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Chapter 5
Methods for Predicting n Values for the Manning Equation

5-1. Introduction

This chapter describes the prediction of the total Manning's roughness coefficient (n value) for a reach by establishing physically based component parts and determining the contribution from each. The following component parts were selected: bed roughness, bank roughness, surface irregularities, obstructions, vegetation roughness, and expansion/contraction losses.

5-2. Approach

Hydraulic roughness is a major source of uncertainty in water surface profile calculations. Field data at each project are required to confirm selected values. When field data are not available, the traditional approach is to use handbook methods or analytical methods to predict the hydraulic roughness values.

a. Handbook method. In this approach the engineer uses "calibrated photographs" and other subjective methods to associate hydraulic roughness values with conditions observed and anticipated in the project reach. Chow (1959) and Barnes (1967) are the dominant sources of calibrated photographs. More recently, Arcement and Schneider (1989) extended the work to include floodplains. Other sources, like hydraulics and agricultural handbooks, add variation but not much additional insight.

b. Analytical methods. A second approach for predicting roughness coefficients is to relate hydraulic roughness to the effective surface roughness and irregularity of the flow boundaries. This approach is called analytical methods in this chapter. The classic example is the Moody-type diagram for hydraulic roughness in open channel flow (Plate 3). The procedure shown in paragraph 2-2*c* is still the state of the art in n values for concrete-lined channels. It is based on the Keulegan equations for velocity distribution (Chow 1959). The Iwagaki relationship has been included in the determination of the coefficients for the roughness equations.

c. Grass-lined channels. Manning's n values for grass-lined channels were reported by the Soil Conservation Service (Chow 1959).

d. Mobile boundary channels. Simons and Richardson (1966) related bed forms in mobile boundary channels to stream power. These data indicate that a significant change can occur in n values as the stream bed changes from ripples to dunes to plane bed to antidune. Subsequently, work by Limerinos (1970) and Brownlie (1983) provided regression equations for calculating bed roughness in mobile boundary channels. Note that channel bed roughness is just one component of the total n value for a reach.

e. Compositing. The procedure for combining different roughnesses across a section into a single value for hydraulic computations is called compositing. The composited value may change if a different method for compositing is chosen. Therefore, the handbook methods are probably more dependable as sources of n values than the analytical methods because the compositing is included in the field observation.

5-3. Hydraulic Roughness by Handbook Methods

Arcement and Schneider (1989) summarize the state of the art in selecting n values for natural channels and flood plains. This work was performed for the U.S. Department of Transportation and subsequently will be called the USDT method in this chapter. The basic approach follows that proposed by Cowan (Chow 1959):

$$n = (n_b + n_1 + n_2 + n_3 + n_4)m \tag{5-1}$$

where

n_b = base n value

n_1 = addition for surface irregularities

n_2 = addition for variation in channel cross section

n_3 = addition for obstructions

n_4 = addition for vegetation

m = ratio for meandering

5-4. Base *n* Values (n_b) for Channels

On page 4 of their report, Arcement and Schneider state, "The values in [their] Table 1 for sand channels are for upper regime flows and are based on extensive laboratory and field data obtained by the U.S. Geological Survey. When using these values, a check must be made to ensure that the stream power is large enough to produce upper

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* regime flow." Although the base n values given in Table 5-1 for stable channels are from verification studies, the values have a wide range because the effects of bed roughness are extremely difficult to separate from the effects of other roughness factors. The choice of n values from Table 5-1 will be influenced by personal judgment and experience. The n values for lower and transitional regime flows are much larger generally than the values given in Table 5-1 for upper regime flow. Also, the vegetation density method of Petryk and Bosmajian (1975) is presented for the vegetation component n_4. Although the work was published in the mid-1970's, it has not received widespread attention in the profession. It has considerable appeal as a design procedure, however, and deserves additional evaluation.

a. Example. Figure 5-1 is the proposed design for a levee project in which the sponsor proposes vegetation along the project. The hydraulic roughness values for this section are estimated from several different handbook sources in Tables 5-1 and 5-2. Note that handbooks divide n values into two categories: channel bed and bank and flood plains.

b. Sensitivity of calculations to n values. The calculated water depth is shown in Table 5-3 using the mean values of both channel and overbank roughness. The mean values are considered to be the best estimate, statistically.

Both n values were increased by adding their standard deviation. The resulting water surface elevation increased about 0.7 ft, from 9.4 ft to 10.1 ft. This standard deviation in n values is really quite small. However, it demonstrates how sensitive water depth is to n value.

5-5. Hydraulic Roughness by Analytical Methods

Investigators continue to explore physically based hydraulic roughness equations. These are the methods in which hydraulic roughness is calculated from the effective surface roughness k_s. The new Hydraulic Design Package (SAM), under development at the U.S. Army Engineer Waterways Experiment Station (WES) (Thomas et al., in preparation), offers nine analytical methods for n values (Table 5-4). None of the n value equations account for momentum or bend losses. Presently, the only technique for bend losses is to increase the n values by a factor. Cowan (Chow 1959) proposed a multiplier in Equation 5-1, and both Chow and the USDT report suggest values to use. Scobey (Chow 1959) proposed increasing the n value by 0.001 for each 20 degrees of curvature. Chow suggested that should not exceed a total of 0.002 even in flumes having pronounced curvature.

a. Effective surface roughness height k_s. For the design of concrete channels, Corps of Engineers values for k_s are shown in Chapter 2 (Table 2-1). Chow (1959) gives a table of k_s values (Table 8-1) for other boundary materials such as k_s for natural rivers. Please note that, at this point in time, the profession has not adopted tables of k_s values as they have Manning's n values. Moreover, there is no generally accepted technique for measuring this property geometrically. Therefore, the use of Table 8-1 is discouraged. Instead, use the Strickler or the Keulegan equations and calculate k_s from available sources of Manning's n value. (Note: These equations do not necessarily give the same results.)

b. Relative roughness. Relative roughness refers to the ratio of the effective surface roughness height, k_s to the hydraulic radius R. The relative roughness parameter is R/k_s.

c. Strickler equation, rigid bed. The Strickler function (Chow 1959) is shown in Figure 5-2. Notice that the effective surface roughness height k_s is correlated with the D_{50} of the bed sediment in this figure. However, k_s can be correlated with other measures of the surface roughness depending on what is representative of the surface roughness height of the boundary materials. For example, riprap research at WES has shown that the Strickler equation (Equation 5-2) will give satisfactory n values when k_s is taken to be the D_{90} of the stone.

$$n = C\, k_s^{1/6} \qquad (5\text{-}2)$$

where

C = 0.034 for riprap size calculations where $k_s = D_{90}$

= 0.038 for discharge capacity of riprapped channels where $k_s = D_{90}$

= 0.034 for natural sediment where $k_s = D_{50}$ (Chow 1959)

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Table 5-1
Hydraulic Roughness, Channel Bed and Banks

Reference	m	n_b	n_1	n_2	n_3	n_4	n
USDT (Arcement and Schneider 1989), pp 4 & 7	1.0	0.024	0.002	0.002	0.001	0.005	0.034
Barnes (1967), p 78	-	0.037	-	-	-	-	0.034
Chow (1959), p 109, Table 5-5, Fine Gravel	1.0	0.024	0.005	0.0	0.0	0.00	0.034
Chow (1959), p 112, Table 5-6, D-1a3	-	0.040	-	-	-	-	0.040
Chow (1959), p 120, Figure 5-5(14)	-	0.030	-	-	-	-	0.030
Brater and King (1976), p 7-17, Natural	-	0.035	-	-	-	-	0.035
Mean							0.035
Standard deviation							0.003

Note:

$n = (n_b + n_1 + n_2 + n_3 + n_4)m$

where

n_b = base n-value
n_1 = addition for surface irregularities
n_2 = addition for variation in channel cross section
n_3 = addition for obstructions
n_4 = addition for vegetation
m = ratio for meandering

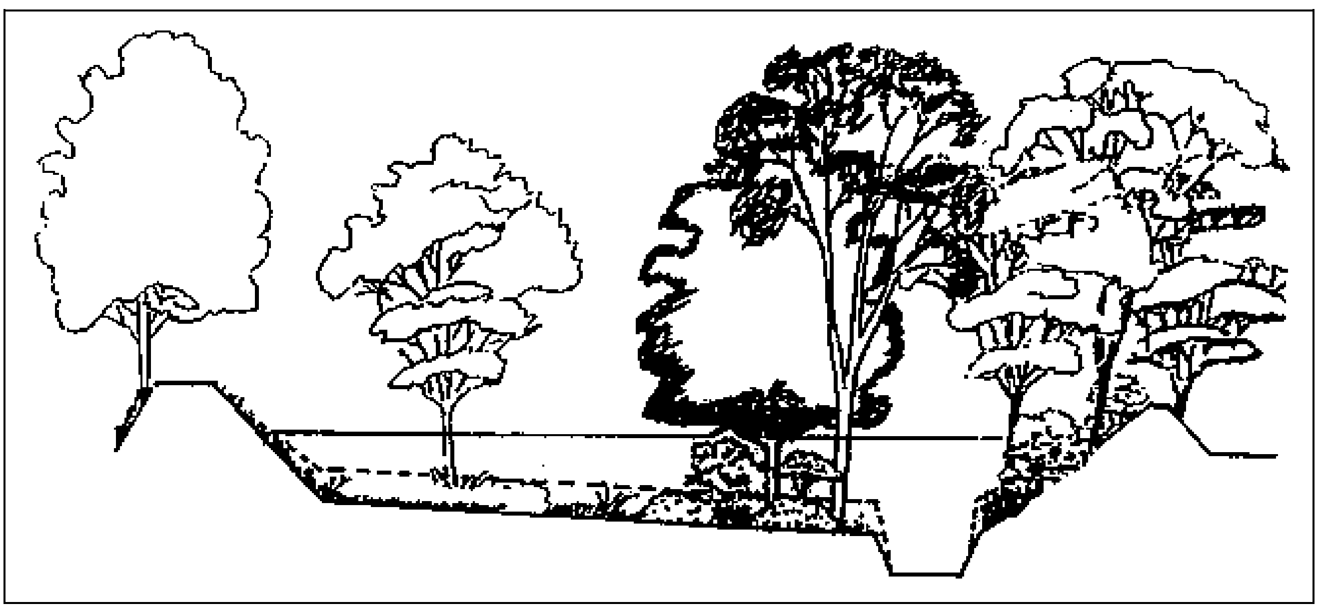

Figure 5-1. Design cross section

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Table 5-2
Hydraulic Roughness, Floodplain

Reference	n_b	n_1	n_2	n_3	n_4	n
USDT (Arcement and Schneider 1989), pp 4 & 9	0.028	0.010	-	0.012	0.050	0.100
Barnes (1967), None Given	-	-	-	-	-	-
Chow (1959), p 113, Table 5-6, D-2c5	0.100	-	-	-	-	0.100
Chow (1959), p 123, Figure 5-5(23)	0.125	-	-	-	-	0.125
Brater and King (1976), None Given	-	-	-	-	-	-
Mean						0.108
Standard deviation						0.012

Note: Same n value equation as channel bed and banks.

Table 5-3
Sensitivity of Depth to n Value

Case	Channel	n Value: Flood-plain	n Value: Water Surface
Mean	0.035	0.108	9.4
+1 Standard Deviation	0.038	0.120	10.1

Table 5-4
n Value Equations and Compositing Methods in SAM

n Value Equations	Methods for Compositing
Manning's n	Alpha Method
Keulegan	Equal Velocity Method
Strickler	Total Force Method
Limerinos	Total Discharge Method
Brownlie	
Grass E[1]	
Grass D[1]	
Grass C[1]	
Grass B[1]	
Grass A[1]	

Note: [1] Grass type described in Table 5-7.

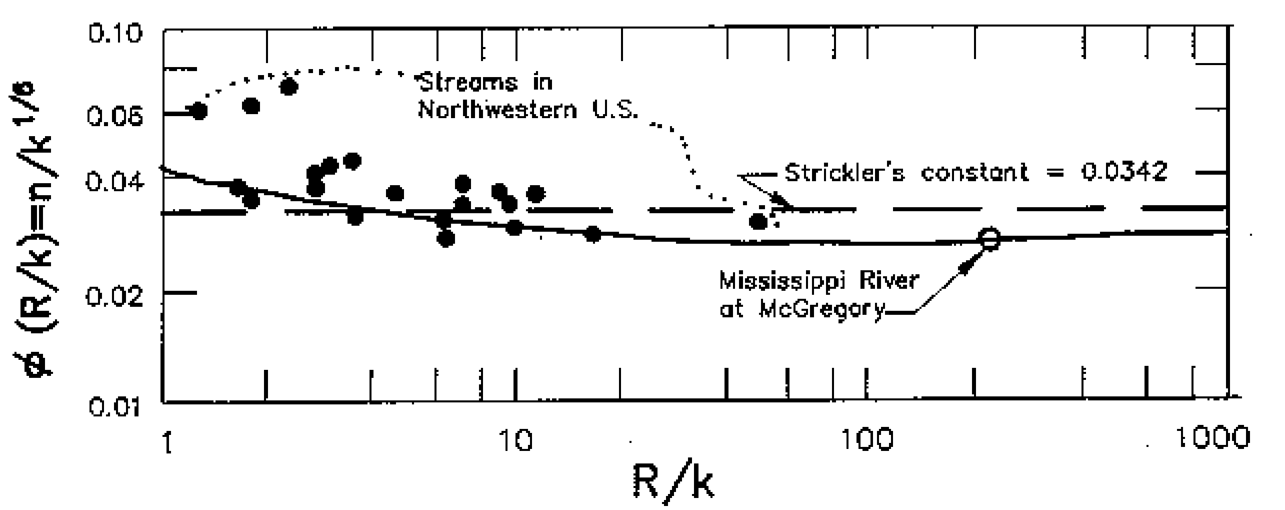

Figure 5-2. The Strickler function (Chow 1959) (courtesy of McGraw-Hill Book Company, Inc.)

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* *d. Keulegan equations, rigid bed.* The procedure in Chapter 2 is still the state of the art in n values for rigid boundary channel design. It is a relative roughness approach based on the Keulegan equations for velocity distribution (Chow 1959). Keulegan classified flow types as hydraulically smooth flow, hydraulically rough flow, and a transition zone. His equations, presented in Chapter 2 and repeated as follows, are written in terms of the Chezy coefficient because of the simpler powers involved. The conversion to Manning's n value follows.

(1) The equation for fully rough flow is

$$C = 32.6 \log_{10}\left(\frac{12.2\ R}{k}\right) \tag{2-6 bis}$$

(2) For smooth flow the equation is

$$C = 32.6 \log_{10}\left(\frac{5.2\ R_n}{C}\right) \tag{2-5 bis}$$

(3) The equation showing the relationship of n value and Chezy C is (see Equation 2-4)

$$n = \frac{1.486}{C} R^{1/6} \tag{5-3}$$

where

R_n = Reynolds number

= 4RV/ν

where

V = average flow velocity

ν = kinematic viscosity of water

and 32.6, 12.2 and 5.2 are empirical coefficients determined from laboratory experiments. These equations, when graphed, produce a Moody-type diagram for open channel flow (Plate 3).

e. The Iwagaki relationship.

(1) Chow presents Keulegan's equation for the average flow velocity V in the following form

$$V = U_* \left[6.25 + 5.75 \log_{10}\left(\frac{R}{k_s}\right)\right] \tag{5-4}$$

where

U_* = boundary shear velocity = $\sqrt{gRS}$

g = acceleration of gravity

S = slope

6.25 = coefficient for fully rough flow

(2) Substituting a variable, A_r, for the constant, 6.25, substituting the Chezy equation for velocity, and substituting $\sqrt{gRS}$ for U_* gives

$$\frac{V}{U_*} = \frac{C}{\sqrt{g}} = A_r + 5.75 \log_{10}\left(\frac{R}{k_s}\right) \tag{5-5}$$

$$C = \sqrt{g}\left[A_r + 5.75 \log_{10}\left(\frac{R}{k_s}\right)\right] \tag{5-6}$$

The form shown in Chapter 2 can be written as follows:

$$C = 32.6 \log_{10}\left[10^{\frac{A_r\sqrt{g}}{32.6}}\left(\frac{R}{k_s}\right)\right] \tag{5-7}$$

where A_r is the Iwagaki coefficient for rough flow.

From Keulegan's study of Bazin's data, the value of A_r was found to have a wide range, varying from 3.23 to 16.92. Thus, a mean value of 6.25 for A_r may be used.

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* "A further study was made by Iwagaki on experimental data obtained from many sources. The results of the study have disclosed that resistance to turbulent flow in open channels becomes obviously larger than that in pipes with increase in the Froude number. Iwagaki reasoned that this is due to the increased instability of the free surface at high Froude numbers" (Chow 1959, p 204).

(3) The Iwagaki relationship is shown in Figure 5-3.

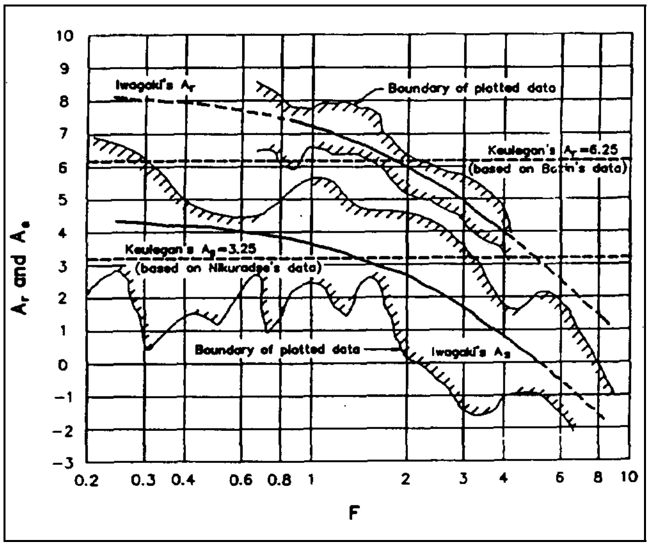

Figure 5-3. The Iwagaki relationship (Chow 1959) (courtesy of McGraw-Hill Book Company, Inc.)

(4) The comparable form of the equation for smooth flow is

$$C = 32.6 \log_{10} \left[10^{\frac{A_s \sqrt{g}}{32.6}} \left(\frac{\sqrt{g} R_n}{4C} \right) \right] \tag{5-8}$$

where A_s is the Iwagaki coefficient for smooth flow.

f. A_r and A_s coefficients.

(1) The A_r and A_s coefficients are shown graphically in Figure 5-3, but the equations for the curves were not provided. It can be shown that the equation for A_r is of the form

$$A_r = -27.058 \log_{10} (F + 9) + 34.289 \tag{5-9}$$

where F is the Froude number. Data ranged from 0.2 < F < 8.0.

(2) Using an equation of the same form, the relationship for A_s is

$$A_s = -24.739 \log_{10} (F + 10) + 29.349 \tag{5-10}$$

(3) When the values of A_r and A_s are 6.2411 and 3.25, the coefficients in the roughness equations are 12.2 and 5.2, respectively. These are the values shown in Equations 2-5 and 2-6. Using Equations 5-9 and 5-10, those values correspond to Froude numbers of 1.88 and 1.35, respectively.

g. Transition zone. The limit of the fully rough zone is

$$\frac{R_n / C}{R / k_s} > 50 \tag{5-11}$$

The roughness equation in the transition zone is a combination of the equations for smooth and fully rough flow as follows:

$$C = -32.6 \log_{10} \left(\frac{4C}{\sqrt{g} R_n 10^{\frac{A_s \sqrt{g}}{32.6}}} + \frac{k_s}{R 10^{\frac{A_r \sqrt{g}}{32.6}}} \right) \tag{5-12}$$

h. Comparison of n-values, from Strickler and Keulegan equations. Table 5-5 is a comparison of n values calculated by the Strickler and Keulegan equations. Flow is fully rough. Notice the Strickler equation uses the effective surface roughness height k_s, and not relative roughness. Therefore, the n value does not vary with hydraulic radius R. On the other hand, the Keulegan equation uses relative roughness, and that requires both k_s and R. The constant in the Strickler equation, 0.034, is that recommended by Chow (1959). The resulting n values match the Keulegan results adequately. For example, the k_s for concrete is 0.007. That converts to an n value of 0.015 using Strickler and to 0.014-0.018 using Keulegan.

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Table 5-5
n Values Calculated by Strickler and Keulegan Equations

Effective Roughness				Keulegan Equation R, ft				
k_s, mm	k_s, ft	Strickler $n = 0.034 \cdot k_s^{1/6}$	F	1	5	10	20	50
			8	0.012	0.013	0.014	0.015	0.016
0.10	0.0003281	0.009	1.88	0.010	0.011	0.012	0.013	0.014
			0.2	0.009	0.011	0.011	0.012	0.013
			8	0.017	0.017	0.018	0.019	0.020
1.00	0.003281	0.013	1.88	0.013	0.014	0.015	0.015	0.017
			0.2	0.012	0.013	0.014	0.015	0.016
			8	0.019	0.019	0.020	0.020	0.021
2.13	0.007	0.015	1.88	0.014	0.015	0.016	0.017	0.018
			0.2	0.013	0.015	0.015	0.015	0.018
			8	0.026	0.025	0.025	0.025	0.026
10	0.03281	0.019	1.88	0.018	0.018	0.019	0.019	0.020
			0.2	0.016	0.017	0.017	0.018	0.019
			8	0.049	0.037	0.035	0.034	0.033
64	0.20997	0.026	1.88	0.026	0.024	0.024	0.025	0.025
			0.2	0.022	0.022	0.022	0.022	0.023
			8	0.060	0.042	0.039	0.037	0.036
100	0.3281	0.028	1.88	0.029	0.026	0.026	0.026	0.027
			0.2	0.024	0.023	0.023	0.024	0.024
			8	0.084	0.048	0.043	0.041	0.039
152.4	0.500	0.030	1.88	0.033	0.029	0.028	0.028	0.028
			0.2	0.027	0.025	0.025	0.025	0.026
			8	____	____	0.092	0.073	0.061
1,000	3.2808	0.041	1.88	____	____	0.043	0.040	0.039
			0.2	____	____	0.036	0.034	0.034

Note:

$$C = 32.6 \log_{10} \left(Coef_2 \cdot R/k_s \right)$$

$$Coef_2 = 10^{\left(\sqrt{g} \cdot A_r/32.6 \right)}$$

$$A_r = 27.058 \cdot \log_{10} \left(F + 9 \right) + 34.289$$

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* *i. Bed roughness in mobile boundary streams.*

(1) In mobile boundary channels the bed roughness is composed of grain roughness and form roughness. The grain roughness refers to the effective surface roughness height of the mixture of sediment particles on the streambed. Form roughness refers to bed features described as ripples, dunes, transition, plain bed, standing waves, and antidunes. These bed features, called bed forms, are grouped into the general categories of lower regime, transitional, and upper regime.

(2) Regime, in this usage of the term, does not refer to whether the flow is sub- or supercritical. The Froude number may remain less than 1, and the bed regime may still shift from lower to upper and back. Neither does it refer to channel dimensions, flow velocity, nor slope. It is simply the category of bed forms that are contributing to the hydraulic roughness. However, the amount of hydraulic loss produced by bed form roughness may exceed that produced by grain roughness. Therefore, it cannot be ignored.

(3) The significant difference between mobile boundary streams and rigid boundary streams is in the requirement to predict when the bed forms change from one regime to another. It seems to be related to flow velocity, flow depth, water temperature, and effective sediment particle size.

(4) Two functions are presented in this chapter for calculating n values in mobile boundary channels: Limerinos (1970) and Brownlie (1983). However, only the Brownlie method includes predicting the change from one bed regime to the other. These relationships are described in more detail in the following paragraphs.

(5) It is important to establish which portion of the channel cross section is bed and which is bank because the bed roughness predictors apply only to the channel bed. That is, typically the vegetation roughness and bank angle do not permit the bed load to move along the face of the banks. Therefore, the Limerinos and Brownlie n value equations should not be used to forecast bank roughness.

(6) On the other hand, the point bar is a natural source-sink zone for sediment transport. Consequently, it is a location at which the Limerinos and Brownlie equations apply.

j. Limerinos n-value predictor, mobile bed.

(1) Limerinos developed an empirical relative roughness equation for coarse, mobile bed streams using field data (Limerinos 1970). He correlated n values with hydraulic radius and bed sediment size. The following equation resulted:

$$n = \frac{0.0926\ R^{1/6}}{1.16 + 2.0\ \log_{10}\left(\frac{R}{d_{84}}\right)} \qquad (5\text{-}13)$$

where

n = Manning's n value. Data ranged from 0.02 to 0.10.

R = hydraulic radius, ft. Data ranged from 1 to 6 ft.

d_{84} = the particle size, ft, for which 84 percent of the sediment mixture is finer. Data ranged from 1.5 to 250 mm.

(2) Data were from relatively wide, straight streams having a simple trapezoidal shape and no overbank flow. There was very little increase in width with depth, and the banks were stable. Irregularity was minimal. The amount of vegetation on the bed and banks was negligible.

(3) Grain sizes in Limerinos's data ranged from very coarse sand to large cobbles. The objective was to select field sites at which the bed forms would not change with flow hydraulics during the measurement. Consequently, it follows that this equation is applicable to gravel/cobble bed streams and to bed regimes similar to those found in such streams.

(4) N values predicted with the Limerinos equation are sufficiently larger than those predicted by the Strickler equation to indicate that some loss other than grain roughness must have been present. However, the Limerinos equation is not applicable to lower regime flow nor does it forecast the transition between upper and lower regimes.

(5) Burkham and Dawdy (1976) showed the Limerinos equation could be used in sand bed streams provided the regime was plain bed. In that analysis they *

* extended the range of the relative roughness parameter as follows:

$$600 < \frac{R}{d_{84}} < 10,000$$

k. Comparison of Strickler and Limerinos n values.

(1) Table 5-6 shows n values calculated by the Strickler and the Limerinos equations. For a hydraulic radius of 1 ft, the Limerinos values are higher than Strickler's by 15 to 57 percent.

(2) Furthermore, for k_s up to about 10 mm the Limerinos n values increase with depth, which is the same trend as seen in the Keulegan n values in Table 5-5. However, the Limerinos n values are larger than Keulegan's by 7 to 52 percent. These consistent differences lead one to suspect some bed irregularities in Limerinos' field data in addition to grain roughness.

(3) Arcement and Schneider (1989, p 6) state, "If a measured d_{84} is available or can be estimated, [Limerinos] may be used to obtain a base n for sand channels in lieu of using Table 1." However, n values calculated by Limerinos, shown in Table 5-6 herein, are considerably smaller than the values shown in Table 1 of Arcement and Schneider even though they state their Table 1 is for upper regime flow.

l. The Brownlie bed-roughness predictor, mobile bed.

(1) In sediment transport calculations it is important to link n values to the bed regime. This is particularly true when hydraulic conditions shift between upper regime and lower regime flow. There are several methods in Vanoni (1975) that express n value in terms of sediment parameters, but Brownlie (1983) is the only method that calculates the transition. This method post-dates Vanoni (1975).

(2) Brownlie sought to reconstitute the most fundamental process--the discontinuity in the graph of hydraulic radius versus velocity (Figure 5-4). In the process of this research, he collected the known sediment data sets--77 in all, containing 7,027 data points. Of the total, 75 percent were from flume studies and 25 percent from field tests. He used 22 of these data sets and demonstrated a significant agreement with both field and laboratory data.

(3) Brownlie's basic equations were modified for SAM to display bed roughness as a coefficient times the grain roughness.

$$n = [\text{BED FORM ROUGHNESS}] \times [\text{STRICKLER GRAIN ROUGHNESS}] \qquad (5\text{-}14)$$

Table 5-6
n Values Calculated by Strickler and Limerinos Equations

Effective Roughness			Limerinos Equation R, ft				
k_s, mm	k_s, ft	Strickler $n = 0.034 \cdot k_s^{1/6}$	1	5	10	20	50
0.10	0.0003281	0.009	0.011	0.013	0.013	0.014	0.015
1.00	0.003281	0.013	0.015	0.016	0.017	0.017	0.019
2.13	0.007	0.015	0.017	0.018	0.018	0.019	0.020
10	0.03281	0.019	0.022	0.022	0.022	0.023	0.024
64	0.20997	0.026	0.037	0.031	0.030	0.030	0.030
100	0.3281	0.028	0.044	0.034	0.033	0.032	0.032
152.4	0.5	0.030	0.053	0.038	0.036	0.035	0.034

Note:

Limerinos Equation: $$n = \frac{0.0926\ R^{1/6}}{1.16 + 2 \cdot \log\ (R/k_s)}$$

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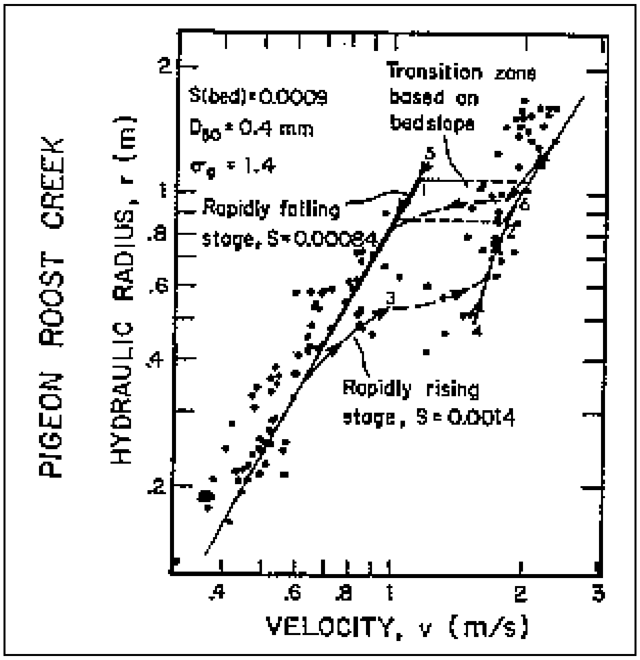

Figure 5-4. Velocity versus hydraulic radius in a mobile bed stream (courtesy of W. M. Keck Laboratories of Hydraulics and Water Resources (Brownlie 1981))

This makes it easy to compare the results with the skin friction for fixed bed systems as presented in Plate 3. The resulting forms of the equations for lower and upper regimes are as follows:

(a) Lower regime flow:

$$n = \left[1.6940 \left(\frac{R}{d_{50}} \right)^{0.1374} S^{0.1112} \sigma^{0.1605} \right] 0.034 (d_{50})^{0.167} \tag{5-15}$$

(b) Upper regime flow:

$$n = \left[1.0213 \left(\frac{R}{d_{50}} \right)^{0.0662} S^{0.0395} \sigma^{0.1282} \right] 0.034 (d_{50})^{0.167} \tag{5-16}$$

where

R = hydraulic radius, ft, of the bed portion of the cross section

d_{50} = the particle size, ft, for which 50 percent of the sediment mixture is finer

S = bed slope. Probably the energy slope will be more representative if flow is nonuniform.

σ = the geometric standard deviation of the sediment mixture (is shown as σ_g in Figure 5-4)

$$\sigma = 0.5 \left(\frac{d_{84}}{d_{50}} + \frac{d_{50}}{d_{16}} \right) \tag{5-17}$$

(c) Transition function: If the slope is greater than 0.006, flow is always upper regime. Otherwise, the transition is correlated with the grain Froude number as follows:

$$F_g = \frac{V}{\sqrt{(s_s - 1)\, g\, d_{50}}} \tag{5-18}$$

$$F_g' = \frac{1.74}{S^{1/3}} \tag{5-19}$$

If $F_g \le F_g'$, then lower regime flow

If $F_g > F_g'$, then upper regime flow

where

F_g = grain Froude number

V = velocity of flow

s_s = specific gravity of sediment particles

The transition occurs over a range of hydraulic radii and not at a point. Over this range, then, it is a double-valued function, and the transition test will give different regimes depending on which equation is being solved for roughness at that iteration. That is realistic since one expects the rising side of a hydrograph to trigger the transition at a different discharge than does the falling side.

m. Soil Conservation Service (SCS) n values, grass cover. Hydraulic roughness curves for five types of grass cover were published by SCS (US Department of Agriculture 1947) (Figure 5-5). Each curve type, A

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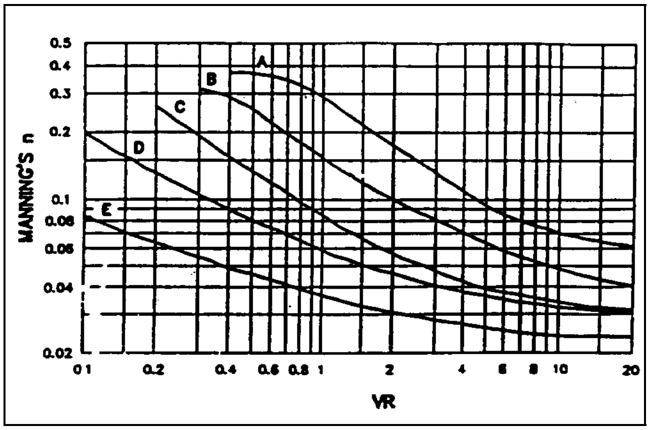

Figure 5-5. n value relationships for grass cover

through E, refers to grass conditions described in Table 5-7.

n. Example. To use analytical methods, the engineer is faced with assigning physically based parameters, like surface roughness or material type, to each subdivided area in a cross section. The subdivided areas are bounded by vertical lines between successive coordinate points on the boundary and the water surface. Table 5-8 illustrates the development of n values for the cross section in Figure 5-1 by the application of analytical equations. The analytical methods are in the Hydraulic Design Package SAM. The cross section is coded as station and elevation starting at the levee on the left, Area 1.

Table 5-7
Characteristics of Grass Cover

Type	Cover	Condition
A	Weeping love grass	Excellent stand, tall (average 30 in.)
	Yellow bluestem *(Andropogon ischaemum)*	Excellent stand, tall (average 36 in.)
B	Kudzu	Very dense growth, uncut
	Bermuda grass	Good stand, tall (average 12 in.)
	Native grass mixture (little bluestem, blue grama, other long and short midwest grasses)	Good stand, unmowed
	Weeping love grass	Good stand, tall (average 24 in.)
	Lespedeza sericea	Good stand, not woody, tall (average 19 in.)
	Alfalfa	Good stand, uncut (average 11 in.)
	Weeping love grass	Good stand, mowed (average 13 in.)
	Kudzu	Dense growth, uncut
	Blue grama	Good stand, uncut (average 13 in.)
C	Crabgrass	Fair stand, uncut (10 to 48 in.)
	Bermuda grass	Good stand, mowed
	Common lespedeza	Good stand, uncut (average 11 in.)
	Grass-legume mixture--summer (orchard grass, redtop, Italian ryegrass, and common lespedeza)	Good stand, uncut (6 to 8 in.)
	Centipede grass	Very dense cover (average 6 in.)
	Kentucky bluegrass	Good stand, headed (6 to 12 in.)
D	Bermuda grass	Good stand, cut to 2.5-in. height
	Common lespedeza	Excellent stand, uncut (average 4.5 in.)
	Buffalo grass	Good stand, uncut (3 to 6 in.)
	Grass-legume mixture--fall, spring (orchard grass, redtop, Italian ryegrass, and common lespedeza)	Good stand, uncut (4 to 5 in.)
	Lespedeza sericea	After cutting to 2-in. height; very good stand before cutting
E	Bermuda grass	Good stand, cut to 1.5-in. height
	Bermuda grass	Burned stubble

*

*

Table 5-8
Hydraulic Roughness from Surface Properties

Area No.	Station	Elevation	n Value	k_s, ft	Comment
	0.0	18.00			
1					Grass D: Bermuda grass cut to 2.5 in. From Soil Conservation Service (Chow 1959, pp 179-184)
	50.0	5.50			
2			0.100		Left Floodplain, (USDT (Arcement and Schneider 1989), Table 3) $n = (n_b + n_1 + n_2 + n_3 + n_4)$ $= (0.028+0.010+0.012+0.050)$
	125.0	2.00			
3				1	Strickler k_s-ft; Assumed (Chow, p 206)
	129.0	0.00			
4					Brownlie bed roughness equations (Brownlie 1983) D_{84} = 6.5 mm, D_{50} = 1.7 mm, D_{16} = 0.4 mm
	154.0	0.00			
5				1	Same as left bank (Area 3)
	158.0	2.00			
6			0.125		Right Floodplain, (USDT (Arcement and Schneider 1989), Table 3) n = (0.028+0.010+0.012+0.075)
	168.0	5.50			
7					Same as left levee (Area 1)
	218.0	18.00			

(1) Area 1 is designed to be a mowed grass surface. The n value will depend on the flow depth and velocity over the panel.

(2) Area 2 is the left floodplain. The best source for n values in large, woody vegetation is the USDT procedure, referenced in Table 5-2. Therefore, that n value will be coded directly.

(3) Area 3 is the left bank of the channel. Roughness will be calculated by estimating a surface irregularity k_s for the bank line to be 1 ft.

(4) For Area 4, the channel bed roughness will be calculated from the bed sediment gradation using the Brownlie bed roughness equations. That method predicts whether the roughness is lower or upper regime. It uses the d_{84}, d_{50}, and d_{16} grain sizes of the bed surface.

(5) Area 5 is the right bank. It will be the same as the left bank.

(6) Area 6 is expected to have a more dense stand of vegetation than on the left side.

(7) Area 7, the right levee, will be the same as the left levee.

5-6. Composite n Values and Hydraulic Radius

The calculations that transform the complex geometry and roughness into representative one-dimensional hydraulic parameters for flow depth calculations are called compositing hydraulic parameters. That is, in a complex cross section the composite hydraulic radius includes, in addition to the usual geometric element property, the variation of both depth and n values. There are several methods in the literature for compositing. The Alpha method, described in Appendix C, was selected as the default for SAM. Two other methods are provided as options: equal velocity and sum of forces.

a. Equal velocity method. Cox (1973) tested three methods for determining the equivalent roughness in a rectangular channel: the equal velocity method, which is sometimes called the Horton or the Einstein method after the developers; the Los Angeles District method; and the Colbatch method.

*

* (1) Perhaps a more rational method for vertical walls is the equal velocity method. It was proposed independently by Horton and by Einstein (Chow 1959), and is one which prevents dividing by zero.

$$\bar{n} = \frac{\left(p_1 n_1^{1.5} + p_2 \; n_2^{1.5} + ... + p_N n_N^{1.5}\right)^{2/3}}{P^{\,2/3}} \qquad (5\text{-}20)$$

where

$\bar{n}$ = the composite n value for the section

p_N = wetted perimeter in subdivided area n

n_N = n value in subdivided area n

N = the last subdivided area in the cross section

P = total wetted perimeter in the cross section

Since only wetted perimeter, and not hydraulic radius, appears in this equation, it is always well behaved.

(2) The equations for the Los Angeles District (Equation 5-21) and Colbatch (Equation 5-22) methods (Figure 5-6) are as follows:

$$\bar{n} = \frac{\left(a_1 n_1 \; + \; a_2 n_2 \; + \; \cdots \; + \; a_N n_N\right)}{A} \qquad (5\text{-}21)$$

$$\bar{n} = \frac{\left(a_1 n_1^{1.5} \; + \; a_2 n_2^{1.5} \; + \; \cdots \; + \; a_N n_N^{1.5}\right)^{2/3}}{A^{\,2/3}} \qquad (5\text{-}22)$$

where

a_N = end area associated with subdivided area n

A = total area in cross section

As a result of these experiments, Cox concluded that Horton's method was not as accurate as the Los Angeles District method or the Colbatch method. Based on one of Cox's figures, the Horton method gave a composite n value as much as 8 percent higher than measured for the combination of rough walls and a smooth bed. One test, a combination of smooth walls and a rough bed, gave an effective n value about 4 percent lower than measured.

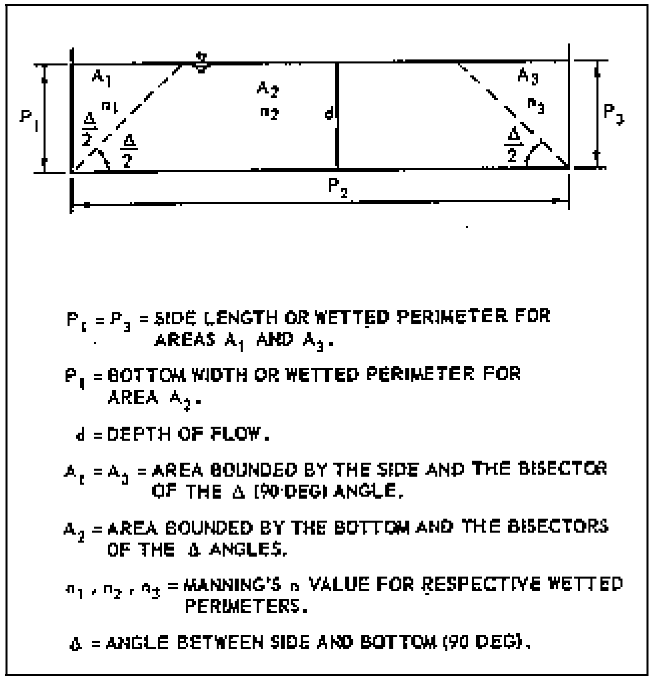

Figure 5-6. Definition sketch for Los Angeles District and Colbatch methods

(3) Horton's method is retained here because of its simplicity. It is adequate for the simple cross-section shapes, and it is programmable for the complex cross-section shapes. The other methods that Cox tested would be very difficult to program for automatic computations in complex cross sections.

b. Alpha method.

(1) The Chezy equation forms the basis for this method. The cross section is subdivided into areas between coordinate points.

(2) Calculations always begin at the first area in the cross section, and the geometric properties are calculated and saved for each wet area across the section. The hydraulic radius and Chezy C are then calculated and the compositing parameters summed. Computations move area by area to the end of the cross section.

(3) The alpha method fails when there is a vertical wall.

(4) James and Brown reported that the "Manning or Chezy equations do not accurately predict the stage-discharge relation in a channel-floodplain configuration for shallow depths on the floodplain (1.0 < Y/D < 1.4; *

* where Y = main channel depth and D = main channel bank full depth) without adjustments to either the resistance coefficient or the hydraulic radius.... the effects of geometry seem to disappear at the higher stages, i.e., for Y/D > 1.4, it no longer became necessary to make any correction to the basic equations" (James and Brown 1977, p 24).

c. Sum of forces method. This method was proposed by Pavlovskii, by Muhlhofer, and by Einstein and Banks (Chow 1959). It is based on the hypothesis that the total force resisting the flow is equal to the sum of the forces resisting the flow in each area. The resulting composite n value is

$$\bar{n} = \frac{\sqrt{p_1 n_1^2 + p_2 n_2^2 + \cdots + p_N n_N^2}}{P^{1/2}} \tag{5-23}$$

d. Conveyance method. The traditional approach to compositing by the conveyance method requires the cross section to be subdivided into subsections between channel and overbanks. Conveyance is calculated for each subsection as follows:

$$K_i = \frac{1.486\, A_i R_i^{2/3}}{n_i} \tag{5-24}$$

where

K_i = conveyance in subsection i

A_i = end area of subsection i

R_i = hydraulic radius in subsection i

n_i = n value in subsection i

The composite n value is calculated from the total conveyance and the hydraulic radius as follows:

$$\bar{n} = \frac{1.486\ AR^{2/3}}{K} \tag{5-25}$$

where

A = total end area of cross section

R = hydraulic radius for the entire cross section

'

= A/P

K = total conveyance of cross section = $K_1 + K_2 + \ldots\ldots\ldots + K_n$

e. Example. Flow depth calculations using n values calculated by the analytical methods are shown in Tables 5-9 through 5-11. Note the column headed "n_i value" in Table 5-10. The value for each area is shown, and at the bottom of that column the composited value for the entire cross section is 0.062. Table 5-11 shows the equivalent n value for the conveyance method to be 0.051. It is important not to mix n values determined by different compositing methods.

5-7. Expansion and Contraction in a 1-D Model

If the handbook approach is used, the expansion and contraction losses are included in the n_2 term. That is the contribution from variation in cross sections. Therefore, if contraction and expansion coefficients are being used, leave that term out.

If the analytical methods are used, no terms for expansion or contraction will be included. They would have to be added separately--perhaps by increasing the k_s value. Values from the n_2 component in the handbook method would be appropriate. They would have to be included in k_s.

5-8. Unforeseen Factors

a. Seasonality. This affects water temperature and vegetation. Both can cause significant changes in n value.

b. Tubeworms and barnacles. The Corps built a concrete channel in Corte Madera Creek only to find that marine creatures called tubeworms were attracted to it. They create a substantial increase in the surface roughness in the zone below sea level. Rather than the usual k_s of 0.007 ft, WES estimated the zone with the tubeworms had a k_s of 0.08 ft (Copeland and Thomas 1989).

c. Roughness from gravel moving in a concrete channel. In recent experiments at WES, gravel movement was modeled along a hard bottom flume to determine how much the n value would increase (Stonestreet, Copeland, and McVan 1991). As long as it moved, the increase was only about 10 percent. That was the case for concentrations up to about 3,000 ppm. When the concentration exceeded that, bed deposits began to form. That effect on *

*

Table 5-9
Water Surface Elevations Using the Alpha Method
Normal Depth Using Composite Properties by Alpha Method

****	N	Discharge cfs	Water Surface Elevation ft	Top Width ft	Composite R ft	Slope ft/ft	Composite n Value	Velocity fps	Froude Number	Boundary Shear Stress psf
****	1	2,300.00	9.58	150.6	7.77	0.000800	0.0621	2.64	0.17	0.39

Table 5-10
Water Surface Elevations Using the Alpha Method
Flow Distribution by Alpha Method, Discharge = 2,300.00 cfs

Station	Percentage Increase Discharge	Area A_i sq ft	Wetted Perimeter p_i ft	$R_i = A_i/p_i$	k_s ft	n_i Value	Velocity fps
0.0							
	3.06	33.2	16.8	1.98	1.179	0.0312	2.11
50.0							
	25.74	437.0	75.1	5.82	624.9	0.1000	1.35
125.0							
	7.10	34.3	4.5	7.67	1.000	0.0342	4.76
129.0							
	51.31	239.4	25.0	9.58	4.563	0.0383	4.93
154.0							
	7.10	34.3	4.5	7.67	1.000	0.0342	4.76
158.0							
	2.64	58.3	10.6	5.50	2,384.0	0.1250	1.04
168.0							
	3.06	33.2	16.8	1.98	1.179	0.0312	2.11
218.0							
	100.00	869.9	153.2	7.77	18.59	0.0621	2.64

Table 5-11
Water Surface Elevations Using the Alpha Method
Equivalent Hydraulic Properties using Conveyance Method

Hydraulic Radius Velocity ft	Manning's n Value	Discharge cfs	Subsection Area sq ft	Velocity fps
5.68	0.0506	2300.00	869.86	2.64

*

* n value is very significant and requires a sedimentation investigation.

d. Bed form roughness in concrete channels. After the Corte Madera Creek channel went into operation, sediment deposited over the smooth concrete bed in the downstream portion. A sedimentation study was conducted, after the fact, using HEC-6 (Copeland and Thomas 1989). They determined the channel n value to be 0.028 using high-water marks and the known water discharge. The calculated depth and gradation of bed deposits matched prototype values very nicely. This n value is not suggested as a design value. It is presented to illustrate surprises that can come from a fixed-bed hydraulic approach.

e. Large woody debris. Large woody debris refers to downed trees and log jams. This is a condition that exists, but its effect on the hydraulic roughness during large floods is not well documented.

f. Wetlands. Measurements by the South Florida Water Management District in connection with the restoration of the Kissimmee River produced n values of 1.011. That coincided with flow depths below the top of the marsh vegetation. They chose to use an n value of 0.3 for the levee design calculations because the flow depth was considerably above the top of the dense marsh vegetation. However, that was judgment rather than experiment. (Once flow depth exceeds the top of vegetation, it seems reasonable to reduce n values.)

g. Marsh. Studies for a flood at Kawanui Marsh, Hawaii, resulted in an n value of 0.95. That is attributed to a dense vine that was growing on the water surface. It was attached to the bed from place to place, but when the flood occurred, it piled the vine into accordion-like folds. Subsequent measurements, on smaller floods, were used to develop the n value.

*