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**Part 654 Stream Restoration Design
National Engineering Handbook**

Chapter 9   Alluvial Channel Design

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Cover photo: In an alluvial channel, there is a continual exchange of the channel boundary material with the flow.

Advisory Note

Techniques and approaches contained in this handbook are not all-inclusive, nor universally applicable. Designing stream restorations requires appropriate training and experience, especially to identify conditions where various approaches, tools, and techniques are most applicable, as well as their limitations for design. Note also that product names are included only to show type and availability and do not constitute endorsement for their specific use.

Chapter 9   Alluvial Channel Design

Contents

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654.0900 Purpose

Alluvial channel design techniques are generally used for movable boundary systems and streams with beds and banks made of unconsolidated sediment particles. In an alluvial channel, there is a continual exchange of the channel boundary material with the flow. Therefore, the design of an alluvial channel as part of a restoration project requires an assessment of sediment continuity and channel performance for a range of flows. A wide variety of sources and techniques are available to the designer for designing stable alluvial channels. This chapter provides an overview and discussion of some of the most common alluvial channel design techniques. The use and application of regime, analogy, hydraulic geometry, and analytical methods are presented and described. Examples have been provided to illustrate the methods.

654.0901 Introduction

The channel geometry and flow conditions in an alluvial stream are interrelated. The river's shape and size are determined by the river itself through the processes of erosion, sediment transport, sedimentation, and resuspension. Alluvial rivers are free to adjust section, pattern, and profile in response to hydraulic changes. Alluvial streams flow through channels with bed and banks made of sediments transported by the stream under-current conditions. In alluvial streams, the independent variables that drive the hydraulic design of the channel are discharge, sediment inflow, and bed and bank-material composition. The dependent or design variables are width, depth, slope, and planform.

Alluvial channel design approaches fall into five general categories: regime, analogy, hydraulic geometry, extremal, and analytical methods. Each method has its advantages and disadvantages, depending on the stream reach being restored.

A summary of alluvial channel design methods described in this chapter is presented in table 9–1. The table summarizes the basic theory and assumptions behind each method, input requirements for using the method, and basic limitations associated with each method. Table 9–1 is a general guide, recognizing that exceptions will be encountered. Designs can become complex, especially in wood-dominated systems or when some of the necessary input data is contradictory or missing. When there is uncertainty regarding the appropriate technique, it is recommended that the designer use several of what appear to be the most appropriate techniques and look for agreement on critical design elements.

Table 9–1   Characteristics of alluvial channel hydraulic design methods

Design method	Theory and assumptions	Requirements	Limitations
Regime	Dependent channel dimensions of width, depth, and slope can be determined from regression relationships with independent variables of channel-forming discharge, bed gradation, and sediment-inflow concentration. Based on the assumption that alluvial canals will evolve to the same stable channel dimensions, given the same independent driving variables	Channel-forming discharge and inflowing sediment concentration must be estimated. Bed and bank characteristics must be determined from field evaluations	Applicability is limited to channels similar to those used to develop the regression equations. Most of the data came from irrigation canals. Froude numbers should be less than 0.3, sediment transport low, and discharge relatively uniform, similar to flow in canals
Analogy	Channel dimensions from a reference reach can be transferred to another location. Based on the assumption that alluvial streams will evolve to the same stable channel dimensions, given the same independent driving variables	Reference reach must be stable and alluvial. Reference reach must have same channel-forming discharge, valley slope, and similar bed and bank characteristics. Watershed conditions must be similar	Difficult to find a suitable reference reach, especially in developed watersheds. Dependent design variables from the reference reach must be used as a combined set
Hydraulic geometry	Dependent channel dimensions of width, depth, and slope can be determined from regression relationships with independent variables. Independent variables may include one or more of the following: channel-forming discharge, drainage area, bed gradation, bank conditions, or sediment-inflow concentration. Based on the assumption that alluvial streams will evolve to the same stable channel dimensions, given the same one or two independent driving variables	Regression curves must be developed from stable and alluvial reaches and from physiographically similar watersheds. Channel-forming discharge must be estimated. Bed and bank characteristics must be determined from field evaluations	Applicability is limited to channels similar to those used to develop the regression equations. There is a high degree of uncertainty associated with the assumptions that (1) channel dimensions can be determined by a single independent variable; and (2) and with the determination of the channel-forming discharge. Design is only for the channel-forming discharge. Modifications may be required to convey higher flows. Sediment transport is typically low

Table 9–1   Characteristics of alluvial channel hydraulic design methods—Continued

Design method	Theory and assumptions	Requirements	Limitations
Extremal hypothesis	Alluvial channels will adjust channel dimensions so that energy expenditure is minimized. Depth and sediment transport can be calculated from physically based equations including continuity, hydraulic resistance and sediment transport. Typically, these equations are based on the assumptions of fully turbulent, hydraulically rough and gradually varied flow	Channel-forming discharge and inflowing sediment concentration must be estimated. Estimates of bed-material gradation and resistance coefficients must be obtained. Appropriate hydraulic resistance and sediment transport equations must be solved simultaneously, which requires a computer program or detailed spreadsheet analysis	Support for the extremal hypothesis is divided. Many stable alluvial channels exist at conditions different from the computed extremal condition
Analytical	Depth and sediment transport can be calculated from physically based equations including continuity, hydraulic resistance, and sediment transport. Typically, based on the assumptions of fully turbulent, hydraulically rough, gradually varied flow	Channel-forming discharge and inflowing sediment concentration must be estimated. Bank characteristics must be determined from field evaluations. Estimates of bed-material gradation and resistance coefficients must be obtained. Appropriate hydraulic resistance and sediment transport equations must be solved simultaneously, which requires a computer program or detailed spreadsheet analysis	A family of solutions is obtained from the hydraulic resistance and sediment transport equations. Another method must be used to obtain the third independent variable

654.0902 Alluvial channel design variables

Alluvial channels are different from threshold channels in that the channel boundary is mobile, and sediment transport is significant. The National Engineering Handbook (NEH) 654.08 presents a basic overview of threshold channel design techniques. In an alluvial channel design, stability depends on both the channel geometry and composition of the boundary materials. Alluvial channels are capable of adjustment. Stable natural alluvial channels typically form their geometry by moving boundary material. Channel-forming discharge is typically used to determine preliminary channel dimensions, but the full range of expected discharges should be used to determine final dimensions. The hydraulic design variables of width, depth, slope, and planform are the primary dependent variables in an alluvial channel (table 9–2). Their magnitudes are determined by the independent variables of sediment inflow, water inflow, and bank composition. The downstream water surface elevation is an independent variable that could have a significant effect on the dependent variables in some cases. Boundary resistance along the channel banks and sometimes along the bed can be both dependent and/or independent, depending on local circumstances.

Design of alluvial irrigation canals has traditionally been accomplished using regime methods. Regime methods rely on regression equations that are used to determine the dependent variables. The independent variables of discharge and sediment concentration are single-valued functions and, therefore, are applicable to cases where the discharge is relatively uniform with time. Regime methods are applicable for low-energy systems with low sediment transport.

The design philosophy for an alluvial channel to be designed as a natural stream, as part of a restoration project, is to employ both geomorphic principles and physically based analytical techniques to determine the design variables. Average magnitudes for width, depth, and slope are determined first. Planform and other features such as riffles, pools, and habitat enhancement structures are added later. The initial or preliminary average channel geometry is determined using a single channel-forming discharge.

Sizing the channel for the channel-forming discharge promotes channel stability. Project constraints may not allow the channel geometry to fit the dimensions suggested by the channel-forming discharge, but an effort should be made to be as close as possible to the stable channel geometry to reduce project maintenance costs. Later in the design process, a full range of discharges is used to evaluate the channel design and emulate the full range of natural discharges. The initial design, however, may need to be adjusted.

Analytical techniques are employed to ensure that the combinations of design variables are compatible. With three unknowns, three equations are required to determine the magnitude of each design variable. A hydraulic resistance equation, such as Manning's equation, can be one design equation. A sediment transport equation, such as Meyer-Peter and Müller's equation can be the second design equation. Resistance and sediment transport equations are well established and can be used with a reasonable level of confidence in the design process. One additional equation is needed. Four alternatives are considered to determine this third equation: analogy methods, hydraulic geometry relationships, constraint of one of the variables, or adopting an extremal hypothesis.

Table 9–2   Alluvial channel design variables

Dependent variables	Independent variables
Width	Sediment inflow
Depth	Water inflow
Slope	Bank composition
Planform	

654.0903 Regime methods

Regime methods were introduced by British engineers in the late nineteenth century to design and operate extensive irrigation systems in India. These canals were excavated into fine sand-bed material and carried their design discharge within the channel. Sediment entered the canals through the canal head works. The objective of channel design was to set the channel dimensions so that the inflowing sediment load would be passed without significant scour or deposition. Channels that carried their design flow without significant degradation or aggradation were said to be in regime. Data collected from these regime channels were used to develop relationships between hydraulic and sediment variables deemed to be significant. Most regime formulations include relationships to calculate channel width, depth, and slope as functions of channel-forming (or dominant) discharge and bed-material size.

One of the major deficiencies of the regime approach is that the equations often contain empirical coefficients that must be estimated primarily using judgment and experience. Regime equations are typically regression equations. They should not be used in cases where the discharge, sediment transport, bed gradations, and channel characteristics of the project channel are significantly different from those used in the development of the regime relationships. In general, regime relationships are applicable to flows at low Froude numbers, in the ripple-dune regime, with low sediment transport, and relatively uniform discharges. Since many of these equations were developed using canals where the flows remain in the channel, they do not reflect the effects of flood plain flows in the channel formation and maintenance in a natural channel. In short, since this theory is based on steady, uniform flows in canals, a sole application of it to unsteady, nonuniform rivers is not necessarily an optimum design method, given other alternatives.

(a) Blench regime equations

Stable channel dimensions may be calculated using the Blench regime equations. These regime equations are also addressed in American Society of Civil Engineers (ASCE) Manual 54 (ASCE 2006). The data used

to develop the Blench regression equations came from Indian canals with sand beds and slightly cohesive-to-cohesive banks. The sediment inflow was affected by sediment exclusion and/or ejection structures and was generally less than 30 milligrams per liter (Federal Interagency Stream Restoration Working Group (FISRWG) 1998). The equations were intended for design of canals with sand beds. The basic three channel dimensions—width, depth, and slope—are calculated as a function of bed-material grain size, channel-forming discharge, bed-material sediment concentration, and bank composition. The regression equations are not dimensionless and must be used with the units used in their derivation.

$$d = \left(\frac{F_S Q}{F_B^2} \right)^{\frac{1}{3}}$$

$$W = \left(\frac{F_B Q}{F_S} \right)^{0.5}$$

$$F_B = 1.9\sqrt{D_{50}}$$

$$S = \frac{F_B^{0.875}}{\frac{3.63g}{\upsilon^{0.25}} W^{0.25} d^{0.125} \left(1 + \frac{C}{2,330} \right)} \qquad \text{(eq. 9–1)}$$

where:
W = channel width (ft)
F_B = bed factor
F_S = side factor
Q = water discharge (ft³/s)
D_{50} = median grain size of bed material (mm)
d = depth (ft)
S = slope
C = bed-material sediment concentration (ppm)
g = acceleration of gravity (ft/s²)
υ = kinematic viscosity (ft²/s)

The results are true regime values only if Q is the channel-forming discharge. However, a width, depth, and slope may be calculated for any discharge by these equations.

Blench suggested that the following values be used for the side factor:

F_S = 0.10 for friable banks
F_S = 0.20 for silty, clay, loam banks
F_S = 0.30 for tough clay banks

(b) Modified regime method

The modified regime method was introduced by Simons and Albertson (1963) and is based on data from canals in India and the United States. Simons and Albertson expanded the range of conditions used in development of previous regime equations, reducing reliance on empirical coefficients. This method is also addressed in Design of Open Channels, TR–25 (U.S. Department of Agriculture (USDA) Soil Conservation Service (SCS) (1977). Regime canals in California's Imperial Valley, the San Luis Valley in Colorado, and canals in Wyoming, Colorado, and Nebraska, were used to develop the equations. Limits of data sets used to derive the modified regime equations are given in table 9–3 (FISRWG 1998). Three sets of equations were developed for three classes of channels based on the composition of streambed and streambanks. The need for computing bed, bank, or sediment concentration factors is eliminated. Inflowing sediment concentration is not an independent variable. The equations are presented in table 9–4. These are not dimensionless equations and must be used with the units used in their derivation.

The following relationships between channel geometry and slope are applicable to all three channel types.

$$d = 1.23R \qquad (1 < R < 7)$$
$$d = 2.11 + 0.934R \qquad (7 < R < 12)$$
$$W = 0.9P$$
$$W = 0.92TW - 2.0 \qquad \text{(eq. 9–2)}$$

where:
Q = channel-forming discharge (ft^3/s)
P = perimeter (ft)
R = hydraulic radius (ft)
A = channel cross-sectional area (ft^2)
V = mean channel velocity (ft/s)
W = average channel width (ft)
d = average flow depth (ft)
TW = channel top width (ft)

According to Simons and Albertson, the channel Froude number must be less than 0.3 to avoid excessive scour.

Procedure for application of the modified regime method

Step 1   Determine the channel-forming discharge. Use methods outlined in NEH654.05. The channel-forming discharge is the primary independent variable in the modified regime equations.

Step 2   Determine the character of the bed and bank materials. Determine characteristics for both the design reach and the upstream reach. Classify the boundary materials as either sand bed and sand banks, sand bed and cohesive banks, or cohesive bed and cohesive banks. Coefficients for the modified regime equations are determined from the boundary classification.

Step 3   Calculate sediment transport rate. Select an appropriate sediment transport equation, and calculate inflow to the design reach.

Step 4   Check to see if the modified regime approach is applicable. Use table 9–1.

Step 5   Determine the channel geometry and acceptable safe slope using the modified regime equations. Use table 9–4.

Step 6   Check the slope calculated with the modified regime equations. Use Manning's equation with a realistic roughness coefficient and cross-sectional geometry consistent with that determined in step 4.

After channel dimensions have been determined, it is prudent to evaluate the sediment transport capacity of the design reach and compare it to the upstream supply reach. This can be accomplished by calculating sediment transport capacity in the two reaches, using an appropriate sediment transport equation. More detail on how to make this evaluation is given in NEH654.13, which addresses sediment budget analysis.

Table 9–3 Limits of data sets used to derive Simons and Albertson modified regime equations

Data source	Median bed-material size (mm)	Banks	Discharge (ft³/s)	Sediment concentration (ppm)	Slope (L/L)	Bedforms
United States and Indian canals	0.318 to 0.465	Sand	100 to 400	<500	0.000135 to 0.000388	Ripple to dunes
	0.06 to 0.46	Cohesive	5 to 88,300	<500	0.000059 to 0.00034	Ripples to dunes
	Cohesive 0.029 to 0.36	Cohesive	137 to 510	<500	0.000063 to 0.000114	Ripples to dunes

Table 9–4 Coefficients for modified regime equations

Q (ft³/s)	Sand bed and sand banks	Sand bed and cohesive banks	Cohesive bed and cohesive banks
$P \text{ (ft)} = C_1 Q^{0.512}$	3.30	2.51	2.12
$R \text{ (ft)} = C_2 Q^{0.361}$	0.37	0.43	0.51
$A \text{ (ft}^2) = C_3 Q^{0.873}$	1.22	1.08	1.08
$V \text{ (ft/s)} = C_4 (R^2 S)^{1/3}$	13.9	16.1	16.0
$W/d = C_5 Q^{0.151}$	6.5	4.3	3.0

Note: * A soil is classed as cohesive if the plasticity index is >7

Example 1: Modified regime method

Given: The channel has a sand bed and cohesive banks. The channel-forming discharge is 600 ft³/s. Use 2H:1V side slopes and $n = 0.022$.

Problem: Design a stable trapezoidal channel using the modified regime approach.

Solution:

Step 1 Compute the channel perimeter, P:

$$P = 2.51Q^{0.512}$$
$$P = (2.51)(600)^{0.512}$$
$$P = 66.4 \text{ ft} \qquad \text{(eq. 9–3)}$$

Step 2 Compute the hydraulic radius, R:

$$R = 0.43Q^{0.361}$$
$$R = (0.43)(600)^{0.361}$$
$$R = 4.33 \text{ ft} \qquad \text{(eq. 9–4)}$$

Step 3 Compute the flow area, A:

$$A = PR$$
$$A = (66.4)(4.33)$$
$$A = 288 \text{ ft}^2 \qquad \text{(eq. 9–5)}$$

or

$$A = 1.08Q^{0.873}$$
$$A = (108)(600)^{0.873}$$
$$A = 288 \text{ ft}^2 \qquad \text{(eq. 9–6)}$$

Step 4 Compute mean velocity, V:

$$V = \frac{Q}{A}$$
$$V = \frac{(600)}{(288)}$$
$$V = 2.08 \text{ ft/s} \qquad \text{(eq. 9–7)}$$

Step 5 Compute the depth, d:

When R < 7 ft

$$d = 1.23R$$
$$d = (1.23)(4.33)$$
$$d = 5.33 \text{ ft} \qquad \text{(eq. 9–8)}$$

Step 6 Compute the Froude number:

$$F = \frac{V}{(gd)^{0.5}}$$
$$F = \frac{(2.08)}{\left[(32.2)(5.33)\right]^{0.5}}$$
$$F = 0.159$$
$$F < 0.3 \qquad \text{(eq. 9–9)}$$

therefore, design meets this requirement for stability.

Step 7 Compute bottom width, BW:

$$W = 0.9P$$
$$W = 0.9(66.4)$$
$$W = 59.8 \text{ ft}$$
$$W = 0.92TW - 2.0$$
$$59.8 = 0.92TW - 2.0$$
$$TW = 67.2 \text{ ft} \qquad \text{(eq. 9–10)}$$

For 2H: 1V side slopes
$$BW = 67.2 - (2)(2)(5.33)$$
$$BW = 45.9 \text{ ft} \qquad \text{(eq. 9–11)}$$

Step 8 Calculate the width-to-depth ratio, W/d:

$$\frac{W}{d} = 4.3Q^{0.151}$$
$$\frac{W}{d} = 4.3(600)^{0.151}$$
$$\frac{W}{d} = 11.3 \qquad \text{(eq. 9–12)}$$

Example 1: Modified regime method—Continued

Step 9 Calculate regime slope and regime hydraulic roughness coefficient:

$$V = C_4 \left(R^2 S\right)^{\frac{1}{3}} \qquad \text{(table 9-4)}$$

$$2.08 = 16.1 \left[\left(4.33\right)^2 S\right]^{\frac{1}{3}}$$

$$S = 0.000115$$

$$n = 1.486 A R^{\frac{2}{3}} \frac{S^{\frac{1}{2}}}{Q} \qquad \text{(Manning's equati}$$

$$n = \left(1.486\right)\left(288\right)\left(4.33\right)^{\frac{2}{3}} \frac{\left(0.000115\right)^{\frac{1}{2}}}{\left(600\right)}$$

$$n = 0.020$$

(eq. 9–13)

Step 10 Calculate the channel slope assuming uniform flow:

$$S = \frac{\left(Qn\right)^2}{\left(2.208 A^2 R^{\frac{4}{3}}\right)}$$

$$S = \frac{\left[\left(600\right)\left(0.022\right)\right]^2}{\left[2.208 \left(288\right)^2 \left(4.33\right)^{\frac{4}{3}}\right]}$$

$$S = 0.000135$$

(eq. 9–14)

Step 11 Select the channel design slope:

At this point in the design process, the designer must decide if there is more confidence in the regime hydraulic roughness coefficient, 0.020, or the assigned hydraulic roughness coefficient, 0.022. If it can be demonstrated that the regime relationship for slope fits existing data in channels physiographically similar to the design channel, the engineer might choose the regime slope. Without such calibration data, there is less uncertainty related to assigning a roughness coefficient and using the slope from the uniform flow equation.

For example:

S = 0.000135 (from step 10)

d = 5.3 ft (from step 5)

BW = 45 ft (from step 7)

Step 12 After channel dimensions have been determined, calculate the sediment transport capacity of the design reach, and compare it to the upstream supply reach. If the sediment transport capacity of the supply reach is greater than the sediment transport capacity of the design channel, either sediment removal must be provided for, or the design must be modified. This step is described in more detail in NEH654.13.

654.0904 Analogy method and reference reaches

Estimates for stable channel design width, depth, and slope in an alluvial channel can be made using channel dimensions from a similar stable channel. The channel reach from which the design dimensions are taken is frequently called a reference reach. The concept is that alluvial streams will evolve to the same stable channel dimensions, given the same independent driving hydraulic variables. To apply the analogy method, the bed and bank materials, sediment inflow, slope, valley type, and annual discharge hydrograph should be close to the same in both the design and reference reaches. When these conditions exist, the reference reach is said to be physiographically similar to the design reach. All three dependent hydraulic design dimensions from the reference reach must be used in the design reach to maintain physiographic similarity. Given these constraints, it can be difficult to find a suitable reference reach, especially in urban or developed watersheds. However, while locating a suitable reference reach can be problematic, many stream restorations have been planned, measured, and designed using this approach.

A reference reach is a site that is able to transport sediments and detritus from its contributing watershed drainage area, while maintaining a consistent profile, dimension, and plan view, over time. The reference reach with the highest level of confidence would be the existing channel in the project reach or just upstream or downstream from the project reach. If the existing channel is used as a reference reach, the channel must be stable, and there should be no significant recent or future changes in the watershed. Urbanization in the watershed can significantly change both the inflow hydrograph and the sediment inflow. The analogy method is inappropriate for streams where the entire fluvial system, or a significant part of it, is in disequilibrium.

A stable historic channel can sometimes be used as a reference reach to obtain estimates for the dependent design variables of channel width and planform. This is feasible if historical width and planform information can be determined from mapping, aerial photos, and/or soil borings. However, this technique is not applicable if the watershed sediment yield and runoff

characteristics have changed over time. It cannot be assumed that the historically stable channel dimensions will continue to be stable with different water and sediment inflow.

An existing pristine or pre-settlement reach may also be used as an analog or reference reach. These reaches are rare, but can sometimes be found on USDA Forest Service or National Park Service land, as well as undeveloped portions of less developed countries. However, the use of these analogs is hindered by the same issues as for the historic channels. As a result, their use is generally not feasible to use in the vast majority of stream restoration projects, unless restoration to the pristine pre-settlement condition is the project goal.

In practice, several reference reaches with relatively similar channel-forming discharges may be used to develop a range of solutions for a single dependent design variable, typically, width. Analytical methods can then be employed to determine the other dependent design variables. Other design features such as planform, riffle and pool spacing, riffle widths, and pool depths can be determined using the reference reaches. The reference reaches must be stable and alluvial. The bed and banks in both the project and reference reaches must be composed of similar sediments. There should be no significant differences in watershed hydrology, channel flows, sediment inflow, or bed-material load between the project and reference reaches.

(a) Limitations of analogy method

It can be very difficult to find a stable alluvial reference reach with characteristics physiographically similar to the reach to be restored. The independent driving variables of sediment inflow, bed and bank material, and channel-forming discharge must be similar. The dependent design variables of slope, depth, and width must be taken together as a set.

654.0905 Hydraulic geometry method

Hydraulic geometry theory is an extension of regime theory. Regime theory was developed to design canals. Hydraulic geometry was developed for analysis of natural streams and rivers. Hydraulic geometry theory is based on the concept that a river system tends to develop in a predictable way, producing an approximate equilibrium between the channel and the inflowing water and sediment (Leopold and Maddock 1953). The theory typically relates a dependent variable, such as width, to an independent or driving variable, such as discharge or drainage area. Herein lies the primary weaknesses of hydraulic geometry theory—dependent hydraulic design variables are assumed to be related only to a single independent design variable and not to any other design variables.

To help overcome this deficiency, hydraulic geometry relationships are sometimes stratified according to bed-material size, bank vegetation, or bank material type. Rosgen (1998) suggests stream classification as an appropriate tool for differentiating hydraulic geometry relationships. Hydraulic geometry relationships are developed from field observations at stable and alluvial channel cross sections and were originally used as descriptors of geomorphically adjusted channel forms. As design tools, hydraulic geometry relationships may be useful for preliminary or trial selection of the stable channel width. Hydraulic geometry relationships for depth and slope are, however, less reliable and not recommended for final channel design.

A hydraulic geometry relationship for width can be developed for a specific river, watershed, or for streams with similar physiographic characteristics. Data scatter is expected about the developed curve, even in the same river reach. An example of a hydraulic geometry relationship between bankfull discharge and bankfull water surface width developed for a mountainous watershed can be found in Emmett (1975). Emmett collected data at 39 gaging stations in the Salmon River Drainage Basin, Idaho. The relationship between bankfull discharge and bankfull width is shown in figure 9–1. Emmett's mean regression line had a regression coefficient (r^2) of 0.92. Nevertheless, a wide range of bankfull widths were found for any specific

bankfull discharge. The data scatter indicates that for a bankfull discharge of 200 cubic feet per second, the bankfull width could reasonably range between 15 and 45 feet. This range does not necessarily indicate instability or different physiographic conditions, but rather the wide range of possible stable widths for a given channel-forming discharge. Some other examples of regional hydraulic geometry studies are Leopold and Maddock (1953); Dunne and Leopold (1978); Charlton, Brown, and Benson (1978); Bray (1982); and Hey and Thorne (1986). Additional guidance for application of hydraulic geometry methods is provided in FISRWG (1998). Table 9–5 provides the range of data used to derive the hydraulic geometry width predictors for gravel-bed rivers shown in table 9–6 (FISRWG 1998; Soar and Thorne 2001).

The more dissimilar the stream and watershed characteristics are in stream reaches used to develop a hydraulic geometry relationship, the greater the expected data scatter around the regression line, and the less reliable the results. It is important to recognize that this scatter represents a valid range of stable channel configurations due to variables such as geology, vegetation, land use, sediment load and gradation, runoff characteristics, and in some geographic areas, woody debris. The composition of the bank is very important in the determination of a stable channel width. The presence and percentage of cohesive sediment in the bank and/or the amount of vegetation on the bank significantly affect the stable alluvial channel width (Schumm 1977; Hey and Thorne 1986).

Figure 9–1   Hydraulic geometry relationship for width for the Upper Salmon River Basin, ID

$$W_b = 1.37 Q_b^{0.54}$$
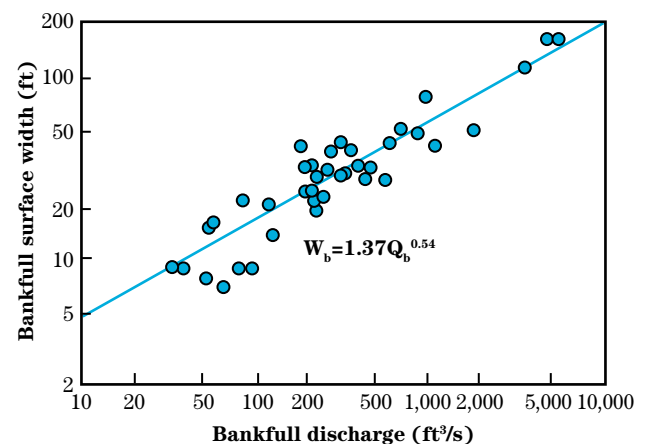
Table 9–5 Limits of data sets used to derive hydraulic geometry equation

Reference	Data source	Median bed material size (mm)	Banks	Discharge (ft³/s)	Sediment concentration (ppm)	Slope (L/L)	Bed forms
Nixon (1959)	U.K. rivers	Gravel		700–18,000	Not measured		
Kellerhalls (1967)	U.S., Canadian, and Swiss rivers of low sinuosity, and laboratory	7–265 bed armored	Noncohesive	1.1–70,600	Negligible	0.00017–0.0131	Plane
Emmett (1975)	Salmon River, ID	11–58	Cohesive, sand and gravel	40–5,100		0.0009–0.0006	
Charlton, Brown, and Benson (1978)	Meandering U.K. rivers	33–113	Sand or gravel	95–5,500	Negligible	0.0009–0.0137	
Bray (1982)	Sinuous Canadian rivers	1.9–145		194–138,400	Mobile bed	0.00022–0.015	
Parker (1982)	British rivers Alberta–single Alberta–braided		Cohesive Little cohesion No cohesion	100–21,200 400–200,000	Active bed	0.0007–0.015 0.0002–0.015 0.0025–0.015	
Hey and Thorne (1986)	Meandering U.K. rivers	14–176	Cohesive and composite	138–15,000	Computed 0–535	0.0012–0.021	

Table 9–6   Hydraulic geometry width equations for gravel-bed rivers

		$W = aQ^b$		
Reference	**Data source**	**Coefficient a** **W = m** **Q = m³/s**	**Coefficient a** **W = ft** **Q = ft³/s**	**Exponent b**
Nixon (1959)	U.K.	2.99	1.65	0.5
Kellerhalls (1967)	U.S., Canada, Switzerland	3.26	1.80	0.5
Bray (1973, 1982)*	Canada	3.83	1.90	0.53
Emmett (1975)	Salmon River, ID	2.86	1.37	0.54
Charlton, Brown, and Benson (1978)	U.K.			
	Type A	3.74	2.47	0.45
	Type A_G	3.37–4.86	2.22–3.21	0.45
	Type A_T	2.62–4.11	1.73–2.71	0.45
	Type B	2.43	1.85	0.41
Parker (1982)	U.K. single channel; cohesive or vegetated banks	3.73	2.50	0.446
	Alberta single channel; little cohesion in banks	5.86	3.99	0.441
	Alberta braided; no cohe- sion in banks	7.08	5.25	0.417
Hey and Thorne (1986)	U.K. rivers			
	Type I	4.33	2.39	0.50
	Type II	3.33	1.84	0.50
	Type III	2.73	1.51	0.50
	Type IV	2.34	1.29	0.50

* Bankfull discharge equated to 2-year recurrence interval discharge

Note:
Type A   = low sediment load
Type A_G = low sediment load and grass-lined banks
Type A_T = low sediment load and tree-lined banks
Type B   = appreciable sediment load
Type I   = grassy banks with no trees or shrubs
Type II   = 1 to 5 percent tree/shrub cover
Type III = 5 to 50 percent tree/shrub cover
Type IV = greater than 50 percent tree-shrub cover or incised into flood plain

When a hydraulic geometry relationship is to be used for a channel design, the first choice is to use one developed from stable alluvial reaches of the project stream. It is required that the stable reaches used to develop the relationship have similar physiographic conditions to each other and the project reach. If there are no stable reaches, or if the range of discharges is insufficient, a second choice is to use other streams or tributaries in the same watershed to develop the hydraulic geometry relationship.

The third choice is to use regional relationships developed for other watersheds in the same physiographic region. The transfer of hydraulic geometry relationships developed for one watershed to another watershed should be performed with care. The two watersheds should be similar in historical land use, physiography, hydrologic regime, precipitation, and vegetation. For example, relationships developed for pristine watersheds should not be transferred to urban watersheds. Relationships developed for areas with snowmelt hydrology should not be transferred to areas dominated by convective storms. Since discharge is the variable that shapes the channel, relationships based on discharge can be transferred with more confidence than those based on drainage area, which is basically a surrogate for discharge.

Urbanized streams present particular problems in both the development and the application of hydraulic geometry relationships. Land use and runoff characteristics usually vary greatly, even within a single watershed. The multiplicity of humanmade structures, such as storm sewers, bridge openings, culverts and stormwater management facilities, and amount of impervious pavement changes the amount, duration, and timing of flows. This would be expected to greatly increase data variability. These factors make discharge more poorly correlated with drainage area and, therefore, would make discharge the better choice than drainage area as an independent variable. Locating stable, alluvial reaches in urban or developed watersheds may be difficult.

In all cases, it must be remembered that data used to develop hydraulic geometry relationships should come from stable reaches and that the watersheds and channel boundary conditions should be similar to the project channel.

(a) Procedure for developing hydraulic geometry relationships

Step 1   Locate gaging stations with long-term records. Making sure that the record is homogeneous (temporally consistent watershed conditions) as described in NEH654.05, calculate an annual peak frequency curve and a flow-duration curve. Preferably, these gaging stations will be on physiographically similar reaches of the same river as the project. A second choice is physiographically similar reaches of streams in physiographically similar watersheds. Make sure that discharge ranges are significantly greater and less than the design reach. Do not rely solely on regional relationships or drainage area versus discharge plots. These are already empirical and may not be appropriate for deriving new relationships.

Step 2   Locate stable alluvial channel reaches that can be associated with the gaging stations. Survey a typical channel cross section or several cross sections in the reach. Determine average channel top width at bankfull flow and average channel depth. Rosgen (1998) suggests gathering data from a reach length associated with two meander wavelengths or 20 top widths. Estimate channel hydraulic roughness. Using surveys or contour maps determine average channel bed slope. If the channel slope is discontinuous, use the cross sections to develop a backwater model, and calculate average energy slope. Determine bankfull discharge by using a normal depth equation or a backwater model.

Step 3   Note channel characteristics. Characteristics such as bank material composition, bed-material gradation, and bank vegetation are of interest. These characteristics may be used to ensure the physiographic similarity of the stream or to develop more refined hydraulic geometry relationships.

Step 4   Determine the channel-forming discharge. Determine the 2-year peak discharge from the annual peak frequency curve. Calculate the effective discharge using the flow-duration curve and a sediment transport curve. Determine the bankfull discharge from field measurements and backwater calculations. From these three discharges, estimate the channel-forming discharge as described in NEH654.05.

Step 5 Develop regression curve. Plot the measured channel top width versus the channel-forming discharge, and develop a power regression curve through the data. Plot confidence limits. The final plot should include the data so that the natural range of data can be observed.

(b) Generalized width predictors

Lacking data to develop more reliable hydraulic geometry relationships, generalized width predictors for various river types with different bank characteristics have been developed (Copeland et al. 2001) and are presented in figures 9–2 through 9–11. The range of data used in the development of these equations is shown in table 9–7 (Soar and Thorne 2001).

These predictors include confidence limits and may be used for general guidance when stream or watershed specific data cannot be obtained.

(c) Hydraulic geometry for meandering sand-bed rivers

Hydraulic geometry width predictors (fig. 9–2) were developed from data collected from 58 meandering sand-bed rivers in the United States (Copeland et al.

2001). These rivers were located mostly in Indiana, Illinois, Iowa, Kansas, Oklahoma, Texas, Arkansas, Louisiana, Mississippi, Kentucky, Virginia, North Carolina, and South Carolina (fig. 9–3).

Sufficient data were collected to determine both bankfull discharge and effective discharge. Data were collected from stable reaches, so bankfull discharge should be the most reliable approximator for the channel-forming discharge. In many of these meandering sand-bed rivers, the effective discharge was significantly less than the bankfull discharge. For design purposes, the bankfull discharge was used to define the width predictor. The data were divided into two sets: type T1, where there was less than 50 percent tree cover on the banks (fig. 9–4) and type T2, where there was greater than 50 percent tree cover on the banks (fig. 9–5).

Figures 9–6 and 9–7 are examples of rivers used in the development of the sand-bed hydraulic geometry relations. All sites were tree-lined to some degree; therefore, the predictors should not be used for grass-lined or thinly vegetated banks. The percentage of silt and clay in the banks was not found to be statistically significant in affecting width for these rivers, possibly because the root-binding properties of the trees were more significant in stabilizing the bank than cohesive forces.

Table 9–7 Limits of data sets used to derive generalized hydraulic geometry equations

River type	Median bed material mm	Banks	Discharge ft^3/s	Sediment concentration ppm	Slope
United States meandering sand-bed rivers	0.12–1.63	Cohesive and noncohesive	630–48,300	Significant	0.00007–0.00088
United States gravel-bed rivers	3–122	Variable	39–18,000	Negligible	0.00062–0.024
United Kingdom gravel-bed rivers	14–176	Variable	95–23,000	Negligible	0.00036–0.0021

Figure 9–2   Best-fit hydraulic geometry relationships for width for U.S. sand-bed rivers with banks typed according to density of tree cover
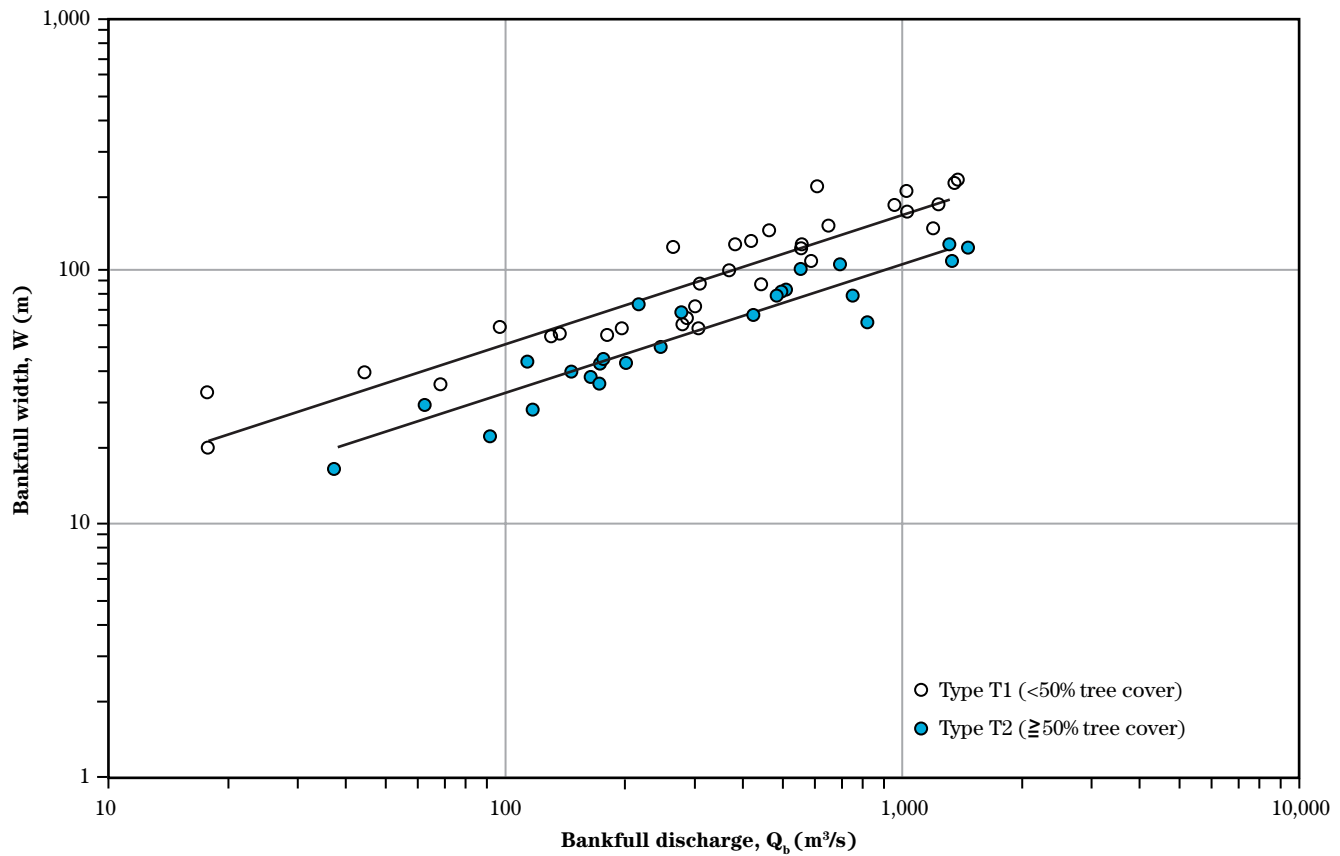
Figure 9–3   Sites used to develop U.S. sand-bed river hydraulic geometry relationships

Figure 9–4   Confidence intervals applied to the hydraulic geometry equation for width based on 32 sand-bed rivers with less than 50% tree cover on the banks (T1). SI units – m and m³/s (English units – ft and ft³/s)
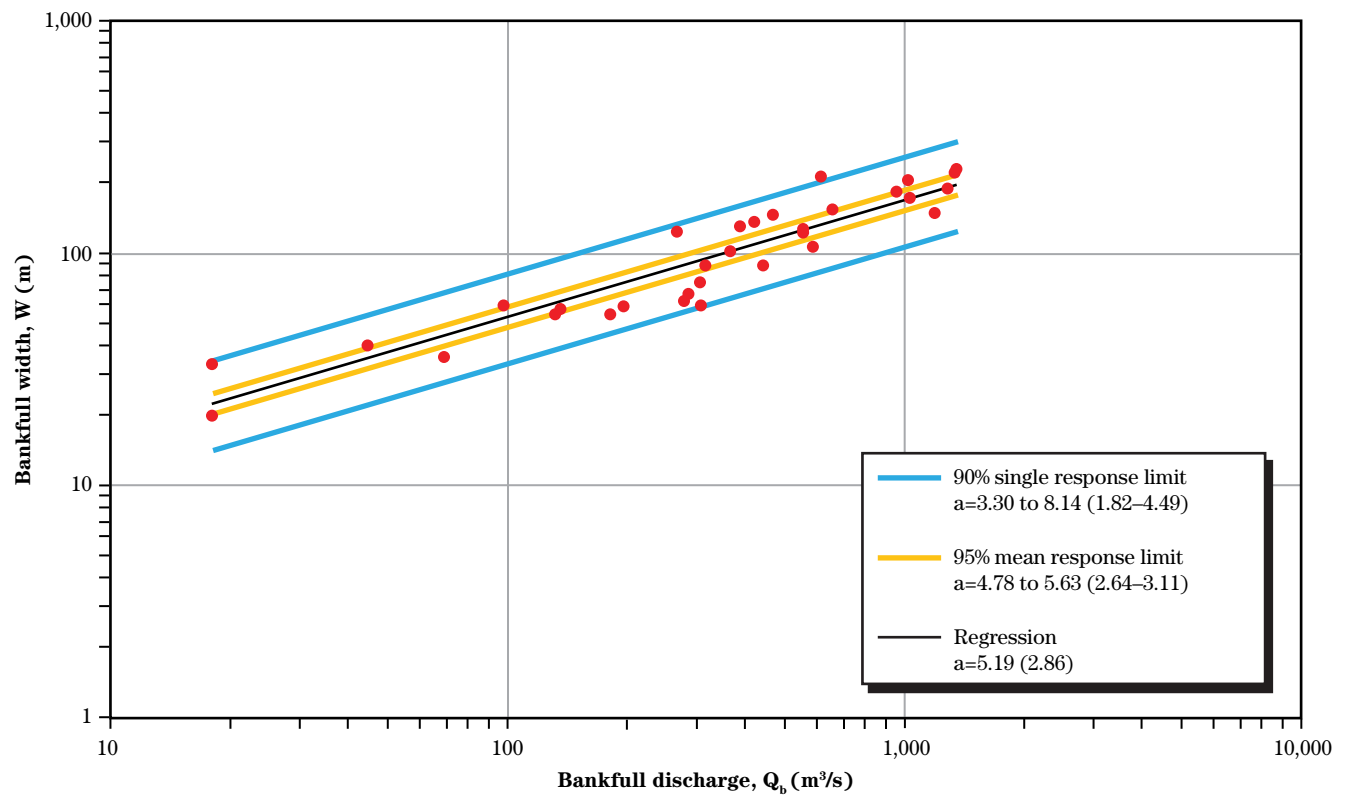
Figure 9–5   Confidence intervals applied to the width hydraulic geometry equation based on 26 sand-bed rivers with at least 50% tree cover on the banks (T2). SI units – m and m^3/s (English units ft and ft^3/s)
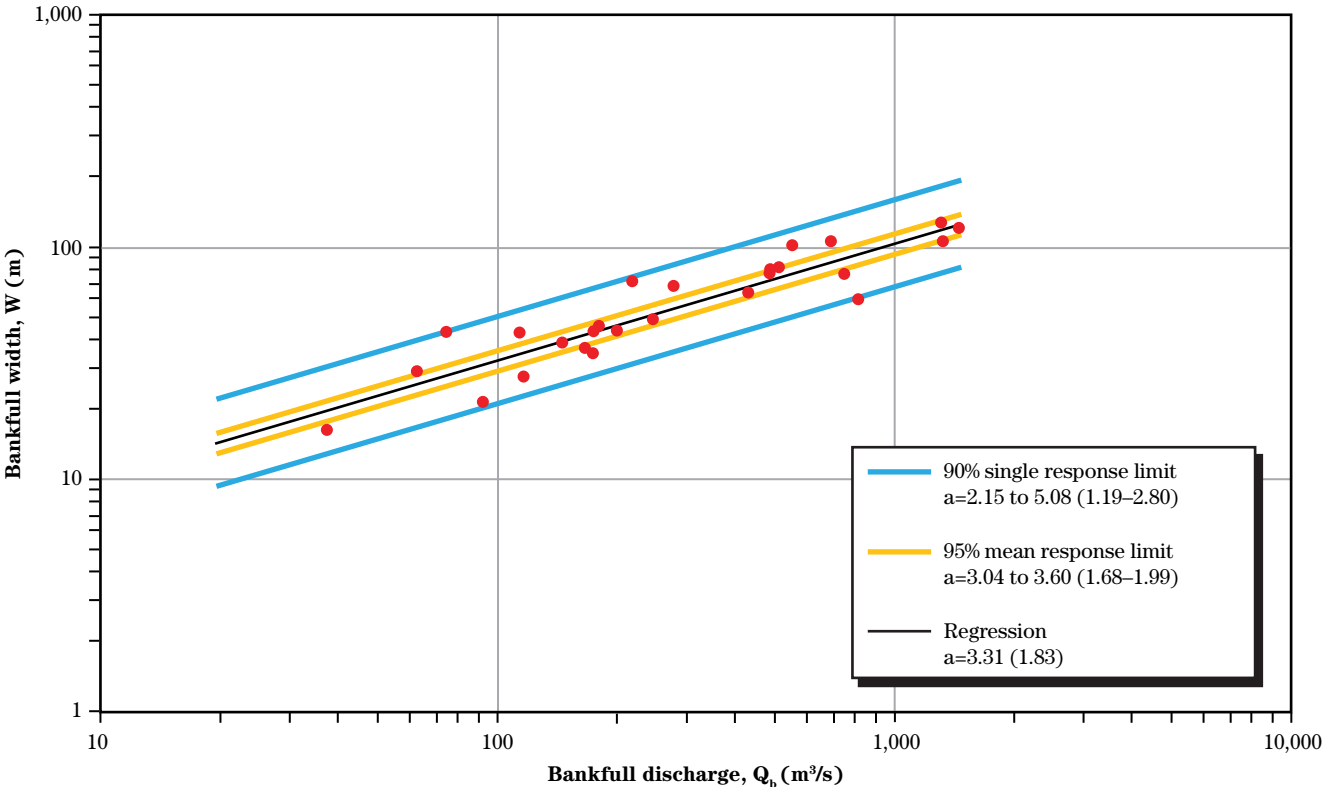
Figure 9–6   Type 1 bankline (T1), less than 50% tree cover

Figure 9–7   Type 2 bankline (T2), greater than 50% tree cover

The hydraulic geometry width predictor is expressed by the general equation:

$$W = aQ^b$$

(eq. 9–15)

where:
W = channel top width
Q = channel-forming discharge
a = see table 9–8
b = see table 9–8

The hydraulic geometry width predictors each include two sets of confidence bands. The 95 percent mean response limit provides the band in which one can be 95 percent confident that the mean value of the width will occur. This is the confidence interval for the regression line and provides the range of average values of width that can be expected for a given discharge. The 90 percent single response limit provides the envelope curves that contain 90 percent of the data points. This is the confidence interval for an individual predicted value and provides the engineer with the range of possible widths that have been observed to correspond to a given discharge. The confidence interval on an individual predicted value is wider than the confidence interval of the regression line because it includes both the variance of the regression line plus the squared standard deviation of the data set.

While the equations given in table 9–8 may be used for preliminary design purposes, they are subject to several limitations. In the absence of stage-discharge relationships at each site, the bankfull discharge was calculated using Manning's equation and is subject to assumptions related to choice of a resistance coefficient. As cross-sectional geometry was used to calculate discharge, discharge is not truly independent of width in this analysis. Furthermore, only one cross section was measured at each site. Identification of the bankfull reference level, although based on field experience and geomorphic criteria, is always subject to a degree of uncertainty. These factors contribute to the observed variability in the width relationships. Finally, small rivers are not well represented in the data set; therefore, the generalized width predictors should not be applied when channel-forming discharge is less than 600 cubic feet per second in type T1 channels and less than 1,300 cubic feet per second in type T2 channels.

Table 9–8   Hydraulic geometry width predictors for meandering sand-bed rivers

			$W = aQ^b$			
			SI units m and m^3/s (English units ft and ft^3/s)			
Data source	**Sample size**	**a**	**90% single response limit for a**	**95% mean response limit for a**	**b**	**r^2**
All sand-bed rivers	58	4.24 (2.34)	2.34–7.68 (1.29–4.24)	3.90–4.60 (2.15–2.54)	0.5	0.76
Type T1: <50% tree cover	32	5.19 (2.86)	3.30–8.14 (1.82–4.49)	4.78–5.63 (2.64–3.11)	0.5	0.87
Type T2: >50% tree cover	26	3.31 (1.83)	2.15–5.08 (1.19–2.80)	3.04–3.60 (1.68–1.99)	0.5	0.85

(d) Hydraulic geometry for gravel-bed rivers

A review of the published gravel-bed river data and hydraulic geometry width predictors for North American and British rivers (Copeland et al. 2001) revealed that North American gravel-bed rivers are generally wider than those found in the United Kingdom, assuming discharge and other conditions are equal. North American data used to develop the hydraulic geometry relationship included data from Brandywine Creek in Pennsylvania (Wolman 1955), Alaskan streams (Emmett 1972), Upper Salmon River in Idaho (Emmett 1975), Colorado, New Mexico, Oregon, Pennsylvania, Tennessee, Utah, West Virginia, and Wyoming (Williams 1978), Alberta, Canada (Annable 1996), and the Rocky Mountain region of Colorado (Andrews 1984). United Kingdom data included data from Nixon (1959), Charlton, Brown, and Benson (1978) and Hey and Thorne (1986). The gravel-bed river data excluded data from braided, anastomosed, and split channel rivers. The hydraulic geometry relationships are shown in fig. 9–8. The difference in these regression curves cannot satisfactorily be explained using the site descriptions given in original publications. A possible explanation is that the United Kingdom sites have on the average more resistant banks than the North American sites. Another plausible explanation is that the North American sites on the average may be flashier. Still, another possibility is that the North American sites may be more active; that is, they may have a higher sediment load. Further research is required to validate these hypotheses.

The hydraulic geometry width predictors for North American and United Kingdom gravel-bed rivers are presented with confidence bands in figures 9–9 and 9–10, respectively. Exponents and coefficients for the hydraulic geometry equation are given in table 9–9. The gravel-bed river data comprise a wide range of bank material types (cohesive, sand, gravel, and composite banks of various strata). However, different width-discharge relationships based on different types of bank material could not be derived for the North American river data from the limited information available.

There were sufficient data available from the United Kingdom gravel-bed rivers to develop distinct width predictors based on erodible banks, low density of trees, and resistant banks, high density of trees (figs. 9–11 and 9–12). These hydraulic geometry relations may be used for preliminary design purposes, recognizing that considerable variability may occur for areas different from the streams used in the development of the equations.

(e) Uncertainty in hydraulic geometry relations

A sufficient number of data points must be measured to ensure that the results from hydraulic geometry analysis are statistically valid. For example, if any three or four random data points were used, a different relation could easily be derived. The fewer and more widely scattered the data points, the less confidence one has in any derived trend. Even with quite a few data points in a relatively homogeneous watershed, there is a great deal of scatter in the data due to natural variability.

Stable natural rivers have morphologies that broadly conform to regime or hydraulic geometry relationships. Therefore, dependent parameters of channel form can be linked to independent controls of flow regime, boundary materials, and riparian vegetation. However, rivers do not follow regime laws precisely. Every river displays local departures from the expected channel form described by morphological equations and possesses inherent variability in space and time. While it is true that natural channel forms are in general predictable, it is also true that each river is in detail unique. Regime dimensions in the natural domain should be interpreted only as representative reach-average, ideal, or target conditions, about which channel morphology fluctuates in time and space.

The coefficient of determination, r^2, in hydraulic geometry analysis numerically represents the amount of variation that can be explained by the selected independent variable. If r^2 is 1.0, there is no variation. The closer the r^2 value is to zero, the less useful the relation, and the wider the scatter in the data. The natural variability of data in a relatively homogeneous watershed such as the upper Salmon River watershed (Emmett 1975) underlines the importance of viewing the data used to develop the curve, not just the curve itself, along with statistical parameters such as r^2 values and confidence limits. If the r^2 value exceeds 95 percent for data collected in natural stream systems, it may indicate autocorrelation or too few data points. Equations given without plotted data points or statistical parameters should be verified for applicability.

Figure 9–8   Downstream width hydraulic geometry for North American gravel-bed rivers, $W = 3.68Q_b^{0.5}$ and U.K. gravel-bed rivers, $W = 2.99Q_b^{0.5}$
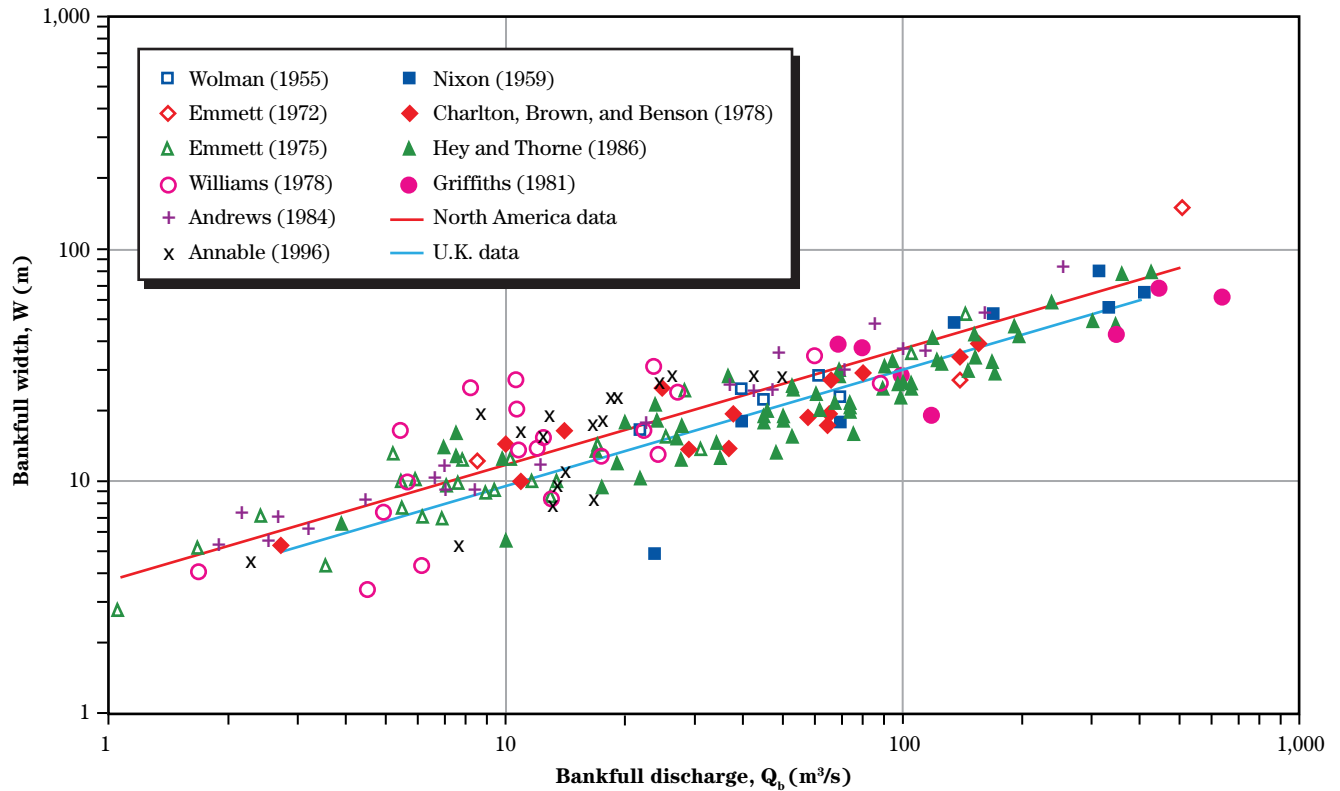
Table 9–9   Hydraulic geometry width predictors for gravel-bed rivers

$$W = aQ^b$$

SI units m and m³/s (English units ft and ft³/s)

Data source	Sample size	a	90% single response limit for a	95% mean response limit for a	b	r^2
All North American gravel-bed rivers	94	3.68 (2.03)	2.03–6.68 (1.12–3.69)	3.45–3.94 (1.90–2.18)	0.5	0.80
All U.K. gravel-bed rivers	86	2.99 (1.65)	1.86–4.79 (1.02–2.64)	2.83–3.16 (1.56–1.74)	0.5	0.80
<5% tree or shrub cover, or grass-lined banks (U.K. rivers)	36	3.70 (2.04)	2.64–5.20 (1.46–2.87)	3.49–3.92 (1.93–2.16)	0.5	0.92
≥5% tree or shrub cover (UK rivers)	43	2.46 (1.36)	1.87–3.24 (1.03–1.79)	2.36–2.57 (1.30–1.42)	0.5	0.92

Figure 9–9 Downstream width hydraulic geometry for North American gravel-bed rivers, $W = aQ_b^{0.5}$ with confidence bands. Based on 94 sites in North America. SI units – m and m^3/s (English units ft and ft^3/s)
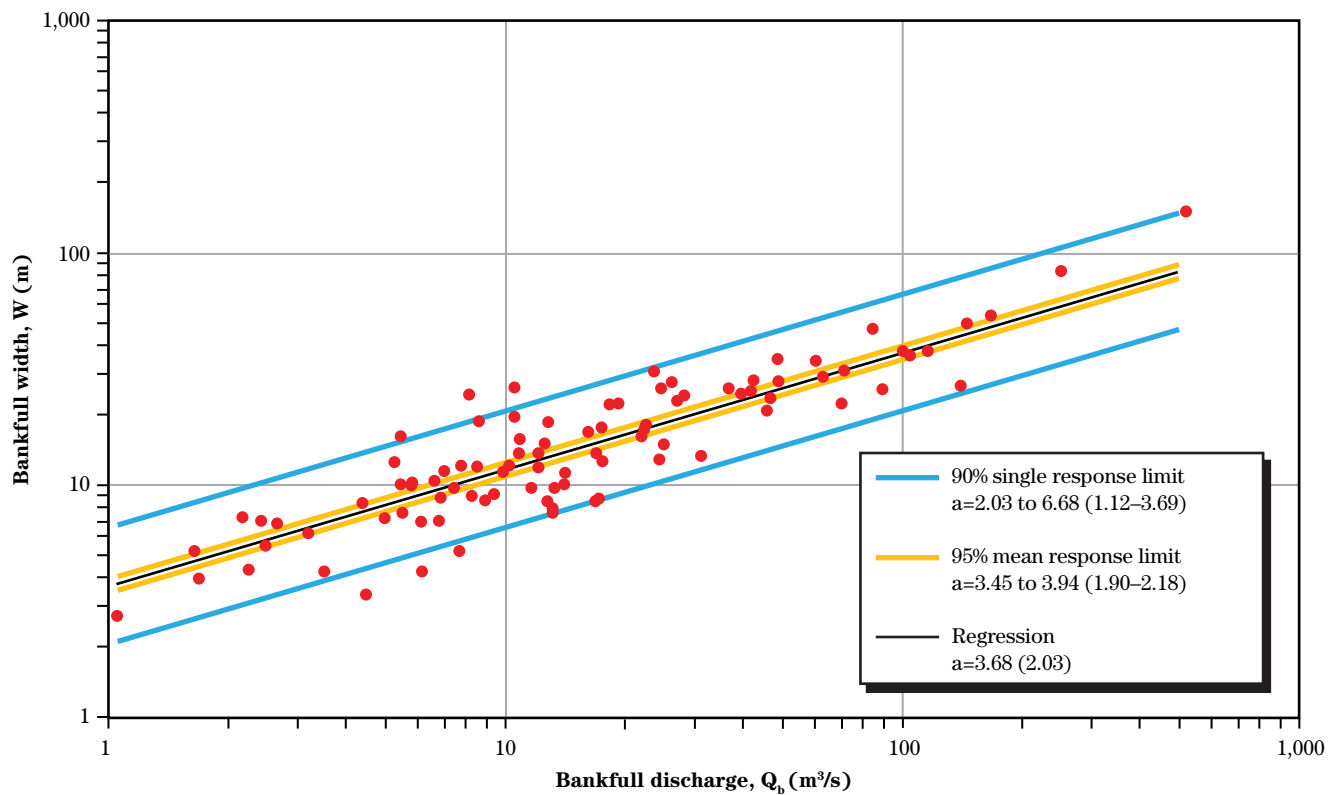
Figure 9–10 Downstream width hydraulic geometry for U.K. gravel-bed rivers, $W = aQ_b^{0.5}$ with confidence bands. Based on 86 sites in the U.K. SI units m and m^3/s (English units ft and ft^3/s)
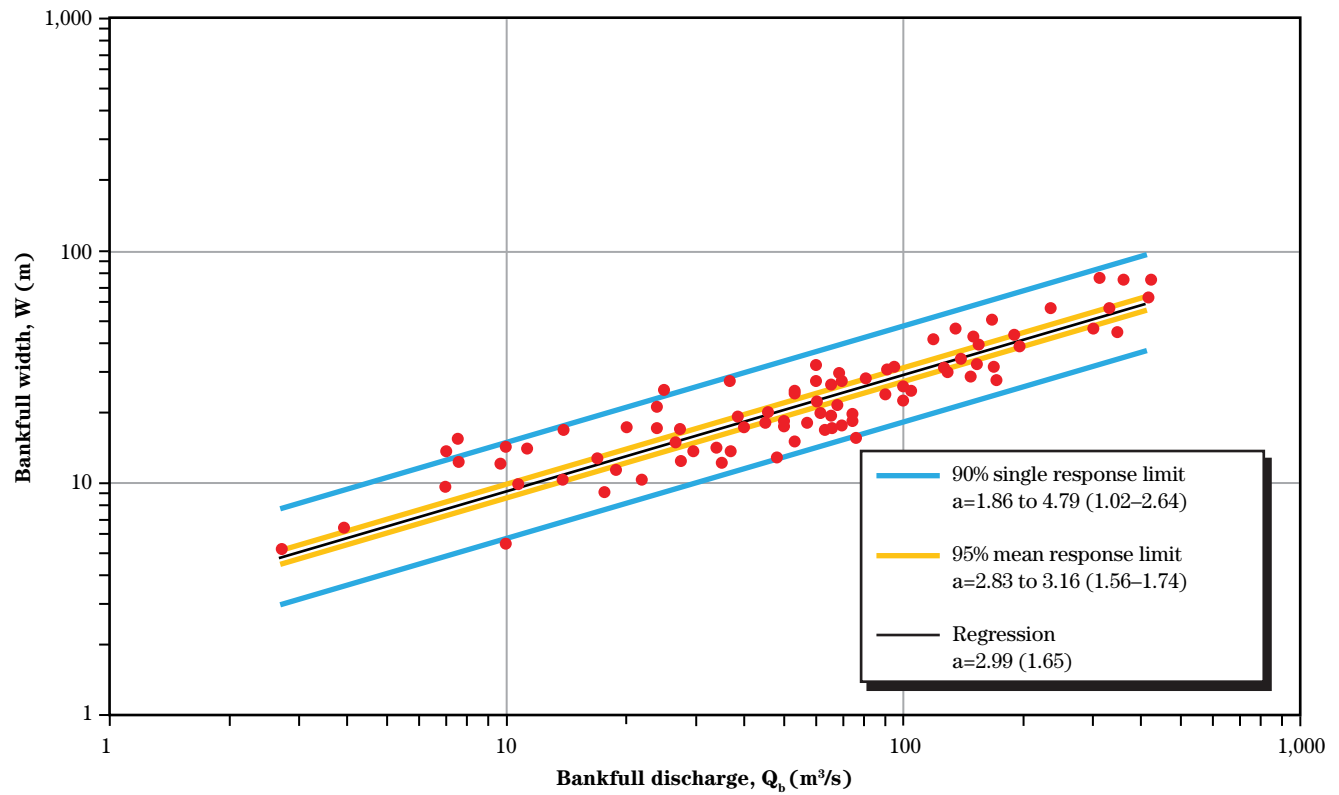
Figure 9–11   Downstream width hydraulic geometry for U.K. gravel-bed rivers, $W = aQ_b^{0.5}$ with confidence bands. Based on 36 sites in the U.K. with erodible banks. SI units m and m^3/s (English units ft and ft^3/s)
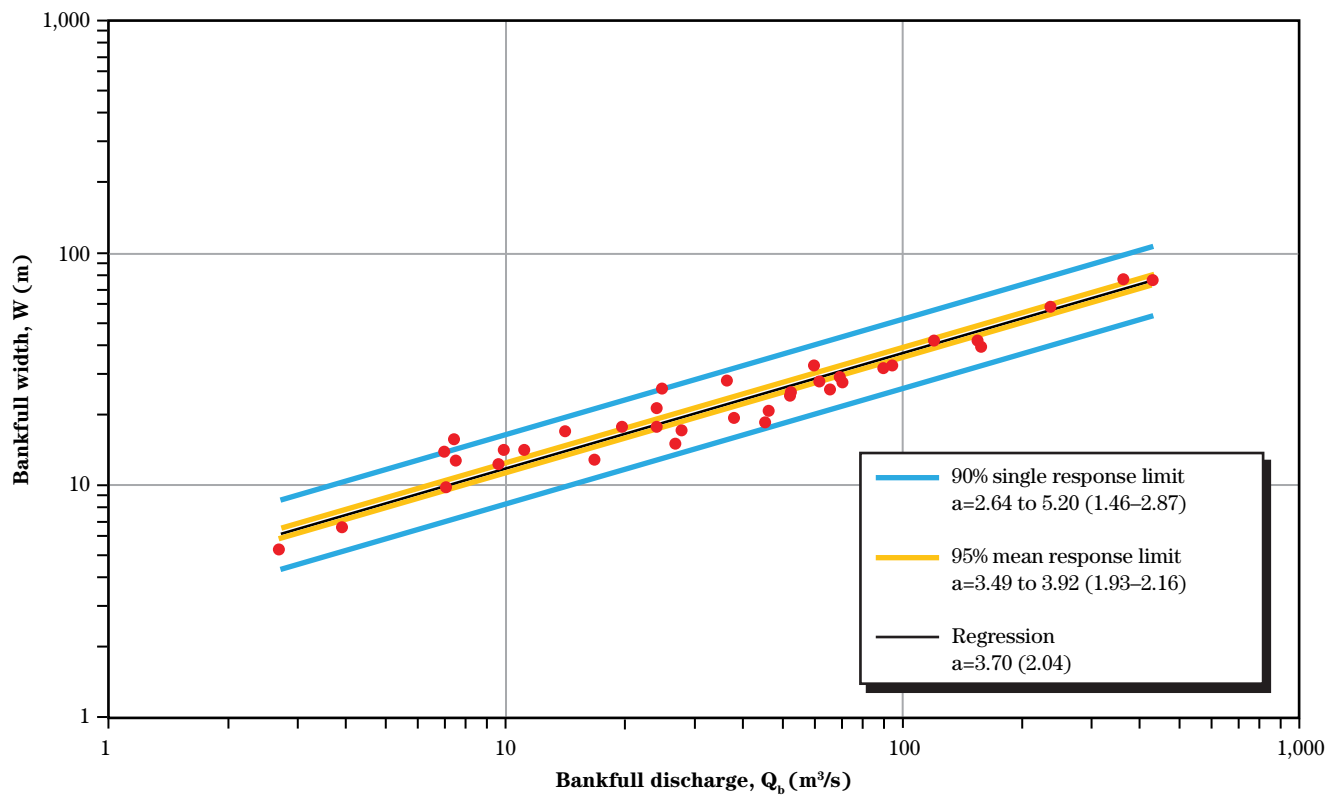
Figure 9–12   Downstream width hydraulic geometry for U.K. gravel-bed rivers, $W = aQ_b^{0.5}$ with confidence bands. Based on 43 sites in the U.K. with resistant banks. SI units m and m^3/s (English units ft and ft^3/s)


90% single response limit
a=1.87 to 3.24 (1.03–1.79)

95% mean response limit
a=2.36 to 2.57 (1.30–1.42)

Regression
a=2.46 (1.36)
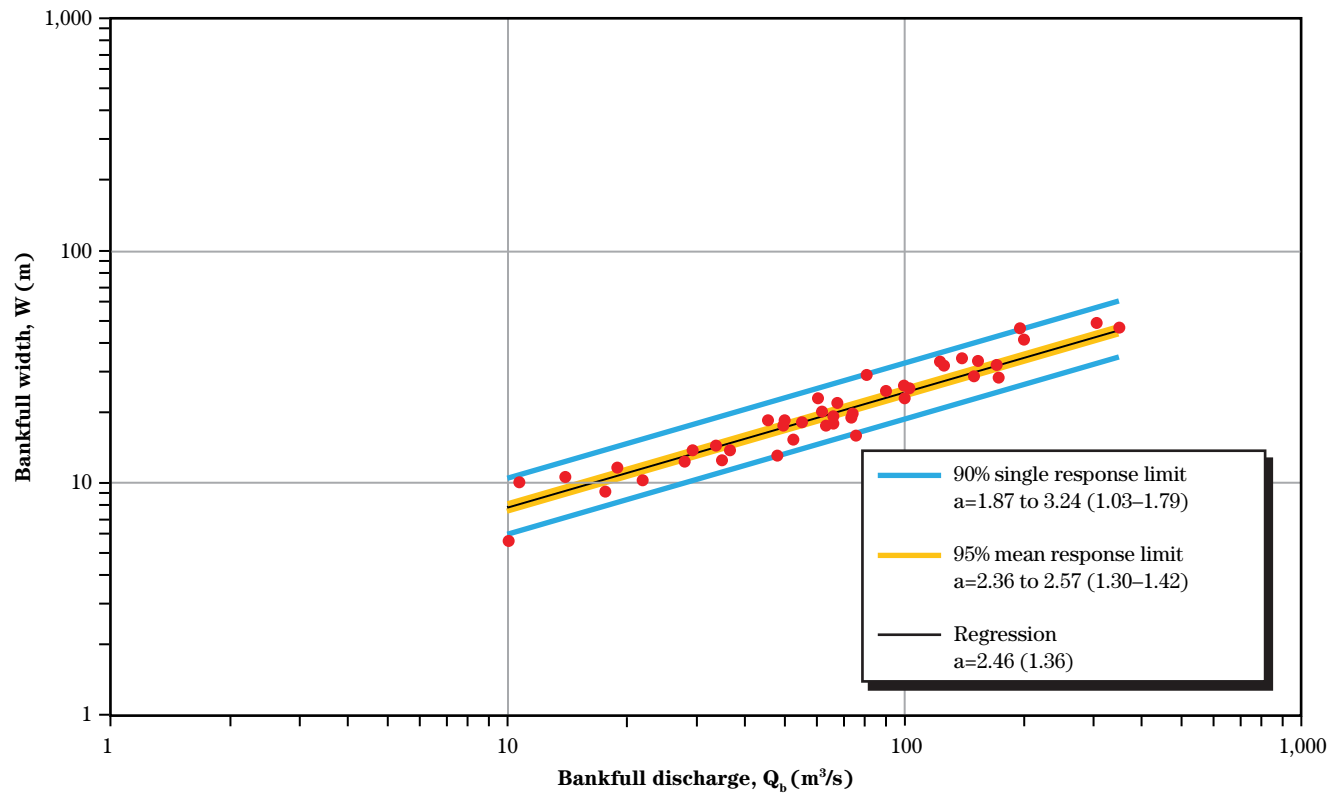
(f) Limitations of hydraulic geometry methods

- The formulas provide design variables only for the channel-forming discharge. These design variables may provide the most stable channel, but modifications may be necessary in final design to account for larger flood discharges.

- When developing a hydraulic geometry relationship from field measurements, it is difficult to determine the water surface elevation at channel-forming discharge. This is especially true in unstable channels.

- The hydraulic geometry equation must be developed from physiographically similar streams; that is, streams with depths, slopes, bed and bank material, and sediment inflow concentrations similar to the design channel.

- The assumption that channel dimensions are related only to one or two independent variables is simplistic. The data scatter associated with hydraulic geometry plots demonstrates that stability can occur at more than one combination of width and discharge. The channel-forming discharge may be the most significant factor affecting channel geometry, but other factors can also affect channel dimensions. These include the shape of the annual hydrograph, the shape and magnitude of the annual hydrograph from previous years, upstream or downstream channel control points, and localized variability in alluvial stratum.

- Hydraulic geometry relationships are assumed to be power functions. This assumption provides for visually comforting plots on log-log graph paper, but the actual data scatter may be too great for reliable final engineering design.

- Hydraulic geometry relationships are regression equations and should not be extended beyond the range of the data used to develop them, even in physiographically similar watersheds.

In summary, hydraulic geometry methods suffer the same limitation as the analogy methods. They both depend on a comparison to a channel that is adjusted in some sense. This is true whether it is a reference reach or a channel whose dimensions are used in a hydraulic geometry relationship. If the channel that is to be designed is disturbed or is likely to change due to changes in water or sediment supply, there is really no exact template that is appropriate. Hydraulic geometry relationships are useful for preliminary or trial selection of channel width. Hydraulic and sediment transport analyses are recommended for final channel design.

654.0906 Extremal hypotheses

If a reliable hydraulic geometry relationship cannot be determined from field data or when sediment transport is significant, analytical methods may be employed to obtain a range of feasible solutions. Analytical methods employ an extremal hypothesis as a third equation. One extremal hypothesis assumes that a channel will adjust its geometry so that the time rate of energy expenditure is minimized (Chang 1980; Copeland 1994). Another extremal hypothesis assumes that sediment transport is maximized within the constraints on the system (White, Bettess, and Paris 1982; Millar and Quick 1993). These are equivalent assumptions. Computer programs or look-up charts are required to solve the resistance, sediment transport, and extremal equations simultaneously. The U.S. Army Corps of Engineers Hydraulic Design Package, SAM (Thomas, Copeland, and McComas 2003), as well as HEC–RAS, contains a program to solve these equations. The program uses the Brownlie (1981) resistance and sediment transport equations for sand-bed streams, the Limerinos resistance equation, and the Meyer-Peter and Müller sediment transport equation for gravel-bed streams.

The advantage of using an extremal hypothesis is that a unique solution can be obtained for the dependent variables of width, depth, and slope. However, extensive field experience demonstrates that channels can be stable with widths, depths, and slopes different from those found at the extremal condition. Also, the sensitivity of energy minima or sediment transport maxima to changes in driving variables may be low, so that the channel dimensions corresponding to the extremal value are poorly defined.

654.0907 Constrained dependent variables

In many cases, project constraints limit the theoretical variability in channel geometry. These constraints can be anthropogenic or geologic. For example, the channel slope cannot be greater than the valley slope for a long reach. The channel width may be limited by available rights-of-way. Flood risks and damages may limit allowable depth. For these and many other reasons, the selection of one of the dependent design variables may be based on established project constraints.

654.0908 Analytical methods

After selecting one of the dependent design variables using geomorphic principles, the other two design variables can be computed using a resistance equation and a sediment transport equation. Appropriate equations can be chosen from those described in the American Society of Civil Engineers (ASCE 2006), USACE (1995a), USACE (1991b), Thomas, Copeland, and McComas (2003), or one of many sediment transport textbooks. The data ranges used in the development of sediment transport functions used in Thomas, Copeland, and McComas are given in tables 9–10 through 9–21. These summaries are based on the authors' stated ranges, as presented in their original papers. Otherwise, the summaries were determined based on the author's description of their database in combination with the data listings of Brownlie (1981) or Toffaleti (1968). A review of this information may serve as guidance in selecting the appropriate function.

The stable channel analytical method in the USACE SAM (Thomas, Copeland, and McComas 2003), provides a computer program that simultaneously solves resistance and sediment transport equations. The program provides a family of solutions from which the unique solution for depth and slope can be determined using the width determined from geomorphic principles or from project constraints. This method is described in detail in the following paragraphs.

(a) Stable channel dimensions using analytical techniques

Stable channel dimensions can be calculated analytically using computer programs or spreadsheets. The USACE SAM (Thomas, Copeland, and McComas 2003) calculates stable channel dimensions that will pass a prescribed sediment load without deposition or erosion. This routine is also available in HEC–RAS. The analytical approach (Copeland 1994) determines dependent design variables of width, slope, and depth from the independent variables of discharge, sediment inflow, and bed-material composition. It solves flow resistance and sediment transport equations simultaneously, leaving one dependent variable optional.

The extremal hypothesis (minimum stream power) can be used as a third equation for a unique solution. Be aware of the cautions associated with using the extremal hypothesis as described in the previous section of this chapter. This method is based on a typical trapezoidal cross section and assumes steady, uniform flow. The method is especially applicable to small streams because it accounts for transporting the bed-material sediment discharge in the water above the bed, not the banks, and because it separates total hydraulic roughness into bed and bank components.

(b) Basic equations for sand-bed streams

For sand-bed streams, the sediment transport and resistance equations developed by Brownlie are recommended because they account for bed-form roughness. There are separate resistance equations for upper and lower regime flow. Upper regime flow is characterized by relatively high velocities and high sediment transport. The bedforms are plane bed, antidunes or chutes, and pools, which do not provide significant form resistance. Lower regime flow is characterized by relatively low velocity and low sediment transport. The bedforms are dunes or ripples, which provide significant form resistance. The equations are dimensionless and can be used with any consistent set of units.

Upper regime

$$R_b = 0.2836 D_{50} q_*^{0.6248} S^{-0.2877} \sigma^{0.0813} \qquad \text{(eq. 9–16)}$$

Relatively high velocities and high sediment transport

Lower regime

$$R_b = 0.3742 D_{50} q_*^{0.6539} S^{-0.2542} \sigma^{0.1050} \qquad \text{(eq. 9–17)}$$

$$q_* = \frac{Vd}{\sqrt{gD_{50}^3}} \qquad \text{(eq. 9–18)}$$

Relatively low velocities and low sediment transport

where:
R_b = hydraulic radius associated with the bed (ft or m)
D_{50} = median grain size (ft or m)
S = slope

σ = geometric bed-material gradation coefficient
V = average velocity (ft/s or m/s)
d = water depth (ft or m)
g = acceleration of gravity (ft/s^2 or m/s^2)

To determine if upper or lower regime flow exists for a given set of hydraulic conditions, a grain Froude number F_g and a variable F_g' were defined by Brownlie. According to Brownlie, upper regime occurs if S>0.006 or if F_g>1.25F_g', and lower regime occurs if F_g<0.8F_g'. Between these limits is the transition zone. In the SAM, $F_g = F_g'$ is used to distinguish between upper and lower regime flow. If a spreadsheet analysis is used, the user may choose a different criterion for determining the break between upper and lower regime flow in the transition zone.

$$F_g = \frac{V}{\sqrt{gd_{50}\left(\dfrac{\gamma_s - \gamma}{\gamma}\right)}} \qquad \text{(eq. 9–19)}$$

$$F_g' = \frac{1.74}{S^{0.3333}} \qquad \text{(eq. 9–20)}$$

where:
γ_s = specific weight of sediment (lb/ft^3 or N/m^3)
γ = specific weight of water (lb/ft^3 or N/m^3)

The hydraulic radius of the side slope is calculated using Manning's equation:

$$R_s = \left(\frac{(V)(n_s)}{(CME)(S^{0.5})}\right)^{1.5} \qquad \text{(eq. 9–21)}$$

where:
R$_s$ = hydraulic radius associated with the side slopes (ft or m)
V   = average velocity (ft/s or m/s)
n_s = Manning's roughness coefficient for the bank
CME = 1.486 (English units) = 1.0 (SI units)

If the roughness height k$_s$ of the bank is known, then it can be used instead of Manning's roughness coefficient to define bank roughness. Strickler's equation can be used to calculate the bank roughness coefficient:

$$n_s = 0.039k_s^{\frac{1}{6}} \qquad \text{(eq. 9–22)}$$

where:
k$_s$ = roughness height (ft)

$$n_s = 0.048k_s^{\frac{1}{6}} \qquad \text{(eq. 9–23)}$$

where:
k$_s$ = roughness height (m)

For riprap, k$_s$ should be set equal to the minimum design d$_{90}$.

Composite hydraulic parameters are partitioned in the manner proposed by Einstein (1950):

$$A = R_b P_b + R_s P_s \qquad \text{(eq. 9–24)}$$

where:
A   = total cross-sectional area (ft^2 or m^2)
P$_b$ = perimeter of the bed (ft or m)
P$_s$ = perimeter of the side slopes (ft or m)

This method assumes that the average velocity for the total cross section is representative of the average velocity in each subsection.

Concentration, C, in parts per million, is calculated using the Brownlie sediment transport equation, which is also a regression equation. The equation is based on the same extensive set of flume and field data used to develop the Brownlie resistance equations. This equation is recommended because of its compatibility with the resistance equations, which are coupled with the sediment transport equation in the numerical solution. The equation is dimensionless and can be used with any consistent set of units.

$$C = 9,022\left(F_g - F_{go}\right)^{1.978} S^{0.6601}\left(\frac{R_b}{D_{50}}\right)^{-0.3301}$$

$$F_{go} = \frac{4.596\tau_{*0}^{0.5293}}{S^{0.1405}\sigma^{0.1606}}$$

$$\tau_{*0} = 0.22Y + 0.06(10)^{-7.7Y}$$

$$Y = \left(\sqrt{\frac{\gamma_s - \gamma}{\gamma}}R_g\right)^{-0.6}$$

$$R_g = \frac{\sqrt{gD_{50}^3}}{\upsilon} \qquad \text{(eq. 9–25)}$$

where:
C   = concentration (ppm)
R$_b$ = bed hydraulic radius (ft or m)
D$_{50}$ = median grain size (ft or m)
σ = bed-material gradation coefficient
γ_s = specific weight of sediment (lb/ft^3 or N/m^3)
γ = specific weight of water (lb/ft^3 or N/m^3)
g   = acceleration of gravity (ft/s^2 or m/s^2)
ν = kinematic viscosity (ft^2/s or m^2/s)
The other variables are dimensionless.

Table 9–10   Ackers-White transport function

Parameter	Flume data
Particle size range (mm)	0.04–7.0
Specific gravity	1.0–2.7
Multiple size classes	No
Velocity (ft/s)	0.07–7.1
Depth (ft)	0.01–1.4
Slope (ft/ft)	0.00006–0.037
Width (ft)	0.23–4
Water temperature (°F)	46–89

Table 9–11   Brownlie transport function

Parameter	River data	Flume data
Particle size range (mm)	0.086–1.4	0.088–1.4
Multiple size classes	No	No
Velocity (ft/s) [calculated]	1.2–7.9	0.7–6.6
Depth (ft)	0.35–57	0.11–1.9
Slope (ft/ft)	0.00001–0.0018	0.00027–0.017
Width (ft)	6.6–3640	0.83–8.0
Water temperature (°F)	32–95	35–102

Table 9–12   Colby transport function

Parameter	Data range
Particle size range (mm)	0.18–0.70
Multiple size classes	No
Velocity (ft/s)	0.70–8.0
Depth (ft)	0.20–57
Slope (ft/ft)	0.000031–0.010
Width (ft)	0.88–3000
Water temperature (°F)	32–89
Correction for fines (ppm)	Yes

Table 9–13   Einstein transport function

Parameter	Flume data
Particle size range (mm)	0.78–29
Multiple size classes	Yes
Velocity (ft/s)	0.9–9.4
Depth (ft)	0.03–3.6
Slope (ft/ft)	0.00037–0.018
Width (ft)	0.66–6.6
Water temperature (°F)	Not reported

Table 9–14   Laursen (Copeland 1994) transport function

Parameter	River data	Flume data
Median particle size range (mm)	0.08–0.70	0.011–29
Multiple size classes	Yes	Yes
Velocity (ft/s)	0.068–7.8	0.70–9.4
Depth (ft)	0.67–54	0.03–3.6
Slope (ft/ft)	0.0000021–0.0018	0.00025–0.025
Width (ft)	63–3640	0.25–6.6
Water temperature (°F)	32–93	46–83

Table 9–15   Laursen (Madden 1985) transport function

Parameter	Data range
Particle size range (mm)	0.04–4.8
Multiple size classes	Yes
Velocity, (ft/s)	0.85–7.7
Depth (ft)	0.25–54
Slope (ft/ft)	0.00001–0.1
Width (ft)	3–3640
Water temperature (°F)	36–90

Table 9–16 Meyer-Peter and Müller (1948) transport function

Parameter	Data range
Particle size range (mm)	0.4–29
Particle specific gravity	1.25–4
Multiple size classes	Yes
Velocity (ft/s)	1.2–9.4
Depth (ft)	0.03–3.9
Slope (ft/ft)	0.0004–0.02
Width (ft)	0.5–6.6
Water temperature (˚F)	Not published

Table 9–17 Parker transport function

Parameter	River data
Median particle size range (mm)	18–28
Total particle size range (mm)	2–102
Multiple size classes	Yes
Velocity (ft/s)	2.6–3.7
Depth (ft)	1.0–1.5
Slope (ft/ft)	0.0097–0.011
Width (ft)	16–20
Water temperature (˚F)	41–44

Table 9–18 Profitt (Profitt and Sutherland 1983) transport function

Parameter	River data
Particle size range (mm)	2.90–12
Multiple size classes	Yes
Velocity (ft/s)	2.00–3.4
Depth (ft)	0.35–0.84
Slope (ft/ft)	0.003
Width (ft)	2.00
Water temperature (˚F)	59–63

Table 9–19 Schoklitsch transport function

Parameter	Data range
Particle size range (mm)	0.3–4.9
Multiple size classes	No
Velocity (ft/s)	0.8–4.5
Depth (ft)	0.037–0.74
Slope (ft/ft)	0.00012–0.055
Width (ft)	0.23–2.0
Water temperature (˚F)	Not published

Table 9–20 Toffaleti transport function

Parameter	River data	Flume data
Median particle size range (mm)	0.095–0.76	0.91–0.45
Total particle size range (mm)	0.062–4	0.062–4
Multiple size classes	Yes	Yes
Velocity (ft/s)	0.7–7.8	0.7–6.3
Hydraulic radius (ft)	0.7–56.7	0.07–1.1
Slope (ft/ft)	0.000002–0.0011	0.00014–0.019
Width (ft)	63–3640	0.8–8
Water temperature (˚F)	32–93	40–93

Table 9–21 Yang transport function

Parameter	Sand data	Gravel data
Particle size range (mm)	0.15–1.7	2.5–7.0
Multiple size classes	No	No
Velocity, (ft/s)	0.8–6.4	1.4–5.1
Depth (ft)	0.04–50	0.08–0.72
Slope (ft/ft)	0.000043–0.028	0.0012–0.029
Width (ft)	0.44–1750	0.7–1.3
Water temperature (˚F)	32–94	Not reported

(c) Basic equations for gravel-bed streams

For gravel-bed streams, equations more appropriate for coarse bed streams should be used in the analytical solution. The Limerinos equation is recommended to calculate grain roughness on the bed. The Meyer-Peter and Müller equation is recommended to calculate sediment transport. The advantage of the Limerinos equation is that it accounts for the decrease in roughness with increasing water depth in cases where the bed roughness is primarily due to the dimensions of the sediment grains (sand, gravel, or cobbles) on the bed. The Manning equation can be used to calculate the roughness on the channel side slope. The Manning equation is appropriate for this case because bank roughness is best estimated using experience and engineering judgment. Additional roughness may be added in the manner suggested by Cowan (1956).

The Limerinos equation accounts for the grain roughness in a uniform reach of a gravel-bed stream that is relatively free of bedforms. The Limerinos equation may be presented in dimensionless units as:

$$\frac{V}{U'_*} = 5.66 \log \left(3.80 \frac{R'_b}{D_{84}} \right)$$

(eq. 9–26)

where:
V   = average velocity (ft/s or m/s)
U'_* = shear velocity associated with grain roughness (ft/s or m/s)
R'_b = hydraulic radius associated with grain roughness (ft or m)
D_{84} = grain size for which 84 percent of the bed is finer (ft or m)

Manning's roughness coefficient associated with grain roughness can be determined from the Limerinos equation:

$$n'_b = \frac{(CME)\left(R'_b{}^{\frac{1}{6}}\right)}{g5.66 \log\left(3.80 \frac{R'_b}{D_{84}}\right)}$$

(eq. 9–27)

where:
n'_b = roughness coefficient associated with bed
CME = 1.486 English units (1.0 SI units)
R'_b = bed hydraulic radius associated with grain roughness (ft or m)

g   = acceleration of gravity (ft/s or m/s)
D_{84} = grain size for which 84 percent of the bed is finer (ft or m)

Additional bed roughness may be added to the grain bed roughness using the Cowan (1956) method. Roughness may be added to account for factors such as surface irregularities, variability in channel shape, obstructions, vegetation, and meandering. Meandering can be accounted for with a meandering coefficient, m. In a straight channel, the meandering coefficient is 1.0. Appropriate values for the meandering coefficient and additions to the grain roughness n value can be found in Cowan (1956) and Chow (1959). Using the Cowan equation the total bed roughness coefficient is:

$$n_b = m\left(n''_b + n'_b\right)$$

(eq. 9–28)

where:
n_b = total bed roughness coefficient
m   = Cowan meander coefficient
n''_b = bed roughness other than grain roughness

Using the Manning equation the hydraulic radius associated with the bed is calculated

$$R_b = \left(\frac{(V)(n_b)}{\left(S^{\frac{1}{2}}\right)(CME)} \right)^{\frac{3}{2}}$$

(eq. 9–29)

where:
S   = channel slope, dimensionless

The hydraulic radius associated with the bank or side slope is

$$R_s = \left(\frac{(V)(n_s)}{\left(S^{\frac{1}{2}}\right)(CME)} \right)^{\frac{3}{2}}$$

(eq. 9–30)

where:
n_s = the roughness coefficient associated with the side slope

Values for m, n''_b and n_s must be selected by the designer. Using the Cowan method provides water surface elevations that account for the total channel roughness and not just the grain roughness.

Sediment transport can be calculated using the Meyer-Peter and Müller equation:

$$\left(\frac{k_r}{k_r'}\right)^{\frac{3}{2}} \gamma_w R_b S = 0.047\left(\gamma_s - \gamma_w\right) D_m + 0.25\left(\frac{\gamma_w}{g}\right)^{\frac{1}{3}}\left(\frac{\gamma_s - \gamma_w}{\gamma_s}\right)^{\frac{2}{3}} g_s^{\frac{2}{3}}$$

(eq. 9–31)

where:
k_r = total bed roughness = $\dfrac{1}{n_b}$

k_r' = particle roughness = $\dfrac{1}{n_b'}$

γ_w = specific weight of water (lb/ft^3 or N/m^3)
γ_s = specific weight of sediment (lb/ft^3 or N/m^3)
D_m = median sediment size (ft or m)
g_s = sediment transport (lb/s-ft or N/s-m)
R_b = bed hydraulic radius (ft or m)

and

$$\left(\frac{k_r}{k_r'}\right)^{\frac{3}{2}} R_b = R_b'$$

(eq. 9–32)

and

$$D_m = \sum_{i=1}^{n} f_i D_i$$

(eq. 9–33)

where:
f_i = fraction of size class "i" in bed
D_i = geometric mean diameter of size class "i" in bed (ft or m)

(d) Calculating sediment discharge and concentration

A typical cross section, with the critical hydraulic parameters labeled, is shown in figure 9–13. The concentration calculated from the sediment transport equation applies only vertically above the bed. Total sediment transport in weight per unit time is calculated by the following equation:

$$Q_s = \gamma CBDV$$

(eq. 9–34)

where:
Q_s = sediment transport (weight/time)
B = base width, length

An average concentration for the total discharge is then calculated:

$$\bar{C} = \frac{Q_s}{0.027Q}$$

(eq. 9–35)

where:
$\bar{C}$ = concentration using the total discharge (ppm)
Q_s = sediment transport (tons/d)
Q = discharge (ft^3/s)

(e) Input requirements

Required input data for the analytical method are sediment inflow concentration, side slope, bank roughness coefficient, additional channel roughness and meandering coefficients for the Cowan method, bed material D_{50}, bed-material gradation coefficient, and water discharge. If sediment inflow is to be calculated, which is the recommended procedure, additional data are required for the supply reach. These are base width, side slope, bank roughness coefficient, bed material

Figure 9–13   Typical cross section used in analytical method
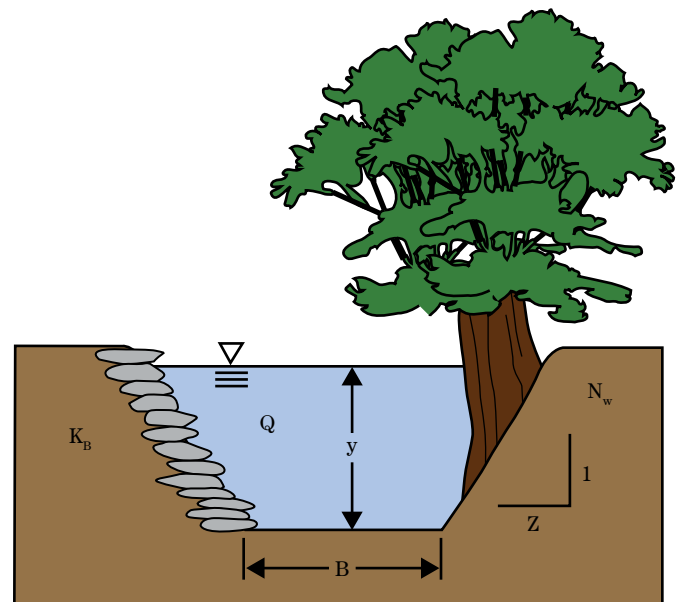
median grain size, geometric gradation coefficient, average slope, and discharge. It is important that the base width be representative of the total movable bed width of the channel. Additional channel roughness due to surface irregularities, variability in channel shape, obstructions, instream structures, vegetation, and meandering are added using the Cowan method. If either the USACE SAM or HEC–RAS program is used, adding additional roughness with the Cowan method is only available with the gravel-bed option. If the sand-bed option is used, only grain and form roughness is included in the Brownlie equations, so additional roughness can only be added by increasing the roughness coefficient assigned to the bank. In this case, the bank roughness should serve as a composite of all additional roughness factors. This can be accomplished using one of the hydraulic compositing methods described in NEH654.06. Only flow over the bed is considered capable of transporting the bed-material sediment load.

Water discharge

The design discharge is critical in determining appropriate dimensions for the channel. The channel-forming discharge will provide the most stable channel, but it is also important to evaluate how the design channel will respond during flood events. The channel-forming discharge is typically used to set channel dimensions, and flood discharges are used to evaluate channel performance at design conditions.

Investigators have proposed different methods for estimating the channel-forming discharge. The 2-year frequency peak discharge is sometimes used for perennial streams. Some have suggested that the 10-year frequency peak discharge is more appropriate for ephemeral and intermittent streams. The bankfull discharge is sometimes suggested. Others prefer using the effective discharge, which is the discharge that transports the most bed-material sediment. Currently, there is no generally accepted method for determining the channel-forming discharge. It is recommended that a range of discharges be used in the analysis to test sensitivity of the solution.

Inflowing sediment discharge

This is the concentration of the inflowing bed-material load. The bed-material load should be calculated using the same sediment transport equation and same hydraulic equations that are used in the analysis of the

design channel. This is automatically done in USACE SAM or HEC–RAS, if the dimensions and bed-material composition of the upstream supply reach are supplied as input data. Measured data may be used to evaluate the applicability of the Brownlie or Meyer-Peter and Müller equations, but measured data should not be used as input to the analytical method.

Valley slope

Valley slope is the maximum possible slope for the channel invert. The valley slope is determined by the local topography, and a channel with a slope equal to the valley slope would be straight. The valley slope is used to test for sediment deposition. If the minimum slope that will transport the incoming bed-material load is greater than the valley slope, it is not possible to design a stable channel, and deposition is inevitable.

Bank slopes and roughness

The analytical method assumes that all bed-material transport occurs over the bed of the cross section and that none occurs above the side slopes. Therefore, the portion of water conveyed above the side slopes expends energy, but does not transport sediment. This makes the selection of base width in the supply reach important. The base width should reflect the entire alluvial boundary of the channel. In the design reach, the designer must select the channel side slope and side slope roughness. As in the supply reach, sediment transport is calculated only above the base width in the design reach. Therefore, sediment discharge will increase with the selection of steeper design bank angles.

(f)   Range of solutions

For each specified combination of water discharge, sediment transport rate, and transport grain size, unique values of slope and depth are calculated. This can be used to evaluate stability in an existing channel or to evaluate stability in a proposed channel. Considering river morphology is important when interpreting these calculated values. Consistent is also important in the selection of channel dimensions; that is, once a width is selected, the depth and slope are fixed. This allows the designer to consider specific project constraints, such as right-of-way, bank height, sinuosity, bend radius, and minimum bed slope. A consistent set of channel dimensions can then be computed.

If the calculations indicate that the slope of the project channel needs to be less than the natural terrain, the calculated slopes can be used to aid in spacing drop structures or in introducing sinuosity into the project alignment.

An example of a family of slope-width solutions that satisfy the resistance and sediment transport equations for the design discharge is illustrated in figure 9–14. Designers typically use these to focus on a range of appropriate width slope combinations, rather than on a single value set. Any combination of slope and base width from this curve will be stable for the prescribed channel design discharge. Combinations of width and slope that plot above the stability curve will result in degradation, and combinations below the curve will result in aggradation. The greater the distance from the curve, the more severe the instability.

Constraints on this wide range of solutions may result from a maximum possible slope or a width constraint due to right-of-way. Maximum allowable depth could also be a constraint. With constraints, the range of solutions is reduced.

Different water and sediment discharges will produce different stability curves. A channel designed for the channel-forming discharge may not be stable at a different discharge. To evaluate the significance of this difference, a stable channel solution is first obtained for the channel-forming discharge. Then, stability curves are calculated for a range of discharges to determine how sensitive the channel dimensions are to variations in water and sediment inflow events. Figure 9–15 shows two stability curves for the same supply reach, but different discharges. The stability curve in this figure is for a channel-forming discharge of 5,000 cubic feet per second. Any width-slope solution along this line will theoretically provide a channel with long-term sediment continuity and stability. If the design channel has a depth constraint for flood control, a width-slope solution is selected from the right end of the stability curve where widths are greater and depths lower.

Conversely, if the design channel has a width constraint due to limited right-of-way, the width-slope solution is selected from the left end of the stability curve. To evaluate channel response for another discharge, a new stability curve is calculated and the design dimensions and compared to the new stability curve. For example, in figure 9–15, a stability curve for a flood discharge of 30,000 cubic feet per second is shown. Width-slope solutions that plot above the flood stability curve indicate that the design channel will degrade during the flood, and width-slope solutions that plot below the flood stability curve will aggrade during the flood. Figure 9–15 shows that degradation will occur during the flood in the channel designed with a depth constraint, and that aggradation will occur during the flood in the channel designed with a width constraint. Note that there is only one combination of width-slope solutions that satisfy sediment continuity for both discharges.

Long-term aggradation and degradation are associated with the channel-forming discharge, but short-term aggradation or degradation can occur during a flood event depending on which channel dimensions are selected from the stable channel stability curve.

Using a spreadsheet or USACE SAM or HEC–RAS, stable channel dimensions can be calculated for a range of widths on either side of a prescribed median value. It is recommended that calculations be made for at least 20 base widths, each with an increment of 0.1 times the median base width. Stability curves can then be plotted from these data. Typically there will be two solutions for each slope.

A solution for minimum stream power can also be calculated. This solution represents the minimum slope that will transport the incoming sediment load. Opinions are divided regarding the use of minimum stream power to uniquely define channel stability.

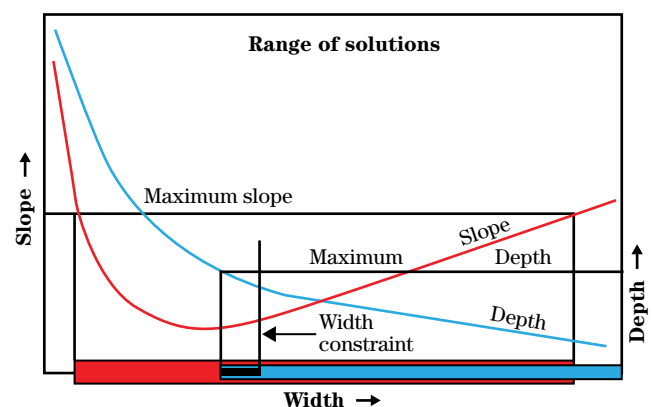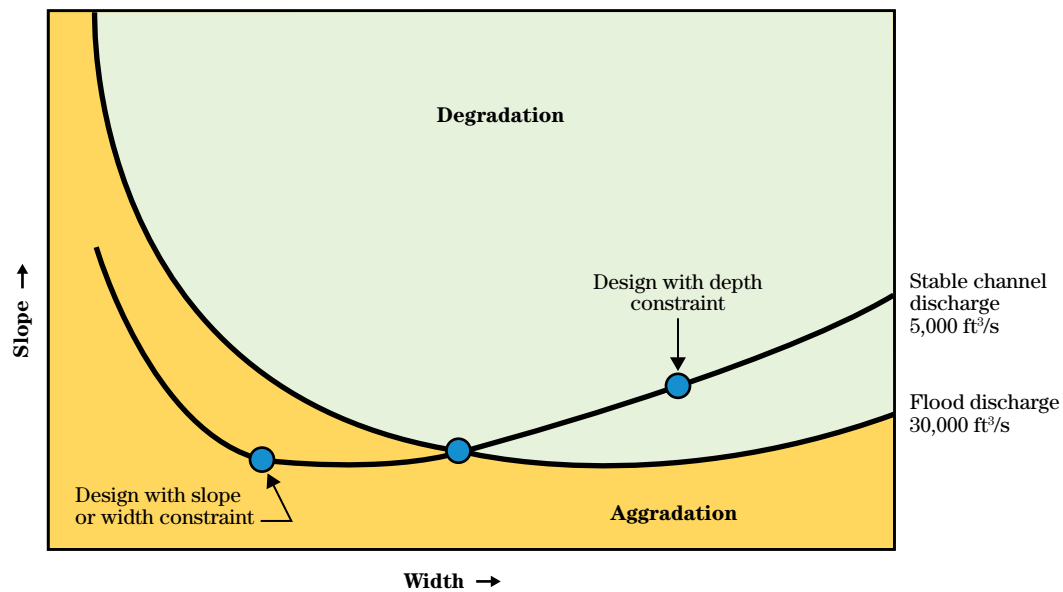**Figure 9–14** Stability curve from stable channel analytical method

Figure 9–15   Stability curves for channel-forming (stable channel) discharge and a flood discharge

654.0909 Sediment impact analysis

Stream restoration projects should not be designed using only a single flow event and the sediment load transported by that event. This approach does not account for potential instability driven by the range of natural flow events. A determination of the potential for aggradation or degradation in a channel reach requires an assessment of the reach-scale sediment budget. The sediment impact assessment is a closure loop at the end of the design procedure to:

- validate the efficacy of the restored channel geometry

- identify flows which may cause aggradation or degradation over the short term (these changes are inevitable and acceptable in a dynamic channel)

- recommend minor adjustments to the channel design to ensure dynamic stability over the medium to long term

This can be accomplished using a sediment budget approach for relatively simple projects or by using a numerical model that incorporates a solution of the sediment continuity equation for more complex projects. More information on this subject is provided in NEH654.13.

654.0910 Basic steps in alluvial channel design

Step 1   Determine the channel-forming discharge—The initial design step is to determine the stable geometry for a single discharge. Use bankfull discharge, effective discharge, or a specific peak frequency as described in NEH654.05.

Step 2   Determine sediment inflow for the project reach—Calculate a sediment transport rating curve for the upstream supply reach. The sediment discharge may be computed based on a typical upstream cross section using a normal depth equation and an appropriate sediment transport equation.

Step 3   Develop a stability curve—Calculate a family of slope-width-depth solutions that satisfy resistance and sediment transport equations for the channel-forming discharge. This step provides a channel geometry that is capable of transporting the inflowing sediment load through the project reach. The equations are used to calculate the design variables of width, slope, and depth from the independent variables of discharge and sediment inflow.

Step 4   Determine channel width—A channel top width for the channel-forming discharge is selected from the stability curve using geomorphic principles or project constraints. Analogy methods, hydraulic geometry curves, or the extremal hypothesis are geomorphic relations that can be used to select width. Depth and slope for the selected width are determined from the stability curve.

Step 5   Conduct an analytical sediment budget analysis—Using the design channel dimensions, calculate a sediment-transport rating curve in the project reach. Using a flow-duration curve that includes some high flood discharges, calculate sediment yield into and out of the project reach. More information is provided in NEH654.13.

Step 6   Determine channel planform—Sinuosity is determined from the calculated channel slope and valley slope. Remaining planform design parameters include the meander wavelength, an appropriate channel length for one meander wave-

length, and the trace of the channel. These can be determined from analogy methods, hydraulic geometry relations, or analytical techniques that assume minimum expenditure of energy. These techniques are described in NEH654.12.

Step 7 Natural variability in cross-sectional shape—Variability in channel width and depth can either be allowed to develop naturally or can be part of the project design. Sand-bed streams have the ability to create natural variability in channel form rather quickly because they are characterized by significant bed-material sediment transport. Gravel-bed streams typically adjust much more slowly. Streams with very little bed-material movement may not adjust at all. If variability is to be included in the project design, dimensions for cross sections in riffles and pools can be obtained from stable reaches of the existing stream or from reference reaches. Thorne (1988) has provided morphologic relationships for channel width for a meandering sand-bed river. Other researchers have correlated variability to riparian and bank conditions. Analogy methods have also been used in the design of variability. Techniques for design of variability in cross-sectional shape are described in NEH654.12.

Step 8 Instream structures—Successful stream restoration often includes the use of bank protection, grade control, and habitat features. To restore a stream with physical habitat features resembling a natural stream, a combined technology approach is required. Sound physical principles and well established engineering formulas are used in the analysis and design of both soft and hard features. Systems composed of living plant materials are often used in association with inert materials, such as wood or rock, and manufactured products. A significant flood event (normally no smaller than the 10-year frequency discharge) is used to size structures and compute scour depths. In addition, the quantity of water and its related hydroperiod largely determines what type of vegetation will grow in an area. The flexibility of these features depends on the project goals, tolerance for project change, and consequences of failure. Consideration is given to the effects that proposed features could have on flooding. For example, vegetation often increases boundary roughness, decreasing velocities, and increasing flood profiles. Additional design considerations

include the level of risk that is acceptable, natural system dynamics, anthropogenic activities in the watershed, the construction time frame, existing infrastructure, and desired speed of improvement, cost, and maintenance. Guidelines for design of instream structures are described in NEH654.14.

Example 2: Stable channel analytical method

Objective: Determine stable channel dimensions for a diversion channel. Upstream natural stream is coming out of a hillside watershed.

Given: Dimensions of the upstream natural channel reach are:

Base width = 22 ft

Side slopes

　　Left bank = 2.2H:1V

　　Right bank = 1.1H:1V

Side slope roughness coefficient = 0.07

Channel slope = 0.0025

Bed material — sandy gravel

　　D_{84} = 22 mm　　　D_{50} = 3.7 mm

　　D_{16} = 0.43 mm

Channel-forming discharge = 2,500 ft^3/s

Design values for the bypass channel:

Side slopes = 3H:1V

Side slope roughness coefficient = 0.045

Valley slope = 0.0020 (maximum design slope)

Solution: Solve the Brownlie resistance and sediment transport equations using the USACE SAM (Thomas, Copeland, and McComas 2003) or another program or spreadsheet. Example output is shown in figure 9–16. The sand-bed equations are chosen because bedforms may occur at the channel-forming discharge. However, the bed gradation is borderline between sand and gravel, and it would be prudent to make computations using both sand and gravel equations. From this table, stability curves for slope and depth as a function of depth can be platted (figs. 9–17 and 9–18).

The stability curves provide a family of solutions for width, depth, and slope that satisfy the resistance and sediment transport equations. Any combination of solutions on these curves will theoretically be stable in terms of aggradation and degradation. If the extremal hypothesis is adopted, a unique solution is provided. In this case:

　Base width　=　67 ft
　Depth　　　=　6.7 ft
　Slope　　　=　0.001879

If a straight channel is desired, then the channel slope would be set equal to the valley slope, 0.0020 (from the stability curves, base width = 38 ft, depth = 8.4 ft).

If a sinuous meandering channel is desired, then the maximum sinuosity for a stable channel can be calculated by dividing the valley slope by the calculated slope at minimum stream power.

$$\frac{0.0020}{0.001879} = 1.06$$

The stable channel dimensions for base width and depth are values calculated at minimum stream power. Any additional sinuosity would result in an aggrading stream. Thus, the only stable solutions occur with sinuosities between 1.0 and 1.06.

If one objective of this channel is flood control then it is best to design a compound channel to achieve both maximum channel stability and flood control benefits. The low-flow channel would be designed based on stability concepts for the channel-forming discharge, while the width and depth of the overflow channels would be based on normal depth or backwater calculations in a compound channel for the design flood (NEH654.06).

The design channel should be checked for the full range of expected natural flow conditions. A sediment budget analysis should be conducted to determine if there will be long-term aggradation or degradation in the channel. A hydraulic analysis should also be conducted at a design flood flow to obtain critical velocities for design on in channel structures and bank protection. It may be necessary to revise the initial design and iterate on a final solution that meets additional project constraints.

Example 2: Stable channel analytical method—Continued

Figure 9–16   Sample output from USACE SAM

```
****************************************************************************
*          SAMwin Software Registered to the US Army Corps of Engineers    *
****************************************************************************
*                        HYDRAULIC CALCULATIONS                            *
*                            Version 1.0                                   *
*        A Product of the Flood Control Channels Research Program          *
* Coastal & Hydraulics Laboratory, USAE Engineer Research & Development Center *
*                           in cooperation with                           *
*              Owen Ayres & Associates, Inc., Ft. Collins, CO              *
****************************************************************************
CALCULATE CHANNEL WIDTH, DEPTH AND SLOPE BY COPELAND METHOD.
CALCULATE INFLOWING SEDIMENT CONCENTRATION, PPM.

 INFLOWING WATER DISCHARGE, CFS =  2500.000
 BASE WIDTH                     =    22.00000
 CHANNEL SLOPE, FT/FT           =     0.00250000


            LEFT BANK          RIGHT BANK
SIDE SLOPE =  2.200               1.100
n-VALUE    =  0.07000             0.07000


 CALCULATE STABLE CHANNEL DIMENSIONS.
 USING BROWNLIE'S RESISTANCE & TRANSPORT EQUATIONS


 MEDIAN BED SIZE ON BED, MM = 3.64849
 GRADATION COEFFICIENT  = 9.950
 VALLEY SLOPE    = 0.00200000


            LEFT               BANK RIGHT BANK
SIDE SLOPE =  3.000             3.000
n-VALUE    =  0.04500           0.04500

 STABLE CHANNELS FOR Q=2500.0, C mgl=210.8, D50=3.648
```

Example 2: Stable channel analytical method—Continued

Figure 9–16   Sample output from USACE SAM—Continued

	BOTTOM WIDTH	DEPTH	ENERGY SLOPE	CMPOSIT n-Value	HYD RADIUS	VEL	FROUDE NUMBER	SHEAR STRESS	BED REGIME*
	FT	FT	FT/FT		FT	FPS		#/SF	
1	5.	10.4	0.004550	0.0456	5.30	6.69	0.37	2.94	LO
2	10.	10.4	0.003240	0.0458	5.64	5.86	0.32	2.10	LO
3	15.	10.1	0.002711	0.0459	5.81	5.45	0.30	1.71	LO
4	20.	9.8	0.002422	0.0460	5.89	5.19	0.29	1.48	LO
5	25.	9.4	0.002241	0.0460	5.92	5.00	0.29	1.31	LO
6	30.	9.0	0.002121	0.0461	5.91	4.86	0.29	1.19	LO
7	35.	8.6	0.002039	0.0461	5.88	4.74	0.28	1.10	LO
8	40.	8.3	0.001981	0.0461	5.82	4.65	0.28	1.03	LO
9	45.	8.0	0.001940	0.0461	5.75	4.56	0.29	0.96	LO
10	50.	7.6	0.001913	0.0460	5.67	4.49	0.29	0.91	LO
11	55.	7.3	0.001895	0.0460	5.57	4.42	0.29	0.87	LO
12	60.	7.1	0.001884	0.0460	5.48	4.36	0.29	0.83	LO
13	65.	6.8	0.001880	0.0459	5.38	4.31	0.29	0.80	LO
14	70.	6.6	0.001880	0.0459	5.27	4.26	0.29	0.77	LO
15	75.	6.3	0.001883	0.0458	5.17	4.21	0.30	0.74	LO
16	80.	6.1	0.001890	0.0457	5.06	4.16	0.30	0.72	LO
17	85.	5.9	0.001899	0.0457	4.96	4.12	0.30	0.70	LO
18	90.	5.7	0.001911	0.0456	4.85	4.08	0.30	0.68	LO
19	95.	5.5	0.001924	0.0456	4.75	4.04	0.30	0.66	LO
20	100.	5.4	0.001939	0.0455	4.65	4.01	0.30	0.65	LO
	RESULTS AT MINIMUM STREAM POWER								
21	67.	6.7	0.001879	0.0459	5.32	4.28	0.29	0.78	LO

* REGIMES: LO=LOWER, TL=TRANSITIONAL-LOWER, TU=TRANSITIONAL-UPPER, UP=UPPER

Example 2: Stable channel analytical method—Continued

Figure 9–17　　Stability curve slope versus base width, Q = 2,500 ft³/s, bed-material sediment concentration = 211 mg/L. Brownlie resistance and sediment transport equations

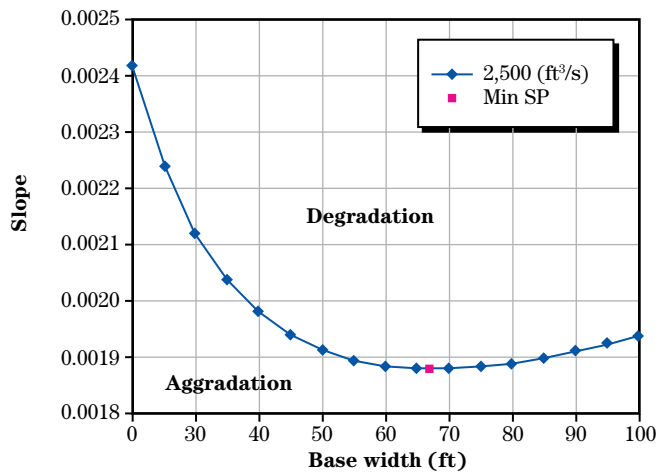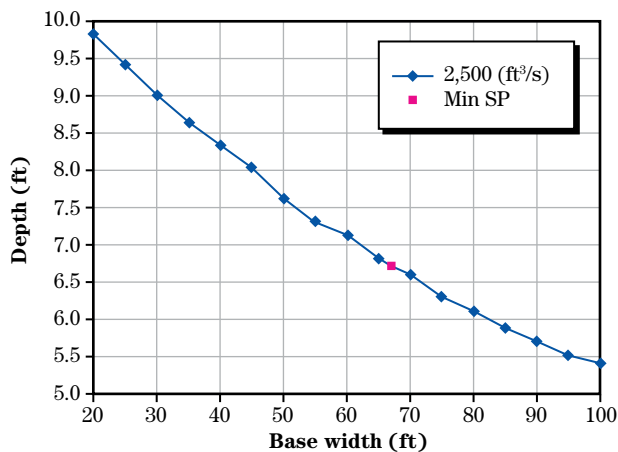**Figure 9–18**　　Stability curve depth versus base width, Q = 2,500 ft³/s, bed-material sediment concentration = 211 mg/L. Brownlie resistance and sediment transport equations

Example 3: Hydraulic design with hydraulic geometry and the stable channel analytical method

Objective: Determine stable channel dimensions for single thread meandering channel that maximizes habitat benefits on an existing braided alluvial fan. The upstream natural stream comes out of a hillside watershed. The project reach is an alluvial fan with a braided channel that flows into a larger river downstream from the project reach. Note that the braided alluvial fan may be a naturally stable channel, but due to the wide shallow flow, water temperature is too high for certain fish species. Cross-sectional variability is negligible due to the lack of pools and riffles.

Given: Dimensions of the upstream natural channel reach are:

    Base width = 55 ft
    Side slopes
        Left bank = 1.5H:1V
        Right bank = 1.5H:1V
    Side slope roughness coefficient = 0.08

Use Cowan method and add 0.01 to upstream channel roughness to account for channel irregularity.

Channel slope = 0.0065
Bed material—gravel
    D_{84} = 19.7 mm
    D_{50} = 6.9 mm
    D_{16} = 0.76 mm
Channel-forming discharge = 1,500 ft³/s

Design values for the single-thread channel:

        Side slopes = 1V:2.5H
        Side slope roughness coefficient = 0.05
        Use Cowan method and add 0.005 to account for channel irregularity
        Valley slope = 0.0055 (maximum design slope)

Solution: Solve the Limerinos resistance and Meyer-Peter and Müller sediment transport equations using the USACE SAM (Thomas, Copeland, and McComas 2003), HEC–RAS, or another program or spreadsheet. Example output is shown in figure 9–19. The gravel-bed equations are chosen because bedforms are not expected to be a factor. From this table, stability curves for slope and depth as a function of depth can be plotted (figs. 9–20 and 9–21).

The stability curves provide a family of solutions for width, depth, and slope that satisfy the resistance and sediment transport equations. Any combination of solutions on these curves will theoretically be stable in terms of aggradation and degradation. Note that a wide range of width solutions will satisfy sediment continuity requirements with a slope of about 0.0045. Selecting a design slope in this range will provide for a stable channel. If the extremal hypothesis is adopted, a unique solution is provided. In this case:

    Base width = 150 ft
    Depth   = 1.9 ft
    Slope   = 0.004488

These channel dimensions also provide the maximum sinuosity with a stable channel. The sinuosity is calculated by dividing the valley slope by the calculated slope at minimum stream power. In this case:

$$\text{Sinuosity} = \frac{0.0055}{0.004488} = 1.23$$

Any additional sinuosity would result in an aggrading stream. Thus, the only stable solutions occur with sinuosities between 1.0 and 1.23.

If a straight channel is desired, then the channel slope would be set equal to the valley slope, 0.0055. Base width and depth, at a slope of 0.0055, can be read from the stability curves:

    Base width   = 60 ft
    Depth   = 3.4 ft

Hydraulic geometry relationships may be used to select an appropriate width. Ideally, a hydraulic geometry relationship could be developed from the study watershed or a regional hydraulic geometry relationship from physiographically similar watersheds might be available. Lacking one of these, figure 9–9, developed from North American gravel-bed rivers, can be used. Converting 1,500 cubic feet per second to 42.5 cubic meters per second, a bankfull width of 24 meters (79 ft) is obtained from the mean regression line. The hydraulic geometry relationship refers to top width, while the stable channel analytical method in the USACE SAM calculates base width. Figure 9–22 is a top width versus base width curve developed from

Example 3: Hydraulic design with hydraulic geometry and the stable channel analytical method—Continued

the SAM output for the design channel which has side slopes of 1V:2.5H. A top width of 79 feet corresponds to a base width of 62 feet. Going back to the stability curve, this width would require a channel slope of about 0.0055. This is equal to the valley slope and, therefore, would be a straight channel. Using the maximum 90 percent single response limit from figure 9–9, a bankfull width of 43.5 meters or 142 feet is calculated. This corresponds to a base width of 132 feet and a stable channel slope of 0.0045. The design base width should be between 62 feet and 132 feet to satisfy both hydraulic geometry relationships and sediment continuity requirements. Decreasing the width provides for greater depths and more shade from trees on the banks, but it also decreases the sinuosity and channel variability that accompanies meandering. The following mean channel dimensions would be appropriate:

Base width = 80 ft
Slope = 0.005
Depth = 2.9 ft
Sinuosity = 1.1

The analogy method is another means of selecting an appropriate channel width. The reference reach used for the analogy method must be from a physiographically similar watershed. In this case, the upstream channel is not appropriate because the channel slope is significantly different from the slope in the design reach. The reference reach would need to be from a watershed with a similar sized drainage area that originates in the hills and flows onto an alluvial plain similar to the project reach. The key stability factor here is the abrupt change in slope between the upland stream and the alluvial fan stream. The reference reach would need to be stable and should have the favorable habitat characteristics desired in the project reach.

Other possible criteria for selecting the channel width could be constrained rights-of-way or minimum flow depths for habitat preservation. Minimum flow depths for a specified percent exceedance discharge can be determined by calculating normal depth for the proposed width.

The design channel should be checked for the full range of expected natural flow conditions. A sediment budget analysis should be conducted to determine if

there will be long-term aggradation or degradation in the channel. A hydraulic analysis at a design flood flow should also be conducted to obtain critical velocities for design on inchannel structures and bank protection if necessary. It may be necessary to revise the initial design and iterate on a final solution that meets additional project constraints.

This example provides average channel dimensions of width, depth, and slope for the project channel. The planform layout is the next design parameter and is described in NEH654.12. Channel variability (riffles and pools) is also addressed in this chapter.

Example 3: Hydraulic design with hydraulic geometry and the stable channel analytical method—Continued

Figure 9–19   Sample output from USACE SAM

```
****************************************************************************
       SAMwin Software Registered to the US Army Corps of Engineers       *
****************************************************************************
*                      HYDRAULIC CALCULATIONS                             *
*                          Version 1.0                                    *
*       A Product of the Flood Control Channels Research Program          *
* Coastal & Hydraulics Laboratory, USAE Engineer Research & Development   *
* Center *in cooperation with Owen Ayres & Associates, Inc., Ft. Collins, CO *
****************************************************************************
**************************
 CALCULATE CHANNEL WIDTH, DEPTH AND SLOPE BY COPELAND METHOD.
 CALCULATE INFLOWING SEDIMENT CONCENTRATION, PPM.

 INFLOWING WATER DISCHARGE, CFS  =  1500.000
 BASE WIDTH, FT               = 55.00000
 CHANNEL SLOPE, FT/FT         = 0.00650000

            LEFT BANK     RIGHT BANK
 SIDE SLOPE  = 1.500        1.500
 n-VALUE     = 0.08000      0.08000

 CALCULATE STABLE CHANNEL DIMENSIONS.
 USING MEYER-PETER-MULLER & LIMERINOS EQUATIONS

 MEDIAN BED SIZE ON BED, MM   =  6.87789
 GRADATION COEFFICIENT        =  5.971
 VALLEY SLOPE                 =  0.00550000

            LEFT BANK     RIGHT BANK
 SIDE SLOPE  = 2.500        2.500
 n-VALUE     = 0.05000      0.05000

 STABLE CHANNELS FOR Q=1500.0, C,mgL=1917., D50=6.878mm
```

Example 3: Hydraulic design with hydraulic geometry and the stable channel analytical method—Continued

Figure 9–19   Sample output from USACE SAM—Continued

	BOTTOM WIDTH	DEPTH	ENERGY SLOPE	CMPOSIT n-Value	HYD RADIUS	VEL	FROUDE NUMBER	SHEAR STRESS
1	10.	6.0	0.020654	0.0494	3.54	10.03	0.72	7.71
2	20.	5.3	0.011705	0.0450	3.64	8.45	0.65	3.89
3	30.	4.7	0.008537	0.0416	3.54	7.67	0.62	2.49
4	40.	4.2	0.006974	0.0389	3.36	7.16	0.62	1.81
5	50.	3.7	0.006084	0.0368	3.16	6.77	0.62	1.42
6	60.	3.4	0.005534	0.0353	2.97	6.46	0.62	1.17
7	70.	3.1	0.005177	0.0341	2.79	6.20	0.62	1.00
8	80.	2.9	0.004938	0.0332	2.63	5.98	0.62	0.89
9	90.	2.7	0.004775	0.0325	2.48	5.79	0.62	0.80
10	100.	2.5	0.004664	0.0320	2.35	5.61	0.62	0.73
11	110.	2.4	0.004588	0.0316	2.24	5.46	0.63	0.68
12	120.	2.2	0.004539	0.0312	2.13	5.32	0.63	0.64
13	130.	2.1	0.004509	0.0309	2.04	5.19	0.63	0.60
14	140.	2.0	0.004493	0.0307	1.96	5.08	0.63	0.57
15	150.	1.9	0.004488	0.0305	1.88	4.97	0.63	0.55
16	160.	1.9	0.004492	0.0304	1.81	4.87	0.63	0.52
17	170.	1.8	0.004503	0.0302	1.75	4.78	0.63	0.50
18	180.	1.7	0.004519	0.0301	1.69	4.70	0.63	0.49
19	190.	1.7	0.004539	0.0300	1.63	4.62	0.63	0.47
20	200.	1.6	0.004563	0.0300	1.58	4.55	0.63	0.46
	RESULTS AT MINIMUM STREAM POWER							
21	150.	1.9	0.004488	0.0305	1.88	4.97	0.63	0.54

Example 3: Hydraulic design with hydraulic geometry and the stable channel analytical method—Continued

Figure 9–20　Stability curve slope versus base width, Q = 1,500 ft^3/s, bed-material sediment concentration = 1,917 mg/L. Limerinos resistance and Meyer-Peter and Müller sediment transport equations
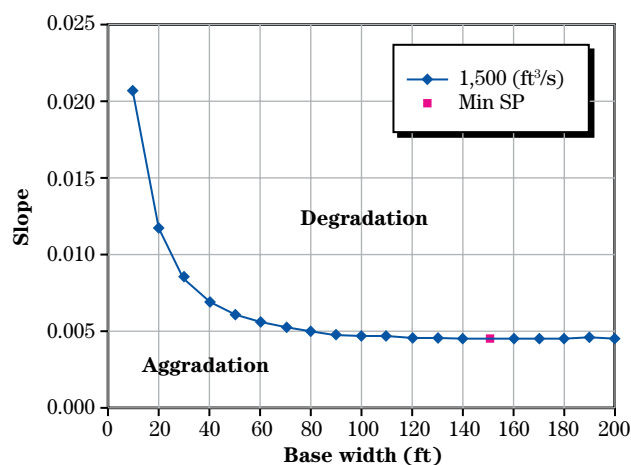

Figure 9–21　Stability curve depth versus base width, Q = 1,500 ft^3/s, bed-material sediment concentration = 1,917 mg/L. Limerinos resistance and Meyer-Peter and Müller sediment transport equations
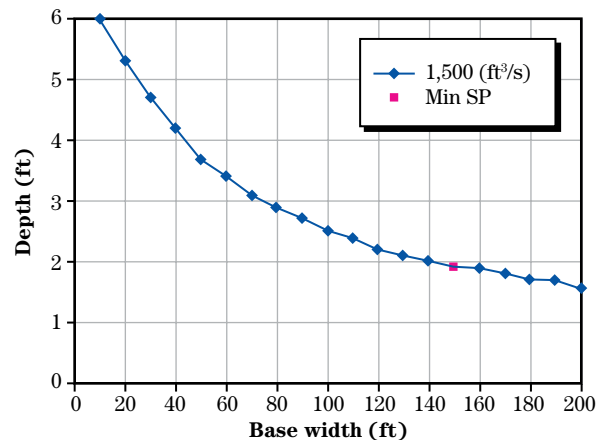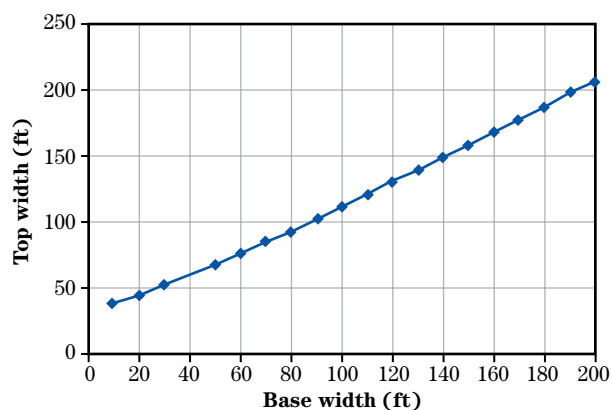

Figure 9–22　Top width verses base width for example problem which has 1V:2.5H side slopes

654.0911 Conclusion

Channels in which there is expected to be an exchange of the inflowing sediment load with the channel boundary should be designed using alluvial design methods. The design goal in an alluvial channel is to pass the inflowing sediment load without significant aggradation, degradation or planform change. Several techniques are available for the design of channels in an alluvial environment. They are the regime method, the analogy method, hydraulic geometry method, and the analytical method. All of these techniques have both advantages and disadvantages.

The analogy method is used to select design elements that are based on the premise that conditions in a reference reach with similar characteristics can be copied to the project reach. The hydraulic geometry method is similar to the analogy method insofar as that it is based on the premise that a river system tends to develop in a predictable way. The theory typically relates a dependent variable, such as width or slope, to an independent or driving variable, such as channel-forming discharge or drainage area. The regime method is similar to the hydraulic geometry, but is more appropriate for canal or drainage ditch type systems. The analytical method uses bed resistance and sediment transport equations to approximate a family of curves for width, slope, and depth for a range of potential stable configurations. These can be used indirectly with project constraints or in conjunction with the analogy or hydraulic geometry methods to estimate critical design elements.

All of the methods presented have advantages and disadvantages. Due to the high degree of uncertainty which is inherent to the nature of alluvial channels, many designers opt to use several methods. For example, during the assessment and design of proposed realignment, the family of curves calculated with the aforementioned analytical techniques can be used to provide another line of evidence which may give the designers more confidence in the chosen section, profile and planform.

All alluvial channel designs require analysis of channel stability. A stream is defined as stable when it has the ability to pass the incoming sediment load without significant degradation or aggradation and when its width, depth, and slope are fairly consistent over time. For design in an alluvial channel, it is suggested that an analytical sediment budget/assessment be conducted to compare the supply capabilities of the upstream reach to the sediment transport capacity of the design reach. Since bed-material sediment transport is significant under flows below, at, and above design flow in an alluvial channel, a sediment assessment should be done for a range of flows in any proposed realignment. Preparing sediment budgets is presented in NEH654.13.