slope (ft/ft)

   α = energy coefficient

   k_e = eddy head loss coefficient (ft)

   y_c = critical depth (ft)

   y_n = normal depth (ft)

2. Record the location of measured channel cross sections and the trial water surface elevation, z, for each section in column 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 ft, and hydraulic radius, R, in ft. 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 ft/s. Record the result in column 6.

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

7. Compute the total head, H, in ft, 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 4.24 and record the result in column 9.

9. Determine the average friction slope, 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, S, (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.

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)	S_f (9)	$\bar{S}_f$ (10)	Δx (11)	h_f (12)	h_e (13)	H (14)
1.													
2.													
3.													
4.													
5.													
6.													
7.													
8.													
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18.													
19.													
20.													
21.													
22.													

(9) $S_f = \dfrac{n^2 v^2}{2.22\,R^{4/3}}$

(12) $h_f = \Delta x\,\bar{S}_f$

(13) $h_e = \dfrac{k_e v^2}{2g}$

Water Surface Profile Computation Form for the Standard Step Method

Table 4–6

Standard Step Method (continued)

12. Compute the eddy loss using the equation:

$$h_e = (k_e v^2)/2g \qquad (4.26)$$

Where: h_e = eddy head loss (ft)

k_e = eddy head loss coefficient (ft) (for prismatic and regular channels, k_e = 0; for gradually converging and diverging channels, k_e = 0 to 0.2; for abrupt expansions and contractions, k_e = 0.5)

v = average velocity (ft/s) (column 6)

g = acceleration due to gravity (32.2 ft/s^2)

13. 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 results in column 14.

14. 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.

4.12 Gradually Varied Flow- Example Problems

Example 1
4.12.1

Direct Step Method

Use the direct step method (Section 4.11.2) to compute a water surface profile for a trapezoidal channel using the following data:

Q = 400 cfs

B = 20 ft

M = 2

S = 0.0016 ft

n = 0.025

α = 1.10

A dam backs up water to a depth of 5 ft 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 4-7. Values in each column of the table are briefly explained below.

1. Depth of flow, in ft, arbitrarily assigned values ranging from 5 to 3.4 ft.
2. Water area in sq ft corresponding to the depth, y, in column 1.
3. Hydraulic radius, in ft, corresponding to y in column 1.
4. Mean velocity, in ft/s, obtained by dividing 400 cfs by the water area in column 2.
5. Velocity head, in ft, calculated using the mean velocity from column 4 and α = 1.1.
6. Specific energy, E, in ft, obtained by adding the velocity head in column 5 to the depth of flow in column 1.
7. Change of specific energy, delta E, in ft, 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 4.24, with n = 0.025, v as given in column 4, and R as given in column 3.
9. Average friction slope between the steps, $\bar{S}_f$, equal to the arithmetic mean of the friction slope computed in column 8 and that of the previous step.

DIRECT STEP METHOD

y (1)	A $by+My^2$ (2)	R A/P (3)	v Q/A (4)	$\alpha v^2/2g$ (5)	E (1)+(5) (6)	ΔE (7)	S_f (8)	$\bar{S}_f$ (9)	$S_o-\bar{S}_f$ (10)	Δx (7)/(10) (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	1146
3.70	101.38	2.77	3.948	0.2664	3.9664	0.0826	0.001132	0.001076	0.000524	158	1304
3.60	97.92	2.71	4.085	0.2856	3.8856	0.0808	0.001244	0.001188	0.000412	196	1500
3.55	96.21	2.68	4.158	0.2958	3.8458	0.0398	0.001310	0.001277	0.000323	123	1623
3.50	94.50	2.65	4.233	0.3067	3.8067	0.0391	0.001382	0.001346	0.000254	154	1777
3.47	93.48	2.63	4.278	0.3131	3.7831	0.0236	0.001427	0.001405	0.000195	121	1898
3.44	92.45	2.61	4.326	0.3202	3.7602	0.0229	0.001471	0.001449	0.000151	152	2050
3.42	91.80	2.60	4.357	0.3246	3.7446	0.0156	0.001500	0.001486	0.000114	137	2187
3.40	91.12	2.59	4.388	0.3292	3.7292	0.0154	0.001535	0.001518	0.000082	188	2375

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)

Example 1
(continued)

10. Difference between the bottom slope, S_o, 0.0016 and the average friction slope, S_f, in column 9.

11. Length of the reach, Δx, in ft, between the consecutive steps, computed by equation 4.25 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 2
4.12.2

Standard Step Method

Use the standard step method (see Section 4.11.3) to compute a water surface profile for the channel data and stations considered in the previous example. Assume the elevation at the dam site is 600 ft.

Results of the calculations, as obtained from Chow (1959), are reported in Table 4-8. 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 the previous example 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 ft and the height of the dam is 5 ft, the first entry is 605.00 ft. 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 ft, 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.

   $605.048 - 600.00 - (155)(0.0016) = 4.80$ ft

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

5. Hydraulic radius, R, in ft, 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 ft, corresponding to the velocity in column 6 and $\alpha = 1.1$.

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

Station	z	y	A	R	v	$\alpha v^2/2g$	H	S_f	$\bar{S}_f$	Δx	h_f	h_e	H
			$by+My^2$	A/P	Q/A		(2)+(7)						
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(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)

Example 2
(continued)

9. Friction slope, S_f, computed by equation 4.24, with n = 0.025, v from column 6, and R from column 5.

10. Average friction slope through the reach, $\bar{S}_f$, between the (continued) 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, Δ x, 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 column 10 and 11.

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

14. Elevation of the total head, H, in ft, 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.

4.13 Approximate Flood Limits

Introduction
4.13.1

For streams and tributaries with drainage areas smaller than one square mile, analysis may be required to identify the 100-year flood elevation and building restriction floodline. This requires a backwater analysis to determine the stream flow depth. Both HEC-2 and WSPRO methods are acceptable.

Floodline Restrictions
4.13.2

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. The maximum allowable distance between cross sections is 100 feet.

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

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.

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