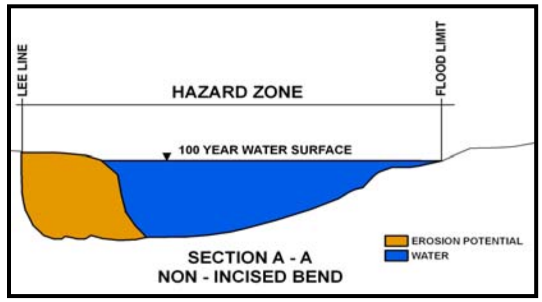
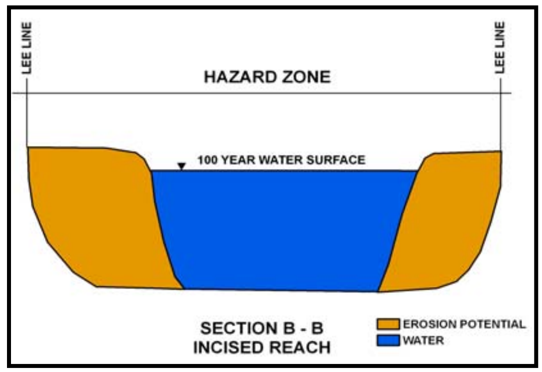
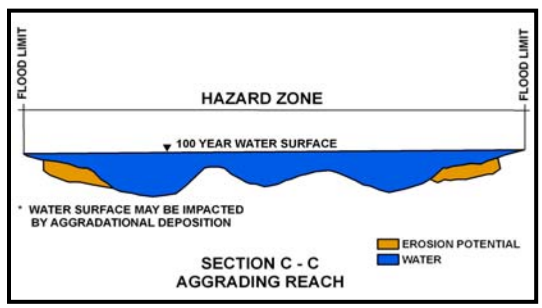
SEDIMENT AND EROSION DESIGN GUIDE







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Disclaimer

This document is provided as a guidance document in the SSCAFCA and City of Rio Rancho area. The intent of this document is to assist regulatory authorities, engineers, developers, and other interested parties in understanding the dynamic behavior of arroyos and other natural or naturalistic drainageways, and identifying appropriate means of protecting people and property from flooding, erosion and sedimentation during storm events up to and including the 100-year event. The user should report any errors in this document promptly to SSCAFCA and the City of Rio Rancho. The user accepts full responsibility for application of the guidance information contained herein to their particular situations.

TABLE OF CONTENTS

Page

LIST OF FIGURES

LIST OF TABLES

CONSTANTS AND CONVERSIONS

mm	= ft * 304.8 = in * 25.38
Unit Weight of Water (γ_w)	= 62.4 lbs/ft^3
Density of Water (ρ_w)	= 1.94 slugs/ft^3
Specific Gravity of Sediment	= 2.65 (Quartz)
Sediment Porosity (in-situ) (η)	= 0.4 (typical for in-situ soils in the SSCAFCA area)
Solid Unit Weight of Sediment (γ_s)	= $S_G\gamma_s$ = 62.4S_G = 165.36 lb/ft^3 (Quartz) = 2.65 γ_w
Bulked Unit Weight of Sediment (dry)	= (1 η) γ_s = 100 lb/ft^3 (η = 0.4)
Sediment Yield: 1 ac-ft/mi^2 (bulked)	= 3.4 tons/ac (η = 0.395)
Sediment Transport:	1 cfs = 43.2γ_s tons/day

Concentration by volume (C_v) = $$\left(\frac{V_s}{V_w + V_s}\right) * 10^6$$

Concentration by weight (C_w) = $$\left(\frac{W_s}{W_w + W_s}\right) * 10^6 = \left(\frac{W_g V_s}{V_w + S_g V_s}\right) 10^6$$

$$C_v = \frac{C_w}{S_G - C_W (S_G - 1)} \quad or \quad C_w = \frac{S_G C_v}{1 + C_v (S_G - 1)}$$

Bulking Factor (BF) = $$\frac{V_w + V_s}{V_w} = 1/(1 - C_v)$$

Fluid density (ρ_m) = $\rho_o [1 + (S_G - 1) C_v]$

LIST OF VARIABLES

Variable	Definition
A	cross-sectional area of the channel
a	pier width (ft)
a	abutment and embankment or wall length projecting into main channel
a and b	regression coefficients using the estimated transport capacities
A^*	the dimensionless scour geometry,(h_s/D, W_s/D, L_s/D, or V_s/D^3) and (h_s/y_e, W_s/y_e, L_s/y_e, or V_s/y_e^3)
a', b, c	coefficient and exponents of the power function bed-material transport capacity relationship $q_s = a'V^bY^c$ (see Appendix C)
B_f	bulking factor
C	Chezy's discharge coefficient (Chezy's C)
C_c	coefficient
c	cohesion
C_s	total sediment concentration by weight
D	diameter
D_{16}, D_{50}, and D_{84}	particle sizes for which 16, 50, and 84 percent of the material is smaller
D_{50}	median sediment size
d_m	tailwater depth (ft)
Δ_{max}	maximum lateral erosion distance
d_p	height of the embankment crest above the original bed
d_s	local scour depth for a free overfall, measured from the streambed downstream of the drop (ft)
D_s	characteristic sediment size
ΔV_s	volume of sediment stored (+) or lost (-) in the reach
Δw	horizontal toe erosion
D_x	Grain size for which x percent of the bed material is finer (ft)
ΔZ	difference in water-surface elevation between the channel centerline and the bank on the outside of the bend (ft)
Δz	average change in bed elevation in the reach
Δz_{max}	maximum allowable degradation
f	Darcy-Weisbach friction factor
F_D	width-depth ratio at dominant discharge (Q_D)
ϕ	angle of the channel alignment with the down-valley direction at location s
ϕ	submerged angle of repose on friction age of the sediment (assumed to 25.5º)
F_g	grain Froude Number
F_r	Froude Number
F_{r1}	Froude Number = $V_1/\sqrt{gy_1}$ based on conditions just upstream of the pier
γ	unit weight of water (62.4 lb/ft^3)

Variable	Definition
g	acceleration of gravity (32.2 ft/s^2)
γ_s	in-situ unit weight of soil
γ_s	unbulked unit weight of the sediment
η	porosity of the bed material (~0.4)
h_a	antidune height
H_c	critical or maximum stable bank height
h_l	head loss
H_t	total drop in head, measured from the upstream to the downstream energy grade line (ft)
K_v	coefficient in Veronese Equation (Equation 3.51) = 1.32
K_1	correction for pier nose shape
K_2	correction for angle of attack of flow
K_2	correction factor
K_3	correction for bed condition (i.e., bedform)
K_4	correction for armoring by bed-material size
K_u	coefficient in critical velocity equation for pier scour (Equation 3.56 (11.17 English Customary units)
L	channel length
λ	meander wavelength
L	length of pier (ft) (Equation 3.51)
L	effective length of spur, or the distance between arcs describing the toe of spurs and the desired bankline (ft) (Equation 5.4)
λ_s	angle of the emergency spillway with respect to horizontal
L_s	maximum spacing of the structures
L_v	downvalley length of arroyo
m	correction factor for sinuosity of the channel
M	total channel length along the meander
n	Manning's roughness coefficient
n_1	Manning's *n* -value for surface irregularities in the cross section
n_2	Manning's *n* -value for variations in shape and size of the channel
n_3	Manning's *n* -value for obstructions
n_4	Manning's *n* -value for vegetation and flow conditions
n_b	base Manning's *n* -value for a straight, uniform channel
n_b	base Manning's *n* -value for a bare soil surface
n_c	composite *n* -value
N_g	geotechnical stability number of bank height
N_h	hydraulic stability number
P	total wetted perimeter
P_F	probability of occurrence of that flood in one year
P_i	wetted perimeter in section i
Q	water discharge (cfs)
q	water discharge per unit width of channel (cfs/ft)
θ	angle between the flow direction and the floodwall (Equation 3.63)

Variable	Definition
θ	expansion angle downstream of spur tips, degrees (Equation 5.4)
Q_D	dominant discharge (cfs)
Q_s	sediment discharge (Equation 3.1)
$q_{s(supply)}$	bed-material supply per unit width of channel
Q_s	sediment-transport capacity (cfs)
$Q_{s\ total}$	total sediment load (cfs)
R	hydraulic radius (ft)
ρ	unit density of water 61.84 slug/ft^3
R_c	radius of curvature at the centerline of the stream (ft)
R_{cmin}	minimum radius of curvature
S_e	energy slope
S	bed slope
S	spacing between spurs at the toe (ft) (Equation 5.4)
S_c	critical slope
S_{eq}	equilibrium slope
S_{ex}	existing channel slope
S_f	friction slope
S_g	specific gravity of the sediment (2.65 for quartz sand)
S_o	existing bed slope
S_s	maximum stable slope
T	attack of the flow
t	duration of the flow
t_0	base time used in the experiments for Equation 3.64
U_0	velocity of the jet
V	average channel velocity (fps)
V_1	approach velocity just upstream of the pier (ft/s)
V_{cDx}	critical velocity for incipient motion of grain size D_x (ft/s)
V_{icDx}	approach velocity to initiate scour of grain size D_x (ft/s)
$V_{s(inflow)}$	volumetric sediment-transport rate into the reach
$V_{s(outflow)}$	volumetric sediment-transport rate out of the reach
W	channel topwidth (ft)
W	average width of the channel bed
W_B	width of the channel
ω	maximum angle of the meander relative to the general direction of the channel
W_D	channel width associated with the dominant discharge
X	cross-sectional area, wetted perimeter, or topwidth
Y_m	depth of flow at the thalweg (maximum depth in the cross section)
Y	hydraulic depth
Y_0	thickness of the jet as it impinges into the downstream tailwater
Y_i	flow depth just upstream of the pier (ft)
Y_{si}	sediment yields associated with each storm event
Y_s	equilibrium depth of scour
Y_D	tailwater depth with respect to the original bed elevation
Z	bed elevation referenced to common datum (ft)

INDEX

1. INTRODUCTION

Development in southern Sandoval County, where arroyos are the primary conduit for passage of overland runoff and watershed-derived sediment, poses complex problems for planning and regulatory agencies and developers. Due to the dynamic nature of arroyos, it is imperative that appropriate steps be taken to protect adjacent structures and facilities from damage due to flooding and erosion. Lining of the arroyos with nonerosive or erosion-resistant material is a common protection method; however, the cost associated with this "hard lining," in terms of construction and maintenance, as well as degradation of the natural environment, may be unacceptable. The Southern Sandoval County Flood Control Authority (SSCAFCA) desires to maintain arroyos within their area in a condition that is as safe as possible and in a natural or naturalistic condition, protecting the local environment and meeting policy goals of other governmental agencies. By use of setbacks and selective stabilization, natural and naturalistic arroyos and watercourses can provide protection to adjacent property similar to that provided by lined channels.

1.1. Purpose and Scope of Manual

The purpose of this manual is to provide guidance for the analysis of sediment areas and arroyos in the SSCAFCA jurisdictional area, including methods for evaluating the potential effect of proposed structures or activities on the vertical and lateral stability of the arroyos and drainageways, and for establishing the Lateral Erosion Envelope (LEE) line. The LEE line is a boundary along an arroyo or drainageway that would have a low possibility of being disturbed by erosion, scour, or lateral migration of a natural (unlined) arroyo by storms up to and including the 100-year storm.

This Design Guide describes basic concepts related to the physical processes that control arroyo behavior, provides engineering tools that can be used to quantitatively analyze specific processes, and provides guidance on combining these tools to predict short- and long-term arroyo behavior. The manual also provides guidance on selecting countermeasures to mitigate potential erosion or flooding problems in key areas, **Chapter 2** describes the basic geomorphic and watershed processes that are important in the SSCAFCA area. **Chapter 3** provides the individual tools with which to analyze specific aspects of arroyo dynamics, including the following:

- the Incised Channel Evolution Model that explains the progression through which most arroyos evolve in an urbanizing environment,
- methods for assessing hydraulic and sediment transport conditions in the arroyos,
- methods for using that information to assess vertical and lateral adjustability,
- a method for identifying the location of the LEE line, and
- methods for quantifying local scour at various types of structures that are common in the SSCAFCA area,

Chapter 4 describes the general solution procedures that should be used to combine the specific tools that are provided in Chapter 3 to perform an overall assessment of arroyo stability, progressing from qualitative evaluation of pertinent information through quantitative analysis. Chapter 4 also provides guidance in interpreting the results of the quantitative analysis to identify the nature of the hazard (i.e., flooding erosion or sedimentation) to developed properties along the arroyo, and methods of identifying appropriate methods to mitigate those hazards. Finally,

Chapter 5 provides general guidance on selecting and designing erosion barriers that may be incorporated with the LEE line to accomplish the dual goals of maintaining natural or naturalistic conditions while limiting the width of the erosion limit corridor. A set of example problems are provided in **Appendix E** to illustrate the use of the various relationships.

1.2. Arroyo Process—A General Statement of the Problem

Arroyos (or gullies) are ephemeral flow stream channels characterized by steeply sloping or vertical banks of fine sedimentary material and flat, generally sandy beds (Fairbridge, 1968). The American Geological Institute Glossary (1972) defines an arroyo as "a term applied in the arid and semiarid regions of southwestern U.S. to the deep, flat-floored channel or gully of an ephemeral stream or of an intermittent stream usually with vertical or steeply cut banks of unconsolidated material at least 60 centimeters *(2 feet)* high, that is usually dry, but may be transformed into a temporary water course or short lived torrent after heavy rains." This definition contains most of the elements of the arroyo problem in the southwest;

1. Arroyos occur at many different scales, ranging in size from small, gully-like features to major channels such as the Rio Puerco (Schumm et al., 1984).

2. Arroyos are incised channels, and as such, their morphologic, erosional (vertical and lateral), depositional and sediment-transport characteristics are temporally and spatially variable, and will change (evolve) systematically through time (Schumm et al., 1984).

3. Arroyo processes are episodic; therefore, the evolution of an arroyo system to an equilibrium form will take longer than channels in more humid regions because arroyos depend on the stochastic distribution of runoff-producing events (Schumm and Gellis, 1989; Gellis et al., 1991).

4. Although all natural channels tend toward an equilibrium form, arroyos typically do not reach equilibrium because of the discontinuous, ephemeral nature of the flows (Thornes, 1976; Wolman and Miller, 1960). Unlike perennial flow systems in which the channel form is the product of the continuous interaction between relatively frequent flows and the channel boundary sediment, channel form and process in arroyos are driven primarily by infrequent flood flows (Baker, 1977).

The causes for arroyo development were reviewed by Cooke and Reeves (1976) who concluded that, for many arroyos, the initial cause of erosion was the development of roads and trails or other activities that confined the flows and permitted incision to occur. However, cycles of arroyo incision and backfilling have occurred in the past, perhaps as a result of climate change (Love, 1979) or the exceedence of geomorphic thresholds (Schumm and Hadley, 1957; Schumm, 1973; 1977).

Although they do not typically reach an equilibrium form, most arroyos will eventually evolve to a relatively stable condition if the watershed conditions that control runoff and sediment delivery remain constant. In an urbanizing watershed, lateral and vertical erosion can occur very rapidly in response to either a single large storm event or a series of smaller storms during the development of this relatively stable condition, endangering adjacent property and delivering large quantities of sediment to downstream reaches. Even after the arroyo reaches the relatively stable condition, bank erosion and lateral migration will continue in local areas. The evolutionary process by which this occurs is described by the Incised Channel Evolution Model (ICEM) proposed by Schumm et al. (1984) and others (**Figure 1.1**). According to this model, when an otherwise unincised channel (Figure 1.1, Class I) begins to downcut due to any of several factors, including channelization, an increase in water discharge, reduction in sediment supply or other

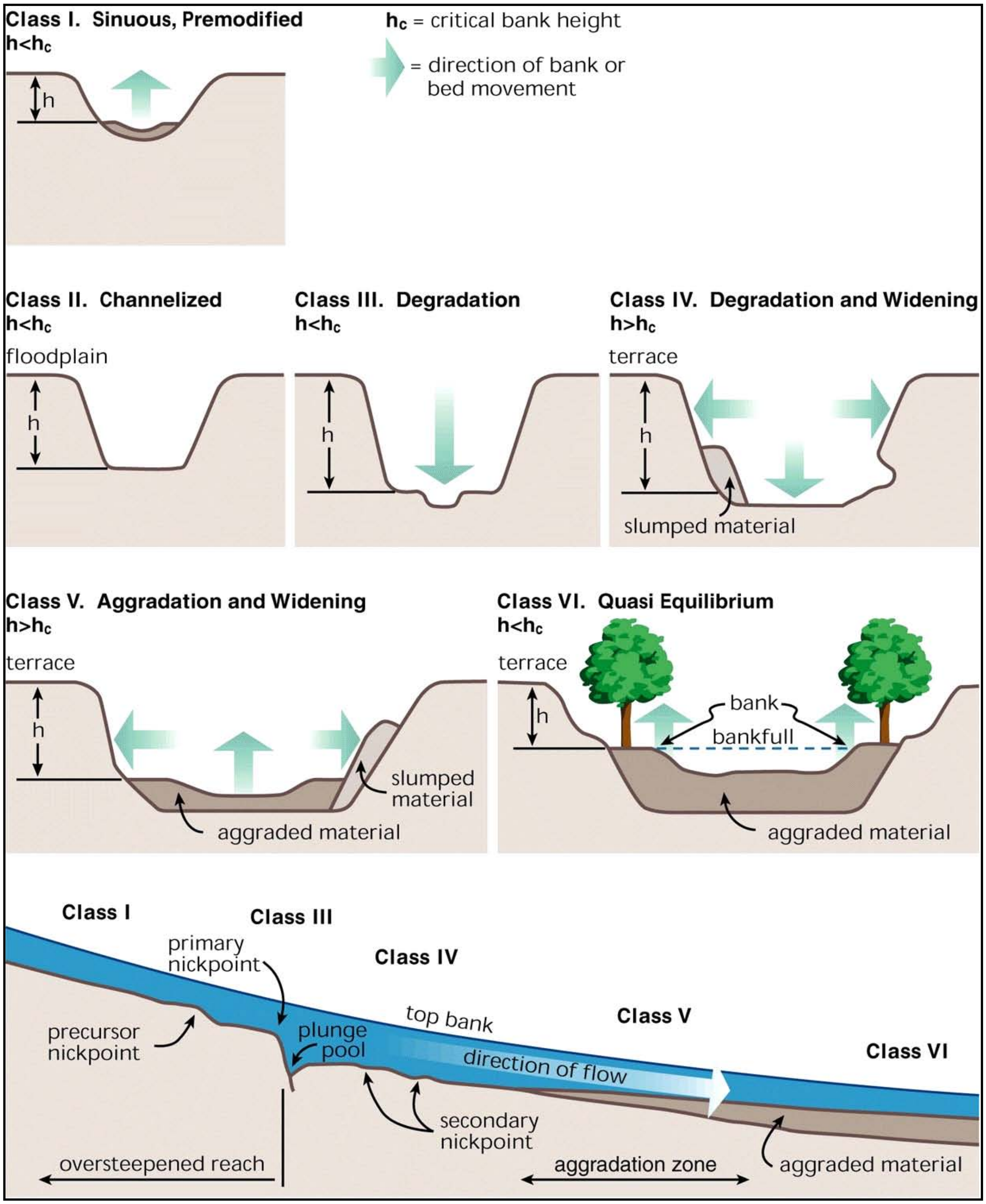


Figure 1.1. Incised Channel Evolution Model (from FISWRG, 1998; after Schumm et al., 1984). It is important to note that the Class VI quasi-equilibrium form may never develop in arroyos such as those in the SSCAFCA area.

disturbance, the channel undergoes a predictable evolutionary sequence. The channel initially downcuts (or degrades) (Figure 1.1, Class III). During this initial downcutting phase, the width of the active channel remains similar to the unincised width. When the downcutting progresses to the point where the bank height exceeds the threshold for geotechnical stability, the downcutting slows and eventually stops, and the incision widens through bank failure that introduces additional sediment into the channel (Figure 1.1, Class IV). In most cases, the introduced sediment from the bank failures eventually exceeds transport capacity of the arroyo, causing aggradation in the bottom of the incision (Figure 1.1, Class V). In systems that are subject to frequent flows, this eventually leads to the development of a quasi-equilibrium form with a narrow floodplain surface between the active channel banks and the edges of the incision (Class VI in Figure 1.1). As noted above, arroyos that are driven by infrequent, intense storms may never reach the equilibrium state illustrated by Class VI. In assessing this process, it is important to understand that, even in channels that are in dynamic equilibrium with the upstream water and sediment supply, erosion, deposition, and lateral migration will continue to occur. As a result, channels that exhibit long-term stability can, and do, migrate against the sides of the incision in local areas, which can lead to local instability and erosion.

The consequences of arroyo development and evolution can be severe. Incision can lead to the failure and loss of bridge crossings and damage to utility crossings (Shen et al., 1981). Incision-induced channel widening leads to land loss and damage to channel-margin structures. Channel erosion as a result of incision and widening of the primary arroyo, and erosion of tributaries as a result of baselevel lowering, both cause increased sediment delivery downstream that often leads to increased flooding frequency or loss of reservoir capacity (Schumm et al., 1984; Harvey and Watson, 1986). In cases where the ground water is relatively shallow, arroyo incision can lead to lowering of the water table, which in turn, can threaten the survival of floodplain vegetation that might otherwise increase the resistance of the channel to lateral erosion (Gellis et al., 1991). A beneficial consequence of arroyo incision and development is the reduction in frequency of overbank flooding (Wilson, 1973).

The arroyo evolution process has not been initiated in many drainageways in undeveloped or sparsely developed drainages within the SSCAFCA area. Because of the combined effects of land surface disturbance during construction, and increased runoff and reduced sediment delivery from the watershed resulting from urbanization, it should be assumed for design purposes that the development process will create conditions where the threshold for arroyo incision will be exceeded, initiating the evolutionary cycle illustrated in Figure 1.1. As this process occurs, significant deposition, loss of channel capacity and channel instability can be expected in local downstream areas. In either the incising or aggrading condition, the potential for lateral instability channel may increase. Definition of the LEE Line and design of protection measures must, therefore, consider the existing stage of the drainageway in the evolutionary cycle and the likely effect of the proposed development.

This Design Guide provides guidelines for evaluating the stage of existing arroyos within the evolutionary process and determining the impact of future development plans on the threshold conditions that may lead to arroyo development for those drainageways that are currently below the threshold. It also presents analysis techniques and guidelines for establishing the LEE Line. Criteria are also presented for design of erosion-control barriers (countermeasures) that can, under appropriate circumstances, reduce the amount of land area required within the LEE Line while maintaining the objective of a natural or naturalistic arroyo.

2. GEOMORPHIC AND WATERSHED PROCESSES IN TRIBUTARIES OF THE MIDDLE RIO GRANDE IN AND NEAR THE SSCAFCA JURISDICTIONAL AREA

2.1. Geomorphology of the Area

2.1.1. Landforms

A number of landforms occur in and near the SSCAFCA jurisdictional area that have a bearing on channel stability and sediment yield. The landforms include: alluvial fans, pediments, alluvial terraces, floodplains of the major channels, caliche-capped mesa, and volcanic plateau.

2.1.1.1. Alluvial Fans

Bull (1977) defined an alluvial fan as "*a deposit whose surface forms a segment of a cone that radiates downslope from the point where the stream leaves the source area*." Smaller fans with slopes in excess of 20 degrees have been referred to as alluvial cones, but they, in fact, fall within Bull's primary definition. In general, the fans are constructed by fluvial and mass-wasting processes (Lecce, 1990), and they occur in a very wide range of climatic zones, including arid/semiarid, humid-glacial, humid-temperate and humid-tropical (Kochel and Johnson, 1984). Alluvial fans are generally located along mountain fronts, and they may be coalesced, forming bajadas (Eckis, 1928). Deposition on the fan is primarily the result of rapid flow expansion rather than reduced slope (Reading, 1978). The thickness of fan deposits is variable and generally related to the type of baselevel control. Local control results in thin fans; whereas, tectonic control results in thick fans (Bull, 1977). Fans may be distinguished from other piedmont geomorphic features, such as pediments, by their thickness-length ratio (Doehring, 1970).

2.1.1.2. Pediments

Pediments are defined as broad, flat or gently sloping, rock-floored erosion surfaces that are typically formed by fluvial processes in an arid or semiarid region. They are typically located at the base of an abrupt and receding mountain front or plateau escarpment, and underlain by bedrock that is generally mantled with a thin veneer of alluvial sediments that are derived from upslope erosion. The longitudinal profile of a pediment is generally slightly concave upward. The development of multiple pediment surfaces is generally the result of baselevel lowering. Within the Middle Rio Grande basin, the best defined pediment surface is the Ortiz pediment of early Pleistocene age that has developed on the surface of the Santa Fe formation south of Tijeras Arroyo on the southeast side of Albuquerque. High level pediment surfaces have been identified east of Tramway Road north of Tijeras Arroyo.

2.1.1.3. Alluvial Terraces

Alluvial terraces are abandoned floodplains that were formed when the Rio Grande flowed at a higher elevation than at present. The surface of the terrace is no longer related to the modern hydrology of the river in that it is no longer inundated as frequently as the active floodplain. Topographically, the terrace consists of two parts: (1) a tread which is the flat surface that represents the level of the former floodplain, and (2) the scarp that is the steep slope that connects the tread to any surface standing at a lower elevation. The presence of a terrace always indicates that the river has downcut. The tread surface is generally underlain by alluvium of variable thickness. Well-developed terraces border the Rio Grande on both the east and west sides. The highest terrace is approximately 200 feet higher than the floodplain, an intermediate terrace is located about 100 feet above the floodplain, and a lower terrace is located about 50 feet above the floodplain.

2.1.1.4. Floodplains of the Major Channels

A floodplain is the surface of relatively smooth land adjacent to the river or arroyo that has been constructed by the river in its existing hydrologic and sediment regimen and covered by water when the channel overflows its banks. The floodplain is constructed of alluvium that is transported by the river. A river can only have one floodplain, but a number of former floodplain surfaces (terraces) can border the channel. Floodplains are located along the Rio Grande and along the floor of Tijeras Arroyo and other larger arroyos within the Middle Rio Grande basin. Floodplains are typically not present in the smaller arroyos that occur in the basin.

2.1.1.5. The Ceja Mesa and Volcanic Plateau

The western margin of the Middle Rio Grande Basin south of the Calabacillas arroyo watershed and north of Interstate 40 is composed of the Ceja Mesa. The mesa, a relatively flat-topped erosional landform, is underlain by sediments of the Santa Fe formation. The surface layers of the mesa are cemented with caliche that has formed a relatively erosion-resistant caprock.

2.1.2. Geomorphology of the SSCAFCA Area

The Middle Rio Grande Basin in the greater Rio Rancho and Albuquerque area can be informally divided into four quadrants on the basis of landscape components, channel characteristics and underlying geology. A convenient north-south dividing line is the Rio Grande, and an east-west dividing line is defined by I-40 to the west and Tijeras Arroyo to the east of the river (**Figure 2.1**). As shown in Figure 2.1, **The SSCAFCA jurisdictional area falls within the northern portion of the Northwest Quadrant.** The four quadrants are described in Mussetter, et al (1994), and a more detailed description of the portion of the Northwest Quadrant in the SSCAFCA area is provided below.

The northwest quadrant includes the drainage basins of Calabacillas, Montoyas, Baranca and Venada Arroyos, and is bounded to the south by I-40, to the west by the Albuquerque Volcanic field and Rio Puerco escarpment, and to the east by the Rio Grande. The quadrant is primarily underlain by the Santa Fe formation. However, southeast of the Calabacillas Arroyo, the Santa Fe formation is overlain by the late Pleistocene basaltic sheets that emanated from the seven volcanic centers and form what is referred to as the Volcanic Escarpment. The basalt acts as a caprock that overlies the erodible sediments of the Santa Fe formation. The arroyos within SSCAFCA's jurisdictional area drain portions of Santa Fe formation that are unaffected by the volcanic field. The drainage basins are underlain by poorly consolidated sedimentary rocks of the Santa Fe Group, and most of the area is covered by large expanses of aeolian sand and alluvium associated with the arroyo systems that drain generally southeasterly into the Rio Grande (Personius et al., 1999). In most cases, the bed of the arroyos is composed of loose sands, with a small amount of silt and fine gravel. Outcrops of the weakly-cemented Santa Fe formation rocks occur in isolated locations in the bed and banks of active arroyos. These outcrops are composed primarily of sandstone and mudstone, with minor amounts of silt/clay and gravel. The outcrops are erosion-resistant, but are erodible when subjected to high velocity flows. This erosion-resistant material provides a measure of temporary vertical and lateral stability. Alluvial materials on the surface in the overbank areas generally contain sufficient gravel-sized particles to produce a desert pavement that inhibits overland erosion during rainstorm events that are of sufficient intensity and duration to cause sheet flow.

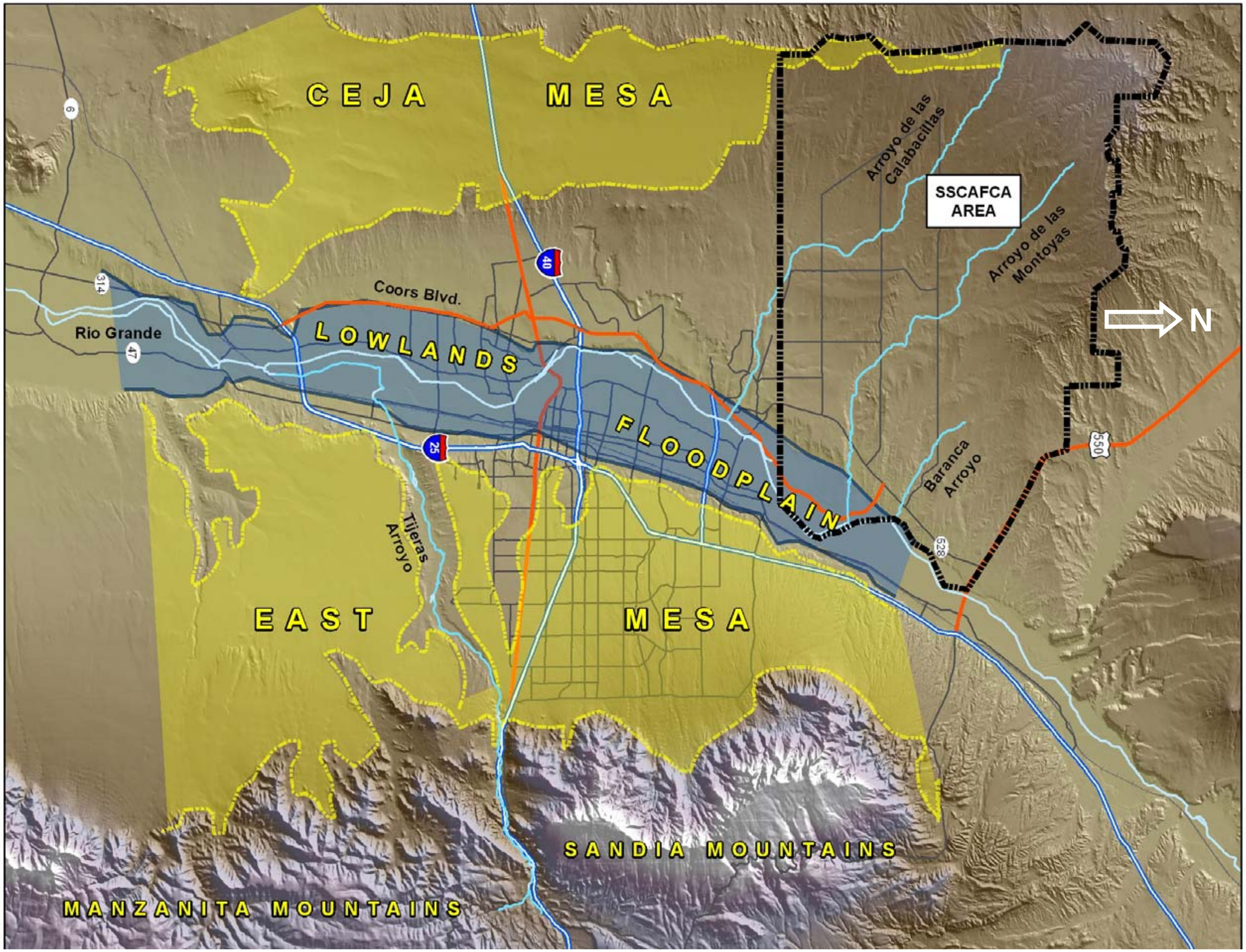


Figure 2.1. Physiographic map showing lowlands, mesas, and mountains in the greater Albuquerque/Rio Rancho area.

The larger drainage basins within SSCAFCA's jurisdictional area, including Montoyas Arroyo, typically contain four general zones consisting of the upland areas that represent the Rio Grande terraces which are underlain by Santa Fe Formation materials, the active arroyos that are underlain by modern alluvium, an alluvial fan that extends from the vicinity of NM Highway 528 to the Corrales Main Canal that is composed primarily of modern alluvium, and the historic Rio Grande floodplain between the Corrales Main Canal and the river. The limits of the present Rio Grande floodplain are defined by the Rio Grande levees which run along the east side of the Upper Corrales Riverside Drain, approximately 2,000 feet east of the Corrales Main Canal. The alluvial fan areas were historically subjected to flooding and significant sediment deposition during large rainstorms (**Figure 2.2**). Channelization on some of the arroyos, including the Harvey Jones channel on Montoyas Arroyo, control flooding and sedimentation in the downstream portion of the fan. Where the arroyos are affected by urbanization, they tend to incise and widen, consistent with the ICEM (Figure 1.1).

2.2. Rainfall-Runoff Processes

Arroyos in the SSCAFCA area are ephemeral, flowing only in response to specific storm events. Analysis of arroyo stability must, therefore, consider the runoff characteristics of the individual storms. Applications of the analysis procedures discussed in this Design Guide require estimates of the hydrographs associated with a range of return period storm events up to and including the design storm. Since the lateral and vertical response of the arroyo channels to these storm events is related to the volume of sediment that can be carried by the flows, estimates of the duration of the flows as well as their magnitude are required. Specific procedures for estimating storm hydrographs in the SSCAFCA area are described in DPM 22.2.

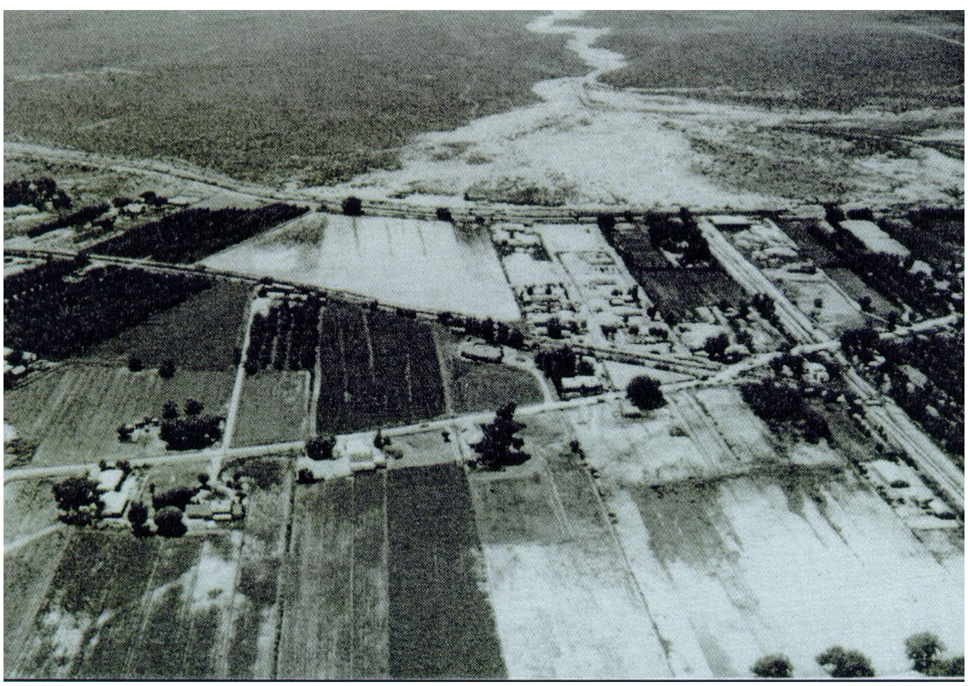


Figure 2.2. Oblique aerial photo of the mouth of Montoyas Arroyo looking westerly (upstream) showing the extent of flooding and sedimentation after the August 1975 storm.

2.3. Sediment Yield

The total sediment yield at a specific location along an arroyo consists of sediment delivered to the channel from overland areas and sediment eroded from the arroyo boundaries by the flowing water. The most accurate method of estimating the quantity of sediment eroded from the channel boundary involves bed-material transport capacity computations that are described in Chapter 3. The amount of sediment derived from overland areas can be estimated using empirical relationships involving various parameters that describe the characteristics and condition of the watershed, or from applicable sediment yield data from existing retention structures. **For purposes of analyzing arroyo stability, watershed sediment yield computations are typically performed to estimate the fine sediment (or wash load) component of the total sediment yield.** (See Section 3.3 for a discussion of the various modes of sediment transport.) Methods that have been used successfully in a large number of drainage basins along the middle and lower Rio Grande are described in Appendix A.

A general permit for storm water discharges must be obtained from the State of New Mexico for construction sites (http://www.nmenv.state.nm.us/swqb/PSRS/NMR150000-Info.html). The permit requires a Surface Water Protection Plan that includes *site-specific interim and permanent stabilization, managerial, and structural solids, erosion, and sediment control best management practices (BMPs) and/or other controls that are designed to prevent to the maximum extent practicable an increase in the sediment yield and flow velocity from pre-construction, pre-development conditions to assure that applicable standards in 20.6.4 NMAC, including the antidegradation policy, or WLAs are met. This requirement applies to discharges both during construction and after construction operations have been completed*. The selection of the BMPs must be based on an appropriate soil loss prediction model. One of the suggested models that is commonly used in the greater Albuquerque area is the Revised Universal Soil Loss (RUSLE) procedure developed by the Natural Resources Conservation Service Details of the procedure, including the RUSLE2 software procedures, can be found on the NRCS website (http://www.ars.usda.gov/Research/docs.htm?docid=5971). A spreadsheet that implements the RUSLE procedure is also available from the NMED.

3. CHANNEL DYNAMICS

This chapter presents background and recommended techniques for analyzing geomorphic, hydraulic and sediment-transport conditions in arroyos and drainageways within the SSCAFCA jurisdictional area. The specific techniques presented in this chapter form the basis for the integrated arroyo stability and LEE Line analysis procedures in Chapter 4.

3.1. Arroyo Geomorphology

The arroyo is one type of incised channel that ranges in size from rills that are a few inches deep to major entrenched streams that may be up to 50 feet in depth. Incised channels have characteristically lowered their beds, thereby setting in motion a period of considerable channel instability. This instability has the potential for serious erosion, dewatering of riparian zones, downstream delivery of sediment to reservoirs, destruction of aquatic habitat, damage to infrastructures such as bridges and utility crossings, and damage to urban development. The causes of incision are highly variable, but the response of incised channels, regardless of scale or location, follows a very similar pattern (Figure 1.1; Schumm et al., 1984). Geomorphic models of channel evolution following incision have been used to develop cost-effective engineering solutions that incorporate an understanding of the system dynamics (Schumm et al., 1984; Harvey and Watson, 1986; Watson et al., 1988a; 1988b).

It should be understood that, while arroyos and drainage channels typically incise in response to urbanization, localized areas of deposition will also occur, particularly in areas of reduced energy such as channel expansions, bends and reaches with slopes that are significantly flatter than the upstream channel. Deposition may also occur in reaches where land disturbance results in the delivery of large quantities of sediment to the arroyo in excess of its transporting capacity.

3.1.1. Channel Incision

The characteristic morphology of incised channels is the result of the force exerted by concentrated flowing water that exceeds the resistance of the material in which it is flowing. The development of an incised channel at a given location may depend on controls acting at that site, or the up- or downstream changes that affect the site. In addition, the response can be the result of extrinsic (external) controls which are imposed on the system, such as climatic fluctuations (Knox, 1972), change of base level (Schumm and Parker, 1973), change of land use (Graf, 1979), or channel modification (Schumm et al., 1984). Incision may also be the result of intrinsic (internal) controls that are inherent in the system, such as the steepening of valley floors by deposition of sediment which, if it is of sufficient magnitude, can initiate channel incision (Schumm and Hadley, 1957; Patton and Schumm, 1975; Begin and Schumm, 1984). Regardless of whether the incision is caused by external or internal controls, the presence of the incision indicates that a threshold of stability has been exceeded (Schumm, 1977).

Once the threshold of stability is exceeded, bed degradation and channel incision will follow. Degradation is the morphologic expression of a sediment-transport imbalance, since sediment-transport capacity exceeds the supply (Pickup, 1977). The bed and banks of the channel then become sources of sediment. This degradation sets in motion a complex series of interacting events through which the system ultimately adjusts to a new state of dynamic equilibrium. Attainment of a new state of dynamic equilibrium involves interdependent adjustments between channel slope and cross-sectional area in response to the imposed discharge and sediment load (Leopold et al., 1964). The time required to attain the new state of dynamic equilibrium in incised channels varies with climatic region. In the humid Southeastern U.S., dynamic equilibrium is typically restored within a period of about 30 years (Schumm et al., 1984); whereas, in the arid and semiarid Southwestern U.S., the time required can be 80 to 100 years or longer (Gellis et al., 1991).

The concept of dynamic equilibrium is expressed very simply by the Lane (1974) relation:

$$QS_e \sim Q_s\, D_{50} \tag{3.1}$$

where Q = water discharge
S_e = energy slope
Q_s = sediment discharge
D_{50} = median sediment size

This relation means that alluvial channels tend toward a state of equilibrium in which the discharge and slope are in balance with the sediment-transport capacity and bed-material size. If, for example, the sediment supply to a reach that was previously in equilibrium is reduced with no change in discharge or particle size, the channel will flatten (or degrade) to achieve a new state of equilibrium between the sediment-transport capacity and supply. Conversely, if the sediment supply is increased, all other factors being the same, the channel will attempt to steepen (or aggrade) to achieve a new state of equilibrium. Equation 3.1 is a useful relation to **qualitatively** evaluate the response of an alluvial channel to changed conditions associated with natural or man-induced changes in the watershed or upstream reaches (see Richardson et al., 1990 for applications).

3.1.2. Channel Widening

Once a critical bank height has been exceeded, bed degradation usually precedes and predisposes channel widening through mass failure of the banks (Little et al., 1982; Harvey and Watson, 1986; Watson et al., 1988a). In most incised channels, fluvial detachment of individual particles does not play a major role in bank retreat. However, fluvial removal of previously failed bank material can trigger continued bank erosion (Thorne, 1982). The critical bank height depends on the geotechnical properties of the bank materials. Identification of the critical bank height can be done with a formal geotechnical stability analysis (Osman and Thorne, 1988, Section 3.4.4), or it can be estimated based on observations of bank heights and angles at sites where bank failure has taken place in similar materials (Biedenharn et al., 1991).

Channel widening as a result of bed degradation will continue until such time as the failed bank materials are no longer removed by fluvial action. Localized erosion at bendways will continue to occur, but system-wide bank failure will eventually cease. The ultimate channel width can be related to the degree of incision and the redevelopment of an equilibrium width-depth ratio that depends on the size of the drainage basin (Harvey and Watson, 1986) which is a surrogate for discharge (Leopold et al., 1964).

Channel widening may also occur in depositional zones due to the increase in stress on the channel banks as the material deposits in the bed. Under these conditions, avulsion and abrupt realignment of the channel is possible.

3.1.3. Arroyo Evolution

Numerous geomorphological studies have used data from different locations to assess and predict landform development through time. This technique is referred to as location-for-time substitution, in which is assumed that the processes that occur along the arroyo at any point in time are the same as those that would occur at a particular location over a period of time. This technique permits results from short-term studies to be used to predict conditions in the arroyo at particular locations over longer periods of time. The ICEM that was described in Chapter 1 (Figure 1.1) was originally developed using this technique for the channelized streams in the Southeastern U.S. (Schumm et al., 1984), and it has subsequently been adapted to the arroyos of the Southwestern U.S. (Schumm and Gellis, 1989; Gellis et al., 1991)

The ICEM identifies and integrates four important facets of the evolution process: (1) bank stability, (2) magnitude and frequency of the range of dominant discharges, (3) hydraulic energy of those discharges, and (4) the morphological adjustments of the channel. These factors can be further reduced to two dimensionless stability numbers, N_g, the geotechnical stability number, and N_h, the hydraulic stability number (Watson et al., 1988a; 1988b).

The geotechnical stability number N_g is defined as the ratio of the actual bank height (H) at a given bank angle to the critical bank height (H_c) (i.e., the maximum bank height that is geotechnically stable):

$$N_g = \frac{H}{H_c} \tag{3.2}$$

When N_g is less than 1, the bank is geotechnically stable; when N_g is greater than 1, the bank is unstable, and bank failure and channel widening are likely. In simple terms, this relationship means that, when the bank height is less than H_c, the bank should remain stable, and when the bank height is greater than H_c, instability can be expected.

The hydraulic stability factor (N_h) is defined as the ratio of the desired sediment supply to the sediment-transport capacity. N_h can be interpreted as a ratio of energy parameters. An example would be the ratio of shear stress or shear intensity at the effective or dominant discharge to the same parameter at conditions of equilibrium between sediment-transport capacity and sediment supply. It is important to note that N_h includes both sediment transport and supply. This is in contrast to most channel design procedures, which are generally based on fixed boundary approximations. N_h provides a rational basis for determining the equilibrium sediment transport/sediment supply relationship that will be required to achieve a state of dynamic equilibrium. Hydraulic stability in the channel is attained when N_h=1. If N_h>1, the channel will degrade, and if N_h<1, the channel will aggrade.

When N_g and N_h are combined, they provide a set of design criteria that define both geotechnical and hydraulic stability in the channel. Overall channel stability is attained when N_g<1 and N_h = 1. Since sediment supply to a channel fluctuates through time, it is prudent to design for a hydraulic condition that is marginally aggradational; therefore, a more conservative approach is to allow for N_h<1.

The relationship between the ICEM and the stability numbers is illustrated in **Figure 3.1**. The points labeled I through VI, which correspond to the classes shown in Figure 1.1 can be viewed as individual locations along an incising arroyo, or as a sequence of conditions through time at a given location. If the geotechnical and hydraulic calculations place a reach of channel at point A on the diagram, the strategy should be to prevent the channel depth from increasing to the point where the critical bank height is exceeded. In contrast, if the reach is located at point E, there will be no need to treat the channel because it is in a condition of quasiequilibrium. If no action is taken when a reach is in a condition represented by point A, the sequence of channel incision and widening will move from point A to point F through time as the channel evolves.

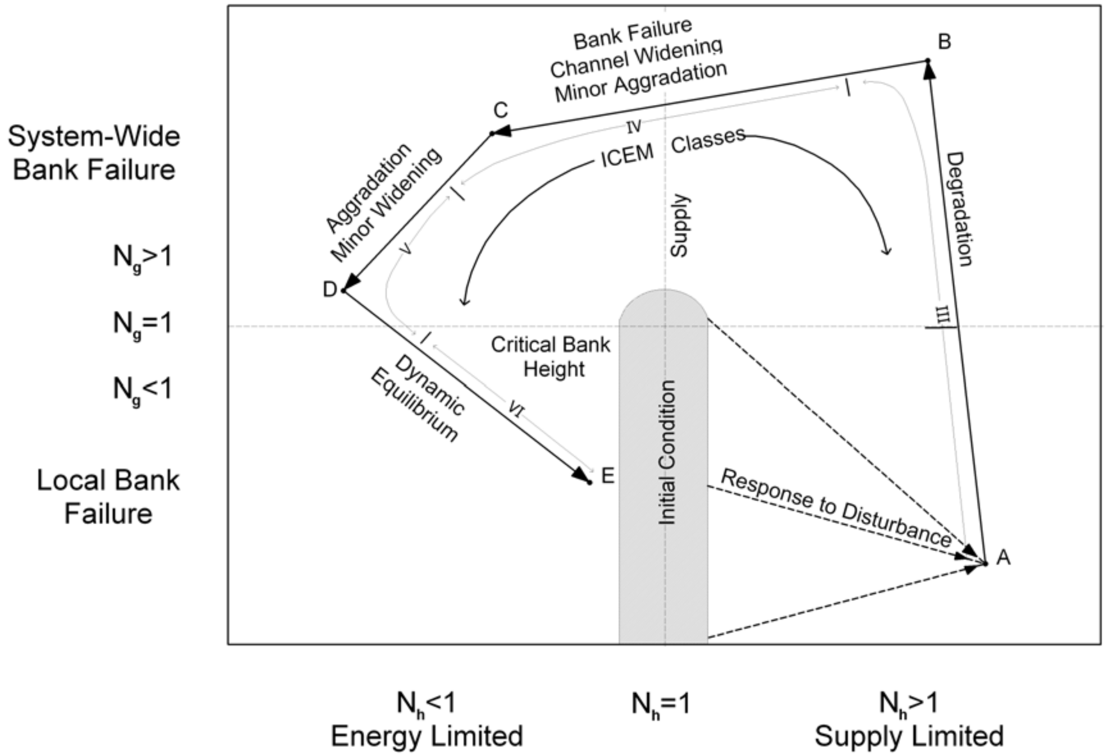


Figure 3.1. Stability number (N_g/N_h) diagram showing the thresholds of bank stability and hydraulic stability for an incised channel, and the corresponding ICEM stages. Note that the ICEM reach types form a continuum and the Type boundaries are gradational.

As the channel evolves from a state of disequilibrium (A) to a state of dynamic equilibrium (E), the reach types move from the lower right to the lower left quadrant via the upper right and upper left quadrants. Management of the channel should strive to keep the channel in the lower right quadrant, or force it to move directly to the lower left quadrant, thereby eliminating the evolution cycle that is an inevitable consequence of bed degradation exceeding the critical bank height. Forcing the channel to move directly into the lower left quadrant generally requires the use of grade-control structures and bank protection. As noted in Chapter 1, the effects of urban development will, for practical purposes, preclude maintenance of a given arroyo in the lower right quadrant under most conditions.

Use of ICEM and the dimensionless stability numbers N_g and N_h enables equilibrium reaches to be identified (i.e., $N_g<1$: $N_h<1$), facilitates identification of reaches that are at risk, and provides a process-based rationale for selecting appropriate treatments. Further, this approach also facilitates evaluation of the potential effects of changed land use (runoff and sediment supply) in the context of channel stability.

3.2. Hydraulic Factors and Principles

3.2.1. Introduction

Knowledge of the hydraulic conditions that will occur during a given storm event is an essential element of a channel stability and LEE Line analysis. The maximum depth of flow during the peak of the 100-year flood determines the location of the regulatory flood boundaries. The erosive power of the flow and thus, the potential for channel erosion or deposition is directly related to the flow velocities and depths for the range of flows in the hydrograph.

Typically, flood analyses consider the channel to have a rigid boundary with no deformation of the bed or banks during the passage of a flood. In erodible channels typical of many arroyos and drainageways in the SSCAFCA jurisdictional area, interaction between the flow and boundary can have a significant influence on both the hydraulic characteristics of the flow and the form of the channel. This interaction is an important consideration when evaluating the potential response of the channel to storm flows.

In most applications, the hydraulic analyses are performed using standard computer programs which employ one-dimensional, step-backwater calculations (e.g., the Corps of Engineers HEC-RAS). These programs contain routines to estimate energy losses through expansions and contractions, bridge openings, and culverts as well as a variety of other options for evaluating energy losses along the channel.

Depending on the required level of accuracy for a specific study, in cases where the channel is uniform in slope and cross section, it may be acceptable to estimate the hydraulic conditions directly by assuming uniform flow and applying an empirical velocity equation such as the Manning or Chezy equation.

This Design Guide assumes that the user has a working knowledge of open-channel hydraulics. Numerous references are available to those needing to review open- channel flow concepts (e.g., Chow, 1959; Henderson, 1966). The purpose of this chapter is to discuss the techniques appropriate for analysis of hydraulic conditions in the steep, highly erodible channels characteristic of the SSCAFCA jurisdictional area.

3.2.2. Uniform Flow Relationships

For uniform and gradually varied flow, channel velocity and depth are normally estimated using either the Manning or Chezy equations. These equations are empirical in nature and are given by:

Manning:

$$V = \frac{1.486}{n} R^{2/3} S_f^{1/2} \tag{3.3}$$

where V = average channel velocity, in feet/second
R = hydraulic radius, in feet
S_f = friction slope
n = Manning's roughness coefficient

and Chezy:

$$V = C R^{1/2} S_f^{1/2} \tag{3.4}$$

where C = Chezy's discharge coefficient (Chezy's C).

Equating the two relationships, the Manning's *n* and Chezy's C coefficients are related by the following equation:

$$C = \frac{1.486}{n} R^{1/6} \tag{3.5}$$

Another common method of representing the resistance to flow caused by grain roughness on the channel boundary in both open channels and closed conduits is the Darcy-Weisbach friction factor (f). The Darcy-Weisbach formula is given by:

$$h_l = \frac{f\,L\,V^2}{D\,2g} \tag{3.6}$$

where h_l = head loss
L = channel length
D = diameter of the conduit

By noting that h_l/L is equivalent to the energy slope (S_e), the hydraulic radius (R) is related to D by:

$$R = \frac{A}{P} = \frac{\pi D^2 / 4}{\pi D} = \frac{D}{4} \tag{3.7}$$

The bed shear stress (τ_0) is given by:

$$\tau_o = \gamma R S_e \tag{3.8}$$

It can be shown that the bed shear stress, in terms of the Darcy-Weisbach friction factor, is:

$$\tau_o = \frac{f \rho V^2}{8} \tag{3.9}$$

In the above relations, γ and ρ are the unit weight and density of water (62.4 lb/ft^3 and 1.94 slugs/ft^3, respectively). By manipulating the above equations, it can be shown that Manning's roughness coefficient (*n*), Chezy's discharge coefficient (C) and the Darcy-Weisbach friction factor (f) are related by:

$$C = \sqrt{\frac{8g}{f}} = \frac{1.486}{n} R^{1/6} \tag{3.10}$$

The accuracy of the result from any of the above relationships depends on the resistance coefficient selected for use in the computation. The following sections discuss factors to be considered in this regard.

3.2.3. Hydraulics in Steep, Alluvial Channels

3.2.3.1. Bedforms and Flow Regime

Natural arroyos in the SSCAFCA jurisdictional area are typically steep and highly erodible. Interaction between the flowing water-sediment mixture and the bed during runoff events creates different bed configurations that change the resistance to flow, velocity, water-surface elevation, and sediment- transport rates. Consequently, an understanding of the different types of bedforms that may occur under differing flow conditions and knowledge of the resistance to flow and sediment transport associated with each is important in selecting appropriate boundary roughness values.

Flow in alluvial channels is divided into two regimes separated by a transition zone (Richardson et al., 1990). Forms of bed roughness in sand channels are shown in **Figure 3.2a**. **Figure 3.2b** shows the relationships between water surface and bed configuration. The flow regimes are:

> Lower regime, where resistance to flow is large and sediment transport is small. The bedform is either ripples or dunes or some combination of the two. Water-surface undulations are out of phase with the bed surface, and there is a relatively large separation zone downstream from the crest of each ripple or dune. The velocity of the downstream movement of the ripples or dunes depends on their height and the velocity of the grains moving up their backs.
>
> The transition zone, where the bed configuration may range from that typical of the lower flow regime to that typical of the upper flow regime, depending mainly on antecedent conditions. If the antecedent bed configuration is dunes, the depth or slope can be increased to values more consistent with those of the upper flow regime without changing the bedform; or, conversely, if the antecedent bed is plane, depth and slope can be decreased to values more consistent with those of the lower flow regime without changing the bedform. Resistance to flow and sediment transport also have the same variability as the bed configuration in the transition. This phenomenon can be explained by the changes in resistance to flow and, consequently, the changes in depth and slope as the bedform changes.
>
> Upper regime, where resistance to flow is small and sediment transport is large. The usual bedforms are plane bed or antidunes. The water surface is in phase with the bed surface except when an antidune breaks, and normally the fluid does not separate from the boundary.

Due to the steepness and erodibility of most arroyos, it can generally be assumed that upper regime flow will occur during significant storm events.

3.2.3.2. Resistance to Flow—General

The general approach for estimating resistance to flow in a stream channel is to select a base value for materials in the channel boundaries assuming a straight, uniform channel, and then to make corrections to the base value to account for channel irregularities, sinuosity, and other factors which affect the resistance to flow (Richardson et al., 1990; Arcement and Schneider, 1984). The following equation is used to compute the equivalent total Manning's roughness coefficient (n) for a channel using this approach:

$$n = (n_b + n_1 + n_2 + n_3 + n_4)m \tag{3.11}$$

where n_b = the base value for a straight, uniform channel
n_1 = value for surface irregularities in the cross section
n_2 = value for variations in shape and size of the channel
n_3 = value for obstructions
n_4 = value for vegetation and flow conditions
m = correction factor for sinuosity of the channel

Arroyos and overbanks in the SSCAFCA jurisdictional area frequently contain varying amounts of chamisa. For relatively small flows and in locations where it is not expected to be taken out by the flows, the chamisa can be treated as a minor obstruction(s) and n_3 used to adjust the base *n*-value to obtain total roughness. It should be noted that under high flow conditions, the chamisa may be bent over in the flow, reducing the effective roughness. Other vegetation has similar properties. The reader is referred to Simons, Li & Associates, Inc. (SLA, 1982) for further discussion of this process.

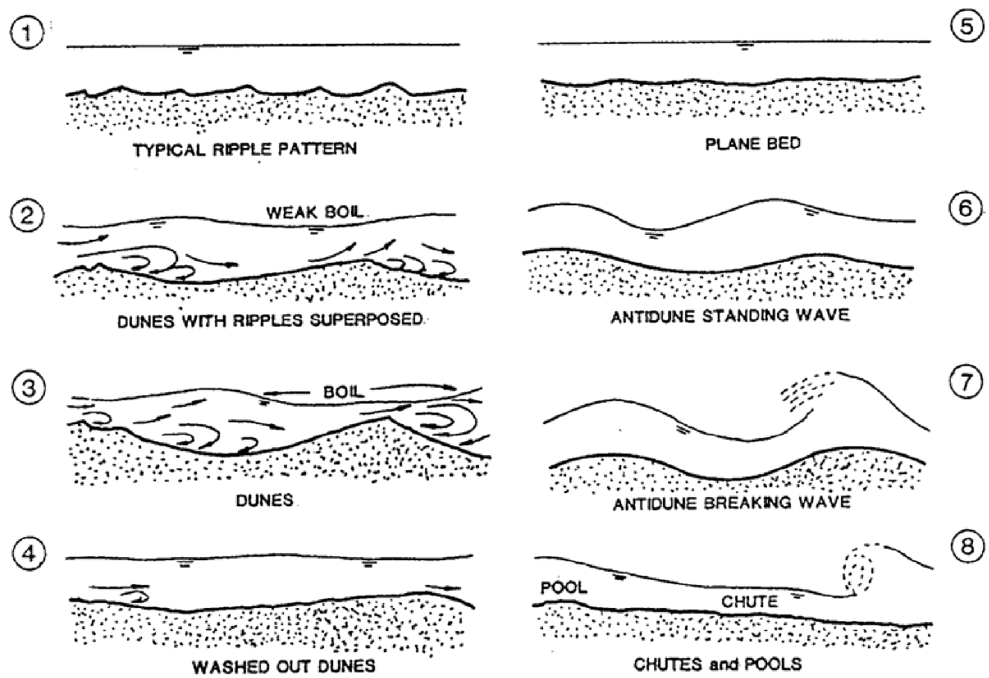


Figure 3.2a. Forms of bed roughness in sand channels.

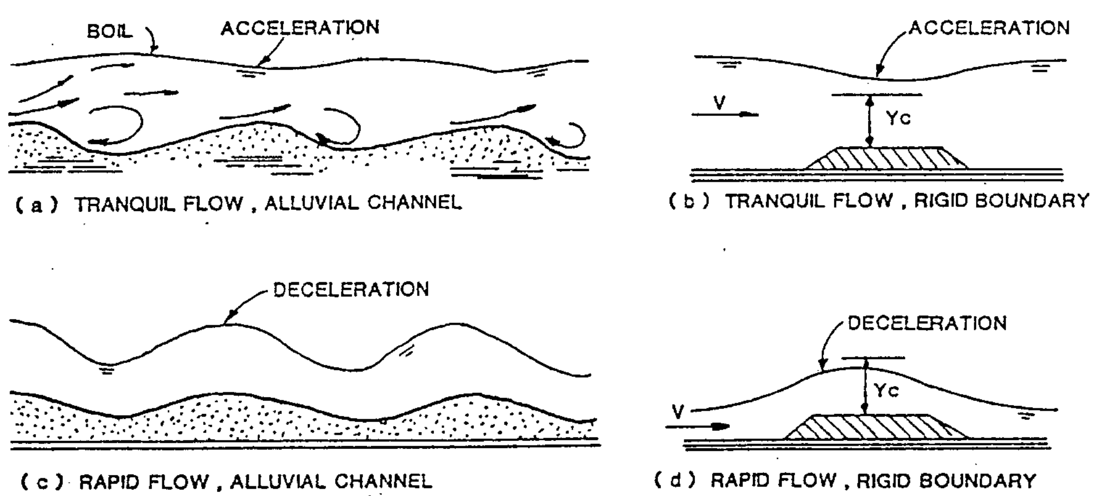


Figure 3.2b. Relation between water surface and bed configuration.

Table 3.1 provides base *n*-values for sand channels and stable channels that are lined with immobile material, while **Table 3.2** provides adjustment factors for use in Equation 3.11. Richardson et al. (1990) and Arcement and Schneider (1984) provide more detailed descriptions of conditions that affect the selection of appropriate values.

3.2.3.3. Resistance to Flow in Sand-bed Channels

The value of n varies significantly in sand-bed channels because of the varying bedforms that occur with different flow regimes. Flow resistance increases with increasing stream power to a maximum value at the upper end of the lower flow regime, decreases rapidly in the transition zone between lower and upper regime and again increases with increasing stream power in the upper flow regime (**Figure 3.3**).

Brownlie (1983) developed relationships for the flow depth in terms of the hydraulic conditions and bed material characteristics for a large set of flume and river data. The USACE Waterways Experiment Station (WES) rearranged Brownlie's relationships to directly solve for the Manning's *n*-value (USACE, 1991).

LOWER REGIME

$$n = [1.6940 \{ \frac{R}{D_{50}} \}^{0.1374} S^{0.1112} G^{0.1605}] \, 0.034 \, D_{50}^{0.167} \tag{3.12}$$

UPPER REGIME

$$n = [1.0213 \{ \frac{R}{D_{50}} \}^{0.0662} S^{0.0395} G^{0.1282}] \, 0.034 \, D_{50}^{0.167} \tag{3.13}$$

where R = hydraulic radius in feet
S = channel slope
G = gradation coefficient of the bed material given by:

$$G = \frac{1}{2}\left(\frac{D_{84}}{D_{50}} + \frac{D_{50}}{D_{16}} \right) \tag{3.14}$$

and D_{16}, D_{50}, and D_{84} are the particle sizes for which 16, 50, and 84 percent of the material is smaller. For slopes greater than about 0.6 percent, the flow will always be in upper regime; thus, Equation 3.13 should be used and this value substituted for n_b in Equation 3.11. For flatter slopes, the *n*-value for the transition between lower and upper flow regime is estimated based on the grain Froude Number (F_g) and a transition relationship given by:

$$F_g = \frac{V}{\sqrt{(S_g - 1)gD_{50}}} \tag{3.15}$$

and

$$F_g' = \frac{1.74}{S_g^{1/3}} \tag{3.16}$$

where S_g = specific gravity of the sediment
g = acceleration of gravity

Table 3.1. Recommended base values of Manning's *n* (n_b)			
Channel or Floodplain Type	Bed Material Median Size (mm)	Base *n*-value	Reference
Sand Channels			
Lower regime flow	0.2 - 2.0	0.030	Chow (1959); Barnes (1967)
Upper regime flow	0.2	0.012	Benson and Dalrymple (1967)
Upper regime flow	0.3	0.017	Benson and Dalrymple (1967)
Upper regime flow	0.4	0.020	Benson and Dalrymple (1967)
Upper regime flow	0.5	0.022	Benson and Dalrymple (1967)
Upper regime flow	0.6	0.023	Benson and Dalrymple (1967)
Upper regime flow	0.8	0.025	Benson and Dalrymple (1967)
Upper regime flow	1.0	0.026	Benson and Dalrymple (1967)
Stable channels and floodplains			
Tined concrete		0.018	DPM 22, Table 22.3 B-1
Shotcrete		0.025	DPM 22, Table 22.3 B-1
Reinforce concrete pipe		0.013	DPM 22, Table 22.3 B-1
Trowled concrete		0.013	DPM 22, Table 22.3 B-1
No-joint cast in place concrete pipe		0.014	DPM 22, Table 22.3 B-1
Reinforced concrete box		0.015	DPM 22, Table 22.3 B-1
Reinforced concrete arch		0.015	DPM 22, Table 22.3 B-1
Streets		0.017	DPM 22, Table 22.3 B-1
Flush grouted riprap		0.020	DPM 22, Table 22.3 B-1
Corrugated metal pipe		0.025	DPM 22, Table 22.3 B-1
Grass-lined channels (sodded & irrigated)		0.025	DPM 22, Table 22.3 B-1
Earth-lined channels (smooth)		0.030	DPM 22, Table 22.3 B-1
Wire-tied riprap		0.040	DPM 22, Table 22.3 B-1
Medium weight dumped riprap		0.045	DPM 22, Table 22.3 B-1
Grouted riprap (exposed rock)		0.045	DPM 22, Table 22.3 B-1
Jetty type riprap (D_{50} > 24”)		0.050	DPM 22, Table 22.3 B-1

Table 3.2. Adjustment factors for the determination of n-values for channels (Arcement and Schneider, 1984).

Conditions	n -value	Remarks
n_1 - Cross-section Irregularity		
Smooth	0	Smoothest channel
Minor	0.001-0.005	Slightly eroded sideslopes
Moderate	0.006-0.010	Moderately rough bed and banks
Severe	0.011-0.020	Badly sloughed and scalloped banks
n_2 - Variation in Cross-sectional Shape and Size		
Gradual	0	Gradual changes
Alternating Occasionally	0.001-0.015	Occasional shifts from large to small section
Alternating Frequently	0.010-0.015	Frequent changes in cross-sectional shape
n_3 - Obstructions		
Negligible	0-0.004	Obstructions < 5% of cross-section area
Minor	0.005-0.015	Obstructions < 15% of cross-section area
Appreciable	0.020-0.030	Obstructions 15-50% of cross-section area
Severe	0.040-0.060	Obstructions > 50% of cross-section area
n_4 - Vegetation		
Small	0.002-0.010	Flow depth > 2x vegetation height
Medium	0.010-0.025	Flow depth > vegetation height
Large	0.025-0.050	Flow depth < vegetation height
Very Large	0.050-0.100	Flow depth < 0.5 vegetation height
M - Sinuosity*		
Minor	1.0	Sinuosity < 1.2
Appreciable	1.15	1.2 Sinuosity < 1.5
Severe	1.30	Sinuosity > 1.5

*Sinuosity is the ratio of channel length to down-valley length.

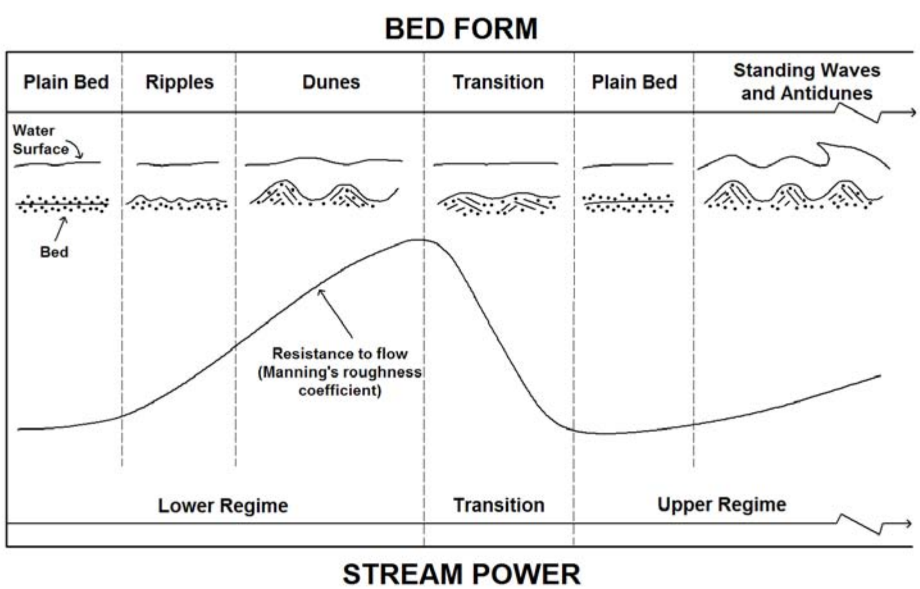


Figure 3.3. Relative resistance in sand-bed channels (after Arcement & Schneider, 1984).

When $F_g \le F'_g$, use the lower regime equation (Equation 3.12) and when $F_g > F'_g$, use the upper regime equation (Equation 3.13). **For arroyos in the SSCAFCA jurisdictional area, upper regime flow can be expected under most conditions.**

Resistance to flow on floodplains. Arcement and Schneider (1984) modified Equation 3.11 for use in estimating *n*-values for floodplains. The correction factor for sinuosity, m, becomes 1.0 for this case and the correction for variations in channel size and shape (n_2) is assumed to be zero. Equation 3.11, adapted for use on floodplains, becomes:

$$n = n_b + n_1 + n_3 + n_4 \qquad (3.17)$$

where n_b = base value of n for a bare soil surface.

Selection of the base value for floodplains is the same as for channels. It is recommended that the user of this Design Guide refer to Arcement and Schneider (1984) for a detailed discussion of factors that affect flow resistance in floodplains.

Resistance to flow in concrete-lined channels. For concrete-lined channels carrying little or no sediment, the boundary roughness values can be selected from Table 3.1, or other appropriate references for concrete roughness (e.g., USACE, 1991). When the channel carries a significant sediment load, the bed roughness may increase to values consistent with an alluvial channel. If the sediment-transport capacity is significantly greater than the supply, most of the sediment particles are expected to be suspended above the bed and no adjustment to the roughness is required. As the sediment supply approaches the transport capacity, a layer of sediment will deposit and move along the bed which will impact channel roughness similar to an alluvial channel. If the sediment supply is greater than the transport capacity, aggradation will occur and the channel capacity and water surface must be adjusted accordingly. For this case, a composite n-value can be estimated using either the conveyance weighting or equal velocity methods. The conveyance-weighting method is described by:

$$n_c = \frac{A_t R_t^{2/3}}{\sum \dfrac{A_i R_i^{2/3}}{n_i}} \qquad (3.18)$$

where the subscripts i and t refer to individual subsections across the cross section and the total cross section (see **Figure 3.4**), respectively, and n_c is the composite *n*-value. The conveyance weighting method given by Equation 3.18 is recommended for purposes of this Design Guide when the Manning's equation is used for the hydraulic computations. Other compositing methods are also available; details can be found in USACE (1991). Of these, the Equal Velocity Method, proposed by Horton and independently by Einstein (Chow, 1959) provides a good approximation for trapezoidal channels, although the method may not be as accurate as Equation 3.18. The Horton or Einstein method is given by:

$$n_c = \left[\frac{\sum_{i=1}^{N} P_i\, n_i^{1.5}}{P}\right]^{\frac{2}{3}} \qquad (3.19)$$

where P = total wetted perimeter
P_i = wetted perimeter in section i
n_i = *n*-value in section i

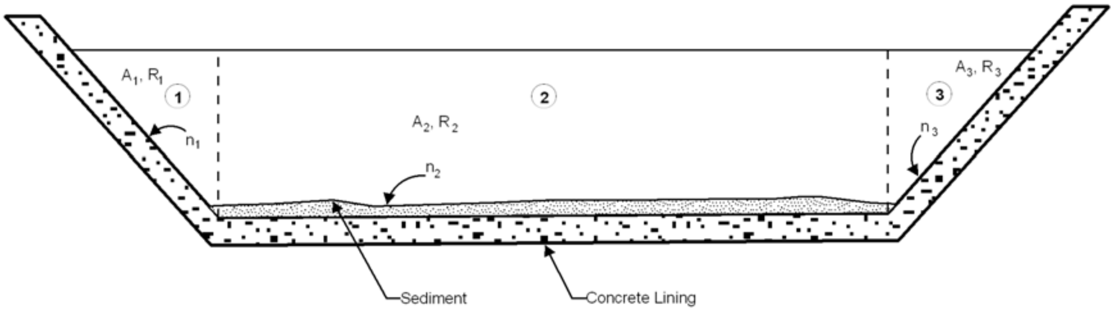


Figure 3.4. Schematic of a concrete-lined channel with significant sediment load.

3.2.3.4. Normal Depth Calculations

As discussed above, when the flow is uniform along the channel, hydraulic conditions can be computed using Manning's or Chezy's equation (Equations 3.3 and 3.4). For these conditions, it is often reasonable to describe the variation in channel geometry (i.e., area, wetted perimeter, hydraulic radius, and topwidth) as single-valued functions of the total flow depth which can simplify the hydraulic and sediment-transport computations. For prismatic cross sections (e.g., trapezoidal channels), relationships between the depth of flow and wetted perimeter can be substituted into Manning's equation and solved for the depth. For natural channel cross sections, two options are available for describing the cross-sectional geometry. The first is to develop relationships between the flow depth and the cross-sectional area, wetted perimeter and topwidth using least squares regression techniques. Power function relationships of the following form usually provide a sufficiently accurate definition of the cross-sectional geometry for most applications where normal depth calculations are appropriate:

$$X = aY_m{}^b \tag{3.20}$$

where X = cross-sectional area, wetted perimeter, or topwidth
Y_m = depth of flow at the thalweg (maximum depth in the cross section)
a, b = constants

Of course, other forms of the equation can also be used. The power function form given by Equation 3.20 provides for easy solution of the normal depth equation.

Another alternative for natural channels is to compute the conveyance within each channel segment across the cross section incrementally and sum to obtain the total flow in the section. This method assumes a constant energy grade line across the channel and requires significantly more computational effort than other methods. For highly irregular cross sections, it may however, provide more accurate results.

Appendix D shows the derivation of the normal depth relationships for the case of a trapezoidal channel and natural channel using the power function relationship given by Equation 3.20.

Incised arroyo channels tend to have relatively flat bottoms and shallow flow depths. For this case, the wide rectangular channel assumption is usually appropriate, eliminating the need for more detailed analysis of the cross-sectional shape for the hydraulic calculations.

3.2.3.5. Water-surface Profiles

The water-surface profile in a channel is a combination of gradually varied flow over long distances, and rapidly varied flow over short distances. Due to various obstructions in the flow (i.e., bridges, drop structures, well established vegetation such as chamisa), the actual flow depth over longer reaches is either larger or smaller than the normal depth defined by Manning's uniform flow equation. In the immediate vicinity of the obstruction, the flow can be rapidly varied.

Gradually varied flow. In gradually varied flow, changes in depth and velocity take place slowly over a large distance, resistance to flow dominates and acceleration forces are neglected. The calculation of a gradually varied flow profile is well defined by analytical procedures, most commonly implemented with computer programs such as HEC-RAS (USACE, 2005).

Rapidly varied flow. In rapidly varied flow, changes in depth and velocity take place over short distances, acceleration forces dominate and resistance to flow may be neglected. The calculation of certain types of rapidly varied flow are well defined by analytical procedures, such as the analysis of hydraulic jumps, but analysis of other types of rapidly varied flow, such as flow through bridge openings are a combination of analytical and empirical relationships. The FHWA document "*Hydraulics of Bridge Waterways*" (Bradley, 1978) provides a procedure for manual calculation of the backwater created by certain types of flow conditions at bridge openings. Gradually varied flow computer programs, such as HEC-RAS , include analysis of bridge backwater, but do not calculate undular jump conditions or flow through the bridge when flow accelerations are large (i.e., large changes in velocity either in magnitude or direction).

3.2.3.6. Superelevation of Water Surface at Bends

The change in flow direction through a channel bend results in centrifugal forces that cause the water surface to be higher along the outside of the bend than along the inside (**Figure 3.5**). The resulting transverse slope can be evaluated quantitatively for both subcritical (tranquil) and supercritical (rapid) flow using the following relationship (USACE, 1970a).

$$\Delta Z = C\,\frac{V^2 W}{g R_c} \tag{3.21}$$

where g = acceleration of gravity (ft/sec^2)
W = channel topwidth (ft)
R_c = radius of curvature at the centerline of the stream (ft)
ΔZ = the difference in water surface elevation between the channel centerline and the bank on the outside of the bend (ft)
V = average velocity (ft/sec)
C = Coefficient (see **Table 3.3**)

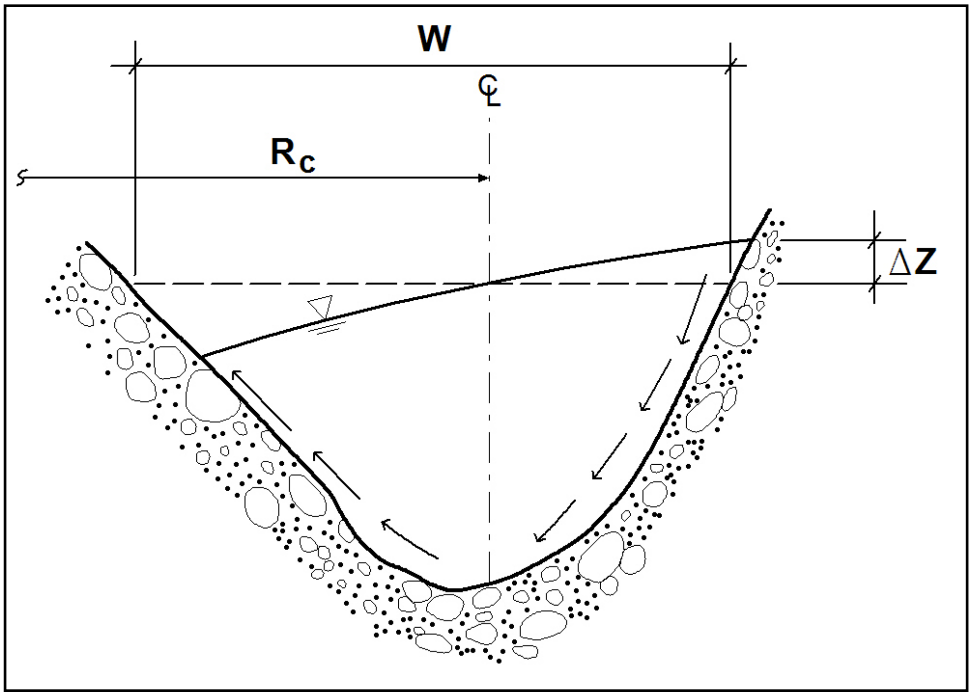


Figure 3.5. Superelevation of water surface in a bend.

Table 3.3. Superelevation formula coefficients (USACE, 1970a).			
Flow Type	Channel Cross Section	Type of Curve	Value of C
Tranquil	Rectangular	Simple circular	0.5
Tranquil	Trapezoidal	Simple circular	0.5
Rapid	Rectangular	Simple circular	1.0
Rapid	Trapezoidal	Simple circular	1.0
Rapid	Rectangular	Spiral transitions	0.5
Rapid	Trapezoidal	Spiral transitions	1.0
Rapid	Rectangular	Spiral banked	0.5

Other equations for superelevation are given in Richardson et al. (1990).

3.2.3.7. Supercritical Flow in Natural Channels

Rigid-boundary hydraulic analysis in arroyos often indicates supercritical flow. Due to the interaction of the water, sediment, and other boundary roughness elements, natural channels typically do not sustain supercritical flow conditions, except over very short distances and very short timeframes (Trieste, 1992). The energy loss associated with antidunes (or chutes and pools) and natural obstructions in the channel can create localized hydraulic jumps. For this reason, the sequent depth (depth to which the water surface will rise through a hydraulic jump) is recommended for determining flood elevations and floodplain limits. The following equation can be used to compute the water-surface elevation associated with the sequent depth ($CWSEL_{seq}$), based on the computed supercritical water-surface elevation from a rigid-boundary analysis (CWSEL):

$$CWSEL_{seq} = CWSEL + 0.5(A/W)\left(\sqrt{1+8F_r^2} - 3\right) \tag{3.22}$$

where A = cross-sectional area of the channel
W = channel width
F_r = Froude Number given by

$$F_r = \frac{V}{\sqrt{gY}} = \frac{Q}{A\sqrt{Ag/W}} \tag{3.23}$$

where V = average channel velocity
Y = hydraulic depth
g = acceleration of gravity

3.3. Sediment Transport in Steep, Erodible Channels

3.3.1. Introduction

The rate and magnitude at which arroyo channels adjust depend on the ability of the flowing water to erode and transport the material that makes up the channel boundary. In channels where the bed and banks are composed primarily of sand-sized material, the bed adjusts to essentially the entire range of flows to which it is subjected. Channel size, shape, and gradient, therefore, depend on the magnitude and duration of the flow and the supply of sediment from upstream sources. In channels where a significant percentage of the bed or bank material consists of coarse gravel and cobble particles, adjustability may be limited.

Incipient motion analysis provides a means for determining the ability of the flows to move the larger particles. In cases where there is a sufficient amount of coarse material in the bed or banks, an armor layer may develop that will act as a partial or complete control on the adjustability of the channel bed. The rate at which adjustment will occur depends on both the capability of the flows to transport the material and the quantity of material brought in from upstream sources. In addition to its utility in evaluating the tendency for channels to armor, the incipient motion concept is an integral part of many bed-material transport relationships.

An understanding of the modes of sediment transport that occur in natural channels is important in selecting appropriate analysis techniques and interpreting the results. Sediment transport can be separated into two general categories: wash load and bed material load (**Figure 3.6**). Wash load is carried in suspension, and is made up of particle sizes finer than those found in significant quantities in the bed. The quantity of wash load carried by a stream is determined by the availability of material from banks and upstream areas of the watershed. Since it is controlled by the availability of material, wash load is considered to be supply-limited because it is generally carried at less than the capacity of the stream. Based on the flow energy, streams are typically capable of carrying considerably more wash load-sized sediment if it were available to the flow.

Bed material load is that part of the total sediment load that is made up of sediment sizes represented in the bed, and it is carried both in suspension and as bed load. The bed load component of the bed material load moves in contact with the bed by various means including rolling, sliding, saltation (brief periods of suspension followed by longer periods of rest) and surface creep (gradual movement of groups of particles by a combination of gravity and fluid forces).

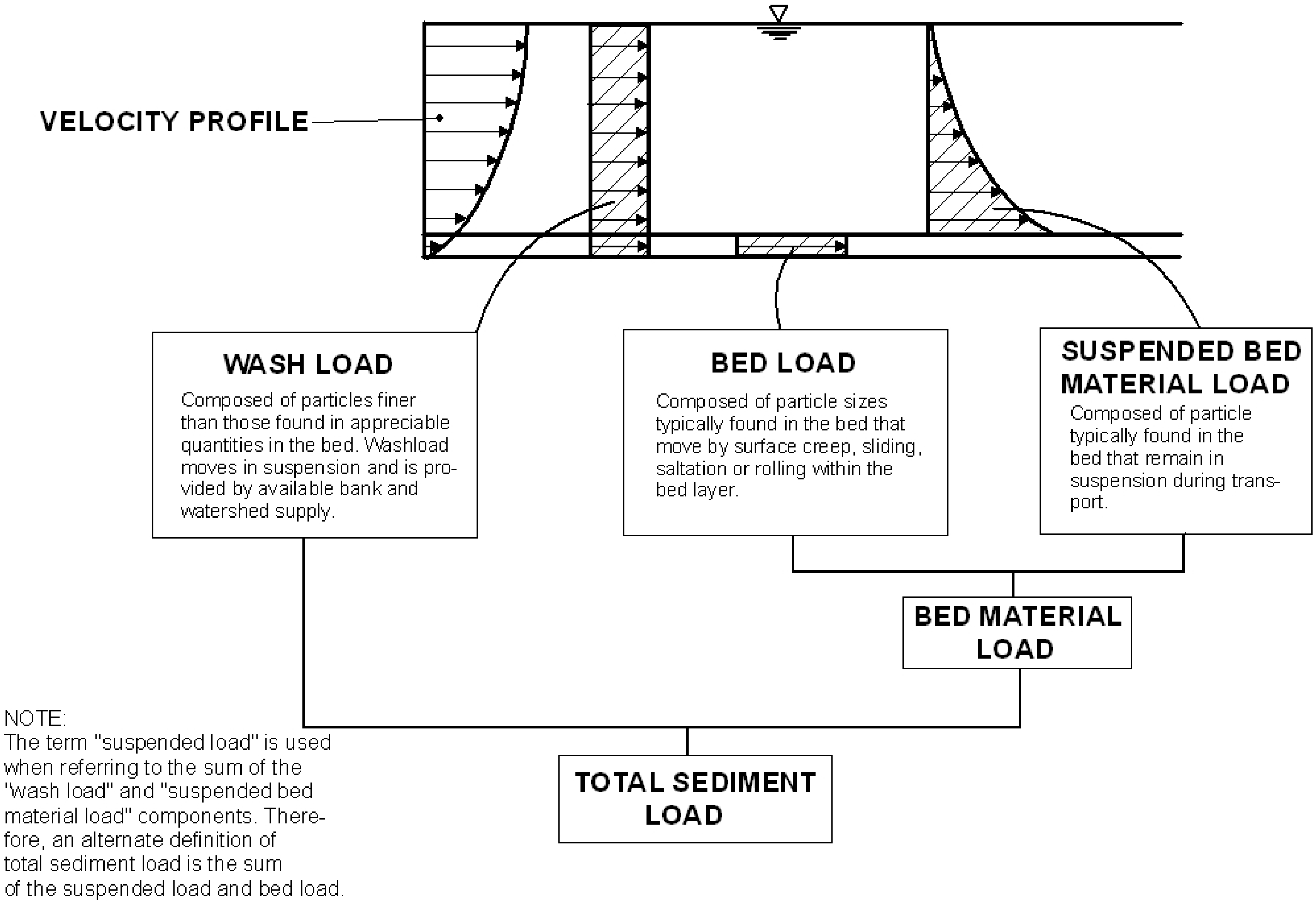


Figure 3.6. Modes of sediment load transport.

Since it is controlled by the availability of material from the watershed, the wash load is not directly related to the hydraulic conditions in the stream at any given time. The bed-material load, on the other hand, is directly related to both the character of the bed material and the hydraulic conditions. As a rule of thumb, it is often assumed that wash load consists of silt and finer material (i.e., less than 0.0625 mm). The maximum size of sediment that can be considered as wash load can, however, vary depending on the characteristics of the stream. In coarse-bed streams, for example, the wash load may consist of material as large as coarse sand. In general, it can be reasonably assumed that sediment finer than about the D_{10} of the bed material (size for which 10 percent of the material, by weight, is finer) makes up the wash load (Richardson et al., 1990, Einstein, 1950).

3.3.2. Analysis of Bed and Bank Material

Bed material is the sediment mixture of which the streambed is composed. Bed material can range in size from large boulders to fine clay particles. Knowledge of the bed-material size gradation is necessary for most sediment-transport analyses, including incipient motion, armoring potential, sediment-transport capacity, and scour.

Bank material usually consists of particles the same size as, or smaller than, the bed particles. As a result, banks are often more easily eroded than the bed, unless protected by vegetation, cohesion, or channel lining.

Of the various sediment properties, size has the greatest significance to the hydraulic engineer since it is the most readily measured and related to other properties that affect the ability of the water to erode and transport the sediment particles. The size of individual particles is less important to the analysis of the channel stability than the distribution of sizes making up the bed and banks.

Particle size gradations consisting of significant quantities of clay material are cohesive in nature. The strength of the cohesive bond will affect the resistance of that material to erosion. Particle size gradations having significant amounts of coarse gravel and cobbles may inhibit the ability of the channel to erode through the armoring process.

The characteristics of the bed and bank material are generally determined by collecting and analyzing representative samples. Important factors to consider in determining where and how many bed-and-bank material samples to collect include:

1. Size and complexity of the study area
2. Number, lengths, and drainage areas of tributaries
3. Evidence of or potential for armoring
4. Structural features that can impact or be significantly impacted by sediment transport
5. Bank failure areas
6. High bank areas
7. Areas exhibiting significant sediment movement or deposition (i.e., bars in channel)

Tributary sediment characteristics can be very important to channel stability, since a single major tributary or tributary source area could be the predominant supplier of sediment to a system.

The depth of bed-material sampling depends on the homogeneity of surface and subsurface materials. Where possible, it is desirable to include material to some depth in establishing bed material characteristics. For example, in sand/gravel-bed systems the potential existence of a surface layer of coarser sediments on top of relatively undisturbed subsurface material must be considered. Samples containing material from both layers would contain materials from two populations in unknown proportions, and thus it is typically more appropriate to sample each layer separately. If the purpose of the sampling is to evaluate hydraulic friction or initiation of bed movement, then the surface sample will be of most interest. Conversely, if bed-material transport during a flood large enough to disturb the surface layer is important, then the underlying layer may be more significant. It should be noted that the material found in bars is a good indication of the sizes of material being transported by the flow as bed-material load.

Methods of sampling depend on the characteristics of the material being sampled and the intended use of the resulting gradation. The most common sampling method consists of collecting a quantity of material from the appropriate location (grab sample) and using standard laboratory analysis to determine the gradation of the material. **Table 3.4** provides guidelines for the minimum weight of samples to insure that the sample is large enough to be representative of the material being tested.

When the material is so coarse that a representative grab sample would be prohibitively large, the surface gradation can be estimated using the photographic grid technique. This method can be accomplished by laying a 2-foot by 2-foot grid subdivided into 0.2-foot grids on a representative location, photographing the grid, and analyzing the distribution of sizes falling under the intersection of the grid points. Details of this method are discussed in Kellerhals and Bray (1971).

Table 3.4. Minimum recommended sample weights for sieve analysis (USACE, 1970b).

Maximum Particle Size	Minimum Weight of Sample	
	g	lb
3-inch	6,000	13
2-inch	4,000	9
1-inch	2,000	4
1/2-inch	1,000	2
Finer than No. 4 sieve	200	0.5
Finer than No. 10 sieve	100	0.25

When the material is too large to be sampled by the grid method, the sample area is under water or a sample over a larger area is desirable, the pebble count technique can be used. This method involves pacing back and forth across the area to be sampled in a grid pattern, picking up and measuring the intermediate axis of the stone at the toe of the boot at each pace, and analyzing the size distribution of those stones. This method is described in Wolman (1954) and Kellerhals and Bray (1971). In analyzing the size gradation of the samples obtained using the pebble count technique, it has been demonstrated (Kellerhals and Bray, 1971) that the percent-by-count method is the most directly equivalent to the percent-by-weight that would be obtained by laboratory sieving.

3.3.3. Bed Material in the SSCAFCA Area

The bed material in arroyos throughout the SSCAFCA area is composed primarily of sand with varying amounts of gravel and small cobbles in some locations. To provide users of this manual with general, quantitative information on the range of bed material gradations that occurs through the area, twenty three (23) grab samples were collected by the author on January 2008 and the particle size gradations were determined using standard laboratory sieve analysis (**Figure 3.7**). With the exception of the two samples that were collected in Lomitas Negras

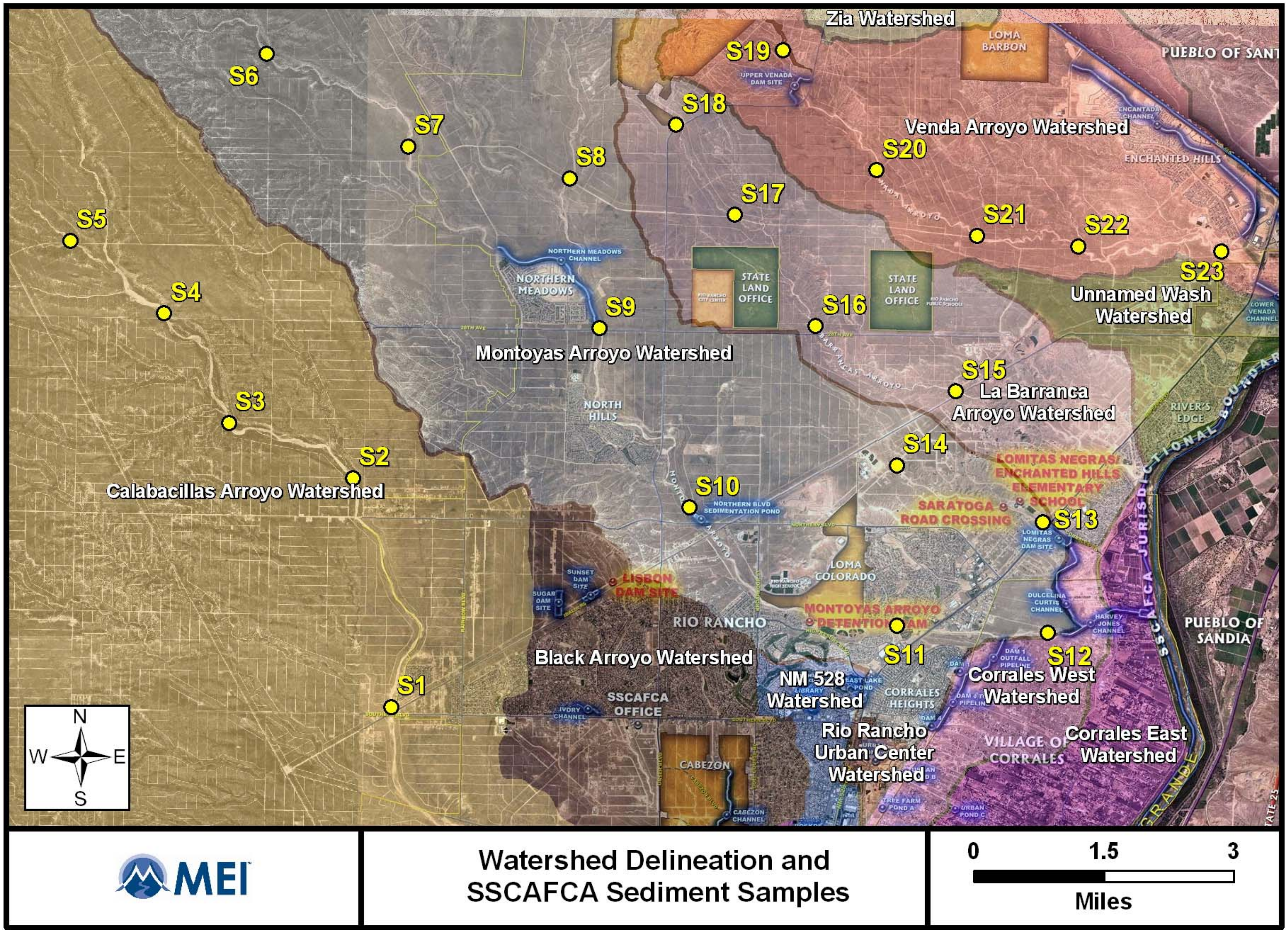


Figure 3.7. Map of SSCAFCA area showing the locations of bed-material samples collected in January 2008.

Arroyo and the most downstream sample in Venada Arroyo, there is a general tendency for the material to fine in the downstream direction (**Table 3.5**). The complete gradation curves for the samples shown in Figure 3.8 are provided in **Appendix G**.

Table 3.5. Summary of median (D_{50}), D_{16} and D_{84} sizes and percent of sand in bed material samples collected in the SSCAFCA area in January 2008.

Sample ID	Arroyo	D_{84} (mm)	D_{16} (mm)	D_{50} (mm)	Percent Gravel (%)
S1	Calabacillas	5.46	0.24	0.69	28
S2	Calabacillas	2.68	0.27	0.77	21
S3	Calabacillas	2.91	0.26	0.74	22
S4	Calabacillas	4.46	0.25	0.83	28
S5	Calabacillas	5.46	0.32	0.91	28
S6	Montoyas	7.68	0.45	1.93	49
S7	Montoyas	4.75	0.29	1.13	36
S8	Montoyas	3.55	0.31	0.81	25
S9	Montoyas	1.48	0.18	0.39	12
S10	Montoyas	2.91	0.18	0.46	21
S11	Montoyas	5.03	0.22	0.96	37
S12	Montoyas	0.50	0.16	0.29	3
S13	Lomitas Negras	7.38	0.21	1.31	42
S14	Lomitas Negras	0.38	0.09	0.19	4
S15	Barranca	2.17	0.20	0.50	17
S16	Barranca	2.65	0.20	0.58	21
S17	Barranca	4.23	0.19	0.67	28
S18	Barranca	13.88	0.18	0.75	36
S24	Barranca	2.36	0.13	0.30	17
S19	Montoyas	3.35	0.19	0.55	24
S20	Montoyas	3.09	0.20	0.51	24
S21	Venada	1.31	0.18	0.40	12
S22	Venada	1.41	0.15	0.33	14
S23	Venada	5.06	0.20	0.80	33

The median (D_{50}) size of the five (5) samples collected along Calabacillas Arroyo ranged from 0.9 mm in the vicinity of King Boulevard to 0.7 mm in the vicinity of Idalia Road, and about 25 percent of the material was consistently in the gravel size-range (i.e., coarser than 2 mm). Six samples were collected along Montoyas Arroyo, and the D_{50} of these samples ranged from 1.9 mm near the westerly extension of Bernende Street at the upstream end (S6) to only about 0.3 mm in the depositional area just upstream from the entrance to the Harvey Jones Channel (S12). The amount of gravel in the Montoyas Arroyo samples ranged about half at the upstream location (S6) to less than 2.5 percent at the downstream location (S12).

The sample collected in Lomitas Negras Arroyo just upstream from NM Highway 528 (S13) had a D_{50} of about 1.9 mm and contained slightly more than 40 percent gravel, and the sample collected just downstream from Idalia Road (S14) had D_{50} of about 0.2 mm and contained about 4 percent gravel. The samples from Barranca Arroyo showed the typical downstream fining trend, with D_{50} ranging from 0.75 mm just upstream from Unser Boulevard (S18) at the upstream end to 0.3 mm just upstream from Highway 528 (S24). These samples contained 17 percent to 36 percent gravel. The upstream four (4) samples from Venada Arroyo also showed the typical downstream fining trend with D_{50} ranging from 0.55 mm just downstream from the Mariposa Development at the upstream end (S19) to 0.33 mm in the vicinity of Progress Boulevard (S 21), and these samples contained 12 percent to 24 percent gravel. The most downstream sample from Venada Arroyo that was collected just upstream from the Encantado outfall had D_{50} of 0.8 mm and contained 33 percent gravel. The arroyo is somewhat incised and confined on both sides by terraces that contain significant gravel. The relatively coarse gradation is likely due to a combination of preferential sediment sorting associated with the incision and supply of coarse sand and gravel-sized material from the bounding terraces.

3.3.4. Incipient Motion and Armoring

Incipient motion refers to the condition where the hydrodynamic forces acting on a grain of sediment are of sufficient magnitude that, if increased even slightly, the grain will move. Under critical conditions (i.e., the point of incipient motion), the hydrodynamic forces acting on the grain are just balanced by the resisting forces of the particle. In cases where there is a significant amount of coarse material (gravel and cobbles) in the bed, the relative susceptibility of the channel bed to degradation can be evaluated by analyzing the magnitude of flows required to produce incipient motion conditions. The bed material in most arroyos in the SSCAFCA area is primarily in the sand size-range; thus, armoring is typically not an important factor in assessing arroyo stability. Methods for evaluating armoring potential in cases where significant gravel is present in the bed material are provided in **Appendix B**.

3.3.5. Methods for Computing Bed Material Transport Capacity

Detailed, quantitative analysis of the aggradation/degradation and lateral migration potential requires knowledge of both the sediment-transport capacity of the stream and the sediment supply. Numerous equations are available for estimating the bed-material transport capacity based on the hydraulic conditions (i.e., velocity, depth, width, shear stress, stream power) and the size characteristics of the bed material. These equations are described in a variety of commonly available books, including Vanoni (1977), SLA (1982), and Chang (1988).

Several of the more commonly used equations have been implemented in the SAMwin software (USACE, 2003). This software also includes guidance on the range of conditions for which the various equations are applicable. At the time of this manual, the USACE is also in the process of implementing several of the more frequently used relationships into a new version of the HEC-RAS software. A beta-test version of the software (Version 4.0) is available on the Hydrologic Engineering Center website (http://www.hec.usace.army.mil/). Official release of the software has not been scheduled, but is expected to occur by the end of 2007. While this software provides a convenient means of performing sediment transport computations, it should be used with caution because it has not been fully tested.

The conditions for which most of the available relations were developed are significantly different from those encountered in the arroyos in the SSCAFCA area, due to the combination of steep gradients and relatively small sediment sizes. Two other relatively simple relationships that are not available in the SAMWin software or HEC-RAS 4.0, but that have been used successfully on projects in the greater Albuquerque area are the MPM-Woo (Mussetter, 2000) and Zeller and Fullerton (1983) equations. The MPM-Woo equation is most applicable to steep, sand-bed channels that carry high concentrations of wash load, and it typically predicts conservatively high transport capacities. The Zeller-Fullerton (1983) equation was developed from transport capacities computed using the Meyer-Peter (MPM)-Einstein method (SLA, 1982) for channels that carry relatively low wash-load concentrations. Results from this equation are often in the lower range of realistic values. Because they are simple and have a history of successful use in the greater Albuquerque area, these equations are described in more detail in **Appendix C**.

3.3.6. Bulking Factors for the SSCAFCA Area

Discharges estimated using standard rainfall-runoff procedures typically do not account for the presence of sediment in the flow. At high sediment loads, the total volume of the water/sediment mixture, and thus, the peak design discharges, can be substantially higher than the corresponding clear-water values. The following relation provides a means of adjusting the clear-water discharges for the presence of the transported sediment if the sediment load is known:

$$B_f = \frac{Q + Q_{s\,total}}{Q} = \frac{1}{1 - \dfrac{C_s / 10^6}{S_g - \left(C_s / 10^6\right)\left(S_g - 1\right)}} \tag{3.24}$$

where B_f = bulking factor,
Q = clear-water discharge,
$Q_{s\,total}$ = **total** sediment load (i.e., combination of bed material and wash load),
C_s = **total** sediment concentration by weight, and
S_g = specific gravity of the sediment.

This relationship indicates that the bulked discharge for a water/sediment mixture at the upper limit of concentrations for water floods (200,000 ppm by volume or 410,000 ppm by weight) would be about 25 percent greater than the clear water discharge (i.e., a bulking factor of 1.25) (**Figure 3.8**).

Because specific knowledge of the sediment load is often not available, conservative estimates of the bulking factor that can be applied to a range of potential design discharges were made by applying the MPM-Woo procedure for a typical, rectangular cross section with width-depth ratio (F_D) at the dominant discharge (Q_D) of 40, assuming critical flow conditions and a range of median (D_{50}) particle sizes. (Dominant discharge is defined, and a method for estimating its magnitude is provided in the text box on the next page.) The assumed width-depth ratio (F) of 40 is based on data from a variety of existing, naturally adjusted arroyos (Leopold and Miller, 1956; Harvey et al., 1985). The assumption of critical flow is based on the observation that average Froude Numbers (F_r) in stable sand-bed streams rarely exceed 0.7 to 1.0 (Richardson, personal communication) at high discharges. It should also be noted that current FEMA procedures for evaluating hydraulic conditions on alluvial fans is based on the assumption of critical flow ($F_r = 1$). Based on analysis of a wide range of arroyos in the greater Rio Rancho and Albuquerque area, the dominant discharge typically has a recurrence interval in the range of 5 to 10 years under relatively undeveloped conditions, and this decreases to 3 to 5 years under highly developed conditions due, primarily, to the increase in runoff during frequently occurring storms. The peak discharge associated with other recurrence interval flows was estimated using average ratios for conditions in the greater Rio Rancho and Albuquerque area. The 100-year peak discharge, for example, averages about five times the dominant discharge. Bulking factors estimated using the above assumptions for the 100-year peak are shown in **Figure 3.9** for channels with dominant discharge ranging from 50 to 1,000 cfs and median (D_{50}) bed-material sizes ranging from 0.5 to 4 mm. As shown in the figure, the bulking factors range from about 1.01 for small arroyos (W_d< = 50 cfs) with relatively coarse bed material (D_{50} = 4 mm) to a maximum of 1.19 for larger channels (Q_D> = 500 cfs) and relatively fine bed material (D_{50} <= 0.5 mm). Estimated bulking factors for other recurrence interval events for the same range of channel and median bed-material sizes are provided in **Table 3.6**.

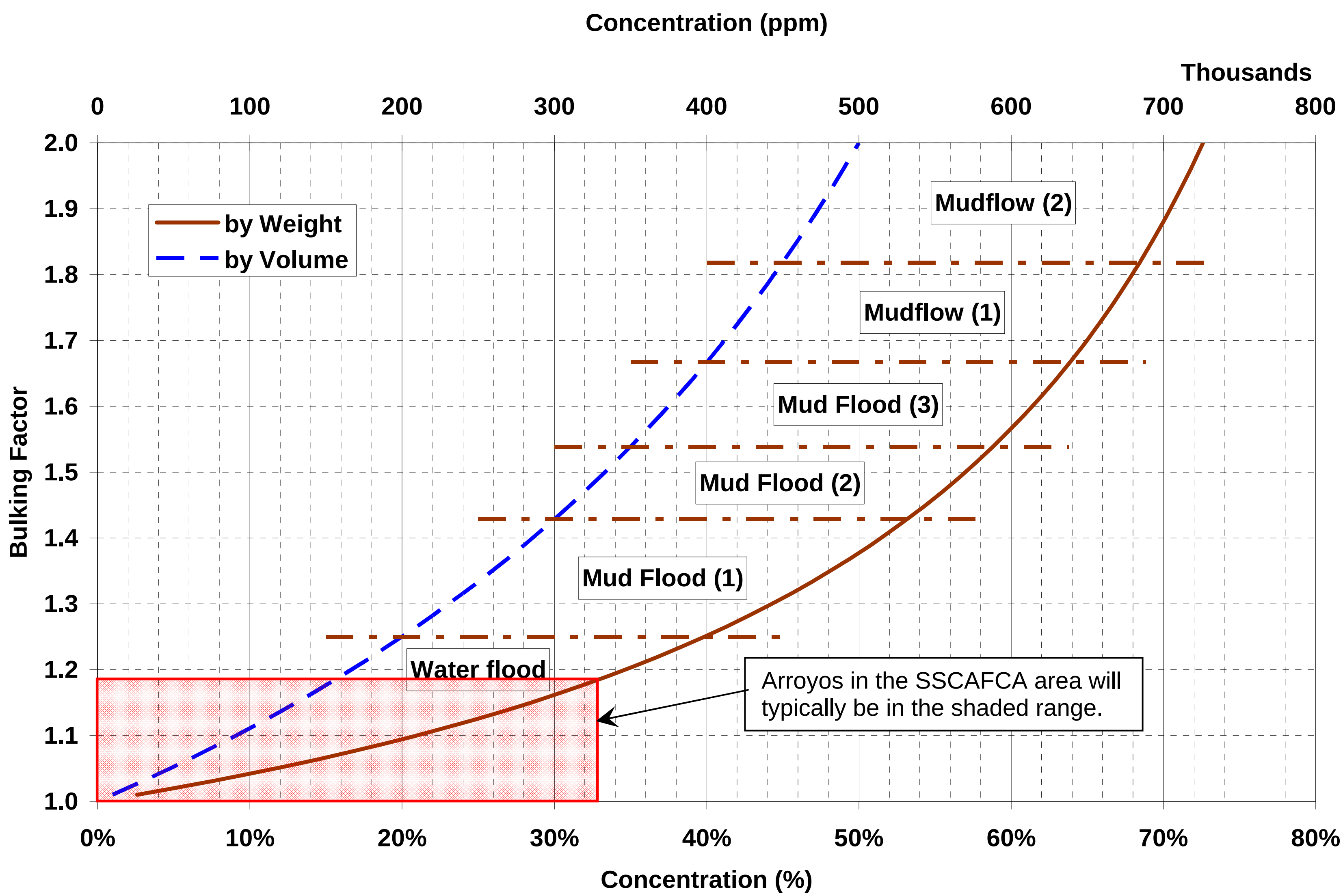


Figure 3.8. Relationship between total sediment concentration and bulking factor.

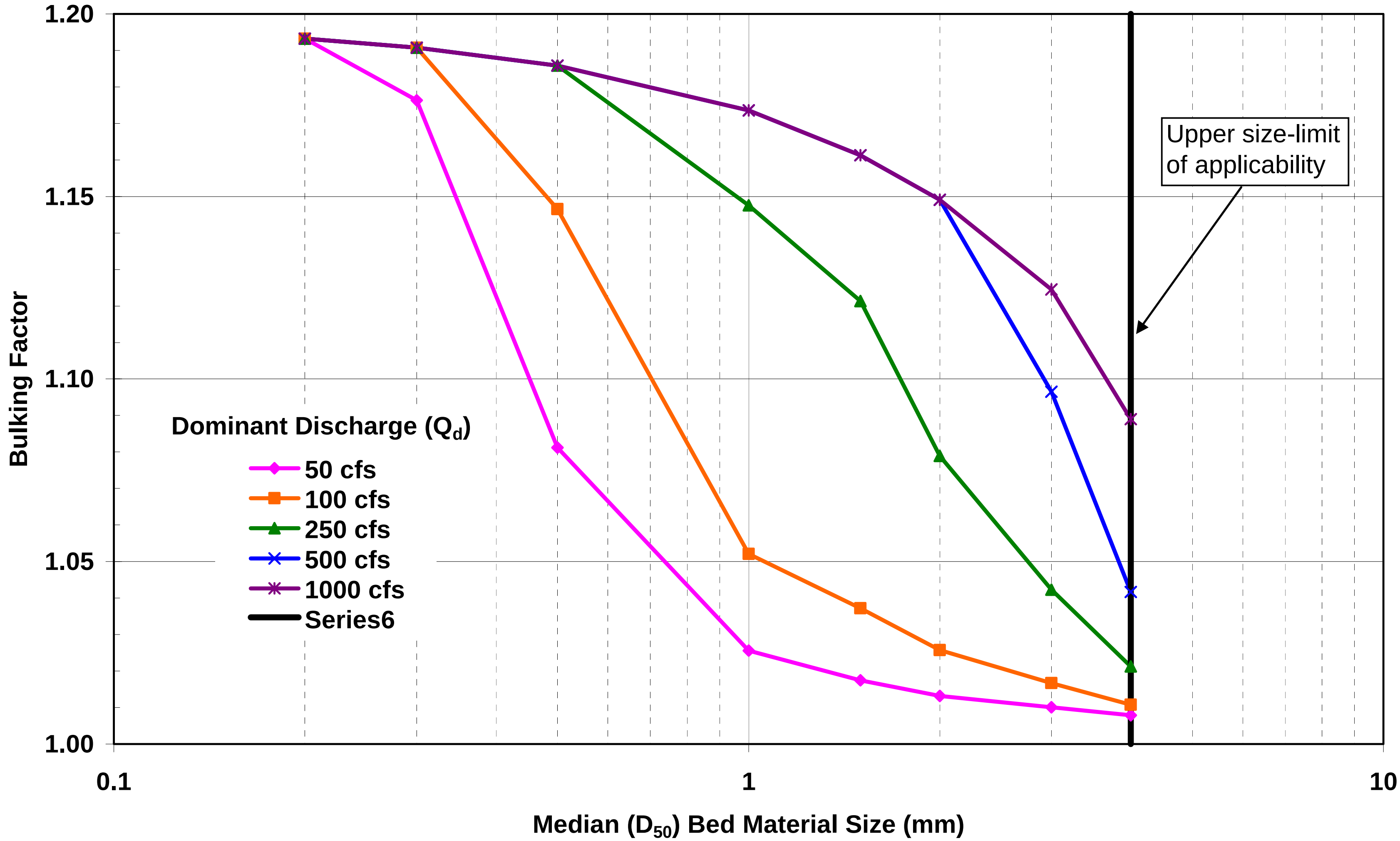


Figure 3.9. Bulking factors for the 100-year peak discharge for natural channels with dominant discharges ranging from 50 to 1,000 cfs and bed-material sizes ranging from 0.2 to 4.0 mm.

Table 3.6. Estimated sediment bulking factors for arroyos in the SSCAFCA jurisdictional area.					
Recurrence Interval (yrs)	Dominant Discharge (cfs)				
	50	100	250	500	1,000
D_{50} (mm) = 0.5 mm					
2	1.01	1.01	1.01	1.01	1.02
5	1.02	1.02	1.05	1.08	1.14
10	1.03	1.05	1.10	1.19	1.19
25	1.05	1.09	1.19	1.19	1.19
50	1.07	1.12	1.19	1.19	1.19
100	1.08	1.15	1.19	1.19	1.19
D_{50} (mm) = 1.0 mm					
2	1.01	1.01	1.01	1.01	1.01
5	1.01	1.01	1.01	1.03	1.05
10	1.01	1.01	1.03	1.07	1.16
25	1.02	1.03	1.08	1.17	1.17
50	1.02	1.04	1.12	1.17	1.17
100	1.03	1.05	1.15	1.17	1.17
D_{50} (mm) = 1.5 mm					
2	1.01	1.01	1.01	1.01	1.01
5	1.01	1.01	1.01	1.02	1.04
10	1.01	1.01	1.02	1.05	1.13
25	1.01	1.02	1.06	1.14	1.16
50	1.01	1.03	1.09	1.16	1.16
100	1.02	1.04	1.12	1.16	1.16
D_{50} (mm) = 2.0 mm					
2	1.01	1.01	1.01	1.01	1.01
5	1.01	1.01	1.01	1.01	1.03
10	1.01	1.01	1.02	1.04	1.08
25	1.01	1.01	1.04	1.09	1.15
50	1.01	1.02	1.06	1.15	1.15
100	1.01	1.03	1.08	1.15	1.15
D_{50} (mm) = 3.0 mm					
2	1.01	1.01	1.01	1.01	1.01
5	1.01	1.01	1.01	1.01	1.02
10	1.01	1.01	1.01	1.02	1.04
25	1.01	1.01	1.02	1.05	1.11
50	1.01	1.01	1.03	1.07	1.12
100	1.01	1.02	1.04	1.10	1.12
D_{50} (mm) = 4.0 mm					
2	1.01	1.01	1.01	1.01	1.01
5	1.01	1.01	1.01	1.01	1.01
10	1.01	1.01	1.01	1.02	1.03
25	1.01	1.01	1.02	1.03	1.06
50	1.01	1.01	1.02	1.04	1.10
100	1.01	1.01	1.03	1.06	1.10

3.4. Evaluation of Channel Adjustments

3.4.1. Sediment Continuity

Vertical adjustments occur in stream channels due to removal of sediment from the channel bed (degradation or scour) or deposition of sediment on the channel bed (aggradation). Within a given reach, the magnitude of vertical adjustment is a function of the difference between the amount of sediment that is carried into the reach by the flow (i.e., the supply) and the amount that is carried out of the reach (i.e., the transport capacity). This concept is a simple statement of the law of conservation of mass. Commonly referred to as the continuity concept, it forms the basis for estimating the magnitude of adjustments to the channel cross section that may occur in response to a given sequence of flows.

The sediment-continuity concept, illustrated in **Figure 3.10**, can be expressed by the following equation:

$$\Delta V = V_{s(inflow)} - V_{s(outflow)} \tag{3.25}$$

where ΔV = volume of sediment stored (+) or lost (-) in the reach,
$V_{s(inflow)}$ = volumetric sediment-transport rate into the reach from upstream and lateral sources, and
$V_{s(outflow)}$ = volumetric sediment-transport rate out of the reach.

It is important to note that the sediment transport and sediment volume in Equation 3.25 relates only to the bed-material load when applied to aggradation/degradation estimates. For most conditions, wash load is assumed to pass through the reach since it is carried in suspension and has little interaction with the channel bed. The sediment-transport rate out of the reach ($Q_{s(outflow)}$) is, therefore, assumed to be the bed-material transport capacity, which can be estimated using the procedures described in a previous section of this manual. The sediment inflow rate is the sum of the bed-material transport capacity (or supply) of the upstream channel and any input of bed material-sized sediment from lateral sources, including tributaries, sheet flow, or bank erosion within the reach.

3.4.2. The Equilibrium Concept

Stream channels tend to adjust toward a state of dynamic equilibrium such that the ability of the channel to carry water and sediment is in balance with the amount of water and sediment delivered from upstream and lateral sources. The condition of dynamic equilibrium with respect to sediment can be expressed by Equation 3.25 by setting the loss or storage term (ΔV) to zero (i.e., the amount of sediment entering the reach is equal to the amount of sediment leaving the reach).

Adjustments to the channel can occur in several ways, including changes in the cross-sectional shape (primarily width), changes in the gradation of the bed material, and changes in the slope. In fact, the adjustments often occur as a combination all of these ways. The relative importance of each depends on the specific characteristics of the channel being analyzed. As described in the previous section if a degrading stream has sufficient coarse material in the bed, winnowing of the fines during degradation will coarsen the bed material, potentially reducing the availability of transportable material. The channel will reach a state of static equilibrium when the bed coarsens sufficiently to balance the inflowing and outflowing sediment loads. If the bed is made up of finer material that is transportable over the entire range of flows and the stream is capable of carrying more material than is being delivered from upstream, the dimensions will adjust and the slope will

Annual Sediment Yield and Dominant Discharge

The **dominant** (or effective) discharge is defined as the increment of discharge that carries the most sediment over a long period of time (Wolman and Miller, 1960; Andrews, 1980; Biedenharn et al., 2000). In perennial, self-adjusted streams, the dominant discharge is often assumed to be same as the bankfull discharge because this represents the long-term condition to which the channel has adjusted, and it is also often assumed to be equivalent to about the mean annual flood peak. Care must be taken in making these assumptions, however, because the dominant, bankfull and mean annual flood peak discharges can be quite different, even in perennial, self-adjusted stream. For ephemeral streams, the dominant discharge tends to be associated with larger, less frequent flood peaks than in perennial streams, due to the absence of sustained flows and the flashy nature of the storm hydrographs. For design purposes, the dominant discharge for lightly developed watersheds in the SSCAFCA jurisdictional area will typically be in the range of the 5- to 10-year peak discharge. In more highly developed watersheds, the frequency of the dominant discharge is typically less because runoff (and sediment transport) associated with the more frequent storms tends to increase dramatically. As a result, the frequency of the dominant discharge is typically in the range of the 3- to 5-year flood peak.

A quantitative method for estimating Q_D in arroyos

If bed-material transport rating curves and storm hydrographs are available, the dominant discharge can be estimated as the peak of the storm event that will produce a bed-material sediment yield equal to the mean annual bed-material sediment yield. The mean annual sediment yield can be estimated by integrating the sediment yield frequency curve (Chang, 1988):

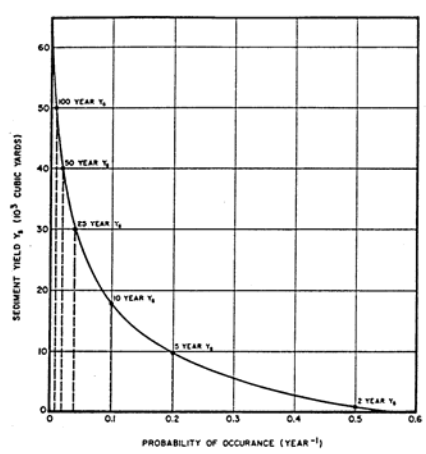
$$Y_{sm} = \int_0^1 Y_s dP_F \tag{3.26}$$

where Y_s is the individual storm sediment yield and P_F is the probability of occurrence of that flood in one year. The product $Y_s \cdot P_F$ represents the contribution of a particular flood to the long-term mean annual yield. For practical purposes, the integration can be accomplished for a series of discrete storm events using the trapezoidal rule. Using the 2-, 5-, 10-, 25-, 50-, and 100-year events, for example, the mean annual sediment yield is approximated by the following relationship:

$$Y_{s_m} = 0.015\,Y_{s_{100}} + 0.015\,Y_{s_{50}} + 0.04\,Y_{s_{25}} + 0.08\,Y_{s_{10}} + 0.2\,Y_{s_5} + 0.4\,Y_{s_2} \tag{3.27}$$

If only the 2-, 10- and 100-year events are used, the following relationship is obtained:

$$Y_{sm} = 0.055\,Y_{s_{100}} + 0.245\,Y_{s_{10}} + 0.45\,Y_{s_2} \tag{3.28}$$



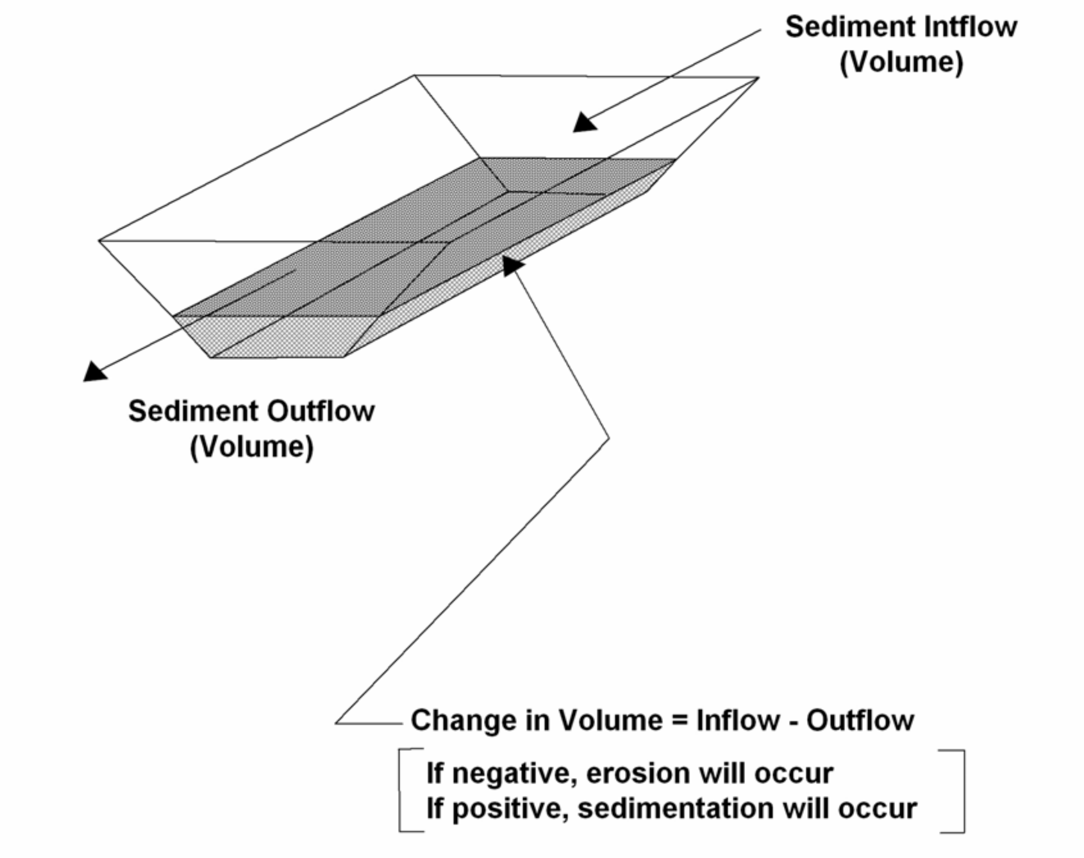


Figure 3.10. Illustration of sediment-continuity concept.

continue to decrease until the transport capacity is the same as the supply. The concept of dynamic equilibrium applies to the condition to which the channel tends over a long period of time, and is the accumulated result of all of the flows to which the channel is subjected.

3.4.3. Evaluation of Vertical Stability

If the channel width remains unchanged, the average change in bed elevation that will result from the change in volume computed from Equation 3.25 can be estimated using the following relation:

$$\Delta Z = \frac{\Delta V}{W_B L(1-\eta)} \tag{3.29}$$

where Δz = average change in bed elevation in the reach,
ΔV = change in sediment volume (see Equation 3.25),
W_B = average width of the channel bed,
L = length of the reach, and
η = porosity of the bed material.

For the type of bed material in arroyos in the SSCAFCA jurisdictional area, the porosity (η) is usually about 0.4.

3.4.3.1. Equilibrium Slope

As indicated by the Lane relation (Equation 3.1), when the sediment delivered to a reach is reduced (or is less than the transport capacity), the channel will flatten its slope to attain an equilibrium condition so that the capacity is in balance with the supply. Conversely, when the sediment delivered to the reach is increased (or is greater than the capacity of the reach), the channel will steepen. The ultimate slope to which the channel will tend for this condition is referred to as the equilibrium slope.

Using hydraulic and sediment-transport relationships and assumptions regarding the channel geometry, it is possible to estimate the equilibrium slope for specific discharge and sediment inflow conditions. Since the analysis is used to evaluate the long-term conditions toward which the channel is adjusting, the analysis should be performed using dominant discharge. In most cases, the equilibrium slope is relatively insensitive to the actual discharge used to compute it, within the range of frequencies typically associated with the dominant discharge.

The magnitude of the sediment supply associated with the dominant discharge is a key element in the equilibrium slope analysis. In the absence of actual sediment supply data, the sediment supply is estimated as the transport capacity of the next upstream reach plus lateral sediment inflows. For natural, undisturbed channels and/or watersheds, this assumption can often be verified through examination of historical data (e.g., profile analysis or aerial photographs). For developed conditions, the impact of the changed land use and upstream channel configuration must be considered in the analysis. Urbanization typically increases the peak discharges and reduces the sediment supply. In addition, unconfined arroyos are typically concentrated into fewer (or single-thread channels, further increasing the peak discharge and volume associated with a given flood event at any point in the channel. If the existing channel has not adjusted to the developed conditions, the transport capacity of the existing supply reach may not accurately reflect the reduced sediment supply that can be expected in the future as the upstream channel adjusts. For this case, it may be necessary to reduce the sediment supply appropriately to reflect future conditions. In general, considerable engineering judgment is required to ensure that the assumptions on which the sediment supply estimates are based accurately reflect conditions controlling the amount of sediment delivered to the reach of concern.

After establishing the upstream sediment supply, the transport capacity of the study reach is evaluated. If the transport capacity and supply for the dominant discharge or individual storm being considered are not equal, the slope and geometry of the channel are adjusted and the transport capacity re-evaluated (assuming armoring is not a consideration). An iterative procedure can be used to obtain a transport capacity that matches the supply. For the equilibrium slope condition, the sediment supply to all reaches being analyzed is the estimated upstream supply to the overall study reach.

Selection of the proper channel geometry is important in equilibrium slope analysis, since sediment transport is proportional to approximately the third to fifth power of velocity and is directly proportional to the channel width. The equilibrium slope is, therefore, very sensitive to these values. Arroyo channels tend to have a rectangular shape with flat bottom and steep banks. The width-depth ratio of the flow is quite large (normally on the order of at least 20 to 40). For such channels (width-depth ratios exceeding about 10), a wide rectangular channel assumption is reasonable for the hydraulic calculations (Chow, 1959). When the wide rectangular channel assumption is valid and significant changes in channel width are not anticipated during the degradation process, the equilibrium slope can be estimated as a function of the unit discharge using the following relation (SLA, 1982):

$$S_{eq} = \left(\frac{a'}{q_{s(supply)}} \right)^{\frac{10}{3(c-b)}} \quad q^{\frac{2(2b+3c)}{3(c-b)}} \quad \left(\frac{n}{1.49} \right)^{2} \tag{3.30}$$

where S_{eq} = equilibrium slope,
$q_{s(supply)}$ = bed-material supply per unit width of channel (cfs/ft),
q = water discharge per unit width of channel (cfs/ft),
n = Manning's roughness coefficient, and
a', b, c = coefficient and exponents of the power function bed-material transport capacity relationship $q_s=a'V^bY^c$ (e.g., Equations C.1 and C.3 in Appendix C).

(Note: The coefficient a'=a in Equation C.1, and a' = a(1-(C_f /10^6) in Equation C.3.)
For cases where the equilibrium slope of a series of reaches having similar roughness, discharge, and channel geometry is of interest, Equation 3.30 can be expressed as a function of the existing channel slope, as follows:

$$S_{eg} = S_{ex} \left(\frac{Q_{s(\text{supply})}}{Q_{s(\text{existing})}} \right)^{\left(\frac{2}{b-x} \right)} \tag{3.31}$$

where

$$x = (3/5)(2b/3 + c) \tag{3.32}$$

and S_{ex} = existing channel slope.

Results of the equilibrium slope calculations are used to predict long-term changes to the channel bed profile. Two important factors that must be considered in evaluating the results of these calculations are the presence of vertical controls, both natural and man-made, and the height of the banks that would develop as the channel flattens in a degradational reach. Vertical controls can be assumed to act as a pivot point where the existing bed will remain fixed, and the equilibrium slope will develop upstream of that point until another control is encountered. Natural controls include bedrock outcrops, sections of channel with an accumulation of large immovable material, or outcrops of caliche or clay. The characteristics of the caliche or clay outcrop should be carefully evaluated since it is probably erodible over time and may act only as a temporary control. Man-made controls include grade-control structures, detention ponds, roadways, dip crossings, culverts that will prevent further lowering of the channel.

When the equilibrium slope calculations indicate significant lowering of the channel bed, the bank heights may become too high to remain stable. When this occurs, bank failure may result, temporarily rejuvenating the sediment supply, slowing or arresting the degradation and resulting in widening and/or lateral migration of the channel consistent with the ICEM described in Chapter 2.

3.4.3.2. An Alternative Approximation of the Equilibrium Slope

From the above relationships, it is apparent that the equilibrium slope depends on the magnitude of the upstream bed-material supply. In many cases, obtaining an accurate estimate of the supply is very difficult or impractical. Factors that contribute to this uncertainty include the timing and magnitude of delivery of bed material-sized sediment from overland and tributary areas, inability to define the long-term condition of the upstream channel (particularly if it has not adjusted to a state of equilibrium), imprecision of available bed-material transport relationships, and uncertainty in the timing and effect of point inputs of bed material due to upstream bank failure.

In such cases where the information necessary to apply the detailed equilibrium slope relationships is not available, an alternative method of estimating the channel slope for equilibrium conditions can be used. This method is based on the observation that stable alluvial channels rarely, if ever, sustain supercritical flow for extended periods of time. Chang (1988), for example, states that "*stable alluvial channels must stay in lower flow regime*" and Trieste (1992) states that *while* [the] *assumption* [of supercritical flow] *may be valid for man-made channels of smooth, nonerosive materials, and some smooth, uniform, bedrock natural channel, it is questionable for most natural channels ...* Since the sediment-transport rate is proportional to the velocity to the third to fifth power, the high velocities associated with supercritical flow would result in extreme rates of bed-material transport in sand-bed streams. The stream will react to this high transport capacity by eroding its bed and banks to a condition that will sustain transport rates consistent with the upstream supply. Average Froude Numbers (F_r) in stable sand-bed streams rarely exceed 0.7 to 1.0 (Richardson, personal communication) at high discharges. In fact, the current FEMA procedures for evaluating hydraulic conditions on alluvial fans is based on the assumption of critical flow (F_r = 1).

Using this argument, the maximum stable slope for a channel of given geometry and dominant discharge can be computed by combining the relationship for the Froude Number with a uniform flow formula. For arroyos in which the wide rectangular channel assumption is valid, the following relationship can be derived based on Manning's formula:

$$S_S = CQ_D^{-0.133} \tag{3.33}$$

where

$$C = 18.28\, n^2 F_D^{0.133} F_r^{2.133} \tag{3.34}$$

The dominant width of the channel (W_D) resulting from these assumptions can be computed from the relation:

$$W_D = 0.5 F_D^{0.6} F_r^{-0.4} Q_D^{0.4} \tag{3.35}$$

In the above equations,
- S_s = maximum stable slope,
- W = width of the channel,
- N = Manning's roughness coefficient,
- F_r = Froude Number (0.7 to 1.0),
- Q_D = dominant discharge, and
- F_D = width-depth ratio of the flowing water.

As previously discussed, data from existing arroyos indicate that a width-depth ratio of the flowing water (F_D) of about 40 is reasonable for conditions at or near the dominant discharge (Q_D), and critical flow conditions (i.e., F_r =1) can be assumed where rigid boundary hydraulics would indicate supercritical flow. For this case, the above equation for the dominant channel width reduces to the following:

$$W_D = 4.6\ Q_D^{0.4} \tag{3.36}$$

The above relationships can be used to estimate the dominant channel width and to obtain an initial approximation of the stable slope for use in locating grade-control structures and bank protection measures.

3.4.3.3. Contraction Scour

Contraction scour occurs when the flow area of a stream at flood stage is locally decreased, either by a natural constriction or by a structure such as a bridge. With the decrease in flow area, the average velocity and bed shear stress increase, resulting in an increase in the bed material transport capacity and, possibly, degradation of the contracted reach. As the bed elevation is lowered, the flow area increases, and the velocity and shear stress decrease, until a state of relative equilibrium is restored. Contraction scour can also be caused by constriction of the floodplain that forces overbank flow back into the channel in the contracted section, increasing the main channel discharge. Overbank flows typically carry very little bed material-sized sediment, further increasing the tendency for scour.

Contraction scour can also be caused by flow around a bend where the concentration of flow on the outside of the bend increases the velocity and shear stress. Other factors that can cause contraction scour include:

1. Change in downstream water surface elevation or drop in baselevel.
2. A natural stream constriction or bend.
3. Street or highway crossings where the crossing narrows the channel.
4. Deposition of lateral and point bars along the channel.
5. Debris blockage.
6. Growth of vegetation within the channel or floodplain.

Contraction scour is typically cyclic. That is, the bed scours during the rising stage of a runoff event, and fills on the falling stage. The maximum contraction scour depth may occur at or slightly after the peak of the runoff event. The elevation of the channel bed after the event has passed may, therefore, not be a good indication of the depth of scour during the event.

Available methods for estimating contraction scour are based on the sediment-continuity principle, and they typically consider two different conditions, depending upon the ability of the uncontracted approach flow to transport bed material into the contracted reach: (1) live-bed scour that occurs when sediment is transported into the contracted reach, and (2) clear-water scour that occurs when bed-material transport in the uncontracted approach flow is insignificant. In the latter case, the scour reaches its maximum depth when the bed shear stress decreases to the critical stress for incipient motion of the bed material. Maximum clear-water scour is about 10 percent greater than the maximum live-bed scour. Detailed procedures for estimating contraction scour, and the technical basis for those procedures, can be found in Richardson and Davis (2001).

3.4.3.4. Antidune Scour

In steep, sand-bed channels typical of arroyos in the SSCAFCA jurisdictional area, antidunes will form on the channel bed during high flows, and the passage of the antidunes can increase the magnitude of scour at any particular location. As a result, the potential scour depth associated with the antidunes must be accounted for in evaluating the total scour near structures for stability analysis. The height of the antidunes can be estimated from the following relation (Kennedy, 1963):

$$h_a = 0.14\,\frac{2\pi\,V^2}{g} = 0.28\,\pi\,Y\,F_r^2 \tag{3.37}$$

where h_a = antidune height,
V = flow velocity,
g = acceleration of gravity,
Y = hydraulic depth of flow, and
F_r = Froude Number.

Scour associated with the antidunes is estimated as one-half the antidune height (Figure 3.11).

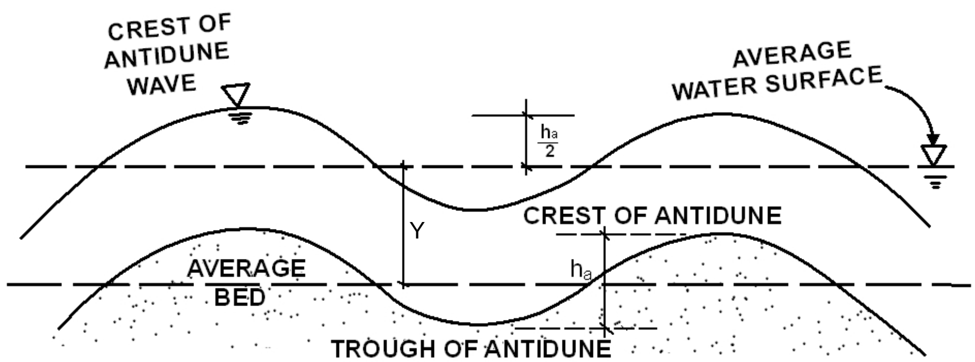


Figure 3.11. Definition sketch for antidune height (SLA, 1985).

3.4.4. Sediment Transport and Channel Stability at Culverts

Culverts can have a significant effect on the stability and dynamics of an arroyo. If backwater or flow expansion occurs upstream of the culvert, sediment will deposit, reducing the flood-carrying capacity of the upstream arroyo and trapping sediment that would otherwise be carried to downstream reaches. Severe deposition can totally plug the culvert, resulting in overtopping of the roadway and flanking of the crossing. Even minor deposition over a period of time can severely reduce culvert capacity. Trapping of sediment upstream (or within) the culvert will also cause degradation in the downstream channel, with the attendant effects on channel stability. The high velocity jet that normally occurs at the outlet of the culvert can cause local scour which may further aggravate channel instability, and may endanger the stability of the crossing. Numerous examples of these effects can be seen throughout the greater Rio Rancho and Albuquerque area. Evaluation of this problem requires consideration of four basic factors:

1. The potential for sediment deposition in the channel upstream of the culvert resulting from backwater due to energy losses at the culvert entrance and/or expansion of the upstream channel,

2. The sediment-transport capacity of the culvert and resulting potential for excessive head loss and/or plugging,

3. Reduction of bed-material sediment supply and resulting bed degradation in the downstream channel, and

4. Local scour at the culvert outlet.

A qualitative discussion of the first three processes is presented below, and method for computing the size of the local scour hole for noncohesive bed material is presented in Section 3.5.

3.4.4.1. Sediment Deposition Upstream of Culverts

The potential for sediment deposition in the channel upstream of a culvert is related to the configuration of the channel and amount of energy loss that occurs as the flow enters the culvert. Backwater caused by the energy loss reduces the flow velocity and thus, sediment-transport capacity, of the upstream channel. The magnitude of the backwater and its effect on the channel velocity can be evaluated using standard computations that consider the hydraulics of the culvert entrance and upstream channel [e.g., "*Hydraulic Design of Highway Culverts*" (FHWA, 1985) and HEC-RAS (USACE, 2005)]. An expansion of the channel upstream of the culvert will also cause deposition. This has occurred, for example, at crossings where the arroyo banks leading to the culvert are not continuous across the roadside drainage ditch. In most such cases, degradation of the channel downstream of the crossing will occur.

The magnitude of the deposition upstream from a culvert must be evaluated on a case-by-case basis using the sediment-continuity concepts presented in previous sections. If the backwater creates a significant ponding effect (i.e., reduce the average channel velocities to less than 1 to 2 fps in the backwater zone), trap efficiency calculations may be necessary to determine the amount of the inflowing sediment load that will be carried into the culvert. If the backwater effect is less significant, sediment routing may be required to evaluate the dynamic effects of backwater and channel bed changes through the storm hydrograph. In certain instances, backwater during the peak flows may cause deposition upstream of the culvert which will steepen the energy gradient into the culvert at lower flows during the recession limb of the hydrograph. This, in turn, will re-entrain all or part of the deposited sediment and return the channel bed toward its original elevation.

3.4.4.2. Sediment Transport in Culverts

The ability of the culvert to carry the sediment passing through the culvert entrance is basically a sediment-transport capacity problem. A comprehensive treatment of sediment transport in closed conduits is beyond the scope of this Design Guide. General information is provided here to introduce the reader to the subject, and to provide guidance regarding the need to employ more sophisticated methods. More detailed treatment of the problem can be found in several references, including ASCE Manual No. 54 (Vanoni, 1977) and Graf (1984).

Figure 3.12 is a conceptual plot taken from ASCE Manual No. 54 (Vanoni, 1977) illustrating the various modes of sediment transport through a closed conduit. This figure shows that the transport mode depends on the particle size and flow velocity in the pipe. Graf (1984) also indicates that the transport mode depends on pipe size. In general, the transport mode can range from an essentially homogeneous mixture the sediment being uniformly distributed throughout the fluid for high velocities and small particle sizes to an armored bottom with no-sediment motion on the bottom of the conduit.

For sand-sized sediment, it is reasonable to assume that the transport mode will be in the range of heterogeneous flow (i.e., solids in suspension or flow with moving bed). For practical purposes, if the velocities are sufficiently high for homogeneous suspension, it can be assumed that the transport capacity is sufficiently high to pass any sediment that enters the culvert and, in the absence of obstructions, the potential for internal culvert blockage is minimal. Data presented in both of the above publications indicate that the transport potential through closed conduits is similar to that in open channels for similar hydraulic and sediment characteristics. Based on this observation, the transport capacity within the culvert can be estimated using one of the previously presented bed material transport capacity equations.

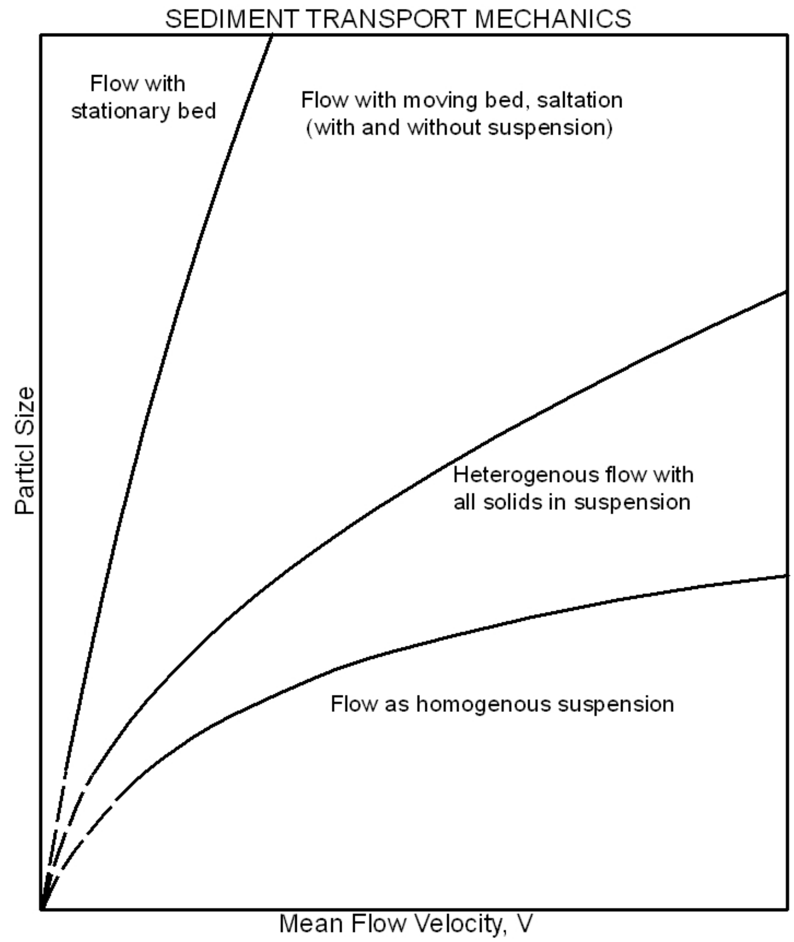


Figure 3.12. Conceptual illustration of modes of sediment transport in closed conduits.

3.4.4.3. Degradation from Culverts

The potential for general degradation of the channel downstream of the culvert can be evaluated using the continuity and equilibrium slope concepts presented in previous sections of this Design Guide. The culvert will act as a control point in the channel, and the dynamics of the downstream arroyo will depend on the amount of sediment entering and passing through the culvert in relation to the transport capacity of the upstream reach.

3.4.5. Evaluation of Lateral Stability and the LEE Line

3.4.5.1. Bank Erosion Processes

Lateral migration and widening of natural channels occurs through bank retreat resulting from two primary mechanisms: grain-by-grain erosion and mass failure. Commonly, mass-wasting and grain-by-grain erosion act in concert; fluvial erosion scours the toe of the bank, and mass failure follows (Simon et al., 1991). Removal of the failed bank material from the bed of the channel occurs through fluvial erosion, and the process is repeated.

The bank erosion process can result from channel incision (degradation), flow around bends, flow deflection due to local deposition or obstructions, aggradation, or a combination of the above. For the case of an incising channel, exceedence of the maximum stable bank height will lead to mass failure and bankline retreat. Flow around a bend can cause erosion at the toe of the bank and subsequent bank failure due to increased shear stress on the outside of the bend. Both local deposition and aggradation over a longer reach create mid-channel bars that can deflect flow into the bank with essentially the same result as flow on the outside of a bend.

The specific failure mechanisms at a given location are related to the characteristics of the bank material that can be broadly classified as cohesive, noncohesive, and composite (**Figure 3.13**). Although the bank material in most arroyos in the SSCAFCA jurisdictional area consists primarily of noncohesive sands and fine gravels, they can contain sufficient amounts of cohesion to influence the bank erosion characteristics. The following discussion relates to the bank failure conditions indicated in Figure 3.13:

1. Noncohesive bank material tends to be removed from the bank grain by grain (Figure 3.13a). The rate of particle removal, and hence the rate of bank erosion, is affected by factors such as particle size, bank slope, the direction and magnitude of the velocity adjacent to the bank, turbulent velocity fluctuations, the magnitude and fluctuations in the shear stress exerted on the banks, seepage forces, piping, and wave forces. Figure 3.13a illustrates failure of noncohesive banks from flow slides resulting from a loss of shear strength because of saturation, and failure from sloughing resulting from the removal of material in the lower portion of the bank.

2. Cohesive material is more resistant to surface erosion and has low permeability, which reduces the effects of seepage, piping, frost heaving, and subsurface flow on the stability of the banks. However, when undercut and/or saturated, such banks are more likely to fail due to mass wasting processes. Cohesive banks typically fail due the development of tension cracks at the top of the slope resulting in a wedge-type failure of the surface block in front of the tension crack or through rotational failure (Figure 3.13b).

3. Composite or stratified banks consist of layers of materials of varying size, permeability, and cohesion. The layers of noncohesive material are subject to surface erosion, but may be partly protected by adjacent layers of cohesive material. This type of bank is vulnerable to erosion and sliding as a consequence of subsurface flows and piping. Typical failure modes are illustrated in Figure 3.13c.

Grain-by-grain erosion can be a significant process in areas of concentrated flow and high shear stress (e.g., on the outside of bends). However, studies of bank erosion processes in both perennial streams and arroyos indicate that mass failure and subsequent fluvial transport of the failed material is the primary mechanism by which the lateral adjustments occur. For example, Leopold and Miller (1956), in a study of arroyos in the Santa Fe area, observed that *Flash floods in arroyos ... appear to do but insignificant amounts of bank cutting as a direct result of impingement of flow on the banks. Wetting of the banks, however, results in subsequent collapse of arcuate slabs of alluvium which tumble into the channel to become important additions to the load of later floods.*

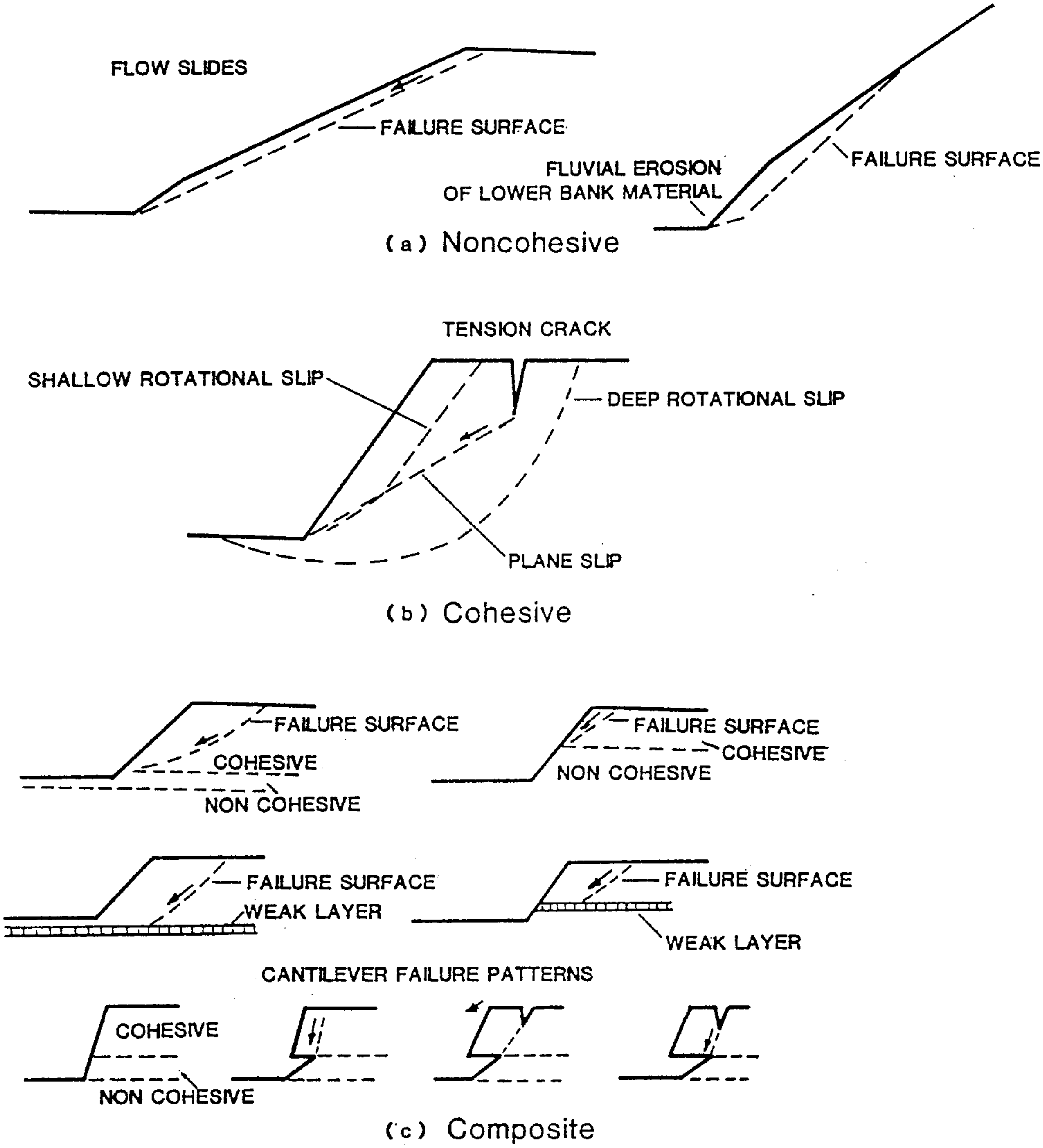


Figure 3.13. Typical bank failure surfaces: (a) noncohesive, (b) cohesive, (c) composite (after Brown, 1985b).

Lohnes and Handy (1968) identified two major failure mechanisms for gully banks that were formed in loess: (1) shear failure along a planar slip surface through the toe of the bank, and (2) slab failure of vertical banks by tension cracking and planar slip. Their analysis investigated the relationship between the height, slope angle, and strength properties of the banks to predict maximum stable bank heights. In their analysis, pore water pressures were neglected; their approach should, therefore, only be used for highly permeable soils with a low degree of saturation.

Bradford and Piest (1980) identified three major mechanisms of gully-wall failure: (1) deep-seated circular arc toe failure, (2) slab failure, and (3) "pop out" failure with shear failure of the remaining cantilevered bank section. Their study indicated that slab failures were associated with vertical banks whereas circular arc failures were associated with lower angle banks. They also concluded that fluvial erosion of *intact* bank material (grain-by-grain erosion) appears to contribute very little to bank retreat. Similar observations have been made by Little et al. (1982) and Schumm et al. (1981), reinforcing the idea that most incised channel bank failures are due to gravitational forces which are primarily controlled by the degree of channel incision. It is important to note, however, that fluvial erosion of *previously failed* bank material does play a significant role in determining the rates of bank retreat. As observed by Thorne (1981), fluvial activity controls the state of *basal endpoint control*; removal of the failed material results in the formation of steeper banks and may induce toe erosion by removing the material along the toe that tends to buttress the bank slope. These factors rejuvenate the process of bank erosion by mass failure. Without basal erosion, mass failure of the bank material would lead to bank slope reduction and stabilization within a relatively short period of time (Lohnes and Handy, 1968; Thorne, 1981).

3.4.5.2. Bank Stability Analysis Techniques

In view of the above discussion, it is apparent that analysis of the potential for lateral migration and widening of the arroyo channels must include bank stability considerations and the resulting potential for mass failure. The geotechnical stability number (N_g), which is the ratio of the actual bank height to the critical bank height, as discussed in Chapter 3, is the basic parameter with which to evaluate the stability of the channel banks. Due to the relatively large number of factors and complexity of the processes, a relationship between bank angle and bank height using field data **on a site-specific basis** provides the most accurate means of evaluating bank stability. Such a relationship could also provide a means of calibrating the parameters in available analytical relationships for evaluating bank stability. Since little or no site-specific data are available with which to develop such a relationship, the analytical relationships must be used with parameters estimated from the geotechnical properties of the bank soils. The appropriate method for determining the critical bank height depends on the assumed shape of the failure surface and the composition and stratigraphy of the banks (Simon et al., 1991).

Ponce (1978) presented a method for analyzing an initially stable bank subject to undermining resulting from baselevel lowering. He assumed a circular failure arc with the failure surface passing through the toe of the bank and used the simplified Bishop method to develop stability curves relating the stable bank height to the soil properties (angle of repose, cohesion, and unit weight) and the bank angle. He neglected the effect of pore water pressure in his analysis. For arroyos and drainageways in the SSCAFCA jurisdictional area, the later assumption may be reasonable since the water table is usually well below the level of the channel bed and the soils are typically well drained. Ponce's approach was adopted for the original Prudent Line analysis developed for Calabacillas Arroyo (Lagasse et al., 1985).

Osman and Thorne (1988) developed a model for wedge-type failures for steep (bank angle >60º) cohesive banks with failure through the toe which is more representative of the bank failure process discussed in the last section. The Osman and Thorne (1988) study included a detailed treatment of the combined effects of fluvial erosion and bank stability for analyzing bank retreat. The geometry of the banks assumed in their analysis is illustrated in **Figures 3.14 and 3.15**.

Figures 3.14a and 3.14b show the assumed initial condition of the bank and its configuration after the first failure, respectively. Figures 3.15a and 3.15b illustrate the initial and failure condition for subsequent failures of the bank during parallel retreat. This method provides relationships to determine the magnitude of degradation (Δz) and horizontal toe erosion (Δw) required to produce a condition of incipient failure.

It is important to note that the soil properties to be used in the analysis are for in-situ conditions, where the cohesion and unit weight may be substantially less than customary values used in geotechnical engineering design for compacted soil conditions. The internal friction angle (ϕ) for most bank soils in the SSCAFCA jurisdictional area vary from about 28º to 32º. The cohesion (c) varies from about 200 to 400 psf, and the in-situ unit weight (γ_s) is about 100 pcf. These values are representative of average conditions. It is recommended the bank material be tested on a site-specific basis prior to performing a detailed stability analysis using this method.

Little et al. (1982) discuss methods for determining the in-situ properties of the bank material. Thorne (1993, personal communication) has found in subsequent work that the effective tensile strength (tensile strength is a function of cohesion (c) and internal friction angle (ϕ) under conditions conducive to bank failure tends to be significantly lower than would be indicated by even the in-situ tests. He believes this is due to a combination of soil moisture effects that occur at failure, but may not be present at the time or exact location of the testing, and other local weaknesses in the soil matrix that may not be indicated by the in-situ test. In general, the strength of the bank at a given location will be controlled by the weakest zone in the bank. For this reason, the low end of the range of reasonable c and Φ values should be used for conservative design purposes.

Since the processes described by Osman and Thorne (1988) are similar to those that have been observed in incising arroyos in the SSCAFCA jurisdictional area, this is the recommended method for estimating the maximum height at which the banks will remain stable and the amount of bank retreat associated with channel incision that may be expected to occur. The maximum stable bank height predicted by this method for a range of internal friction angles (ϕ) and cohesion (c) typical of soils in the SSCAFCA jurisdictional area is shown in **Figure 3.16**. These curves were developed assuming an in-situ unit weight of 100 pcf, bank angle of 70º and tension crack depth of half the bank height.

The assumed bank angle and tension crack depths are consistent with observations of Little et al. (1982) for incised channels in other areas and corroborated by data collected on several incising arroyos in the Albuquerque area.

The method was also used to develop relationships for the amount of bank retreat (BW in Figures 3.14 and 3.15) and the associated volume of material resulting from block failures from the bank as a function of the degradation depth using conservative values for cohesion (c) and internal friction angle (ϕ) of 200 psf and 28º, respectively (**Figures 3.17a and 3.17b**). The bank retreat distance indicated by Figure 3.17a represents the minimum horizontal distance from the existing bankline that would be unstable as a result of block failures from the bank during degradation.

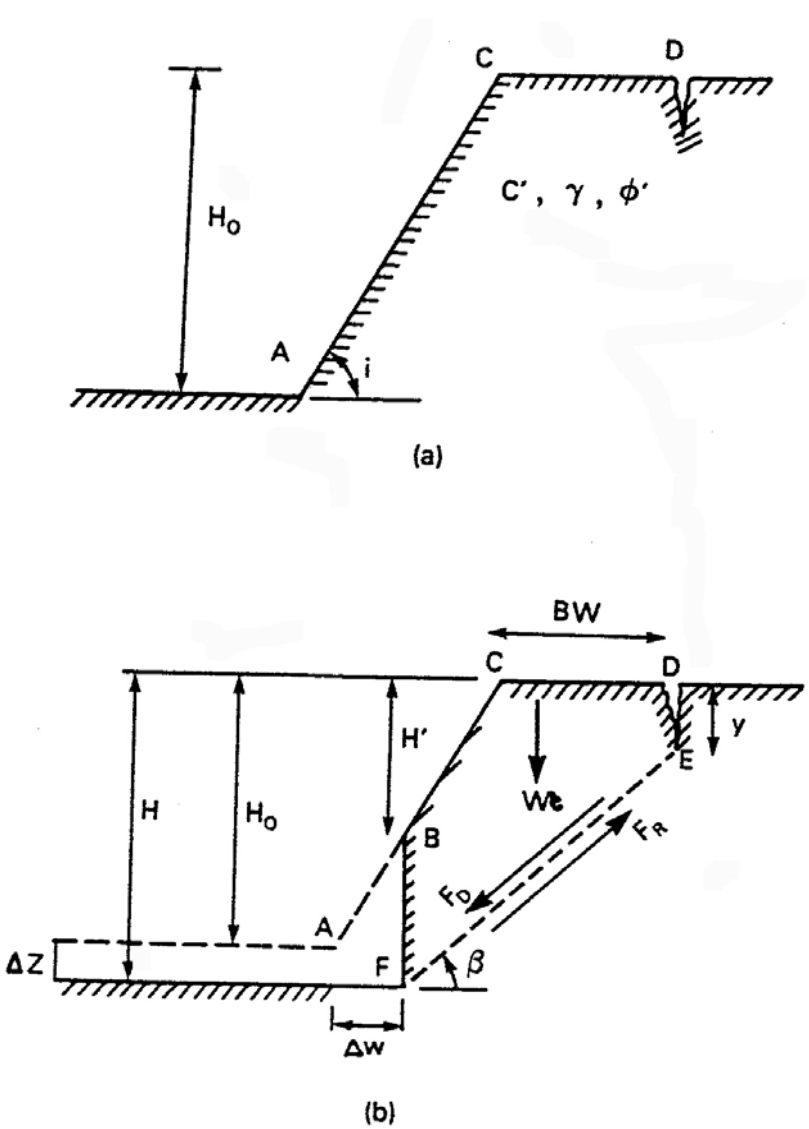


Figure 3.14. (a) Right arroyo bank before erosion (b) right arroyo bank after erosion to point of failure (Osman and Thorne, 1988).

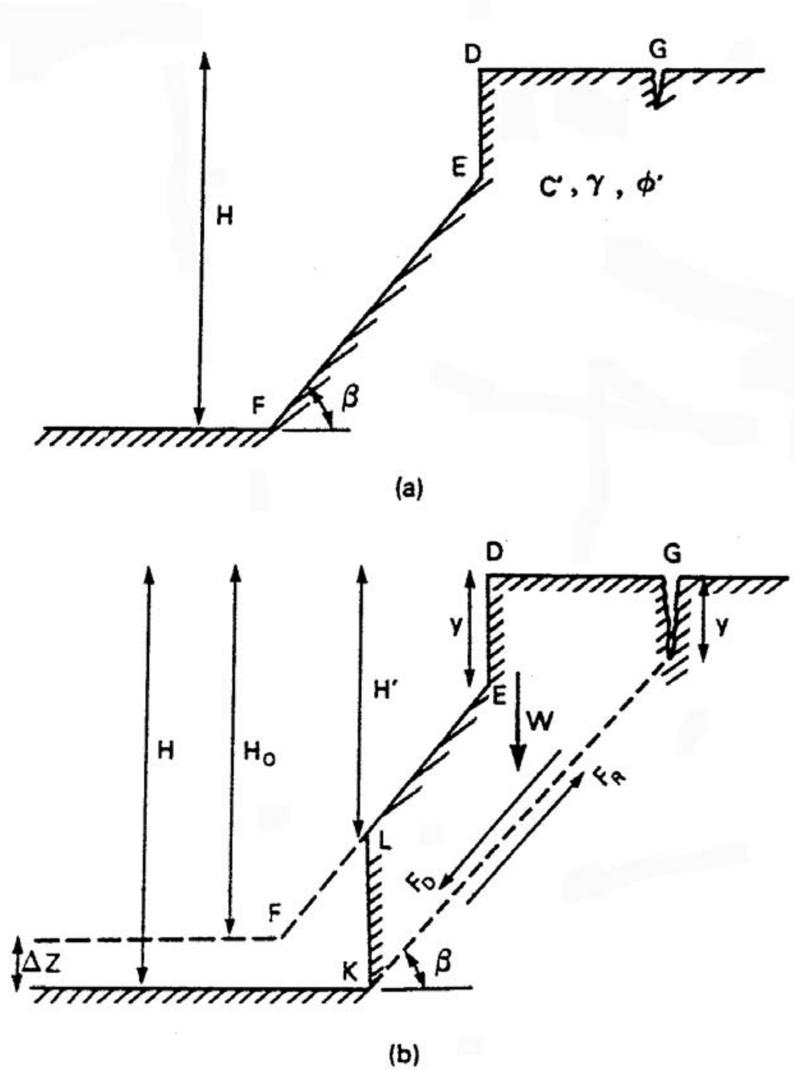


Figure 3.15. Parallel bank retreat: (a) arroyo bank after initial failure; (b) arroyo bank after erosion to point of failure.

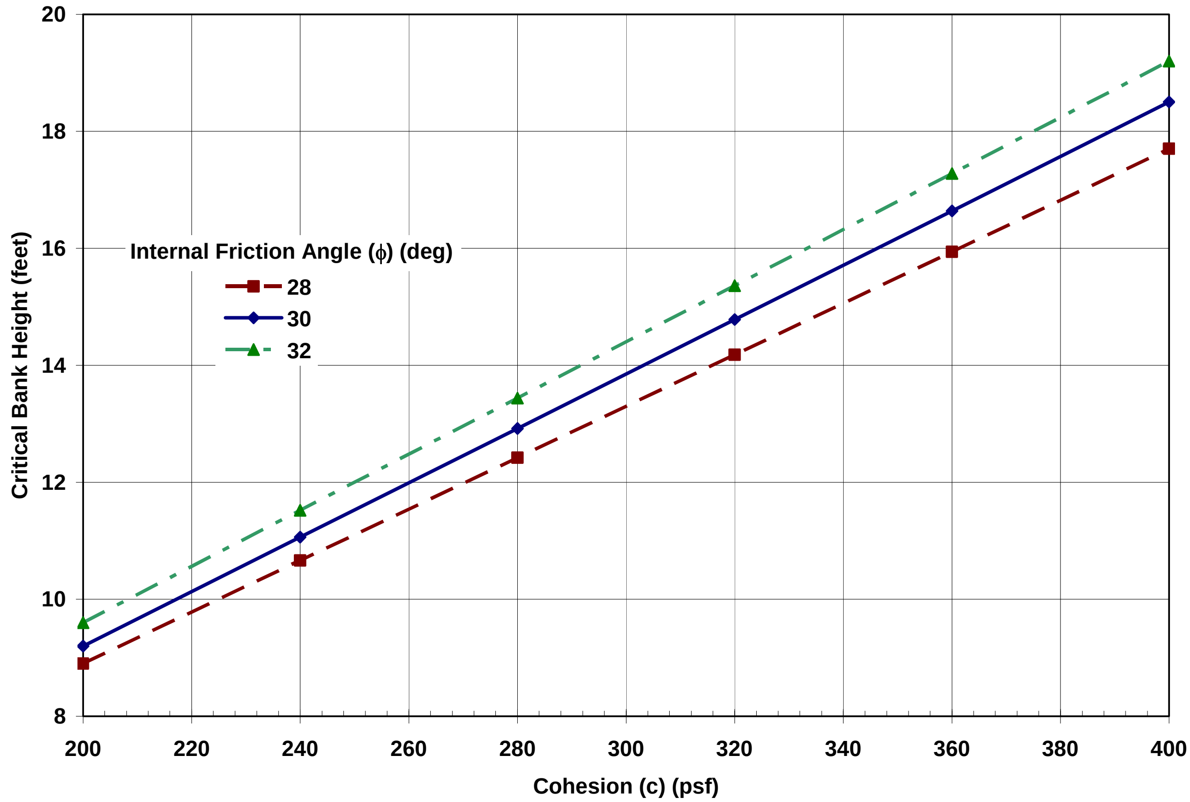


Figure 3.16. Critical bank height predicted by Osman and Thorne (1988) model for varying internal friction angle (ϕ) and cohesion (c) (assumes in-situ unit weight of bank material = 100 pcf).

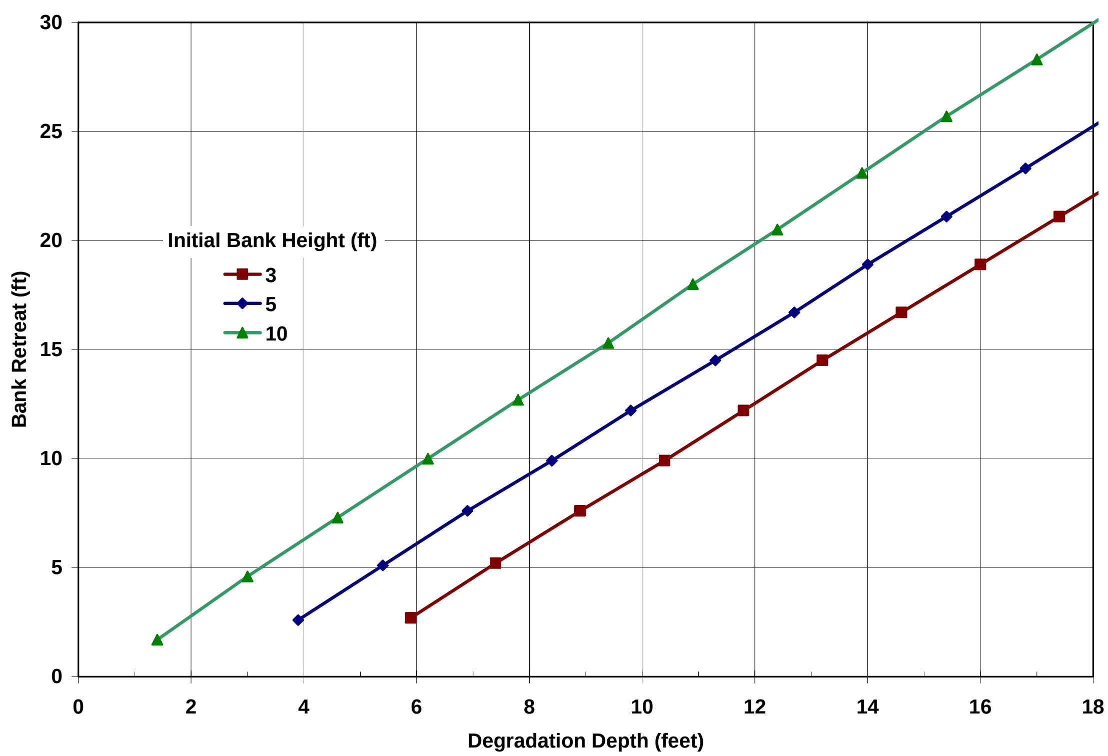


Figure 3.17a. Bank retreat versus degradation depth.

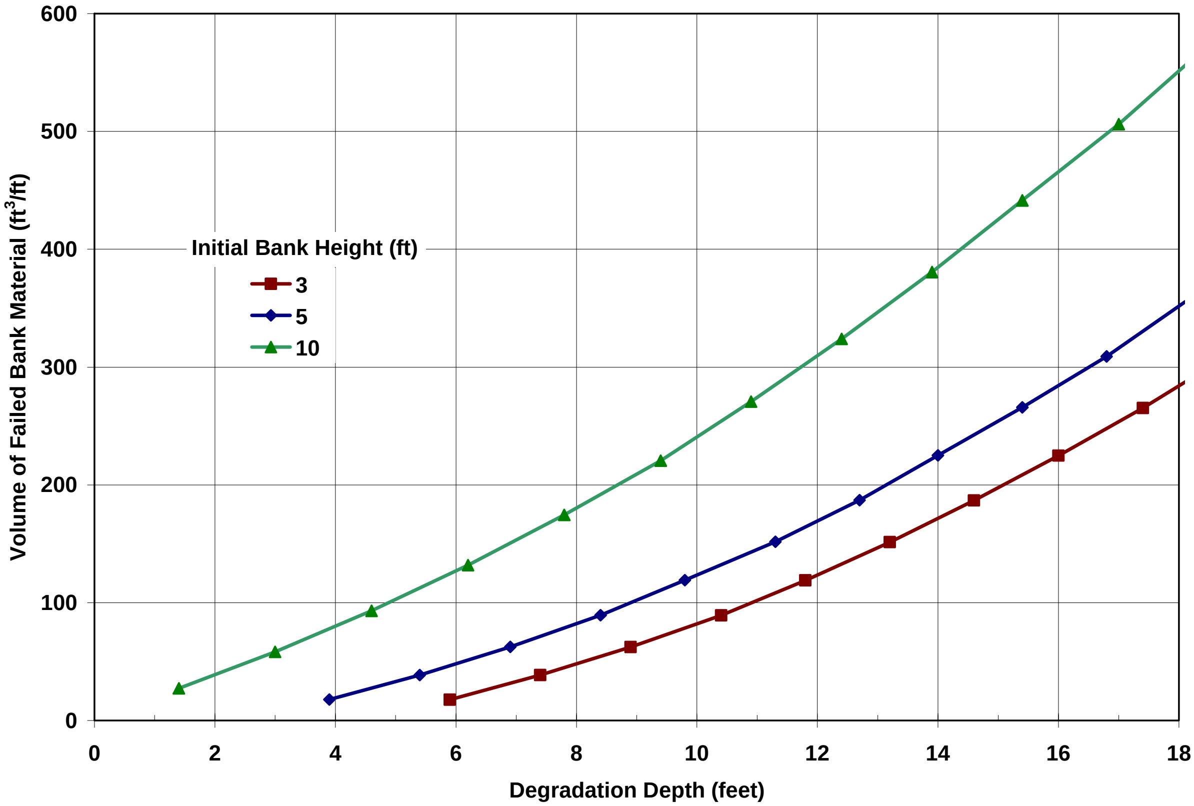


Figure 3.17b. Volume of material eroded from bank versus degradation depth.

3.4.5.3. Lateral Erosion Envelope (The LEE Line)

The processes that control lateral migration depend on the sediment balance in the reach being analyzed. If the reach is degradational, lateral migration will occur primarily through bank failure due to undercutting at the toe or exceedence of the critical bank height, and subsequent fluvial entrainment of the failed material. In vertically stable reaches, lateral migration results from either bank failure due to undercutting of the toe or localized fluvial entrainment of the material directly from the bank, primarily on the outsides of the bends. When future runoff and transport conditions are expected to be similar to historical conditions, empirical data and/or measurements from historical aerial photographs can be an effective means of estimating future migration rates. Detailed methods for applying this technique are presented in Lagasse et al. (2003). Unfortunately, this technique is not applicable to arroyos in developing watersheds where the runoff and sediment load conditions are rapidly changing. A method for estimating lateral migration rates in degrading reaches based on the relative sediment balance and anticipated bank heights, and a computer model for applying the method, were previously developed by Mussetter et al. (1994). This method is complex, there is large uncertainty in the many of the input variables, and significant judgment is also necessary in interpreting the results. In addition, lateral migration over the multiple decade time-frame of applicability for a particular erosion setback line would typically reach the maximum limit, given the size of channel and amount of channel curvature that could reasonably develop.

Based on the above considerations, erosion setback distances along arroyos in the SSCAFCA jurisdictional area that are expected to remain vertically stable or become degradational shall be established based on the maximum lateral migration distance that can be expected over the next

30 to 50 years. The resulting lines along either side of the arroyo define a corridor that is hereinafter referred to as the Lateral Erosion Envelope (LEE), and the individual lines are referred to as LEE lines. The relationships to be used in estimating the location of the LEE lines are developed in the following section.

When the reach is strongly aggradational and the overbank area is relatively flat, the channel alignment and extent of flooding is essentially random (e.g., alluvial fan flooding), and prediction of the erosion boundaries by analytical means is not possible. For this case, however, the flood limits are usually wider than the erosion limits, and they will, thus, control the boundaries of the hazard zone along the arroyo. A detailed discussion of alluvial fan flooding and the associated hazards can be found in National Research Council (NRC, 1996).

A simplified method for estimating the location of the LEE line has been developed based on observations from a large body of research that erosion rates in river bends tend to increase with increasing bend sharpness up to radius of curvature to channel width ratios (R_c/W) in the range of 2 to 4, and the rate decreases sharply at values less than 2 due to energy loss in the bend (**Figure 3.18**; Hickin, 1975; Nanson and Hickin, 1986; Begin, 1981; Odgaard, 1987; Bagnold, 1960). In fact, Carey (1969) and Page and Nanson (1982) showed that in very tight bends (i.e., $R_c/W < 2$), deposition actually occurred on the outside of the bend. Under these conditions, the rate of lateral migration essentially stops, and the bend either cuts off or avulses.

Consistent with the above observations, Leopold and Wolman (1960) and Bagnold (1960) observed that river meanders tend to a constant R_c/W of between 2 and 3. This range of bend sharpness seems to result in a minimum value of resistance to flow, with flow resistance increasing rapidly as R_c/W decreases below 2. Langbein and Leopold (1966) showed that meanders develop to minimize the variance of shear and friction through the bend, and the planform for such a meander follows the approximate shape of a sine-generated curve described by the relation:

$$\phi = \omega \sin\left(2\pi \frac{s}{M} + \frac{\pi}{2}\right) \qquad (3.38)$$

where s = distance along the channel centerline,
ϕ = angle of the channel alignment with the down-valley direction at location s,
ω = maximum angle of the meander relative to the general direction of the channel, and
M = total channel length along the meander (**Figure 3.19**).

The maximum angle (ω) is related to the sinuosity (k) by the approximate relationship:

$$\omega = 2.2\sqrt{\frac{k-1}{k}} \qquad (3.39)$$

The meander wavelength (λ) is related to the sinuosity by the relation:

$$\lambda = kM \qquad (3.40)$$

and the radius of curvature of the bend (R_c) over approximately 75 percent of the length through the main part of the bend is given by the approximate relationship:

$$R_c = \frac{\lambda}{13}\frac{k^{3/2}}{\sqrt{k-1}} \qquad (3.41)$$

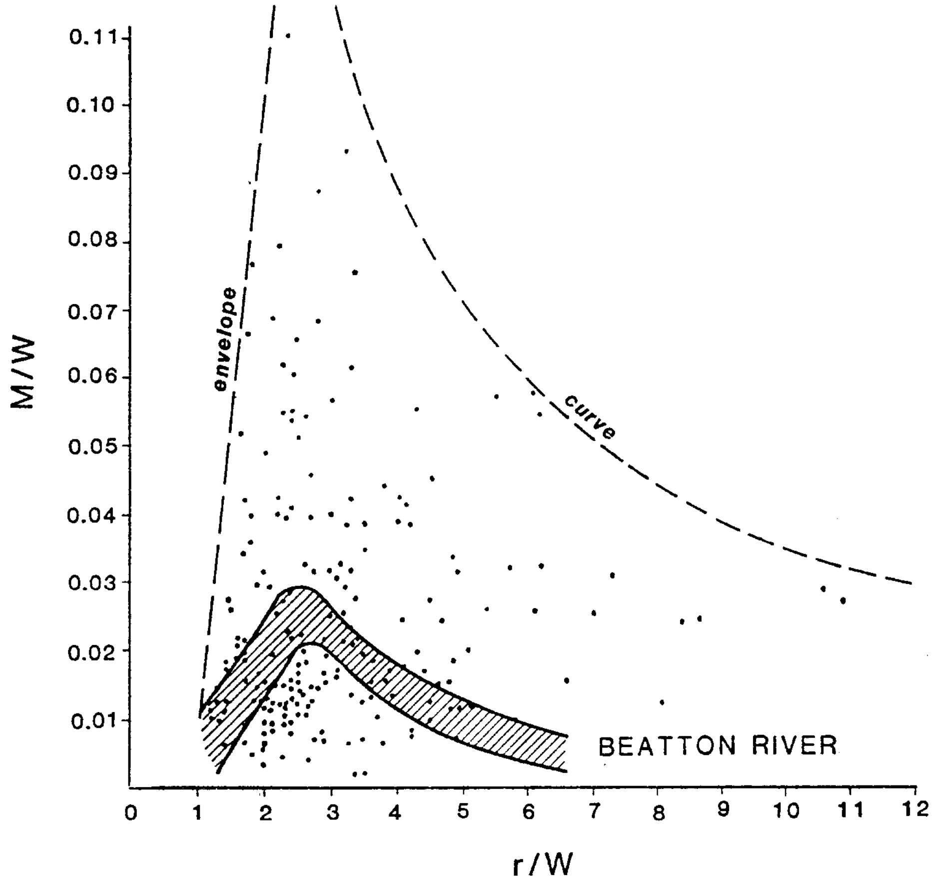


Figure 3.18. The relation between relative migration rate (expressed in channel widths per year) and bend curvature ratio (r/W) for all field sites (from Nanson and Hickin, 1986).

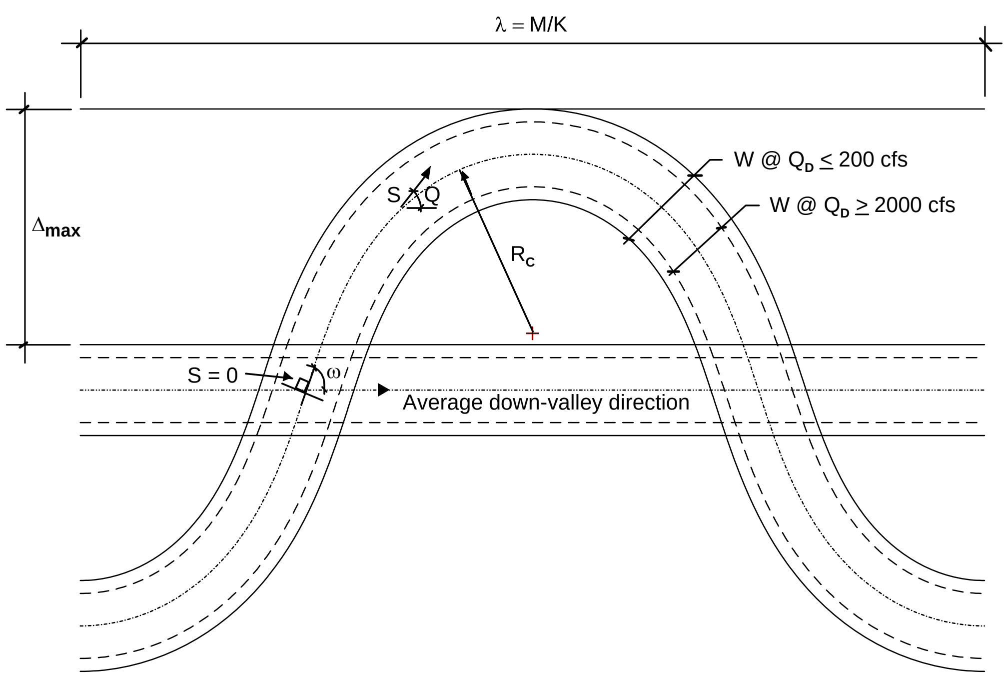


Figure 3.19. Schematic of an idealized meander bend illustrating the variables in Equations 3.38 through 3.44.

The exact radius of the curve at any point in the bend can be computed from

$$R_c = \left[\frac{2\pi\,\omega}{M} \sin\left(\frac{2\pi\, s}{M} \right) \right]^{-1} \tag{3.42}$$

From Equation 3.42, the minimum radius of curvature (R_c) occurs at S = M/4 or R_{cmin} = M/2 $\pi\omega$. Using Equation 3.41, it can be shown that the minimum value of R_c/W (i.e., maximum sharpness) for a sine generated curve occurs at a sinuosity of 1.5.

In self-adjusted channels, the length of a typical meander is typically in the range of 10 to 14 channel widths (Leopold et al., 1964). Although the original data for this relation was derived primarily from perennial streams, the relationship was also verified for arroyos (Leopold et al., 1966). Other data suggests that the ratio of meander wavelength to channel width (λ/W) tends to increase with increasing mean annual discharge Drury (1964). For purposes of this Design Guide, it is, therefore, assumed that λ/W_D in arroyos within the SSCAFCA area will vary according to the following relationships:

$\lambda/W_D=10$	for $Q_D \le 200$ cfs	(3.43a)
$\lambda/W_D=0.8+4\log(Q_D)$	for 200 cfs < Q_D <2000 cfs	(3.43b)
$\lambda/W_D=14$	for $Q_D \ge 2000$ cfs	(3.43c)

where Q_D = dominant discharge
W_D = channel width associated with the dominant discharge

The above information also indicates that the maximum deviation of a channel from a straight line (Δ_{max}) will occur when the ratio R_c/W is at its minimum (i.e., sinuosity of 1.5). The average value of R_c/W over the primary part of the bend varies from 2.0 for λ/W_D = 10 to 2.8 for λ/W = 14. For these conditions, the maximum offset of the channel from a straight line (Δ_{max}) for a sine-generated curve with minimum radius of curvature is approximately one-fourth the meander wavelength, or one-half the distance between the endpoints (crossings) for a given channel bend, thus:

Δ_{max} = 2.5w	for $Q_D \le 200$ cfs	(3.44a)
$\Delta_{max} = [0.2+\log(Q_D)]W_D$	for 200 cfs < Q_D < 2000 cfs	(3.44b)
$\Delta_{max} = 3.5\ W_D$	for $Q_D \ge 2000$ cfs	(3.44c)

The dominant discharge in perennial arroyos can be estimated as the peak discharge of the storm event that would deliver the average annual sediment load. This is accomplished using Equation 3.26:

$$Y_{sm} = .015Y_{s100} + .015Y_{s50} + .04Y_{s25} + .08Y_{s10} + .2Y_{s5} + .4Y_{s2} \tag{3.26}$$

where Y_{sm} is the mean annual sediment load and the values of Y_{si} are the sediment yields associated with each storm event. The value of the dominant discharge (Q_D) is then determined by logarithmic interpolation between the peak discharges for the appropriate storms, or:

$$\log(Q_d) = \log(Q_{R-1}) + [\log(Q_R) - \log(Q_{R-1})]\left[\frac{\log(Y_{sm}) - \log(Y_{s(i-1)})}{\log(Y_{si}) - \log(Y_{s(i-1)})} \right] \tag{3.45}$$

If only the 100-year peak discharge is available, the dominant discharge for arroyos in the SSCAFCA jurisdictional area can be estimated by

$$Q_D = 0.2Q_{100} \tag{3.46}$$

which is a reasonable approximation of the 5- to 10-year peak discharge (C.A. Anderson, 1993, personal communication).

Using a width-depth ratio (F) of 40 that is typical for arroyos and a Froude Number (F_r) of 1, the channel width can be estimated from the relation:

$$W_D = 4.6Q_D^{0.4} \tag{3.36}$$

where Q_D = dominant discharge

For subcritical flow (i.e., F_r <1) and a Manning's *n* of 0.035, the width can be estimated from the relation:

$$W_D = 2.46\, Q_D^{0.375}\, S^{-0.188} \tag{3.47}$$

Equation 3.36 should be used to approximate the width for slopes greater than or equal to the critical slope (S_c), that can be estimated from the following relation based on the above assumptions (wide rectangular channel, uniform flow, *n* = 0.35, F = 40):

$$S_c = 0.037\, Q_D^{-0.133} \tag{3.48}$$

Combining the above relationships, the maximum lateral erosion distance can be estimated, in terms of the dominant discharge (Q_D), from the following relationships, when $S \geq S_c$:

$\Delta_{max} = 11.5Q_D^{0.4}$	for $Q_D \leq 200$ cfs	(3.49a)
$\Delta_{max} = [0.92+4.61\log(Q_D)]Q_D^{0.4}$	for 200 cfs< Q_D <2000 cfs	(3.49b)
$\Delta_{max} = 16.1Q_D^{0.4}$	for $Q_D \geq 2000$ cfs	(3.49c)

or, when $S < S_c$:

$\Delta_{max} = 6.2Q_D^{0.375}S^{-0.188}$	for $Q_D \leq 200$ cfs	(3.50a)
$\Delta_{max} = [0.45+2.51\log(Q_D)]\, Q_D^{0.375}S^{-0.188}$	for 200 cfs< Q_D <2000 cfs	(3.50b)
$\Delta_{max} = 8.6Q_D^{0.375}S^{-0.188}$	for $Q_D \geq 2000$ cfs	(3.50c)

As discussed above, when only the 100-year storm peak is available, Equation 3.46 can be substituted into the above relations to provide an estimate of the maximum lateral erosion distance in terms of the 100-year peak.

In the ideal case, where the channel is assumed to be constant width (W_D) and is aligned along the average downvalley direction, the limits of the erosion envelope are defined by measuring a distance from the approximate **bankline** of the channel prior to the start of meandering. This distance is termed the bankline setback (BSB). In those cases where the bankline is difficult to define, the limits of the erosion envelope can be determined by measuring a distance $\Delta_{max}+W_D/2$ from the approximate centerline of the channel. This distance is termed the centerline setback (CSB). When both distances can be clearly defined, the most conservative (largest) distance defines the erosion setback.

Equations 3.49 and 3.50 define the expected maximum lateral erosion distance from the downvalley direction based on an idealized bend geometry. These relations are believed to provide reasonable, but somewhat conservative, estimates for most cases, particularly where the arroyo is relatively unincised or where the overbanks are relatively flat lateral to the channel. When the arroyo is deeply incised, the overbanks slope upward lateral to the channel, or controls are present, the lateral erosion potential may be reduced at any given location along the arroyo. In addition, for small arroyos (i.e., Q_{100} < ~100 cfs), the relative effects of local cementation of the overbank soils (i.e., caliche), boulders, and manmade structures become more significant. For this reason, engineering judgment, on a site-specific basis, may indicate that a more limited erosion envelope may be acceptable; particularly where the hazard associated with the relatively small discharges in smaller arroyos would be limited even if the erosion envelope were exceeded. The decision to accept a more limited erosion envelope should be made carefully, in consultation with the reviewing agencies.

The following steps summarize the procedure for estimating the maximum lateral erosion distance:

I. Estimate the magnitude of the dominant discharge (Q_D). As noted above, if only the 100-year peak discharge is available, the dominant discharge for arroyos in the SSCAFCA jurisdictional area can be estimated as $0.2Q_{100}$ (Equation 3.46).

II. Estimate the channel width associated with Q_D using Equation 3.36 if $S \geq S_c$ (i.e., supercritical flow), or from Equation 3.47 if $S < S_c$.

III. Estimate the downvalley length of arroyo (L_v) over which the lateral erosion can occur for unconstrained conditions using Equation 3.43 where $L_v = \lambda/2$.

IV. If controls that will prevent lateral erosion of the arroyo (e.g., rock outcrop, bridges, culverts, grade-control structures, spurs, etc.) are present and the controls are less than L_v apart, use the distance between the controls for L_v.

V. Compute the maximum lateral erosion distance (Δ_{max}) as 1/2 L_v if no lateral controls are present or if the spacing between the controls is greater than L_v. By constructing lateral controls at spacings less than L_v, the maximum lateral erosion distance is reduced. **Figure 3.20** shows the relationship between the maximum lateral erosion distance and the downvalley spacing of lateral controls.

VI. Determine the required erosion setback as Δ_{max} from the existing channel bank (BSB), when the bankline is clearly defined or ($\Delta_{max} + 0.5W_D$) when the bankline is not well defined (CSB).

VII. If the channel is expected to degrade such that the critical bank height will be exceeded, an additional distance corresponding to the bank retreat associated with block failure should be added to the erosion setback. The required distance can be found from Figure 3.17.

VIII. Compare the location of the erosion setback line with the location of the edge of the 100-year flood zone. The outer-most line is the required setback boundary. When the hydraulic analysis indicates supercritical flow for the conditions being analyzed, the 100-year flood zone should be defined based on the water-surface elevation associated with the sequent depth (i.e., depth to which the water would rise through a hydraulic jump) (Equation 3.22).

3.4.5.4. Guidelines for Aggradational Channels

As previously discussed, localized aggradational reaches can be expected to occur along arroyos in the SSCAFCA jurisdictional area. These locations will occur in confined reaches downstream from rapidly incising reaches, caused for example by urbanization, which significantly increases the sediment load to the downstream reach above that which occurred prior to urbanization. When significant deposition occurs, channel capacity will be reduced, deposition may cause localized bank erosion and/or channel avulsion, and overbank flooding may increase. In many cases, a multibranched channel will result. The dynamics of the arroyo under aggradational conditions become very uncertain, making analytical estimates of lateral erosion distance impractical or meaningless. The lateral migration potential of the channel is more a function of possible flow paths that may be taken by the overbank flows than that of bank erosion. For this case, the boundary along the arroyo within which it is not considered prudent to develop, is most likely controlled by the limits of flooding rather than migration of the arroyo. The flood limits may be substantially larger than indicated by fixed-bed models without consideration of aggradation.

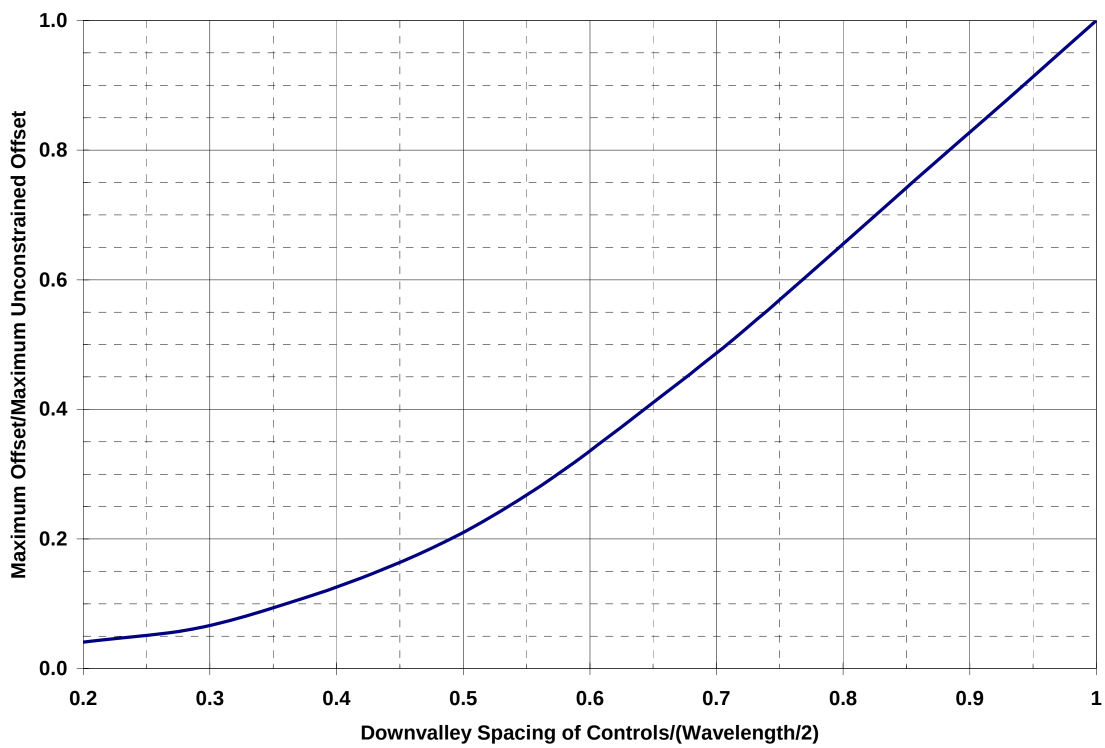


Figure 3.20. Maximum lateral erosion distance for control spaced at less than half the assumed unconstrained meander length.

3.5. Local Scour

Local scour occurs where the flow is accelerated due to obstructions in the flow and involves removal of bed and bank material from around piers, abutments, spurs, embankments, and downstream of grade-control structures or channel drops. The principal erosion mechanism is the creation of vortices by the obstruction and resultant acceleration of the flow. Like contraction scour, local scour is cyclic in nature, scouring during the rising limb with subsequent refilling during the recession of a hydrograph.

3.5.1. Pier Scour at Bridge Crossings

Vortices tend to form around the base of piers placed in the flow, due to a pileup of water on the upstream face of the pier and subsequent acceleration of flow around the pier. The action of the base vortex (otherwise known as the horseshoe vortex—see **Figure 3.21**) removes sediment from the bed of the channel near the pier base resulting in a scour hole. Vertical wake vortices form on the downstream side, which can also remove sediment from around the base of the pier (Richardson and Davis, 2001).

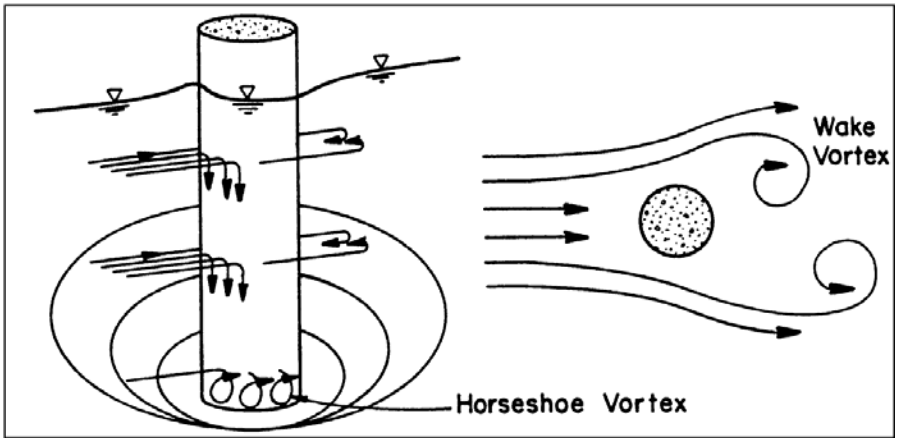


Figure 3.21. Schematic representation of scour at a cylindrical pier.

The following factors influence the size and formation of pier scour holes (Richardson and Davis, 2001):

1. Pier width has a direct influence on depth of local scour. As pier width increases, there is an increase in scour depth.

2. Pier length has no appreciable effect on local scour depth as long as the pier is aligned with the flow. When the pier is skewed to the flow, the length has a significant effect.

3. Flow depth has an effect on the depth of local scour. An increase in flow depth can cause increase scour depth by a factor of 2 or greater for piers.

4. The greater the approach velocity, the deeper the scour.

5. Angle of attack of the flow to the pier has a significant effect on local scour.

6. Shape of the nose of a pier or an abutment has a significant effect on scour. Streamlining the front end of a pier reduces the strength of the horseshoe vortex, thereby reducing scour depth. Streamlining the downstream end of piers reduces the strength of the wake vortices. A square-nose pier will have maximum scour depths about 20 percent greater than a sharp-nose pier and 10 percent greater than either a cylindrical or round-nose pier.

7. Debris can potentially increase the width of the piers, change the shape of piers and abutments, and cause the flow to plunge downward against the bed. This can increase both the local and contraction scour. The magnitude of the increase is still largely undetermined. Debris can be taken into account in the scour equations by estimating how much the debris will increase the width of a pier.

8. Bed-material characteristics such as size, gradation, and cohesion can affect local scour. Bed material in the sand-size range has no effect on local scour depth. Larger-size bed material that can be moved by the flow or by the vortices and turbulence created by the pier

will not affect the maximum scour, but only the time it takes to attain it. Very large particles in the bed material, such as cobbles or boulders, may armor the scour hole. However, the extent to which large particles will minimize scour is not clearly understood.

Fine bed material (silts and clays) will have scour depths as deep as sand-bed streams. This is true even if the fine material is bonded together by cohesion. The effect of cohesion is to influence the time it takes to reach the maximum scour. With sand-bed material, the time to reach maximum depth of scour is measured in hours and can result from a single flood event. With cohesive bed materials, it may take days, months, or even years to reach the maximum scour depth, the result of many flood events.

9. Bed configuration (i.e., bedforms) of sand-bed channels affects the magnitude of local scour. In streams with sand-bed material, the shape of the bed may be ripples, dunes, plane bed, or antidunes. The bed configuration depends on the size distribution of the sand-bed material, hydraulic characteristics, and fluid viscosity and it may change with discharge during a single flood event. The type and change of bed configuration will affect flow velocity, sediment transport, and scour.

For computing pier scour, the following variation of the Colorado State University equation is currently recommended by the Federal Highway Administration (Richardson and Davis, 2001):

$$\frac{Y_s}{Y_1} = 2.0\, K_1\, K_2 K_3\, K_4 \left(\frac{a}{Y_1}\right)^{0.65} F_{r_1}^{0.43} \tag{3.51}$$

where Y_s = equilibrium scour depth (ft),
Y_1 = flow depth just upstream of the pier (ft),
K_1 = correction for pier nose shape from **Figure 3.22** and **Table 3.7**,
K_2 = correction for angle of attack of flow from Equation 3.52 (see below) and **Table 3.8**,
K_3 = correction for bed condition (i.e., bedform) from **Table 3.9**,
K_4 = correction for armoring by bed-material size from Equation 3.53 (see below),
a = pier width (ft),
L = length of pier (ft),
F_{r1} = Froude Number = $V_1/\sqrt{gY_1}$ based on conditions just upstream of the pier, and
g = acceleration of gravity (32.2 ft/s^2).

The correction factor K_2, for angle of attack of the flow, T, is calculated using the following equation:

$$K_2 = (Cos\theta + L/a * Sin\theta)^{0.65} \tag{3.52}$$

If L/a is larger than 12, use L/a = 12 as a maximum in Equation 3.51 and Table 3.7.

Live-bed scour in sand-bed streams with a dune bed configuration fluctuates about the equilibrium scour depth, due to the variability in bed-material sediment transport in the approach flow. In this case, maximum depth of scour is about 30 percent larger than equilibrium depth of scour for large rivers. Normally, arroyos under flood-flow conditions will be in upper regime with either plane bed or antidunes. For these conditions, the maximum scour depth is about 10 percent larger than the equilibrium scour depth computed with Equation 3.51.

The maximum scour depth is the same as the equilibrium scour depth for live-bed conditions with a plane-bed configuration. With antidunes upstream and in the bridge crossing, the maximum scour depth is about 10 percent greater than the equilibrium scour depth of scour. In most cases, arroyos in the SSCFACA jurisdictional area will contain antidunes during flood flows.

Figures and Tables for Pier Scour Equation

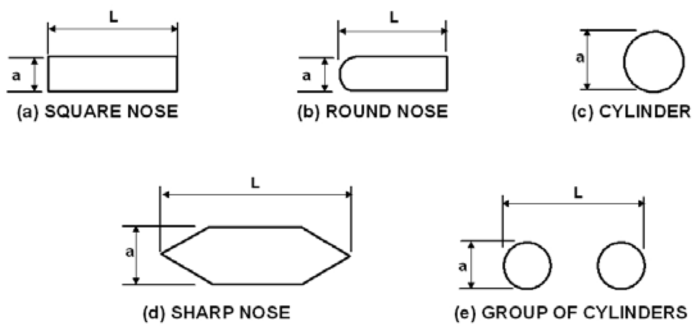


Figure 3.22. Common pier shapes.

Table 3.7. Correction Factor K_1 for pier nose shape.

	Shape of Pier Nose	K_1
a.	Square nose	1.1
b.	Round nose	1.0
c.	Circular cylinder	1.0
d.	Sharp nose	0.9
e.	Group of cylinders	1.0

Table 3.8. Correction Factor K_2 for angle of attack of the flow.

Angle	L/a=4	L/a=8	L/a=12
0	1.0	1.0	1.0
15	1.5	2.0	2.5
30	2.0	2.75	3.5
45	2.3	3.3	4.3
90	2.5	3.9	5.0

Table 3.9. Increase in equilibrium pier scour depths, K_3, for bed condition.

Bed condition	Dune Height, (ft)	K_3
Clear-water scour	N/A	1.1
Plane bed and antidune flow	N/A	1.1
Small dunes	9.8 > H >= 1.97	1.1
Medium dunes	29.5 > H >= 9.8	1.2 to 1.1
Large dunes	H >= 29.5	1.3

GENERAL NOTES

Note 1: The correction factor K_1 for pier nose shape should be determined using Table 3.7 for flow angles of attack up to 5 degrees. For greater angles, K_2 dominates, pier nose shape loses its effect, and K_1 should be considered as 1.0. If L/a is greater than 12, use the values for L/a=12 as a maximum.

Note 2: The correction factor K_2 should only be applied if the entire length of the pier is subjected to the angle of attack of the flow. Use of this factor will result in a significant over-prediction of scour if (1) a portion of the pier is shielded from the direct impingement of the flow by an abutment or another pier; or (2) an abutment or another pier redirects the flow in a direction parallel to the pier. Using judgment, the value of K_2 can be reduced by selecting an effective length of pier that is actually subjected to the angle of attack.

Note 3: The correction factor K_3 results from the fact that for plane-bed conditions, which is typical of most bridge sites for the flood frequencies employed in scour design, the maximum scour may be 10 percent greater than computed with Equation 3.48. The values for K_3 in Table 3.8 indicate that maximum pier scour can increase from between 10 to 20 percent for plane bed and antidunes (common in arroyos) to 30 percent for large dunes (in large rivers). Antidune heights can be computed using Equation 3.37.

The correction factor K_4 decreases scour depths to account for armoring of the scour hole when the median (D_{50}) size of the bed material is greater than or equal to 2.0 mm and the D_{95} greater than or equal to 20 mm. Values for correction factor K_4 should be computed with the following equation:

- If D_{50} < 2 mm or D_{95} < 20 mm, then K_4=1
- If D_{50} >= 2 mm and D_{95} >= 20mm, then:

$$K_4 = 0.4(V_R)^{0.15} \tag{3.53}$$

where:

$$V_R = \frac{V_1 - V_{icD_{50}}}{V_{cD_{50}} - V_{icD_{95}}} > 0 \tag{3.54}$$

and:

V_{icDx} = approach velocity (ft/s) required to initiate scour at the pier for the grain size D_x (ft), given by:

$$V_{icD_x} = 0.645(\frac{D_x}{a})^{0.053} V_{cD_x} \tag{3.55}$$

V_{cDx} = critical velocity (ft/s) for incipient motion for the grain size D_x (ft) given by:

$$V_{cD_x} = K_u Y_1^{1/6} D_x^{\ 1/3} \tag{3.56}$$

where Y_1 = flow depth just upstream of the pier, excluding local scour (ft),
V_1 = approach velocity just upstream of the pier (ft/s),
D_x = Grain size for which x percent of the bed material is finer (ft), and
K_u = 11.17 (English Customary units).

The minimum value of K_4 is 0.4.

Additional methods of estimating scour that account for various complex bridge crossing configurations are described in the Federal Highway Administration (FHWA) Hydraulic Engineering Circular No. 18 (Richardson and Davis, 2001). The specific conditions and calculations that are addressed with these methods include the following:

- Pier Scour Correction Factor for Very Wide Piers
- Scour for Complex Pier Foundations
- Multiple Columns Skewed to the Flow
- Pressure Flow Scour
- Scour From Debris on Piers
- Topwidth of Scour Holes

3.5.2. Scour at Grade-control Structures

Accelerating flow over the crest of grade-control and drop structures induces local scour immediately downstream of these structures, and the resulting turbulence causes the development of a local scour hole. This scour hole can cause an increase in effective bank height that can also cause lateral instability, channel widening and possible outflanking of the structure.

The "*Design Guide for Riprap-Lined Flood Control Channels*" (AMAFCA, 1983) provides information on the evaluation and control of scour at drop structures. The Veronese equation is an alternative, and commonly-used, approach to estimating scour at grade-control structures. This equation is given by:

$$d_s = K_V H_t^{0.225} q^{0.54} - d_m \tag{3.57}$$

where d_s = local scour depth for a free overfall, measured from the streambed downstream of the drop (ft) (**Figure 3.23**),
q = discharge per unit width (cfs/ft),
H_t = total drop in head, measured from the upstream to the downstream energy grade line (ft),
d_m = tailwater depth (ft), and
K_V = 1.32 (English Customary units).

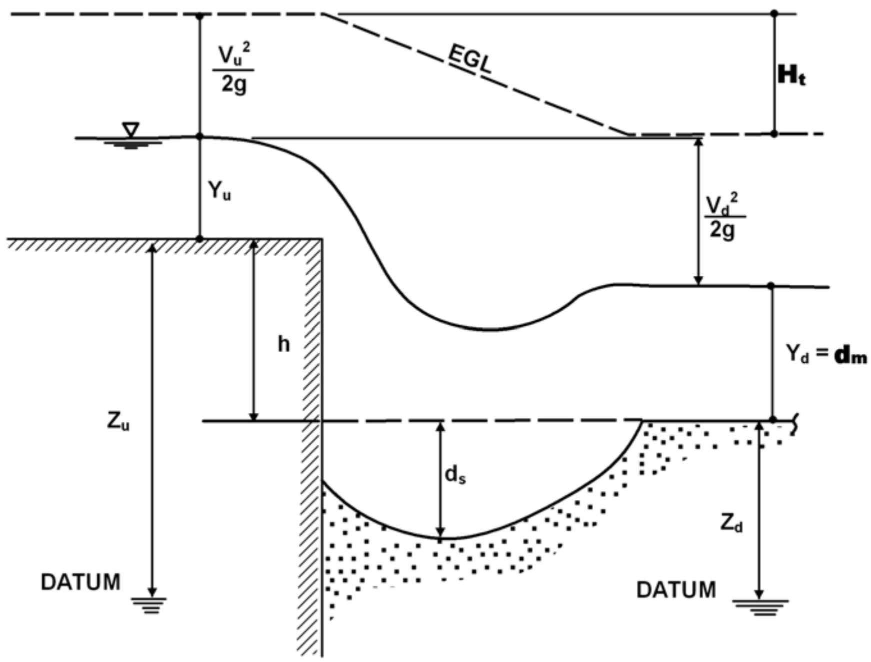


Figure 3.23. Schematic of a vertical drop caused by a check dam.

It should be noted that H_t is the difference in the total head from up- to downstream. This can be computed using the energy equation for steady uniform flow:

$$H_t = \left\{ Y_u + \frac{V_u^2}{2g} + Z_u \right\} - \left\{ Y_d + \frac{V_d^2}{2g} + Z_d \right\} \tag{3.58}$$

where Y = depth (ft),
V = velocity (ft/sec),
Z = bed elevation referenced to common datum (ft), and
g = acceleration due to gravity (32.2 ft/sec^2).

The subscripts u and d refer to up- and downstream of the channel drop, respectively.

The Veronese equation (Equation 3.57) typically provides very conservative results because it only accounts for unit discharge and difference in head across the dam assuming a vertical downstream face. Other factors that affect the magnitude of local scour include dissipation of hydraulic energy as the flow plunges over a sloping downstream face, dissipation of the jet as it passes a relatively deep tailwater flow on the downstream side of the structure, and the characteristics of the material in which the scour hole will develop.

A more complex equation that was developed using a combination of hydraulic theory, laboratory and field data that accounts for these factors were published by Bormann and Julien (1991). This equation is given by the following relationship:

$$D_s = \left\{ \left[\frac{\gamma_s \sin\phi}{2(\gamma_s - \gamma) g \sin(\phi + \beta')} \right]^{0.8} \frac{3.24 Y_0^{0.6} U_0^{1.6}}{D_s^{0.4}} \sin\beta' \right\} - d_p \tag{3.59}$$

$$\beta' = 0.316 \sin\lambda + 0.15 \ln\left(\frac{D_p + Y_0}{Y_0}\right) + 0.13 \ln\left(\frac{Y_D}{Y_0}\right) - 0.05 \ln\left(\frac{U_0}{\sqrt{gY_0}}\right) \tag{3.60}$$

where γ_s = unbulked unit weight of the sediment,
γ = unit weight of water,
ϕ = submerged angle of repose of the sediment (assumed to 25.5º),
Y_0 = thickness of the jet as it impinges into the downstream tailwater,
U_0 = velocity of the jet,
Y_D = tailwater depth with respect to the original bed elevation,
d_p = height of the embankment crest above the original bed,
D_s = characteristic sediment size, and
λ_S = angle of the emergency spillway with respect to horizontal.

It is important to note that a particular drop structure must be designed from a geotechnical and structural perspective to withstand the forces of water and soil assuming the full depth of the potential scour hole predicted by the above equations. In some cases, a series of drops may be employed to minimize drop height and construction costs of foundations, or riprap or energy dissipation could be provided to limit the depth of scour.

3.5.3. Scour at Revetments, Spurs, and Abutments

Local scour must be considered at other obstructions to flow such as revetments, floodwalls, spurs, guidebanks, or bridge abutments . Local scour occurs at the nose of spurs, at the base and ends of revetments and at bridge abutments when either the bridge encroaches into the channel or when overbank flow is forced back into the main channel at the bridge opening. Richardson and Davis (2001) provides guidance for estimating local scour at these types of hydraulic structures.

Field data for scour at abutments and similar structures such as a floodwall for various size streams are scarce, but data collected at rock dikes on the Mississippi River indicate the equilibrium scour depth for large a/Y_1 values can be conservatively estimated by the following equation:

$$\frac{Y_S}{Y_1} = 4\, F_{r1}^{0.33} \tag{3.61}$$

where Y_s = equilibrium depth of scour (measured from the mean bed level to the bottom of the scour hole),
Y_1 = average upstream flow depth in the main channel,
a = abutment and embankment or wall length projecting into main channel, and
F_{r1} = upstream Froude Number.

3.5.4. Scour along a Floodwall

Scour tends to occur along flood walls due to a combination of increased shear stress caused by locally high velocities along the smooth face of the wall and flow impinging directly into the wall.

3.5.4.1. Flow Parallel to the Wall

When the flow runs parallel to a wall that is smoother than the channel bed, the local velocities and boundary shear stresses along the wall tend to be higher than occurs a short distance from the wall due to the reduced roughness. The increased shear stress will, in turn, induce scour until the local flow area has increased sufficiently to reduce the local velocity shear stress to values typical of the rest of the channel cross section.

The distribution of boundary shear stress around the perimeter of a channel is not constant. The distribution in channels of uniform roughness has been quantified both analytically (Olson and Florey, 1952; and Replogle and Chow, 1966) and experimentally (Ippen and Drinker, 1962; Davidian and Cahal, 1963; Rajaratnam and Muralidhar, 1969, Kartha and Leutheusser, 1970; and Schall, 1979). These studies indicate that, in trapezoidal channels, the maximum boundary shear stress occurs near the channel centerline, and a secondary peak occurs about one-third of the distance up the sideslope. On average, the maximum on the bottom is about 0.97 times the average boundary shear stress for the cross section (e.g., as defined by γRS), and the secondary maximum on the sideslope is about 0.76 times the average. The experimental data, however, indicated a range of values, with maximum shear stresses as much as 1.6 times the average. In general, the boundary shear stress distribution is more uniform as the width to depth ratio increases.

Similar information is not available for channel cross sections of nonuniform roughness; however, reasonable conclusions can be drawn from intuitive arguments. For a straight channel with a floodwall that provides less roughness than occurs along the remainder of the channel, the higher shear stresses will tend to be skewed towards the floodwall. In this case, the peak value on the sideslope would be larger, and possibly greater than the peak along the channel bed, which would also be shifted off the centerline toward the floodwall. These effects are more pronounced in narrow channels and/or channels with steep sideslopes. As the channel widens or the sideslope flattens, these effects would diminish.

A first approximation for the potential scour depth along a smooth floodwall due to flow parallel to the wall can be estimated from the following equation (Mussetter et al., 1994):

$$\frac{Y_s}{Y_1} = 0.73 + 0.14\pi F_r^2 \qquad (3.62)$$

The first term in this approximation is based on the observation that the smooth floodwall will increase the conveyance in the vicinity of the wall, which will cause the channel bed to scour so that the local shear stress will be in balance with that in the rougher supply areas immediately upstream. The second term accounts for the effects of antidune scour in the vicinity of the wall.

This equation is applicable only where parallel flow can be assured (e.g., floodwalls along both arroyo banks).

3.5.4.2. Flow Impinging on the Wall at an Angle

As discussed in the previous section, when an obstruction such as an abutment projects into the flow, the depth of scour at the nose of the obstruction can be estimated from Equation 3.61. Considering the physical configuration of the channels from which the data used to develop this equation were derived, scour depths from Equation 3.61 probably represent an upper limit of the scour that could be expected for flow along a floodwall when the flow impinges on the wall at an approximately 90º angle. For cases when the flow impinges on the floodwall at an angle less than 90º, the scour depth will vary as a proportion of that given by Equations 3.61 and 3.62. If it is assumed that the relative significance of the two scour mechanisms is proportional to the change in momentum associated with the change in flow direction from the impingement angle to a direction parallel to the wall (see **Figure 3.24**), the two relations can be combined using a weighting factor based on the sine or cosine of the angle, respectively. The resulting relationship is given by:

$$\frac{Y_s}{Y_1} = \left(0.73 + 0.14\pi F_r^2\right)\cos\theta + 4F_r^{0.33}\ \sin\theta \tag{3.63}$$

where θ = angle between the flow direction and the floodwall.

3.5.4.3. Scour along a Floodwall in Relation to Unconstrained Valley Width

The potential scour that could occur along a floodwall due to changes in planform as the arroyo evolves can be estimated by combining Equation 3.63 with the relationships for ideal meander geometry discussed in Section 3.4. Using these relationships, it can be shown that the maximum angle will vary from zero when the width of the valley is constrained to the width of the arroyo to approximately 71º when the unconstrained valley width is approximately 3.5 times the width of the arroyo. (These values are based on the assumption that the meander wavelength is 14 times the channel width). It is, of course, possible for the channel to impinge perpendicularly to the wall due to local flow deflection or other local factors. For this case, the angle of impingement is no longer related to the valley width and the best estimate of the maximum scour depth can be obtained using Equation 3.61. The dimensionless scour depth as a function of the unconstrained valley width predicted by Equation 3.63, based on the angle between the sine-generated curve and the downvalley direction (see Figure 3.19), is shown in **Figure 3.25** for a range of Froude Numbers (F_r).

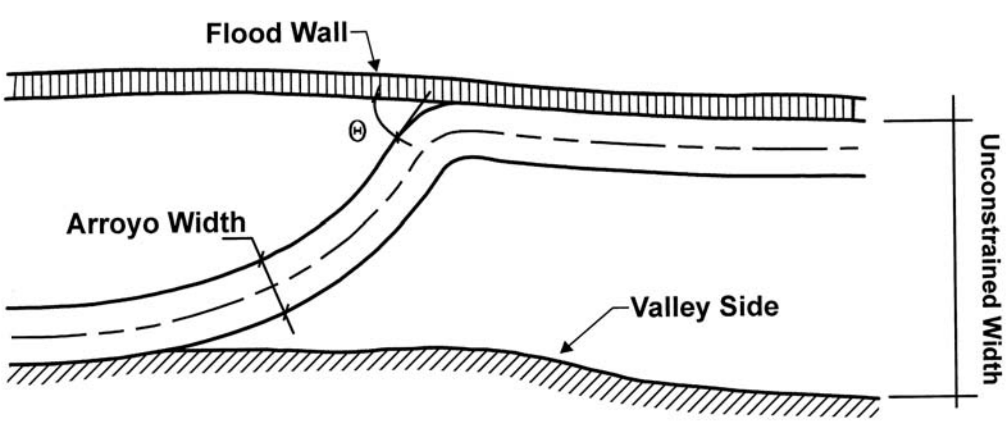


Figure 3.24. Schematic of channel alignment associated with a floodwall.

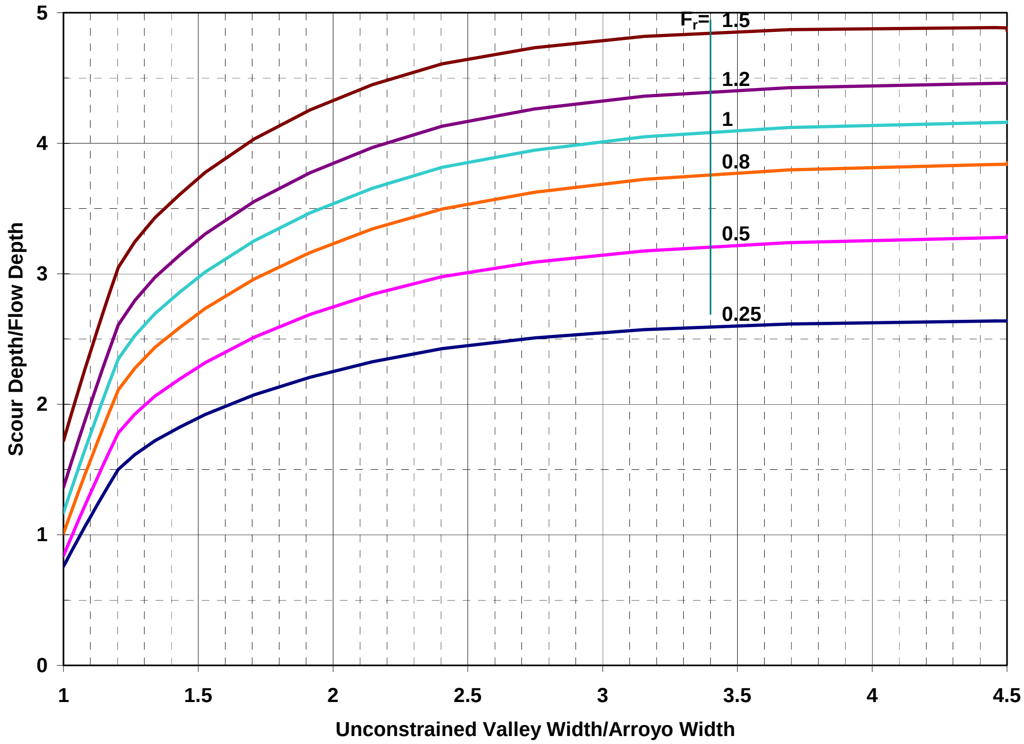


Figure 3.25. Scour along a floodwall as a function of unconstrained valley width.

In using Figure 3.25, it is important to recognize that the relationships are based on an assumed ideal meander geometry and scour relationships that, while they are the best available, are very approximate. Considering the extreme local variability that can occur in a given arroyo, and the approximate nature of the relationships upon which these results are based, engineering judgment is critical in evaluating the reasonableness of the results for a specific problem. In particular, the potential for flow deflection and its effect on the angle of impingement on the wall should be considered, and a conservatively large angle applied in Equation 3.63. If there is any reasonable possibility of flow perpendicular to the wall, an impingement angle of 90º (thus, Equation 3.61) should be assumed.

Equation 3.63 and Figure 3.25 can be used in arroyo applications where lateral migration and bank erosion must occur before channel flows impinge on the flood wall. Lateral erosion can occur rapidly, including the during the early portion of major storm events, changing the cross-sectional geometry of the arroyo. For this case, it is reasonable to use the hydraulic depth (area/topwidth) in estimating the potential scour depth. An estimate of the hydraulic depth may be obtained by using the dominant arroyo width (W_D) and Manning's equation. The scour depth should be measured from channel thalweg, accounting for the potential effects of future degradation (**Figure 3.26**).

3.5.5. Local Scour at Culvert Outlets

Where the bed and banks are not protected on the downstream side of a culvert, high-velocity flow at the culvert outlet will create a local scour hole that could damage or fail the crossing. In many instances, energy dissipation structures are required to prevent excessive erosion in these locations. In assessing the need for protection, the size and depth of the potential local scour hole under unprotected conditions can be estimated using equations presented in "*Hydraulic Design of Energy Dissipators for Culverts and Channels*" (FHWA, 1983). Since most soils in the SSCAFCA jurisdictional area are noncohesive, only the equation applicable to these conditions is presented here. The equation for scour-hole geometry is given by:

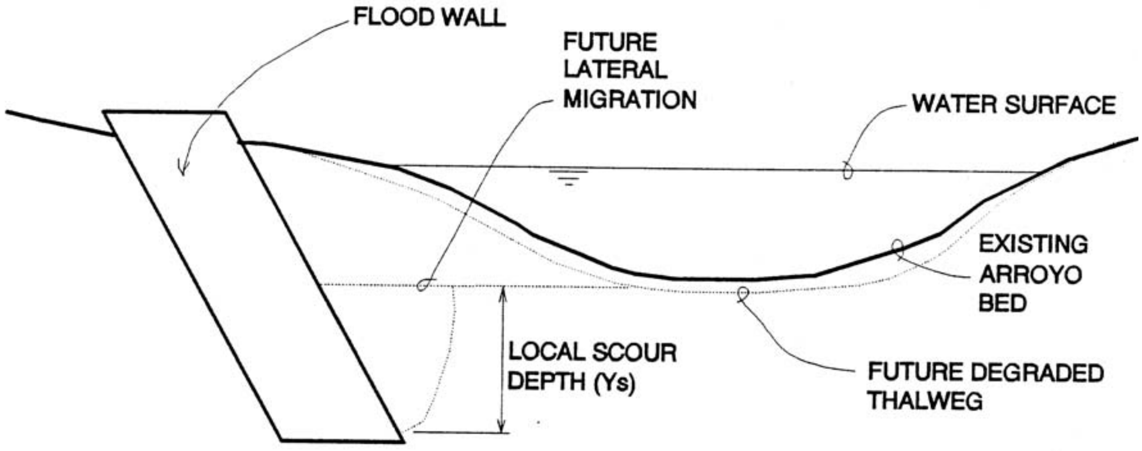


Figure 3.26. Schematic of floodwall scour.

$$A^* = \alpha \left(\frac{Q}{\sqrt{g} D^{5/2}} \right)^{\beta} \left(\frac{t}{t_o} \right)^{\theta} \tag{3.64}$$

where A^* = the dimensionless scour geometry,(h_s/D, W_s/D, L_s/D, or V_s/D^3) and (h_s/y_e, W_s/y_e, L_s/y_e, or V_s/y_e^3),
Q = discharge,
g = acceleration of gravity,
D = culvert diameter,
t = duration of the flow, and
t_0 = 316 minutes (the base time used in the experiments upon which the relations are based).

For noncircular culverts or culverts flowing partially full, the diameter (D) is replaced by the equivalent depth (y_e) defined by:

$$y_e = \sqrt{\frac{A}{2}} \tag{3.65}$$

where A = cross-sectional area of the flow, and the coefficient α is replaced by α_e given by the following relation:

$$\alpha_e = \alpha \quad 0.63^{(2.5\beta - 1)} \tag{3.66}$$

for computing h_s, W_s, and L_s, and by the relation:

$$\alpha_e = \alpha \quad 0.63^{(2.5\beta - 3)} \tag{3.67}$$

for computing V_s. The coefficient (α) and exponents (β and θ) for each parameter and a range of bed material types are summarized in **Table 3.10**. This formulation for scour depth assumes that the elevation of the culvert outlet is at the elevation of the downstream channel. If the culvert outlet is above the channel, or if the downstream channel is lowered by degradation, additional scour will result and the drop structure equations (Equations 3.57 and 3.59) should be used to estimate the scour depth.

Gradation. The cohesionless bed materials presented in Table 3.10 are categorized as either uniform (U) or graded (G), based on the standard deviation of the grain size distribution (σ) given by:

$$\sigma = \left(D_{84} / D_{16}\right)^{1/2} \tag{3.68}$$

If $\sigma \leq 1.5$, the material is considered to be uniform; if $\sigma \geq 1.5$, the material is classified as graded.

Time of Scour. The time of scour is based on the duration of the peak flow. If this duration is not known, a minimum time of 30 minutes is recommended. Tests indicate that approximately two-thirds to three-fourths of the maximum scour occur in the first 30 minutes of the flow.

Headwalls. Installation of a headwall flush with the culvert outlet will move the scour hole downstream. However, the magnitude of the scour and the scour-hole geometry remain essentially the same as for the case without the headwall. If the culvert is installed with a headwall, the headwall should be designed for stability at the maximum depth of scour.

Table 3.10. Experimental coefficients for culvert outlet scour.

Material	Nominal Grain Size D_{50} (mm)	Scour Equation	Depth h_s				Width W_s				Length L_s				Volume V_s			
			α	β	Θ	α_e	α	β	Θ	α_e	α	β	Θ	α_e	α	β	Θ	α_e
Uniform Sand	0.20	V-1 or V-2	2.72	.375	0.10	2.79	11.73	0.92	0.15	6.44	16.82	0.71	0.125	11.75	203.36	2.0	0.375	80.71
Uniform Sand	2.0	V-1 or V-2	1.86	0.45	0.09	1.76	8.44	0.57	0.06	6.94	18.28	0.51	0.17	16.10	101.48	1.41	0.34	79.62
Graded Sand	2.0	V-1 or V-2	1.22	0.85	0.07	0.75	7.25	0.76	0.06	4.78	12.77	0.41	0.04	12.62	36.17	2.09	0.19	12.94
Uniform Gravel	8.0	V-1 or V-2	1.78	0.45	0.04	1.68	9.13	0.62	0.08	7.08	14.36	0.95	0.12	7.61	65.91	1.86	0.19	12.15
Graded Gravel	8.0	V-1 or V-2	1.49	0.50	0.03	1.33	8.76	0.89	0.10	4.97	13.09	0.62	0.07	10.15	42.31	2.28	0.17	32.82

V-1. FOR CIRCULAR CULVERTS. Cohesionless material or the 0.15-mm cohesive sandy clay.

$$\left[\frac{h_s}{D}, \frac{W_s}{D}, \frac{L_s}{D}, or\ \frac{V_s}{D^s}\right] = \alpha\left(\frac{Q}{\sqrt{g}D^{5/2}}\right)^{\beta}\left(\frac{t}{t_o}\right)\theta$$

where t_o = 316 min. (3.69a)

V-2. FOR OTHER CULVERT SHAPES. Same material as above.

$$\left[\frac{h_S}{y_e}, \frac{W_S}{y_e}, \frac{L_S}{y_e}, or\ \frac{V_S}{y_e^3}\right] = \alpha_e\left(\frac{Q}{\sqrt{t}y_e^{5/2}}\right)^{\beta}\left(\frac{t}{t_o}\right)^{\theta}$$

where t_o = 316 min. (3.69b)

4. ANALYSIS PROCEDURES

4.1. General Solution Procedure

This chapter presents guidelines and recommended solution procedures for evaluating arroyo stability and establishing the location of the LEE line. The procedure generally progresses from a qualitative analysis of existing and future conditions (Level 1) to a more detailed quantitative analysis using basic engineering calculation techniques (Level 2). In more complex situations where the consequences of failure are particularly significant, mathematical and/or physical modeling may be required (Level 3). (A Level 3 analysis requires highly specialized knowledge and/or equipment, and is beyond the scope of this Design Guide.) The following discussion provides an organized framework with which to tie the individual components of the analysis together to assess the vertical and lateral stability of a particular reach of arroyo, including establishing an appropriate erosion buffer zone. The solution procedure incorporates the analytical and computational techniques presented in Chapters 2 and 3. Information is also included to assist the user in determining when a more detailed, Level 3 analysis, may be appropriate.

The analysis of any complex problem should begin with an overview or general evaluation, including a qualitative assessment of the problem and potential solutions. This important initial step should provide insight and understanding of significant physical processes that are affecting the behavior of the arroyo. The understanding gained from such a qualitative assessment ensures that subsequent detailed analyses are properly designed.

The progression to a more detailed analysis should begin with application of basic principles, followed as required, by more complex solution techniques. This solution approach, beginning with qualitative analysis, proceeding through basic quantitative principles and then, if appropriate, using more complex or state-of-the-art solution procedures such as mathematical or physical modeling ensures that accurate and reasonable results are obtained while minimizing the expenditure of time and effort.

The 3-Level Analysis Approach

The inherent complexities of arroyo and drainageway stability in arid regions such as those in the SSCAFCA area requires a systematic approach to evaluating arroyo stability, potential long-term evolution in response to both human-induced and natural processes, and methods of mitigating adverse consequences of the evolution. In general, the procedure for assessing channel stability, establishing the location of the LEE Line and identifying appropriate stabilization measures, where necessary, involves up to three levels of analysis, as follows:

- Level 1: Application of Geomorphic Concepts and Other Qualitative Analysis Principals
- Level 2: Application of Basic Hydrologic, Hydraulic and Sediment Transport Engineering Relationships and Concepts
- Level 3: Application of Mathematical or Physical Modeling Studies

4.2. Data Requirements

The types and detail of data required to analyze a sediment area or channel-stability problem are highly dependent on the relative instability of the stream and the depth of study required to obtain adequate resolution of potential problems. More detailed data are needed where quantitative analyses are necessary, and data from an extensive reach of stream may be required to resolve problems in complex and high risk situations.

4.2.1. Level 1: Geomorphic and Other Qualitative Analyses

The data required for preliminary stability analyses include the following items:

- Historical and current maps and aerial photographs,
- Notes and photographs from field inspections,
- Historic channel topography data, including longitudinal profiles and cross sections,
- Bed and bank material characteristics, and historic temporal changes in these characteristics,
- Information on human activities, and
- Temporal changes in stream hydrology and hydraulics.

Historical and current maps of various types, including area, vicinity, site, geologic, soils, and land use maps, each provide essential information for placing the study reach or site into the context of the larger-scale reach of the arroyo. Area maps are needed to locate unstable reaches relative to the site, because unstable up- or downstream reaches can cause instability at the site. Vicinity maps help to identify more localized problems, and should include a sufficient reach of channel to permit identification of geomorphic characteristics of the arroyo or drainageway, and to locate bars, braids, and channel controls. Site maps are needed to determine factors that influence local stability and flow alignment, such as bars and tributaries. Geologic maps provide information on the source material for the alluvium and the presence of controls that will affect the dynamic behavior of the channel such as rock outcrops. Soils and land-use maps provide information on soil types, vegetative cover, and land use that affect the character and availability of sediment supply. Historical aerial photographs record certain ground detail more clearly than maps, and are frequently available at 5- to 10-year intervals, permitting direct measurement of the rate of bend migration, bankline failure, and other channel changes.

Notes and photographs from field inspections are important to gaining an understanding of channel stability problems, particularly local stability. Field inspections should be made during high- and low-flow periods to record the location of bank cutting or slumping and deposition in the channel. Flow directions should be sketched, signs of aggradation or degradation noted, properties of bed and bank materials estimated or measured, and the locations and implications of impacting activities recorded.

Historic longitudinal and cross-section profile data provide a basis for assessing channel stability. Stage trends at gaging stations and comparison of pre- and post-structure bed elevations can provide information on changes in the channel profile. As-built bridge data and cross sections are also frequently useful for this purpose; however, structure-induced local scour should be taken into consideration when such comparisons are made.

Man's activities in a watershed are frequently the cause of channel instability. Information on urbanization, land clearing, sediment removal or placement in arroyo channels, other forms of channelization, bend cutoffs, instream sand-and-gravel mining, dam construction, reservoir operations, and other activities, either existing or planned, are important to evaluate potential impacts to channel stability.

4.2.2. Level 2: Basic Engineering Analyses

Data requirements for basic hydrologic, hydraulic and sediment-transport engineering analysis depend on the types of analysis that must be completed. Hydrologic data needs include flood-flow frequency curves, flood hydrographs, and flow-duration curves. Since long-term stream gages that can provide such information are not available in most, if not all, of the arroyos in the SSCAFCA jurisdictional area, this information is typically developed on a recurrence-interval storm basis using rainfall runoff models. Data required for these models include the mapping discussed in the previous section, as well as rainfall depths and the temporal pattern of typical design storms. Hydraulic data needs include cross sections, channel and bank roughness estimates, channel alignment, and other data for applying step-backwater models such as HEC-RAS, or computing channel hydraulics using other methods. Analysis of basic sediment-transport conditions requires information on land use, soils, and geologic conditions, sediment sizes in the watershed and channel, and measured sediment-transport rates, where available.

4.2.3. Level 3: Mathematical and Physical Model Studies

Application of mathematical and physical model studies requires the same basic data as a Level 2 analysis, but typically in much greater detail. For example, one- and multi-dimensional hydraulic computer models, sediment routing computer models, and physical models all require detailed channel topographic data. The more extensive data requirements for either mathematical or physical model studies, combined with the additional level of effort needed to complete such studies, can be relatively time-consuming and expensive. Due to the uncertainty in the input data and model results, and the inability to represent all aspects of the physical system in a single model, proper interpretation of the results from this type of modeling requires specialized experience.

4.2.4. Data Sources

Preliminary stability data may be available from government agencies such as the USACE, U.S. Geological Survey (USGS), Natural Resource Conservation Service, and other local agencies (**Table 4.1**)

4.3. Level 1: Geomorphic Concepts and Other Quantitative Analysis

A flow chart of the typical steps in a qualitative analyses is provided in **Figure 4.1**. The six steps identified in the figure are generally applicable to most arroyo or drainageway stability problems. These steps are discussed in more detail in the following paragraphs. As shown on Figure 4.1, the qualitative evaluation leads to a conclusion regarding the need for more detailed (Level 2) analysis, or a decision to proceed directly to establishing the LEE Line and/or design of countermeasures (erosion barriers) without further detailed analysis. Guidelines and criteria for selection and design of countermeasures are discussed in Chapter 5.

4.3.1. Step 1: Define Channel Characteristics

The first step in a channel stability analysis is to identify arroyo or drainageway characteristics according to the factors discussed in Chapter 3, Section 3.1, Arroyo Geomorphology. Understanding these characteristics provides insight into current and past channel behavior,

Table 4.1. List of data sources (after Richardson et al., 1990).

Table 4.1. List of data sources (after Richardson et al., 1990).
Topographic Maps: (1) U.S. Geologic Survey 7½" Quadrangle maps. (2) Digital elevation (or digital terrain) models and related, higher resolution topographic maps. (3) Sandoval County GIS mapping. (4) Mapping from local project plans
Planimetric Maps: (1) Public survey plats - U.S. Department of the Interior, U.S. Bureau of Land Management (2) County maps - State Highway Agency. (3) City plats - city or county recorder.
Aerial Photographs: (1) Current color and multi-spectral aerial photographs of the SSCAFCA area. (2) Historic aerial photography, available from the University of New Mexico, Earth Data Analysis Department and the USDA-FSA Aerial Photography Field Office. (3) Sandoval County GIS mapping
Transportation Maps: (1) State highway agency.
Geologic Maps: (1) U.S. Department of the Interior, U.S. Geological Survey, Geologic Division; and New Mexico Bureau of Mines and Mineral Resources at the New Mexico Institute of Mining and Technology. (Note - some regular quadrangle maps also show geological data).
Soils Data: (1) County soil survey reports - U.S. Department of Agriculture, Soil Conservation Service. (2) Land use capability surveys - U.S. Department of Agriculture, Soil Conservation Service. (3) Land classification reports - U.S. Department of the Interior, U.S. Bureau of Reclamation. (4) Hydraulic laboratory reports - U.S. Department of the Interior, U.S. Bureau of Reclamation.
Climatological Data: (1) National Weather Service Data Center. (2) Hydrologic bulletin - U.S. Department of Commerce, National Oceanic and Atmospheric Administration. (3) Technical papers - U.S. Department of Commerce, National Oceanic and Atmospheric Administration. (4) Hydrometeorological reports - U.S. Department of Commerce, National Oceanic and Atmospheric Administration; and USACE. (5) Cooperative study reports - U.S. Department of Commerce, Oceanic and Atmospheric Administration; and U.S. Department of the Interior, U.S. Bureau of Reclamation.

Table 4.1. List of data sources (after Richardson et al., 1990).

Quality of Water Reports: (1) Water supply papers - U.S. Department of the Interior, U.S. Geological Survey, Quality of Water Branch. (2) Reports - U.S. Department of Health, Education, and Welfare, Public Health Service. (3) Reports - state public health departments. (4) Water resources publications - U.S. Department of the Interior, U.S. Bureau of Reclamation. (5) Environmental Protection Agency, regional offices. (6) State water quality agency.

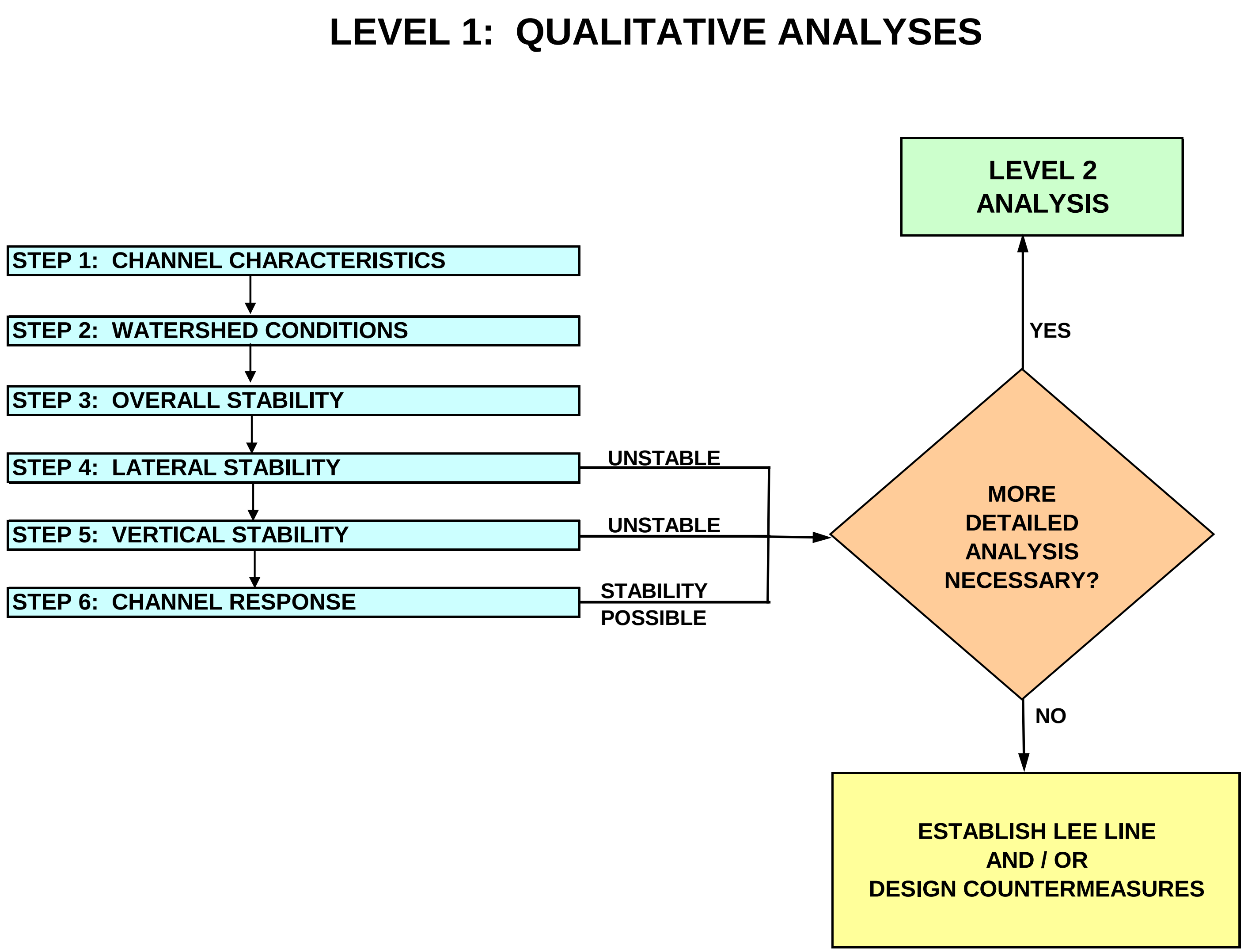


Figure 4.1. Level 1: Qualitative Analyses.

anticipated future channel response, and the natural and man-induced factors that affect channel behavior.

4.3.2. Step 2: Evaluate Watershed Conditions

The water and sediment yield from a particular watershed is a function of natural topographic, soils, and vegetation conditions, and can be strongly influenced by past or proposed land-use practices. As a result, knowledge of the current land-use and historical changes in land use is essential to understanding the conditions that will affect channel stability and potential channel response to natural and man-induced changes.

The presence or absence of vegetative growth can have a significant influence on the runoff and erosional response of a fluvial system. Large-scale changes in vegetation resulting from fire, grazing, land conversion, and urbanization can either increase or decrease the total water and sediment yield. For example, fire and grazing tend to increase water and sediment yield, while urbanization causes the total water yield and peak flows to increase, but tends to decrease sediment yield. Under some conditions, urbanization may cause temporary, but significant, increases in sediment yield.

Information on land-use history and trends can be found in federal, state, and local government documents and reports (i.e., census information, zoning maps, future development plans, etc.). Analysis of aerial photographs can also provide significant insight into historic land-use changes. Land-use change due to urbanization is typically classified based on changes in pervious and impervious cover. The relationship between changes in channel stability and land-use changes can contribute to a qualitative understanding of system response mechanisms.

4.3.3. Step 3: Assess Overall Stream Stability

Table 4.2 summarizes possible channel stability interpretations according to various channel characteristics. **Figure 4.2** is also useful in making a qualitative assessment of channel stability based on channel planform characteristics and the type of sediment load in the channel. This figure shows that straight channels are relatively stable only where flow velocities and sediment load are low. As velocity and sediment load increase, the channel typically transitions to a sinuous planform. Meandering channels become progressively less stable with increasing velocity and bed load. Channels that carry high sediment loads are typically braided.

Bed material transport is directly related to stream power, and relative stability typically decreases with increasing stream power. Stream power is the product of shear stress at the bed and the average velocity in the channel section. As discussed in Section 3.2.2, the shear stress can be estimated for a given set of flow conditions as the product γRS, where γ is the unit weight of water, R is the hydraulic radius, and S is the slope of the energy grade line.

Section 3.1 presents a more detailed discussion of the evolutionary process of arroyo development. The stability diagram in Figure 3.1 is a useful tool for evaluating the relative stability of a given reach of arroyo. In general, an arroyo must be in the lower left quadrant of Figure 3.1 to be considered stable. This implies that the slope and width have adjusted so that the reach is either in equilibrium or mildly aggradational (i.e., $N_h<1$) over a long period of time. It also implies that the banks do not exceed the critical height (i.e., $N_g<1$). It is important to note that, even in this condition, local bank erosion can lead to channel migration in specific areas.

Table 4.2. Interpretation of observed data (after Keefer et al., 1980).

Observed Condition	Channel Response			
	Stable	Unstable	Degrading	Aggrading
Alluvial Fan[1]				
Upstream		•		•
Downstream			•	
Dam and Reservoir[1]				
Upstream		•		•
Downstream		•	•	
Bank Erosion		•	Unknown	Unknown
Vegetated Banks	•		Unknown	Unknown
Head Cuts		•	•	
Diversion				
Clear-water diversion		•		•
Overloaded w/sediment		•	•	
Channel Straightened		•	•	
Clear Watershed		•		
Drought Period	•			•
Wet Period		•	•	
Bed-material Size				
Increase		•		•
Decrease		•	Unknown	•

[1]The observed condition refers to location of the study reach on the alluvial fan, i.e., on the up- or downstream portion of the fan or up- or downstream of a dam.

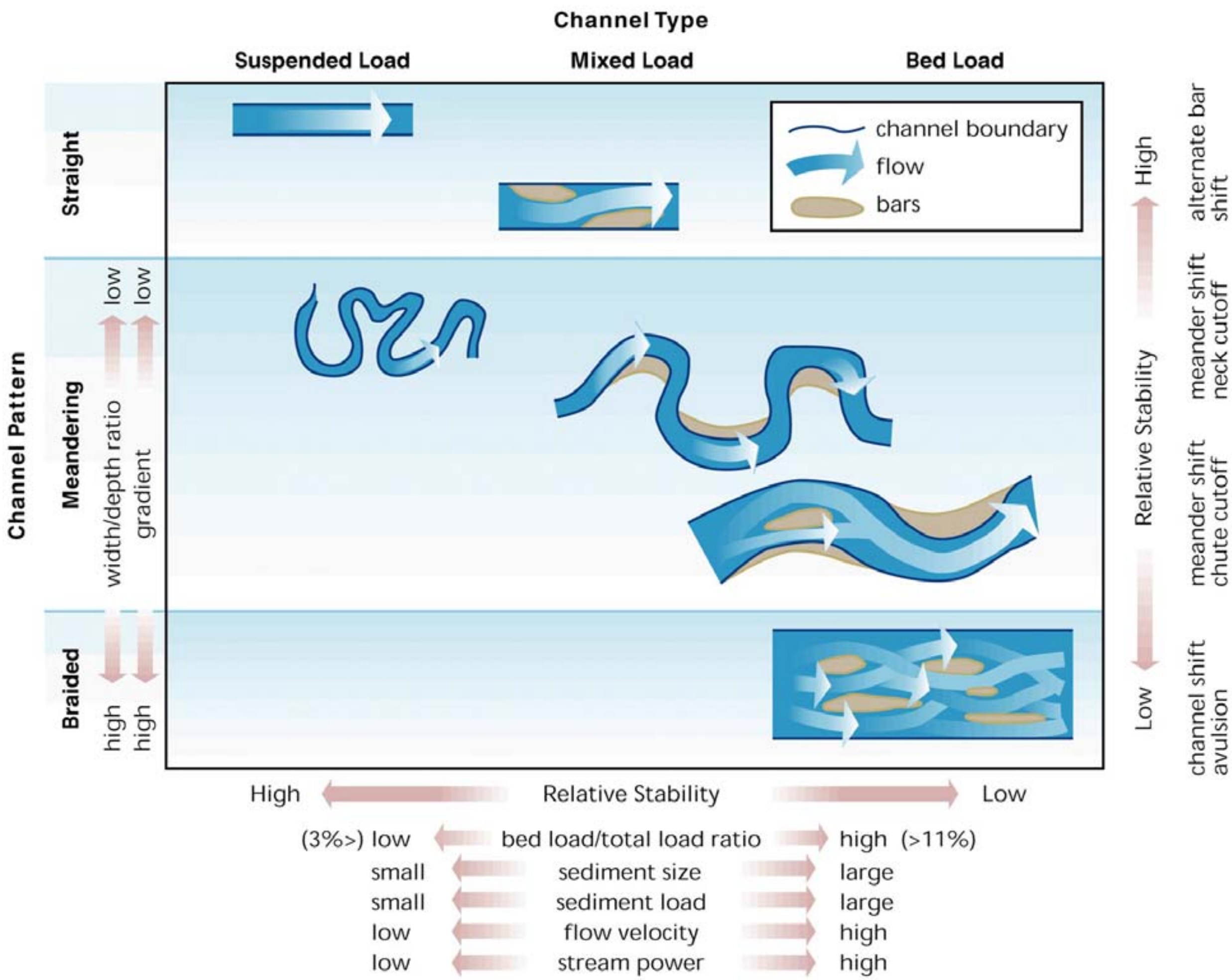
4.3.4. Step 4: Evaluate Lateral Stability

A field inspection is a critical component of a qualitative assessment of lateral stability. A comparison of observed field conditions with the descriptions of stable and unstable channel banks presented in Section 3.4.5 helps to define bank stability. Similarly, field observations of bank material composition and existing failure modes can provide insight into bank stability, based on the descriptions of cohesive, noncohesive, and composite banks given in Section 3.4.5.

A qualitative lateral stability assessment can also be completed from records of the position of a bend or bankline at two or more different times; aerial photographs or maps are usually the only records available. Surveyed cross sections are very useful although rarely available for bend migration in arroyos. Progress is being made on the numerical prediction of loop deformation and bend migration (Level 3 analysis). At present, however, the best available estimates are based on past rates of lateral migration in a particular reach. In using the estimates, it should be recognized that erosion rates may fluctuate substantially over time, and that changes in runoff and sediment load can cause future erosion rates to be quite different from historical rates.

Measurements of bank erosion on two time-sequential aerial photographs (or maps) require the identification of common reference points that are visible on both sets of photographs. Useful reference points include roads, buildings, irrigation canals, bridges, and fence corners. The analysis of lateral stability is greatly facilitated by a drawing of changes in bankline position with time. To prepare such a drawing, aerial photographs are matched in scale and superimposed, holding the reference points fixed.

Sites of potential avulsion (an abrupt shift of channel position to a new flow path) should be identified during this step so that measures can be taken to prevent the avulsion, if appropriate, or



Source: Schumm, The Fluvial System. © 1977. Reprinted by permission of John Wiley and Sons, Inc.

Fig. 7.10 -- Classification of alluvial channels, per Schumm's classification system.
In Stream Corridor Restoration: Principles, Processes, and Practices, 10/98.
Interagency Stream Restoration Working Group (FISRWG)(15 Federal agencies of the US).

Figure 4.2. Channel classification and relative stability as hydraulic factors are varied (after Schumm, 1977).

to mitigate the effects of an avulsion when it occurs, if prevention is not desirable or possible. A careful study of aerial photographs will show where overbank flooding has consistently occurred in the past, and where an overbank channel exists that can capture the flow. In addition, topographic maps and special surveys may show that the channel is perched above the surrounding alluvial surface, making avulsion more likely. Generally, avulsion, as the term is used here, will only be a hazard on alluvial fans.

4.3.5. Step 5: Evaluate Vertical Stability

Problems most commonly associated with degrading channels include bank failure and the undermining of hydraulic structures such as cutoff walls, flow-control structures, and bank protection. Bank sloughing because of degradation often greatly increases the amount of debris carried by the channel and decreases the available conveyance in the channel. The hazard of local scour becomes greater in a degrading stream because of the lower streambed elevation.

Aggradation in a stream channel increases the frequency of backwater that can cause flood-related damage. Lateral erosion, as a result of increased flood stages, can threaten to "outflank" hydraulic structures and bank protection and can also increase the debris load in a channel.

Data records for at least several years are usually needed to detect gradation (bed elevation) changes. In ephemeral channels, gradation changes develop over long periods of time even though rapid change can occur during an extreme flood event. The data needed to assess gradation changes include historic streambed profiles and long-term trends in stage-discharge relationships. Occasionally, information on bed elevation changes can be gained from a series of maps prepared at different times. Bed elevations at railroad, highway, and pipeline crossings monitored over time may also be useful. A qualitative assessment of potential vertical stability problems can be based on the Lane relationship (Section 3.1), the sediment continuity principle (Section 3.4.1), or equilibrium concepts (Section 3.4.2).

4.3.6. Step 6: Evaluate Channel Response to Change

The knowledge and insight developed from evaluation of present and historical channel and watershed conditions, as developed in Steps 1 through 5, provides an understanding of potential channel response to previous impacts and/or proposed changes in the channel or watershed. Additionally, the application of simple, predictive geomorphic relationships, such as the Lane Relationship (see Section 3.1) can assist in evaluating overall channel response mechanisms. **Results from the Level 1 (Qualitative) Analysis can be used to determine whether or not more detailed, quantification of arroyo processes is warranted, and if so, which specific processes are most likely to control the future behavior of the arroyo.**

4.4. Level 2: Basic Engineering Analyses

The Level 2 analysis provides a means of quantitatively evaluating arroyo processes using established engineering relationships and principals. A flow chart of the typical steps in a basic engineering analysis is provided in **Figure 4.3**. The flow chart illustrates the typical steps to be followed if a Level 1 qualitative analysis (Figure 4.1) resulted in a decision that Level 2 analyses is required. The eight basic engineering steps are generally applicable to most arroyo and drainageway stability problems, and are discussed in more detail in the following paragraphs. The basic engineering analysis steps lead to a conclusion regarding the need for more detailed (Level 3) analysis or a decision to proceed to establishing the LEE Line and/or design of countermeasures without more complex studies. Guidelines and criteria for selection and design of countermeasures are discussed in Chapter 5.

LEVEL 2: BASIC ENGINEERING ANALYSES

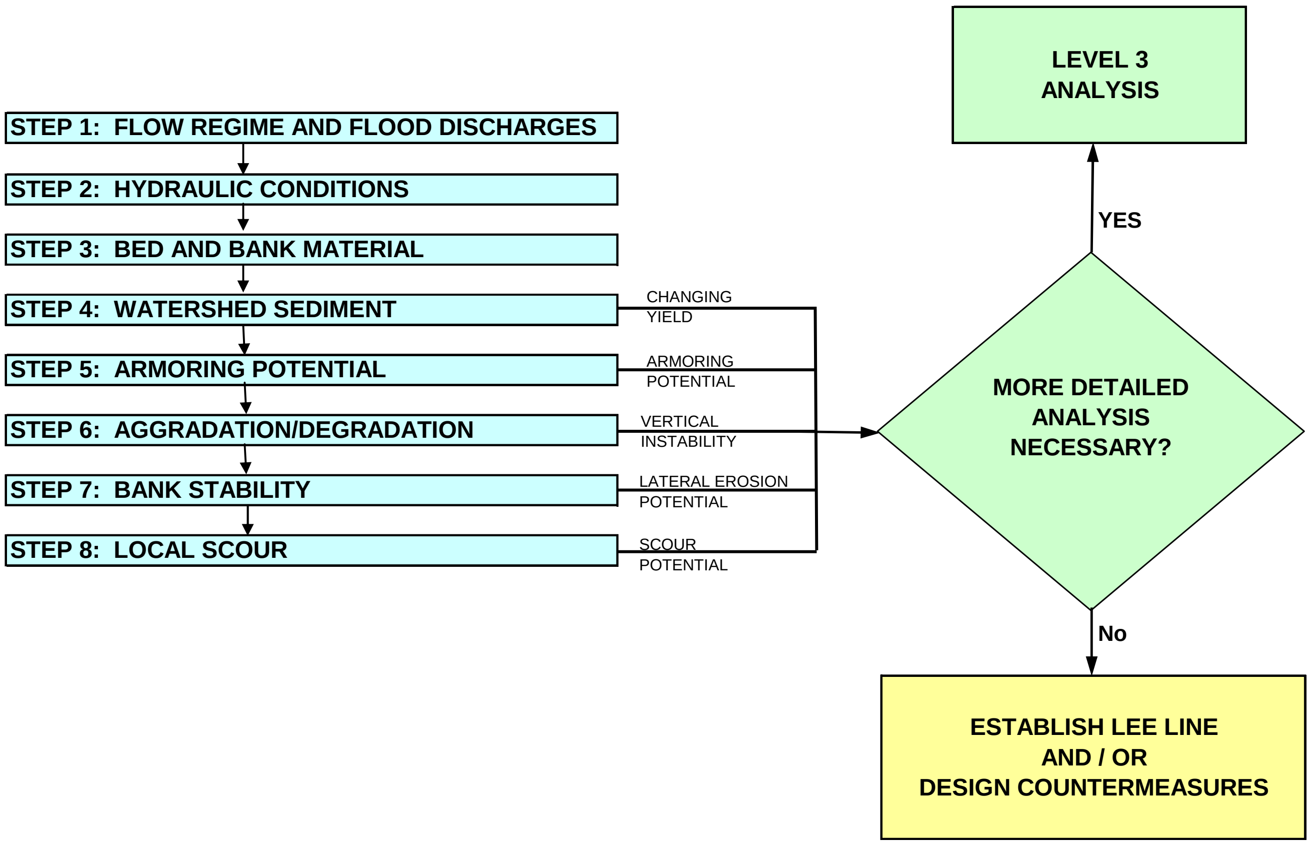


Figure 4.3. Level 2: Basic Engineering Analyses.

4.4.1. Step 1: Quantify Flow Regime and Flood Discharges

Hydrologic analysis techniques recommended for the SSCAFCA jurisdictional area are discussed in Section 2.2. Several additional hydrologic concepts of particular significance to evaluation of arroyo stability are summarized below.

Consideration of flood history is an integral step in attempting to characterize watershed response and morphologic evolution, particularly in arid regions. Many dryland streams flow only in response to significant storms. Leopold et al. (1966), for example, showed that arroyos near Santa Fe have measurable flow only about three times per year, on average. As a consequence, channel response in arid regions is more hydrologically dependent than streams located in a humid environment. Whereas the simple passage of time may be sufficient to cause change in a stream located in a humid environment, time alone, at least in the short term, may not necessarily cause change in an arid system due to the infrequency of hydrologically significant events. The absence of significant morphological changes in an arid region channel over a few to several years should, therefore, not be construed as an indication of system stability.

4.4.2. Step 2: Evaluate Hydraulic Conditions

Knowledge of basic hydraulic conditions, such as velocity, flow depth, top width, for given flood events is essential for completing Level 2 stream stability analysis because incipient motion, sediment transport and scour analyses all require basic hydraulic information.

Section 3.2 provides an overview of techniques that are commonly used in a hydraulic analysis. In some areas, hydraulic information may be readily available from previous studies, such as flood insurance studies, channel improvement projects, etc., and complete re-analysis may not be necessary. However, in other areas, hydraulic analysis using either normal depth calculations or step-backwater analysis will be required prior to completing other quantitative analyses in the Level 2 stream-stability assessment. The most common computer model for analysis of water-surface profiles and hydraulic conditions is the HEC-RAS software (USACE, 2005).

4.4.3. Step 3: Analyze Bed and Bank Material Characteristics

As discussed in Section 3.3.2, quantification of the bed and bank material characteristics in the arroyo is essential to performing a channel stability analysis. Of the various sediment properties, size has the greatest significance to the hydraulic engineer, not only because size is the most readily measured property, but also because other properties, such as shape and fall velocity, tend to vary with particle size. A comprehensive discussion of sediment characteristics, including sediment size and its measurement, is provided in Vanoni (1977) or Richardson et al. (1990). A discussion of sediment-sampling techniques is presented in Section 3.3.2.

4.4.4. Step 4: Evaluate Watershed Sediment Yield

Evaluation of watershed sediment yield, and in particular, the relative change in yield resulting from changes in the watershed due to natural (e.g., fire) or man-induced causes (e.g. urbanization, grazing, road building) can be important factors. Sediment eroded from the land surface can cause silting problems in channels, reservoirs, and detention ponds, resulting in increased flood stage and damage. Conversely, a reduction in sediment supply can also cause adverse impacts to river systems by reducing the supply of incoming sediment, promoting channel degradation and headcutting. A large change in sediment yield as a result of some disturbance, such as a recent fire or long-term land-use changes, would suggest that channel instability either already exist, or are likely to develop.

Quantification of sediment yield is at best an imprecise science. The most useful information is typically obtained from the relative changes in yield that result from a given disturbance or change in the watershed, rather than an analysis of absolute magnitude of the sediment yield. Section 2.3 and Appendix A describe sediment yield processes in the greater Albuquerque area and recommends several methods for estimating sediment yield. For example, both the Pacific Southwest Interagency Committee (PSIAC, 1968) and Modified Universal Soil Loss Equation (MUSLE) are introduced and their applications to the Albuquerque area discussed. Appendix A also summarizes available sediment yield data that may be applicable to the SSCAFCA jurisdictional area.

4.4.5. Step 5: Analyze Potential for Bed Armoring

An evaluation of relative channel stability can be made by performing an incipient motion analysis. The definition of incipient motion is based on the critical or threshold conditions where hydrodynamic forces acting on one grain of sediment have reached a value that, if increased even slightly, will move the grain. Under critical conditions, or at the point of incipient motion, the hydrodynamic forces acting on the grain are just balanced by the resisting forces of the particle. A computational technique for incipient motion is presented in Appendix B. When applied to a sand-bed channel, incipient motion results usually indicate that all particles in the bed material are capable of being moved for even very small discharges, a physically realistic result for most channels in the SSCAFCA jurisdictional area.

4.4.6. Step 6: Evaluate Degradation/Aggradation Potential

The incipient motion analysis (Step 5) provides a relatively simple method of determining whether or not the long-term degradation will be inhibited or eliminated by the presence of coarse bed material. Evaluation of **long-term degradation or aggradation potential** requires a more comprehensive sediment-transport analysis. The sediment-continuity concept (Section 3.4.1) is the basis for estimating degradation or aggradation tendencies in relation to vertical channel dynamics. As shown in Figure 3.19, if the inflow (supply) of sediment to a given reach is known or can be estimated, and the transport capacity of the reach, which is also assumed to represent that sediment outflow, can be calculated, then the storage (aggradation or degradation) in the reach can be estimated for selected discharge levels of a flood hydrograph or cumulatively over the entire hydrograph. A simple sediment-continuity analysis assumes rigid boundary hydraulic conditions and a uniform distribution of the erosion or deposited sediment volume over the reach. Obviously, a more accurate analysis would update channel cross sections at each time step or discharge level of the flood hydrograph. However, as discussed in Section 4.5, extending the analysis to quasidynamic or dynamic water and sediment routing would involve a Level 3 analysis using mathematical models such as HEC-6 (USACE, 1991) or the NCHRP's BRI-STARS program (Molinas, 1993). This level of complexity is not usually required for (**and may not be applicable to**) small arroyo and drainageway problems in the SSCAFCA jurisdictional area.

In performing a Level 2 sediment-continuity analysis, the study reach should be divided into a number of subreaches, generally based on the following factors:

1. Physical characteristics of the channel, such as top width, slope, and sinuosity,
2. Hydraulic conditions, such as depth and velocity,
3. Bed-material sediment characteristics,
4. The presence of hydraulic and sediment transport controls or points of significant flow change such as tributaries or diversions,
5. Areas of particular interest to study objectives, such as bridges or locations of proposed channel improvements,
6. The desire to maintain reach lengths as uniform as possible throughout the system.

Items 1, 2, and 3 are generally selected to provide consistency within the subreach, so that representative average conditions can be determined (SLA, 1985).

After subreach delineation, characteristic geometric and hydraulic information are developed for each subreach for the discharge(s) under consideration. This information may be computed manually through uniform flow or gradually varied flow calculations, or through computer programs such as HEC-RAS (see Section 3.2). The important hydraulic variables, that usually include the velocity, depth and topwidth at individual, representative cross sections within the subreaches are then, typically, averaged to define values representative of conditions in that reach for the given discharge.

After establishing representative hydraulic characteristics in each subreach, the sediment-transport capacity of each subreach is calculated using an appropriate method (see Section 3.3.4 and

Appendix C). The sediment-continuity principle is then applied by comparing transport capacity on a subreach-by-subreach basis, under the assumption that the sediment supply to any given subreach is equal to the transport capacity of the adjacent upstream reach. For long-term conditions, the upstream supply to the study reach is used as the supply to all subreaches under the assumption that the channel must eventually adjust to a state of equilibrium between the overall sediment supply and capacity. The comparison begins at the upstream end of the study reach by designating the first subreach as a supply reach, and progresses in the downstream direction.

To expedite the calculation procedure when evaluating a single storm or several hydrographs, the following analysis procedure is suggested. First, identify five to ten discharges adequate to span the range of discharges up to, and including, the maximum design discharge. Compute the reach-averaged hydraulic conditions within each subreach, and the corresponding sediment-transport capacities, for each discharge. For each subreach, develop a sediment transport capacity rating curve of the form $Q_s = a\ Q^b$ where Q_s is the sediment-transport capacity in cfs, Q is the water discharge in cfs and a and b are regression coefficients using the estimated transport capacities (see Appendix C). The analysis of the discretized hydrographs then proceeds as outlined above, with the sediment-transport capacity for each discharge in any given reach obtained by using the appropriate regression relationship. For more detail on this simplified sediment-continuity procedure, refer to SLA (1985).

As indicated by the Lane relation (Equation 3.1), when the sediment delivered to a reach is reduced (or is less than the capacity of the reach), the channel will tend to flatten its slope to attain an equilibrium condition so that the capacity is in balance with the supply. Conversely, when the sediment delivered to the reach increases (or is greater than the capacity of the reach), the channel will tend to steepen its slope. The ultimate slope for this condition is referred to as the **equilibrium slope**. The equilibrium slope concept provides an alternative means of estimating long-term degradation or aggradation trends, and it also provides a basis for determining appropriate spacing and height requirement for grade control structures to control long-term degradation. The specific relationships to be used for equilibrium slope calculations are discussed in detail in Section 3.4.3.

The equilibrium slope analysis should be performed using the dominant discharge. A reasonable estimate of the dominant discharge is the peak discharge of the flood event that would produce the mean annual sediment yield. The dominant discharge is typically about 20 percent of the 100-year peak discharge. The basis for this procedure, and further discussion of issues associated with estimating the dominant discharge in urbanizing arroyo can be found in Mussetter and Harvey (2005), a copy of which is included in **Appendix E**.

Contraction scour occurs when the flow area of a stream at flood stage is locally decreased, either by a natural constriction or by a structure such as a bridge. With the decrease in flow area, the average velocity and bed shear stress increase, resulting in an increase in the amount of bed material transported and, possibly, degradation of the contracted reach. As the bed elevation is lowered, the flow area increases and the velocity and shear stress decrease until a state of relative equilibrium is reached. The contribution of contraction scour to lowering of bed elevations should be included in an estimate of long-term degradation potential using the following computational sequence:

1. Estimate the hydraulic conditions for the existing channel, assuming fixed-bed conditions.
2. Assess the expected profile (i.e., aggradation or degradation) and planform changes.
3. If the anticipated changes are sufficient to changes the hydraulic conditions, adjust the fixed-bed hydraulics to reflect the expected changes.

4. Estimate contraction scour using the appropriate contraction scour formula and the adjusted fixed-bed hydraulics (see Section 3.4.3).

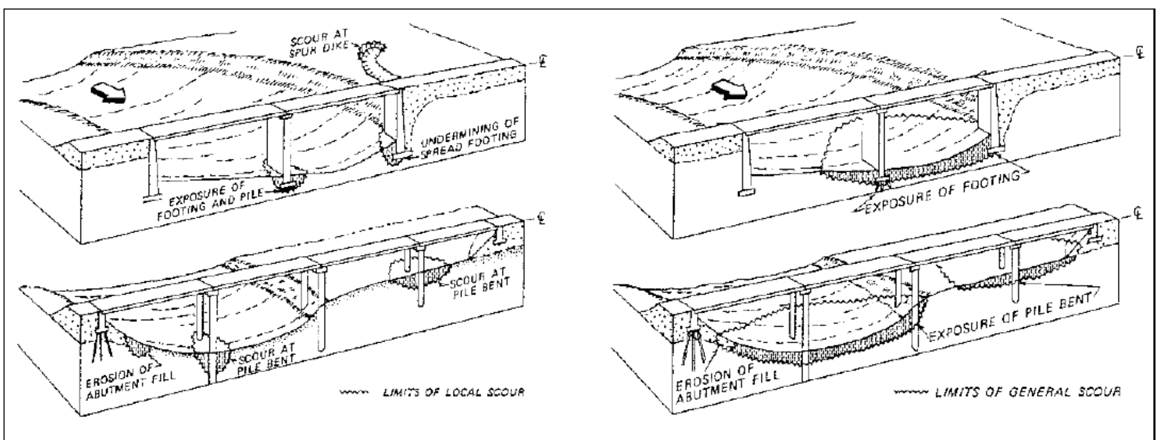
4.4.7. Step 7: Lateral Erosion Potential

Lateral erosion potential can be evaluated using a combination of qualitative (Level 1) and quantitative (Level 2) analysis techniques. As suggested in Level 1, Step 4, field inspection and time-sequenced comparison of aerial photography or mapping can be used to establish historic changes and trends in channel planform alignment. Detailed methods for such analysis are described in Lagasse, et al (2003). A reasonable upper limit on the amount of lateral erosion that can be expected over periods of a few decades can be estimated using the LEE line concept as described in Section 3.4.5.

4.4.8. Step 8: Evaluate Local Scour Conditions

For bridge piers and other objects/obstructions placed in the flow, vortices form around the base of the object. The formation of these vortices results from a pileup of water on the upstream face of the object and subsequent acceleration of flow around the object. The action of the base vortex (otherwise known as the horseshoe vortex, see Figure 3.20) removes sediment from the bed of the channel near the base of the object, resulting in a scour hole. Vertical wake vortices form downstream of the object which can also remove sediment from around the base of the object. **Figure 4.4** illustrates common scour-related problems at a bridge.

Computational procedures for local scour at bridge piers and similar objects are presented in Section 3.5.1. Scour downstream from check dams and other grade-control structures is discussed in Section 3.5.2, and scour at revetments, spurs, abutments, and similar structures is covered in Section 3.5.3. Scour along floodwalls is discussed in Section 3.5.4 and scour at culvert outlets is discussed in Section 3.5.5.



(a) Obstructions to flow

(b) Contraction of the flow or channel deepening at the outside of a bend.

Figure 4.4. Local scour and contraction scour-related hydraulic problems at bridges related to (a) obstructions to the flow or (b) contraction of the flow or channel deepening at the outside of a bend (Brice and Blodgett, 1978a, b).

Total scour is generally considered to be the additive result of the components contributing to channel bed lowering in a given reach. These include:

- long-term aggradation/degradation and contraction scour as estimated in Step 6 (Section 4.4.6), and

- scour associated with antidunes and local scour.

As with contraction scour, the hydraulic parameters to calculate local scour should be determined after the fixed-bed channel hydraulics have been adjusted to reflect any expected long-term profile or planform changes. With significant amounts of contraction scour (e.g., 4 to 5 feet), one could use an iterative procedure to further adjust the fixed-bed channel hydraulics for contraction scour before estimating local scour. However, in most cases, this is not necessary and both local scour and contraction scour estimates can be added independently to the aggradation/degradation potential to provide an estimate of total channel-bed lowering.

Bridge piers and similar objects in the flow are susceptible to the accumulation of debris (trees, brush, trash, etc.) which can substantially increase local scour depths. One approach to simulating debris blockage and its effects on local scour depth is to increase the pier width parameter in the pier scour equation in Section 3.5.1 and recalculate local scour. Judgment must be used, since there is obviously a point beyond which this approach will not provide a realistic estimate of the local scour depth. As a rule of thumb, one should not assume increased pier widths beyond the point where the conveyance of the original channel section (without debris) is reduced by more than 40 to 50 percent.

4.5. Level 3: Mathematical and Physical Model Studies

Detailed evaluation and assessment of stream stability can be accomplished using either mathematical or physical model studies. A mathematical model is simply a quantitative expression of the relevant physical processes involved in stream channel stability. Various types of mathematical models are available for evaluation of sediment transport, depending on the application (watershed or channel analysis) and the level of analysis required [HEC-6 (USACE, 1991); BRI-STARS (Molinas, 1993); see also Chang 1988)]. The use of such models can provide detailed information on erosion and sedimentation throughout a study reach, and allows evaluation of a variety of "what-if" questions.

In applying mathematical models such as HEC-6, however, the user must have a clear understanding of the limitations of the computational procedures being employed to ensure their applicability to the problem at hand. For example, as discussed in Chapters 2 and 3, arroyos adjust in response to individual storm events due to the ephemeral nature of their flows. Most sediment-routing models are designed to analyze long-term scour and deposition. Single event analyses using these models must, therefore, be performed with extreme caution. As noted in the HEC-6 Users Manual (USACE, 1991),

> *HEC-6 assumes that equilibrium conditions are reached within each time step (with certain restrictions ...); however, the prototype is often influenced by unsteady, non-equilibrium conditions during flood events. Equilibrium is never achieved under these conditions because of the continuously changing hydraulic and sediment dynamics. If these situations predominate, single event analysis should be performed only on a qualitative basis.*

This limitation applies to most such sediment-routing models. The conditions being described in the above statements are precisely those that occur in most arroyos during flood flows.

Similarly, physical model studies completed in a hydraulics laboratory can provide detailed information on flow conditions and to some extent, sediment-transport conditions, in a complex study reach. The hydraulic laws and principles involved in scaling physical model studies are well defined and understood, allowing accurate extrapolation of model results to prototype conditions. Physical model studies can often provide better information on complex flow conditions than mathematical models, due to the complexity of the processes and the limitations of 2- and 3-dimensional mathematical models. Often the use of both physical and mathematical models can provide complementary information.

However, the need for detailed information and accuracy available from either mathematical or physical model studies must be balanced by the time and money available. As the analysis becomes more complicated, accounting for more factors, the level of effort necessary becomes proportionally larger. The decision to proceed with a Level 3 type analysis has historically been made only for high risk locations, extraordinarily complex problems, and for forensic analysis where losses and liability costs are high. However, the widespread use of personal computers and the continued development of more sophisticated software have greatly facilitated completion of Level 3-type investigations at a reduced the level of effort and cost, suggesting that Level 3 type analysis techniques may be applied more routinely in the future.

4.6. Application of Analysis Procedures

The Level 1 and Level 2 analysis procedures presented in Figures 4.1 and 4.3 provide guidelines for qualitative and quantitative analyses of arroyo and drainageway stability problems applicable to the SSCAFCA area. The six steps of the Level 1 procedure and eight steps of Level 2 are carried out using the concepts and detailed analysis techniques presented in Chapters 2 and 3. Completion of a comprehensive analysis of arroyo and drainageway stability using these steps provides the necessary information to establish the location of the LEE line, and to identify problem areas associated with channel instability or flooding that may require mitigation using hardened structures or, where appropriate, bioengineering techniques. It should be noted in this regard, however, that bioengineering techniques often used in other areas of the country are unlikely to be successful in the SSCAFCA jurisdictional area due to the soil conditions and very dry climate.

4.6.1. Identification of the Hazard Type

A key step in applying the analysis procedures is to identify the nature (or type) of hazard that could occur as the arroyo evolves. In general, the hazard to developed properties along an arroyo can result from either flooding or lateral erosion, with the specific type of hazard dependant on the aggradational/degradational status of the arroyo. In vertically stable and aggrading reaches, the channel capacity may be limited, and the extent of overbank flooding may be the controlling factor in identifying the limits of the hazard zone. In degrading channels, the lateral erosion potential will most likely control the boundaries of the hazard zone because the in-channel capacity is typically relatively large. If the degrading channel is still in an early phase of evolution according to the ICEM (Figure 1.1), however, flooding limits could also be the controlling factor in the short-term. Because urbanization typically causes channel incision even in arroyos that remain in a naturalistic state, erosion boundaries will most likely be the controlling factor under future conditions.

The scenarios for determining the type of hazard along an arroyo is shown schematically in **Figure 4.5**. Figures 4.5a and 4.5b show a plan view and cross sections of a typical reach of arroyo indicating the relative position of the erosion limit lines and the floodplain boundary that might be expected in non-incised, incised, and aggrading reaches. The 100-year floodplain boundary under existing, rigid-boundary conditions can be established in accordance with Federal Emergency Management Agency (FEMA) flood insurance study and guidance contained in Section 3.2 of this Design Guide. The location of the LEE Line is then determined using the procedures in Section 3.4.5. In general, it should be assumed that the channel planform can eventually change so that

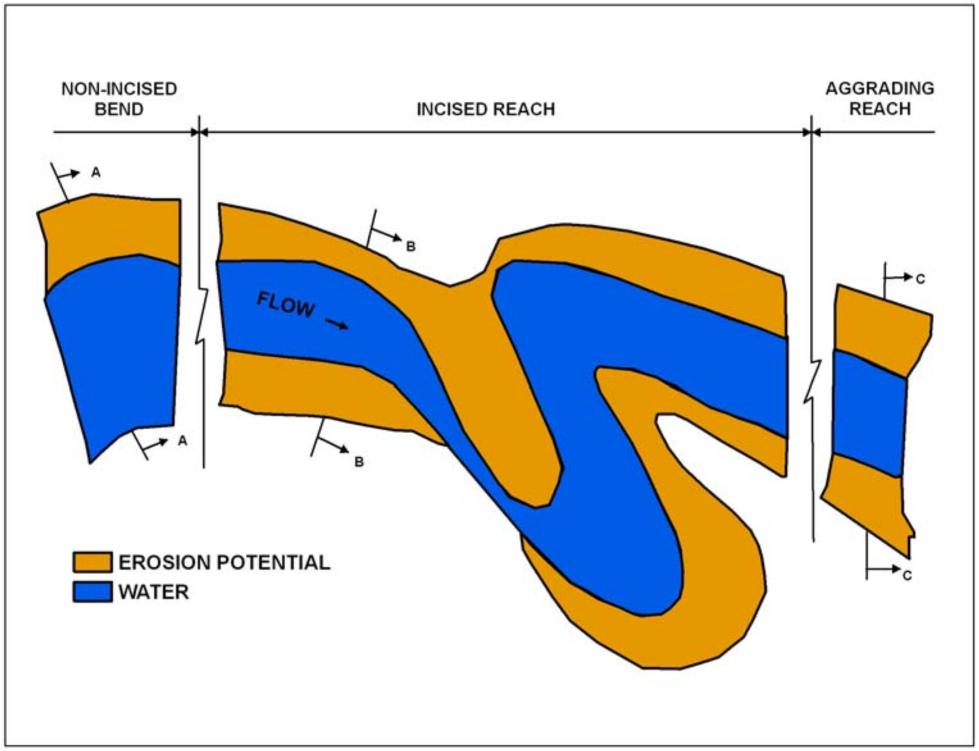


Figure 4.5a. Schematic illustration of flooding and erosion buffer zone.

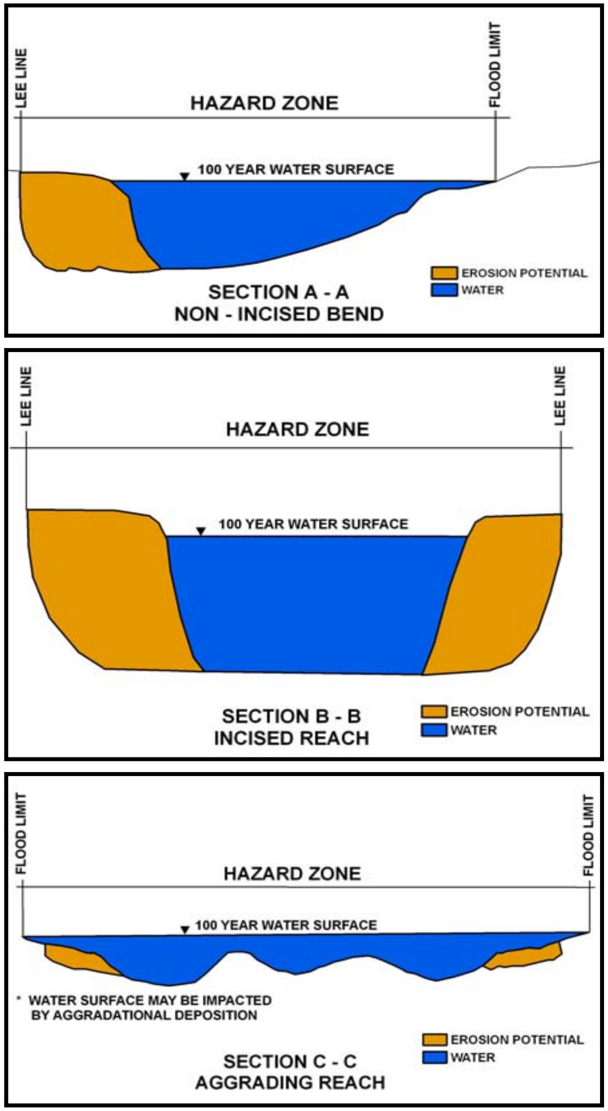


Figure 4.5b. Schematic cross sections non-incised reach, incised reach and aggrading reach showing flooding and erosion buffer zone.

the banks that are currently on the inside of a bend will shift to the outside of a bend through downstream bend migration, or bend cutoff, and the erosion setback indicated by the LEE line should be delineated on both side of the channel. If it can be clearly demonstrated based on the presence of natural or man-made controls that the inflection of a particular bend will not change in the future, it may be acceptable to use the flood boundary on the inside of the bend as the hazard zone limit (Figure 4.5, Section A-A). In straight reaches that are either vertically stable or degrading that have the capacity to convey the 100-year peak discharge without overbank flooding, the LEE line defines the hazard zone limit on both sides of the channel (Figure 4.5, Section B-B). If the overbank flooding occurs along such a reach, the flood boundary may define the hazard limit if it is outside the LEE line. In aggrading reaches, the inundation area associated with potential avulsions will likely control the hazard zone limits (Figure 4.5, Section C-C).

4.6.2. Methods of Controlling the Width of the Hazard Zone

In locations where existing property improvements are within the limits of the LEE line or flood hazard zone, structural measures may be necessary to provide the required level of protection. It is also possible that erosion processes under urbanized conditions, as indicated by the ICEM, would create unacceptable conditions that could include excessive degradation and increased sediment loads that would destabilize downstream reaches. Structural techniques that have been successfully used in the SSCAFCA jurisdictional area and other areas with similar conditions to mitigate such impacts include:

1. Grade-control structures to prevent or minimize degradation.
2. Bank protection to prevent or minimize lateral migration.
3. Erosion barriers to prevent lateral migration beyond a specified boundary.
4. Flow training devices (e.g., guidebanks, spurs, hardpoints) to improve flow alignment through bridge crossings, culverts or channel bends, and to limit erosion on the outside of the bends.
5. Detention ponds to control flood discharges and reduce the sediment delivery to downstream reaches.

A combination of the above measures will be used in most circumstances to maximize the level of protection, while achieving the goal of maintaining a naturalistic arroyo to the extent possible. For example, a series of grade controls can be installed with spacing such that the maximum bank height that would develop through channel degradation (e.g., based on an equilibrium slope analysis) will not exceed the critical height. This concept is illustrated in **Figure 4.6**. The maximum spacing of the structures (L_s) is computed by noting that the maximum allowable degradation (ΔZ_{max}) is the difference between the critical bank height (H_c) (see Figure 3.18) and the existing bank height (H_o), or:

$$\Delta Z_{max} = H_c - H_o \tag{4.4}$$

and the maximum spacing (L_s) is given by:

$$L_s = \frac{\Delta Z_{max}}{\left(S_o - S_{eq}\right)} \tag{4.5}$$

where S_o = the existing bed slope
S_{eq} = the equilibrium slope computed using the procedures in Section 3.4.3.

In designing grade-control structures, it is critical to provide bank protection for an appropriate distance up- and downstream of each structure to prevent flanking. The length of protection should

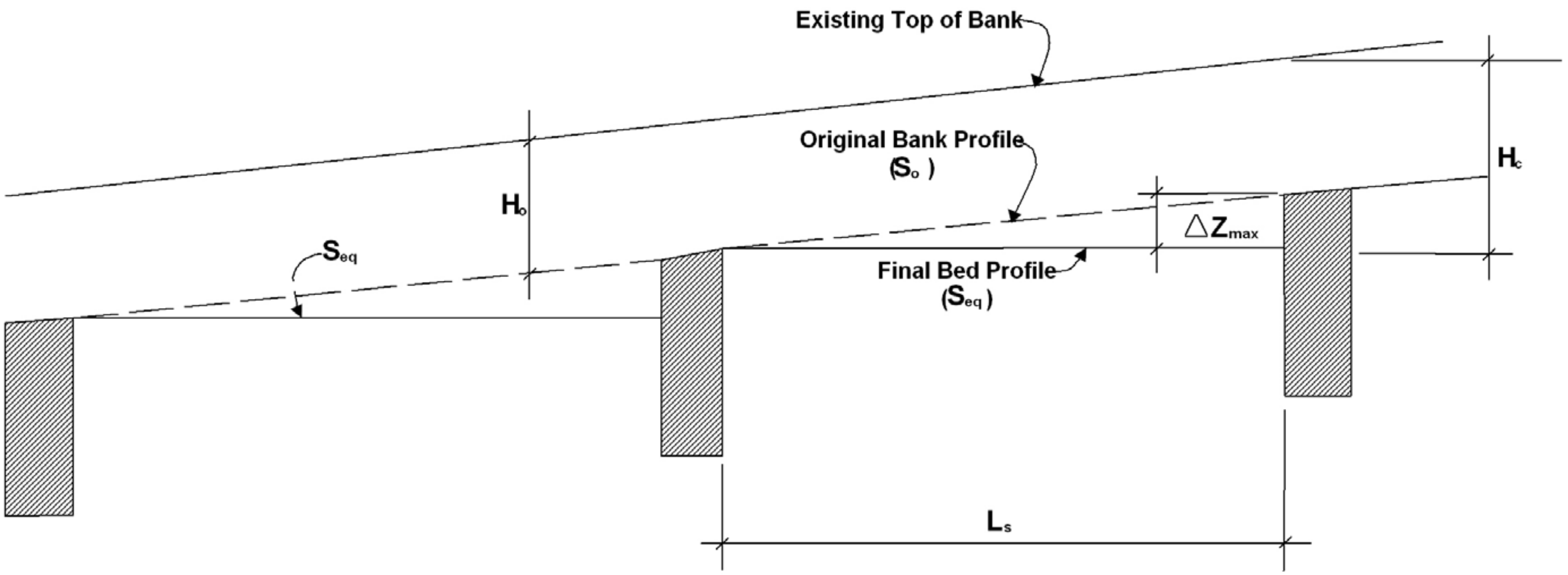


Figure 4.6. Illustration of method for determining the spacing of grade-control structures based on equilibrium slope (S_{eq}) concept and maximum stable (critical) bank height (H_c).

generally be at least two channel widths in each direction from the structure. This, in effect, fixes the lateral location of the channel at the location of each structure, shortening the length of channel available for channel bends to develop, and thus, the potential magnitude of lateral erosion.

Additional bank protection on the outside of bends between the grade-control structures can slow or check the rate of bend migration and further reduce the area within the flood and erosion zone. Figure 3.19 provides a relationship for determining the potential lateral migration distance for controls spaced in distances less than the unconstrained bend length.

In areas where it is impractical or inadvisable to install bank protection directly along the existing arroyo, it may be feasible to install an erosion buffer at or just inside the desired boundary of the stabilized flood zone. This may consist of a cutoff wall or trench fill revetment that will stop further migration of the channel at the desired location. It is important that the design of such an erosion buffer be based on the anticipated future conditions in the arroyo when the buffer is encountered, including the appropriate sizing of material and toedown to prevent undercutting, flanking or failure due to hydraulic forces. Toedown should be based on the potential scour depth along the wall as discussed in Section 3.5.4.

Training devices can be installed to improve flow alignment through bridges, culverts or channel bends to minimize the potential for local scour and prevent deposition of sediment and debris that may plug the opening. For example, the FHWA (Lagasse et al., 2001a) recommends guidebanks at bridges as an effective method of aligning the flow through the bridge opening to minimize the potential for scour around the piers and abutments. Guidebanks are particularly effective in controlling abutment scour since they move the scour hole upstream of the bridge to the end of the guidebank, protecting the bridge abutments from direct attack by the flow. In some cases, installation of hard points or spurs at channel bends can slow or check lateral migration. By improving the flow alignment through the bridge or culvert opening, backwater can be minimized, reducing the tendency for deposition of sediment and debris buildup that may cause channel avulsion or increased upstream flooding. Design criteria for spurs and guidebanks are presented in the next chapter (Section 5.9).

Detention structures can provide relief from flooding in downstream reaches by temporarily storing floodwaters and reducing the peak flows. Detention structures also trap significant quantities of sediment. In some cases, the reduced peak flows result in reduced erosive forces associated with the flood. It is important to recognize, however, that the total volume of water that must ultimately pass through the channel is usually not significantly affected by the detention structure. A modest reduction in flood peak may significantly increase the duration of moderately high flows, actually increasing the overall potential for channel instability. Additionally, in degradational arroyos, trapping of sediment reduces sediment delivery to the downstream reach, increasing the degradational tendency. It is critical to consider both the magnitude and duration of the flows, and the potential effect on the sediment balance in the arroyo before drawing conclusions regarding the ability of a given installation to protect downstream reaches of the channel.

Figure 4.7 illustrates the procedure for combining of the Level 1 and Level 2 analysis approach with natural/naturalistic development concepts. As shown, the Level 1 and Level 2 analyses for existing and future conditions provide the basis for assessing arroyo stability, and identifying the flood and erosion hazard (i.e., LEE Line) limits of a natural arroyo or drainageway. If the project is viable within these limits, the planning and development process can proceed without considering countermeasures or erosion barriers. In many cases, however, stabilization of the flood and erosion risk zone required. As shown in Figure 4.7, some situations may require confining the flood and erosion risk zone by channelizing part or all of the channel or drainageway through the reach proposed for development by the selective use of erosion barriers and other techniques.

Guidelines for these techniques are presented in Chapter 5. In all cases, potential impacts upstream and downstream of the project reach must be considered in evaluating the acceptability of proposed naturalistic channel stabilization techniques before proceeding with the planning and development process.

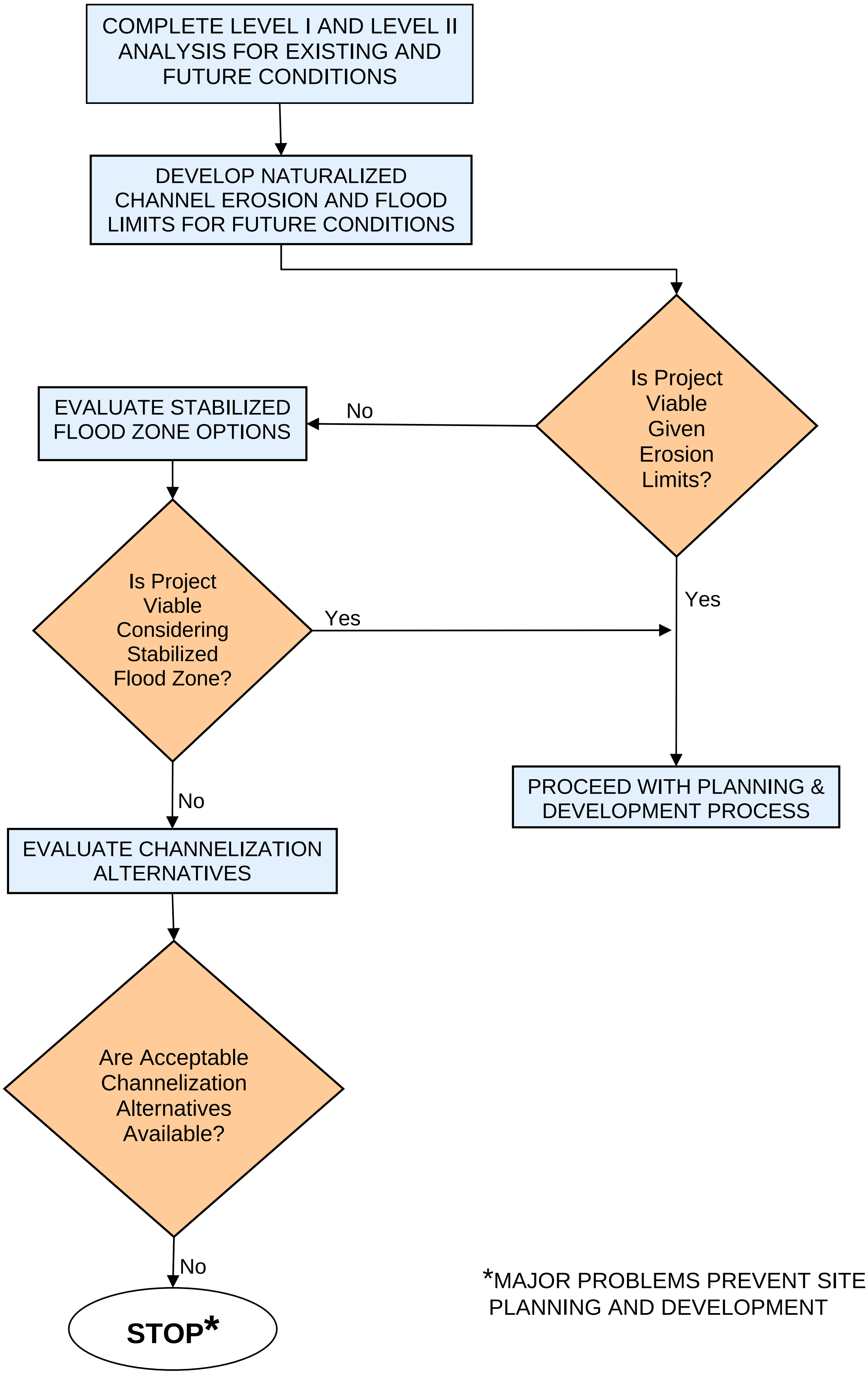


Figure 4.7. Flow chart of analysis procedures.

5. EROSION CONTROL AND COUNTERMEASURE CRITERIA

5.1. Introduction

A countermeasure is defined as a measure incorporated into the arroyo or drainageway to control, inhibit, change, delay, or minimize stream stability problems. Countermeasures may be installed at the time of initial development of the drainageway or retrofitted to resolve stability problems as they develop. Retrofitting can make good economic sense and can be good engineering practice in many locations because the magnitude, location, and nature of potential stability problems are not always discernible at the initial development stage, and indeed, may take a period of several years to develop. In selecting a countermeasure it is necessary to evaluate how the stream might respond, to the countermeasure.

This chapter provides some general criteria for the selection of countermeasures for channel instability. Then, selection and design criteria for countermeasures for specific channel instability problems are discussed. Case history data compiled by the Federal Highway Administration are summarized to provide information on the relative success of various countermeasures for stream stabilization. A more detailed discussion of these and other countermeasures is provided in Lagasse et al., 2001b.

5.2. Criteria for the Selection of Countermeasures

The selection of an appropriate countermeasure for a specific channel erosion problem is dependent on factors such as the erosion mechanism, stream characteristics, construction and maintenance requirements, potential for vandalism, and costs. Perhaps more important, however, is the effectiveness of the measure selected in performing the required function.

Protection of an existing bank line may be accomplished with revetments, spurs, retardance structures, longitudinal dikes, or barrier walls. Spurs, longitudinal dikes, and area retardance structures can be used to establish a new flow path and channel alignment, or constrict flow in a channel. Barrier walls may be used for any of these functions, but because of their high cost, are appropriate for use only where space is at a premium. Channel relocation may be used separately or in conjunction with other countermeasures to change the flow path and flow orientation.

5.2.1. Erosion Mechanism

Bank erosion mechanisms are surface erosion and/or mass wasting. Surface erosion is the removal of soil particles by the velocity and turbulence of the flowing water. Mass wasting is by slides, rotational slip, piping, and block failure. In general slides, rotational slip and block failure result from the bank being under cut by the flow. Also, seepage force of the pore water in the bank is another factor that can cause surface erosion or mass wasting. The type of mechanism is determined by the magnitude of the erosive forces of the water, type of bed and bank material, vegetation, and vertical stability of the stream.

5.2.2. Stream Characteristics

Stream characteristics that influence the selection of countermeasures include: channel width; bank height, configuration, and material; vegetative cover; channel bed sediment-transport condition; bend radii; channel velocities and flow depth; and floodplains.

5.2.2.1. Channel Width

Channel width influences only the use of spur-type countermeasures. On smaller streams, flow constriction resulting from the use of spurs may cause erosion of the opposite bank. However, spurs can be used on small channels where the purpose is to shift the location of the channel.

5.2.2.2. Bank Height

Low banks (<10 feet) may be protected by any of the countermeasures, including barrier walls. Medium height banks (from 10 to 20 feet) may be protected by revetment, retardance structures, spurs, and longitudinal dikes. High banks (>20 feet) generally require revetments used alone or in conjunction with other measures.

5.2.2.3. Channel Configuration

Spurs and jack fields have been successfully used as a countermeasure to control the location of the channel in meandering and braided streams. Also, walls, revetments, and riprap have been used to control bank erosion resulting from stream migration. On multi-branched channels, revetments, riprap, and spurs have been used to control bank erosion and channel shifting. Channels that do not carry large flows can and have been closed off with these types of countermeasures.

5.2.2.4. Channel Material

Spurs, revetments, riprap, or check dams can be used in any type of channel material if they are designed correctly.

5.2.2.5. Bank Vegetation

Vegetation can enhance the performance of structural countermeasures and may, in some cases, reduce the level of structural protection needed. Meander migration and other bank erosion mechanisms are accelerated on many streams in reaches where vegetation has been cleared. In arid and semi-arid areas, rainfall may not provide enough moisture to sustain significant plant density and supplemental irrigation may be required to provide measurable benefit from vegetation.

5.2.2.6. Sediment Transport

In general, sediment-transport conditions can be described as regime, threshold, or rigid. Regime channel beds are those which are in motion under most flow conditions, generally in sand or silt-size noncohesive materials. Threshold channel beds have no bed-material transport at normal flows and become mobile at higher flows. They may be cut through cohesive or noncohesive materials, or an armor layer of coarse-grained material may have developed on the channel bed. Rigid channel beds are cut through rock or boulders and rarely or never become mobile. In general, permeable structures will cause deposition of bed material in transport and are better suited for use in regime and some threshold channels than in rigid channel conditions. Impermeable structures are more effective than permeable structures in channels with little or no bed load, but impermeable structures can also be very effective in mobile bed conditions. Revetments can be effectively used with mobile or immobile channel beds.

5.2.2.7. Bend Radii

Bend radii affect the design of countermeasures. Thus, the cost per foot of bank protection provided by a specific countermeasure may differ considerably on short-radius and longer radius bends.

5.2.2.8. Channel Velocities and Flow Depth

Channel hydraulics affect countermeasure selection because structural stability and induced (local) scour must be considered. Some of the permeable flow retardance measures may not be structurally stable and countermeasures which utilize piles may be susceptible to scour failure in high velocity environments.

5.2.2.9. Debris

Debris (trees, limbs, trash, etc.) can damage or destroy countermeasures and should always be considered during the selection process. On the other hand, the performance of some permeable spurs and area retardance structures is enhanced by debris where debris accumulation causes increased sediment deposition.

5.2.3. Construction and Maintenance Requirements

Standard requirements regarding construction or maintenance such as the availability of materials, construction equipment requirements, site accessibility, time of construction, contractor familiarity with construction methods, and a program of regular maintenance, inspection, and repair are applicable to the selection of appropriate countermeasures. Additional considerations are the extent of bank disturbance which may be necessary and the desirability of preserving stream bank vegetative cover to the extent practicable.

5.2.4. Vandalism

Vandalism is always a maintenance concern in an urban setting since effective countermeasures can be made ineffective by vandals. Documented vandalism includes dismantling of devices, burning, and cutting or chopping with knives, wire cutters, and axes. Countermeasure selection or material selection for construction may be affected by concern for vandalism. For example, rock-filled baskets (gabions) may not be appropriate in some urban environments simply because they become a source for landscaping rock.

5.2.5. Costs

Cost comparisons should be used to study alternative countermeasures. However, it should be understood when comparing costs of existing installations that the measures were probably installed under widely varying stream conditions, that the conservatism (or lack thereof) of the designer is generally not accounted for, that the relative effectiveness of the measures cannot be quantitatively evaluated, and that some measures included in the cost data may not have been fully tested by floods.

5.3. Countermeasure Applications

5.3.1. General

Various devices and structures have been developed to control channel-stability problems and serve as barriers to erosion. Most have been developed through trial-and-error applications, aided in some instances by hydraulic model studies. Case studies which report successes and failures are often the best sources of countermeasure design criteria and can provide valuable guidance in selecting a countermeasure for a specific instability or erosion problem. The following sections discuss, in general, the applicability of common types of countermeasures to typical lateral and vertical instability problems in arroyos and arid region drainageways.

5.3.2. Countermeasures for Meander Migration

Stabilizing the outside of channel bends is a common approach to countering lateral migration problems; however, stabilizing channel banks in a reach of channel can cause a change in the channel cross section and an increase in stream sinuosity upstream of the stabilized banks. **Figure 5.1a** illustrates a natural channel section in a bend with the deeper section at the outside of the bend and a gentle slope toward the inside bank resulting from point bar deposition. **Figure 5.1b** also illustrates the scour which results from stabilizing the outside bank of the channel and the steeper slope of the point bar on the inside of the bend. This effect must be considered in the design of countermeasures. It should also be recognized that the thalweg location and flow direction can change as sinuosity upstream increases.

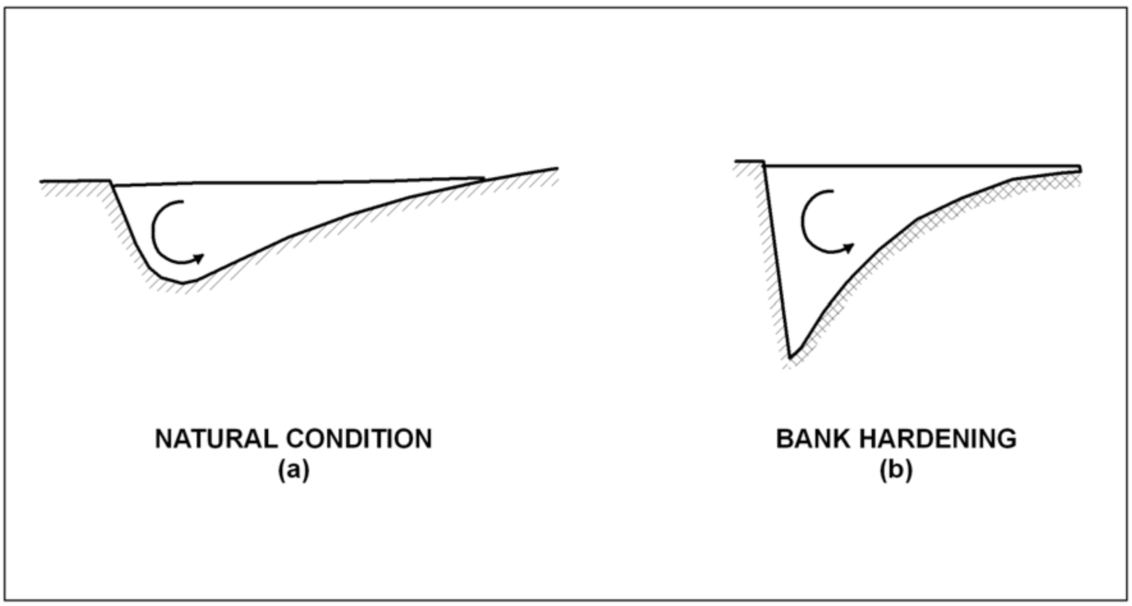


Figure 5.1. Comparison of channel bend cross sections (a) for natural conditions, and (b) for stabilized bend (after Brown, 1985a).

Figure 5.2a illustrates lateral migration in a natural stream and **Figure 5.2b**, the effects of bend stabilization on upstream sinuosity. As sinuosity increases, bend amplitude may increase, bend radii will become smaller, deposition may occur because of reduced slopes, and the channel width-depth ratio may increase as a result of bank erosion and deposition. Ultimately, cutoffs can occur. These changes can also result in changing hydraulic problems downstream of the stabilized bend.

Countermeasures for bend migration include those that:

1. Protect an existing bank line
2. Establish a new flow line or alignment
3. Control and constrict channel flow

The classes of countermeasures identified for bank stabilization and bend control are bank revetments, spurs, retardance structures, longitudinal dikes, barrier walls and channel relocations. These measures may be used individually or in combination to combat bend migration at a site. Some of these countermeasures are also applicable to bank erosion from causes other than bend migration.

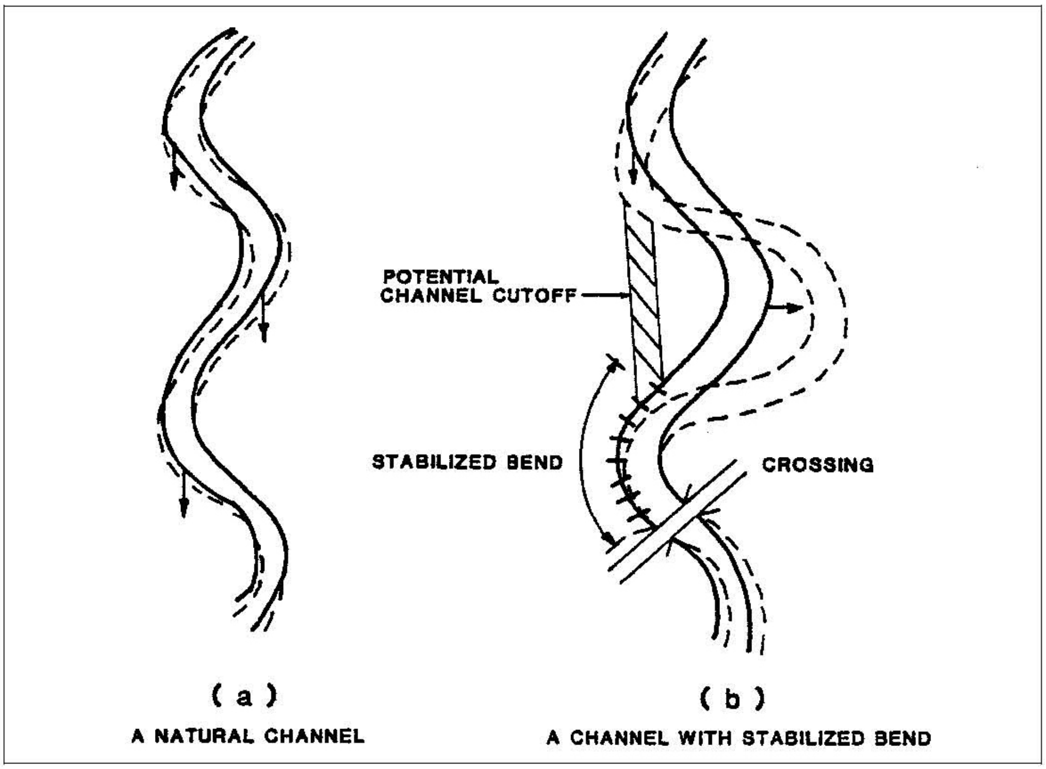


Figure 5.2. Bend migration in (a) a natural channel, and (b) a channel with stabilized bend (after Brown, 1985a).

5.3.3. Countermeasures for Channel Braiding and Multiple Channels

Channel braiding occurs in streams with an overload of sediment, causing deposition and aggradation. As aggradation occurs, the slope of the channel increases, velocities increase, and multiple, interlaced channels develop. The overall channel system becomes wider and multiple channels are formed as bars of sediment are deposited in the main channel. Braiding can also occur where banks are easily eroded and there is a large range in discharge. The channel becomes wider at high flows, and at low-flow forms multiple interlaced channels. Braided channels change alignment rapidly, and are very wide and shallow even at flood flow. In an island braided stream, flow is divided by islands rather than bars, and the branch channels are more permanent than braided channels and generally convey more flow.

Countermeasures used on braided and multi-branched streams are usually intended to confine the multiple channels to one channel. This tends to increase sediment-transport capacity in the principal channel and encourage deposition in secondary channels. These measures usually consist of spur fields or dikes. At bridge crossings, guidebanks (spur dikes) used in combination with revetment on highway fill slopes, riprap on highway fill slopes only, and spurs arranged in the stream channels to constrict flow to one channel, have also been used successfully.

5.3.4. Countermeasures for Degradation and Aggradation

Degradation problems are common on alluvial channels in the southwest. Degradation can cause bankline instability and can contribute to the loss of previously installed countermeasures. Aggradation can increase flooding potential and cause the loss of channel conveyance. Where channels become wider because of aggrading streambeds, existing countermeasures can be "outflanked." At its worst, aggradation may cause streams to abandon their original channels and establish new flow paths.

Countermeasures used to control degradation include check dams and channel linings. Check-dams and structures which perform functions similar to check-dams include drop structures and cutoff walls. A check-dam is a low dam or weir constructed across a channel to prevent degradation.

Linings of the channel banks with concrete riprap or other nonerosive material generally aggravates the tendency of degradation. To protect the lining, check-dams may be necessary to prevent undercutting.

Bank erosion is a common hydraulic hazard in degrading streams (see Section 3.4.4.). As the channel bed degrades, bank slopes become steeper and bank caving failures occur. The USACE found that longitudinal stone dikes, or rock toe-dikes, provided the most effective toe protection of all bank stabilization measures studied for very dynamic and/or actively degrading channels (USACE, 1981).

The following is a condensed list of recommendations and guidelines for the application of countermeasures in drainageways experiencing degradation:

1. Check-dams or drop structures are the most successful techniques for halting degradation on small to medium streams.

2. Channel lining alone with no bed protection is usually not appropriate to counter degradation problems.

3. Riprap or other nonerosive material on channel banks will fail if unanticipated channel degradation undercuts the toe of the riprap.

4. Rock-and-wire mattresses (gabions) are recommended for use only on small (<100-foot) channels experiencing lateral instability and little or no vertical instability. Even in these cases, the mattresses must have adequate burial depth to prevent undercutting by local scour.

5. Longitudinal stone dikes placed at the toe of channel banks are effective countermeasures for bank caving in degrading streams. Precautions to prevent outflanking, such as tiebacks to the banks, are usually necessary.

Currently, measures used in attempts to alleviate aggradation problems include channelization, debris (detention) basins, and/or continued maintenance, or combinations of these. Channelization may include excavating and clearing channels, constructing small dams to form debris basins, constructing channel cutoffs to increase the local slope (Figure 5.2), constructing flow control structures to reduce and control the local channel width, and constructing relief channels to improve flow capacity at the crossing. Except for debris basins and relief channels, these measures are intended to increase the sediment-transport capacity of the channel, thus reducing or eliminating problems with aggradation. Channel cutoffs must be designed with considerable study as they increase local slope and can cause erosion upstream and deposition downstream. These studies would involve the use of sediment-transport relations given in Chapter 3 or the use of sediment-transport models.

A program of continuing maintenance has been successfully used to control problems at hydraulic structures on aggrading streams. In such a program, a monitoring system is set up to survey the affected reach at regular intervals. When some pre-established deposition depth is reached, the channel is dredged or cleared of the deposited material. In some cases, this requires opening a clearing after every major flood. This solution requires surveillance and dedication to the continued maintenance of adequate channel capacity in the aggrading reach. Otherwise, it is only a temporary solution. A debris basin or a deeper channel upstream of the bridge may be easier to maintain. Continuing maintenance is not recommended if analysis shows that other countermeasures are practicable.

Over the short-term, maintenance programs prove to be very cost effective when compared with the high cost of countermeasures such as channelization. When costs over the entire life of the structure are considered, however, maintenance programs may cost more than some of the initially more expensive measures. Also, the reliability of maintenance programs is generally low because the programs are often abandoned for budgetary or priority reasons. However, a program of regular maintenance could prove to be the most cost-efficient solution if analysis of the transport characteristics and sediment supply in a stream system reveals that the aggradation problem is only temporary (perhaps the excess sediment supply is coming from a construction site) or will have only minor effects over a relatively long period of time.

An alternative similar to a maintenance program which could be used on streams with persistent aggradation problems, such as those on alluvial fans, is the use of controlled sand-and-gravel mining from a debris basin constructed upstream of the aggrading reach. Use of this alternative would require careful analysis to ensure that the gravel mining did not upset the balance of sediment and water discharges downstream of the debris basin. Excessive mining could produce a degrading profile downstream, potentially impacting bankline stability or other countermeasures.

Following is a list of guidelines regarding aggradation countermeasures:

1. Extensive channelization projects have generally proven unsuccessful in alleviating general aggradation problems, although some successful cases have been documented. A sufficient increase in the sediment-carrying capacity of the channel is usually not achieved to significantly reduce or eliminate the problem. Channelization should be considered only if analysis shows that the desired results will be achieved.

2. Maintenance programs have proved unreliable, but they provide the most cost-effective solution where aggradation is from a temporary source or on small channels where the problem is limited in magnitude.

3. At aggrading sites on wide, shallow streams, spurs or dikes with flexible revetment have been successful in several cases in confining the flow to narrower, deeper sections.

4. A debris basin and controlled sand-and-gravel mining might be the best solution at alluvial fans and other crossings with severe problems.

5.4. General Design Criteria

Countermeasures can be incorporated into a naturalistic channel design to control, inhibit, delay or minimize stream stability problems. Countermeasures can be used to control vertical instability such as head cuts and channel degradation, as well as to control lateral instability and local scour. Countermeasures should be considered whenever excessive degradation or channel incision is anticipated, and when structures and utilities pertaining to or within the channel right-of-way may be endangered by either lateral migration, channel degradation, or local scour.

The selection of an appropriate countermeasure for a specific erosion problem depends on factors such as erosion mechanism, stream characteristics, construction and maintenance requirements, vandalism, esthetics and costs (see Section 5.2). However, effectiveness is probably the most important consideration in the selection of a specific countermeasure.

5.5. Detention Ponds

A detention pond can be an effective method of reducing flood peaks during storm events. Such structures temporarily detain flows from drains and collectors in urban areas, allowing for a slower, controlled release of flow over a longer time period. Detention structures will also trap sediments, reducing the supply of sediment to the downstream channel, which can create either a beneficial or adverse impact.

The principal benefit of a detention pond as part of a naturalistic arroyo design is that flood peaks and resulting flood water levels and velocities can be reduced. Since the channel size and shape are a function of the channel forming discharge, the reduction of peak discharge achieved with these structures can reduce lateral and vertical erosion in the arroyo. A problem with detention ponds is the ongoing maintenance required to clean the ponds after floods to keep the structures from filling with sediment.

5.6. Riprap

One of the most effective and versatile erosion-control countermeasures is riprap. Riprap can be used to control lateral migration of channel banks and vertical degradation of channel beds, as well as mitigation of local scour at spurs, abutments, guidebanks, grade-control structures, and/or channel drops.

An advantage to the use of riprap is that this countermeasure is somewhat flexible and porous, allowing the stones to shift if there is subsidence. Riprap can prevent buildup of pore water pressure in the soil, which can cause more rigid stabilization measures such as concrete pavement to fail. Local failures of the riprap protection can be easily repaired by placement of more stone. Additionally, riprap can be aesthetically pleasing, and over a period of time, vegetation can establish itself between the stones, increasing the stability of this countermeasure.

A riprap design guide is also available from AMAFCA (1983). This guide discusses placement and filter requirements for protection of channel banks, channel bottoms, and locations of local scour such as drop structures, baffled aprons and grade-control structures. In addition, the Federal Highway Administration Hydraulic Engineering Circular No. 11, "*Design of Riprap Revetment*" (HEC-11) provides a supplemental source of design guidelines for use of riprap and other materials for bankline protection (Brown and Clyde, 1989). Also, Engineering Manual EM 1110-2-1601, "*Hydraulic Design of Flood Control Channels*" (USACE, 1991) contains a chapter and numerous plates to guide riprap design.

If suitable sizes of riprap are not available, wire baskets filled with smaller stones (gabions) can be utilized in much the same fashion as riprap. This countermeasure is suitable for bankline stabilization, protection of approach embankments, and can be used to form grade control or drop structures. It is important to note that the wire baskets are subject to clipping if coarse material (gravel sizes and larger) is transported by the flows in the channel. Furthermore, gabions are more vulnerable to vandalism and are not as flexible as riprap. As a result, they can withstand less settlement than riprap.

As with riprap, care must be exercised in the design to prevent piping of subgrade material from under the gabion mattresses. Suitable filter material must be placed under these structures, and edges must be tied into existing ground to prevent undermining and outflanking by the flow. HEC-11 (Brown and Clyde, 1989) provides detailed design guidelines for use of wire enclosed rock, including gabions, as bank protection (see **Figures 5.3 and 5.4**). Gabion manufacturers also provide guidelines and technical advisories on their particular products.

5.7. Soil Cement

Soil cement can also be used for a wide range of stability problems in arroyos. Soil cement can be used to stabilize access roads, grade-control structures, and channel banks. A unique advantage of soil cement for channel protection and grade control is that a large mass of material can be provided relatively cost effectively to counter erosion forces. As with any rigid countermeasure, soil cement installations must be protected against degradation and outflanking. Since soil cement is relatively impervious it is not recommended for areas where pore water pressure in the underlying soil could cause failure.

A stair-step soil cement construction is recommended on channel banks with relatively steep slopes. Best results have been achieved on slopes no steeper than 3:1 as shown in **Figure 5.5**. However, in the arid southwest a steeper slope up to 1:1 can be used for stair-stepped soil cement. The material is placed in 6- to 12-inch lifts similar to compacted earth. Special care should be exercised to prevent raw soil seams between successive layers of soil cement. A sheeps-foot roller should be used on the last layer at the end of a day to provide an interlock for the next layer. The completed soil-cement installation must be protected from drying out for a 7-day hydration period. After completion, the material has sufficient strength to serve as a roadway along the embankment. Procedures for constructing soil-cement slope protection by the stair step method can be found in Portland Cement Association (PCA, 1984a and 1984b).

Precautions must be taken to prevent undermining at the toe and ends for any soil-cement installation. Protection at the toe can be provided by extending the installation below estimated scour depth, by a riprap launching apron, or by a concrete or sheet pile cutoff-wall extending to bedrock or well below the anticipated scour elevation. Weep holes or subsurface drains for relief of hydrostatic pressure are required for some situations.

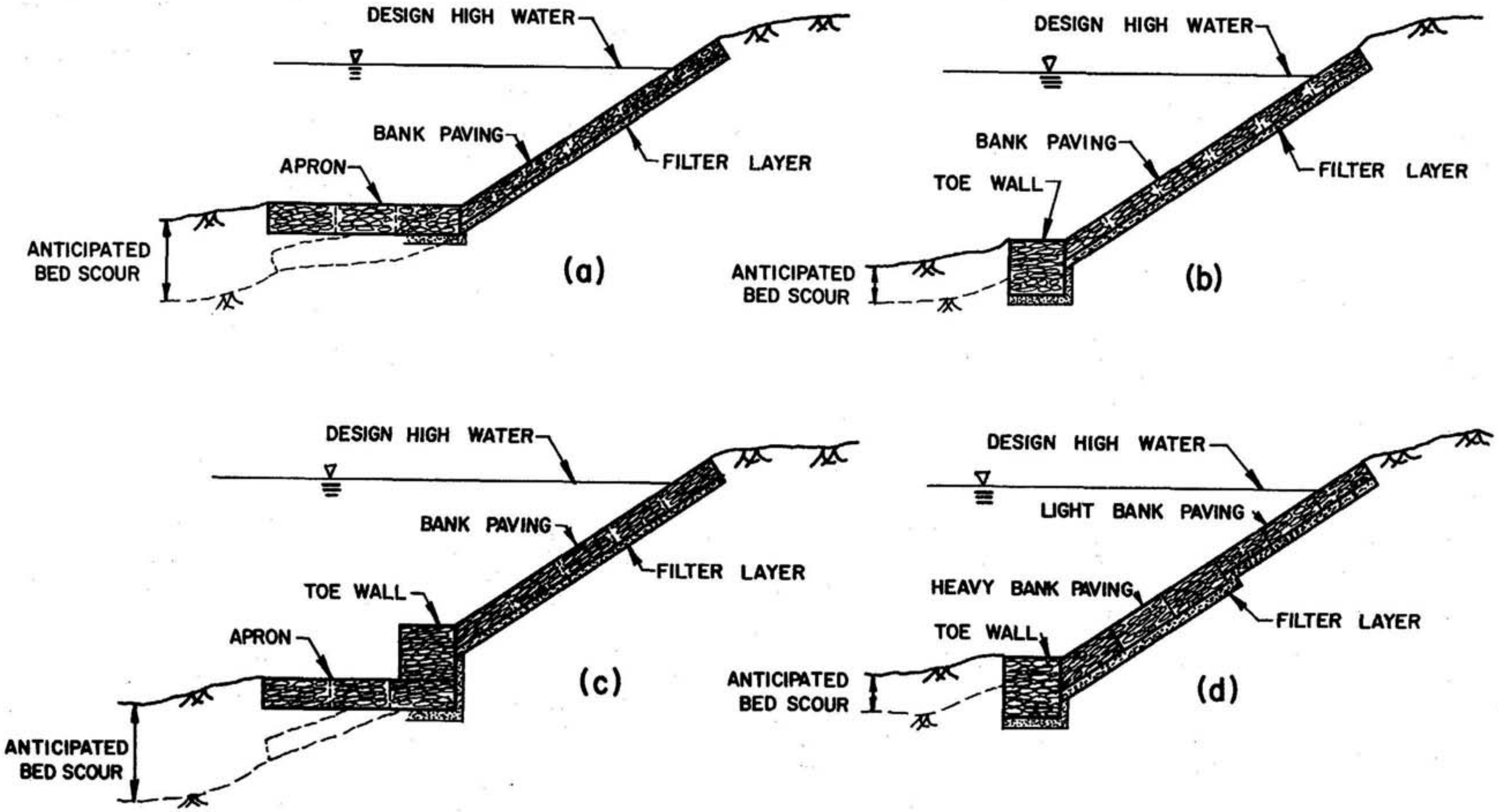


Figure 5.3. Rock and wire mattress configurations: (a) mattress with toe apron; (b) mattress with toe wall; (c) mattress with toe wall; (d) mattress of variable thickness (after Brown and Clyde, 1989).

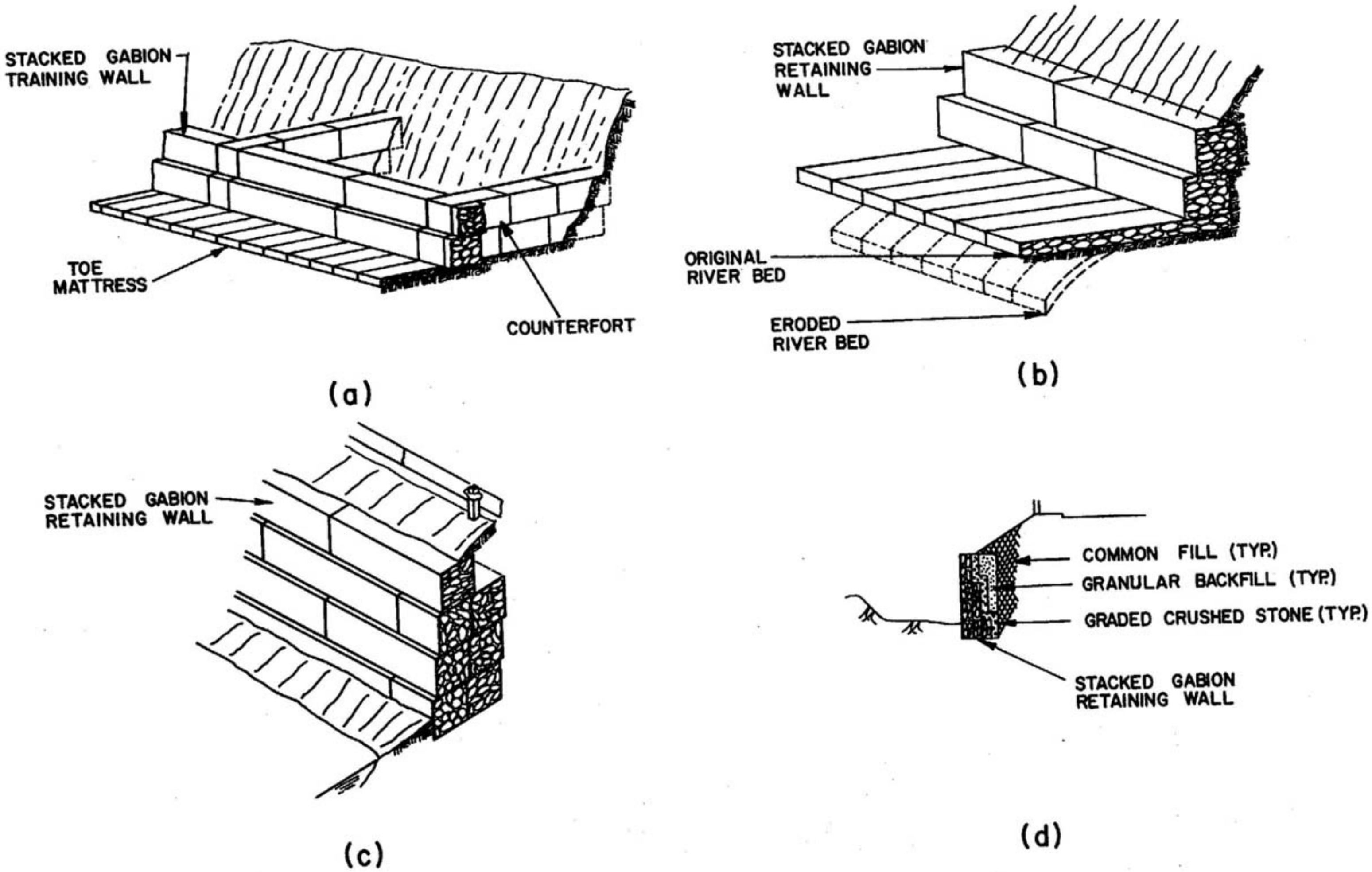


Figure 5.4. Typical stacked block gabion revetment details: (a) training wall with counterforts; (b) stepped back low retaining wall with apron; (c) high retaining wall, stepped-back configuration; (d) high retaining wall, batter type (after Brown and Clyde, 1989).

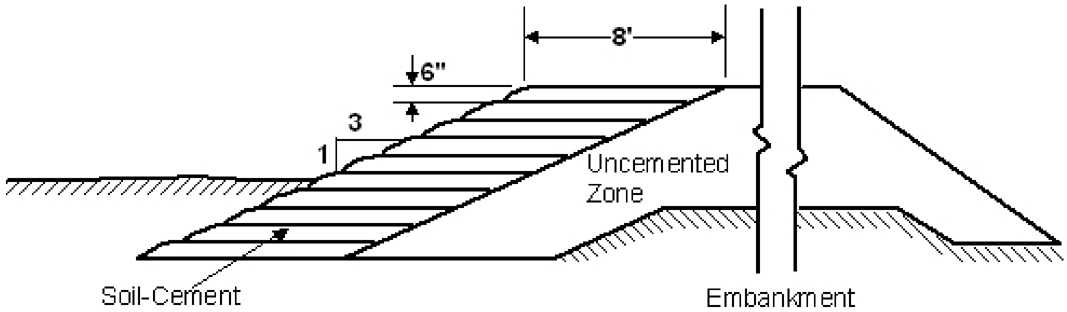


Figure 5.5. Typical soil-cement bank protection (after Richardson and Davis, 2001).

5.8. Revetment and Bankline Stabilization

Revetments are either flexible or rigid structures placed longitudinally along a channel to protect banklines or to establish a new bankline in the arroyo. A comprehensive discussion of revetments is presented in FHWA's HEC-20 (Lagasse et al., 2001a) and detailed design guidelines are available in FHWA's HEC-11 (Brown and Clyde, 1989).

5.8.1. Flexible Revetment

Dumped rock riprap and gabions are the most widely used flexible revetments (see Section 5.6).

5.8.2. Rigid Revetment

Rigid revetments include concrete pavement, sacked concrete, concrete-grouted riprap, concrete-filled fabric mat, and soil cement. Special precautions are warranted in the design to prevent undermining at the toe and termini as well as failure from unstable soils or hydrostatic pressures.

5.9. Spurs and Guidebanks

5.9.1. Spur Description

Spurs are pervious or impervious structures which project perpendicularly from the streambank into the channel. Spurs deflect flowing water away from the bank, reducing velocities and mitigating bank erosion. They can also be used to establish a more desirable channel alignment or width. Sediments carried by the flow can deposit in the low velocity area behind the spur, thus increasing the stability of the spur and bank protection. Local scour can occur at the nose of spurs and therefore must be protected with revetment material.

Spurs are generally used to halt meander migration at a bend. They are also used to channelize wide, poorly defined streams into well-defined channels. Spurs are classified based upon their permeability as retarder spurs, retarder/deflector spurs, or deflector spurs. The permeability of spurs is defined simply as the percentage of the spur surface area facing the streamflow that is open. Deflector spurs are impermeable spurs (<30-percent permeability) which function by diverting the primary flow currents away from the bank. Retarder/deflector spurs are moderately permeable and function by retarding flow velocities at the bank and diverting flow away from the bank. Retarder spurs are more permeable (>70-percent permeability) and function by retarding flow velocities near the bank. Spurs can be constructed using rock and wire basket (gabion) techniques or rock riprap (see **Figures 5.6 and 5.7**) or soil cement.

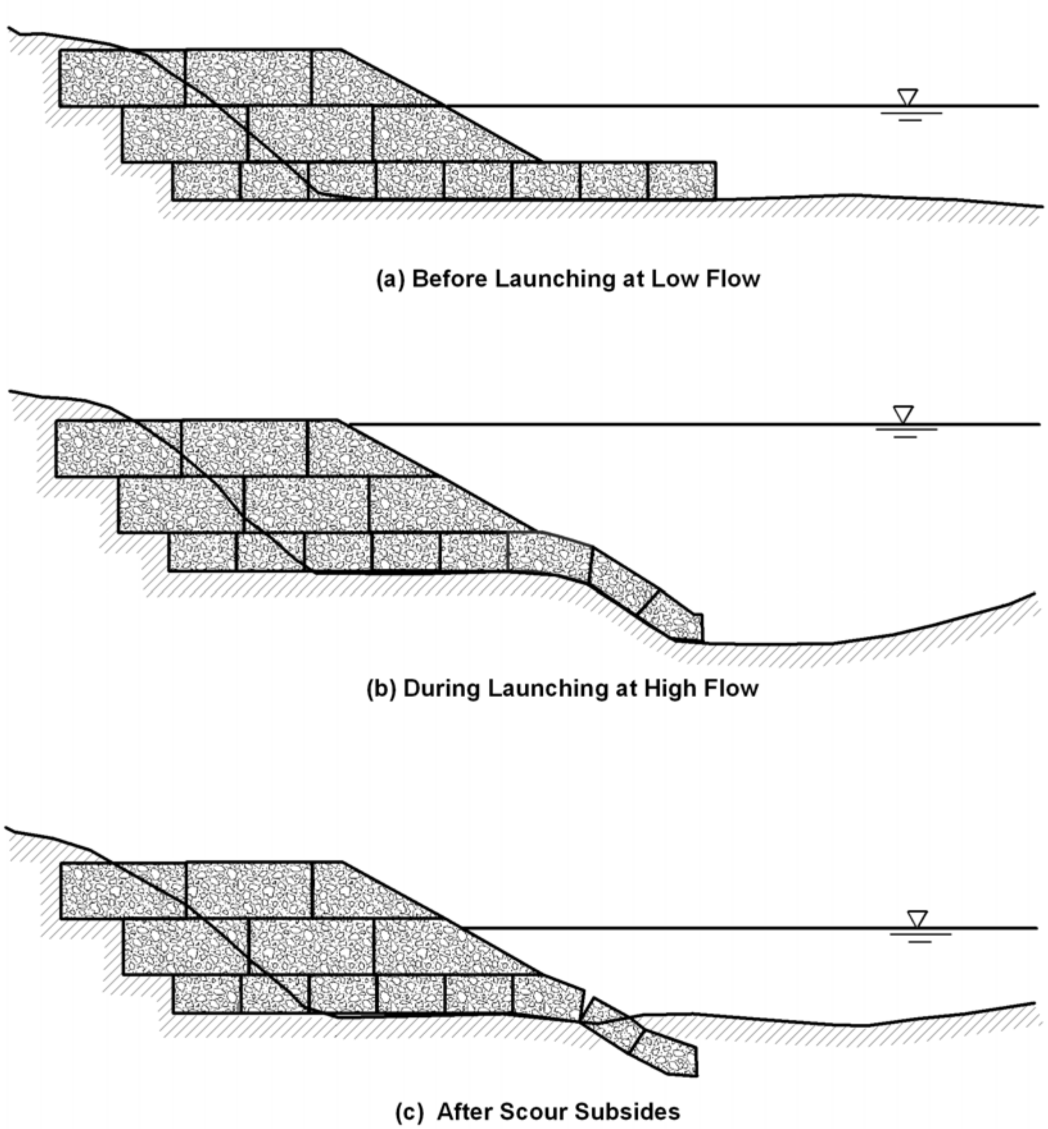


Figure 5.6. Gabion spur illustrating flexible mat tip protection: (a) before launching at low flow, (b) during launching at high flow, and (c) after scour subsides (after Brown, 1985).

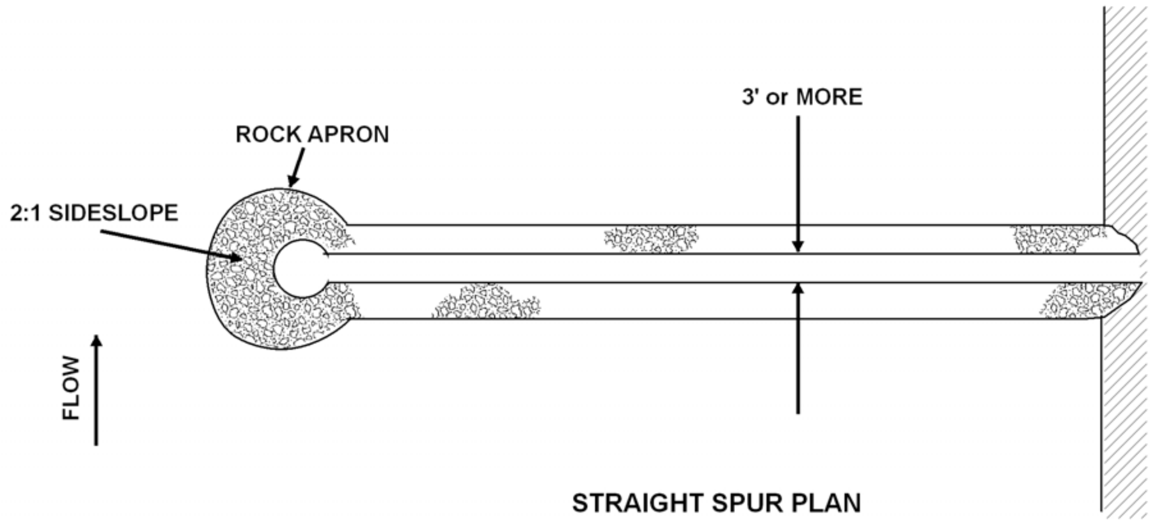


Figure 5.7. Typical straight, round nose spur (Lagasse et al., 2001a).

5.9.2. Spur Design Considerations

Spur design includes setting the limits of bank protection required, selection of the spur type to be used, and design of the spur installation including spur length, orientation, permeability, height, profile, and spacing. Defining the limits of bank protection is subjective and very little quantitative guidance, particularly for arroyo applications, is available. As a minimum, the length of spur field should be equal to the observed or expected length of erosion or bank instability. Evaluation of installations in meandering streams suggests that the protection often extends too far upstream and not far enough downstream, which is valuable insight for any spur field design.

5.9.2.1. Spur Length

Spur length is the projected length of the spur from the bankline normal to the main flow direction. When the bankline is irregular, spur length should be adjusted to provide for a curvilinear flow path. If the spur length is too great, relative to channel width, erosion of the opposite bankline can occur. Generally, impermeable spur length should be less than 15 percent of the channel width, and permeable spurs less than 25 percent of the channel width.

5.9.2.2. Spur Orientation

Spur orientation, relative to the main flow direction, can affect spur spacing, the degree of flow control achieved and the scour depth at the tip of the spur. However, most evidence indicates that spurs normal to the bankline perform adequately and are most cost-effective. Only the spur furthest upstream should be angled downstream to provide a smoother transition into the spur field and minimize scour at the nose of the leading spur.

5.9.2.3. Spur Height

The height of impermeable spurs should not exceed the bank height to minimize scour in the overbank and potential outflanking at high stream stages. When the design water surface is equal or greater than the bank height, impermeable spurs should equal the bank height. When the water surface is lower than the bank height, impermeable spurs should be designed so that overtopping will not occur. Permeable spurs are typically designed so that debris can pass over the top of the spur.

5.9.2.4. Scour Potential

Spur permeability influences scour potential at the streambank and spur tip. Impermeable spurs, in particular, can create erosion of the streambank at the spur root if the spur becomes overtopped, while such erosion is less likely for permeable spurs. Similarly, more scour at the spur tip would be expected for impermeable spurs. Scour depths at spur tips for permeabilities up to about 35 percent are given by **Figure 5.8**. For design purposes, the same figure will provide conservative results for spurs with permeabilities greater than 35 percent.

5.9.2.5. Spur Spacing

Spur spacing is a function of spur length, spur angle, permeability, and the degree of curvature of the bend. For smaller watercourses, such as the arroyos in the Albuquerque area, the spur spacing is also a function of the shape of the meander flow path as described in Section 3.4.5. The flow expansion angle, or the angle at which flow expands toward the bank downstream of a spur, is a function of spur permeability and the ratio of spur length to channel width. The shape of the meander flow path is a function of the dominant discharge, dominant channel width and channel sinuosity. Spur spacing is determined by the following steps:

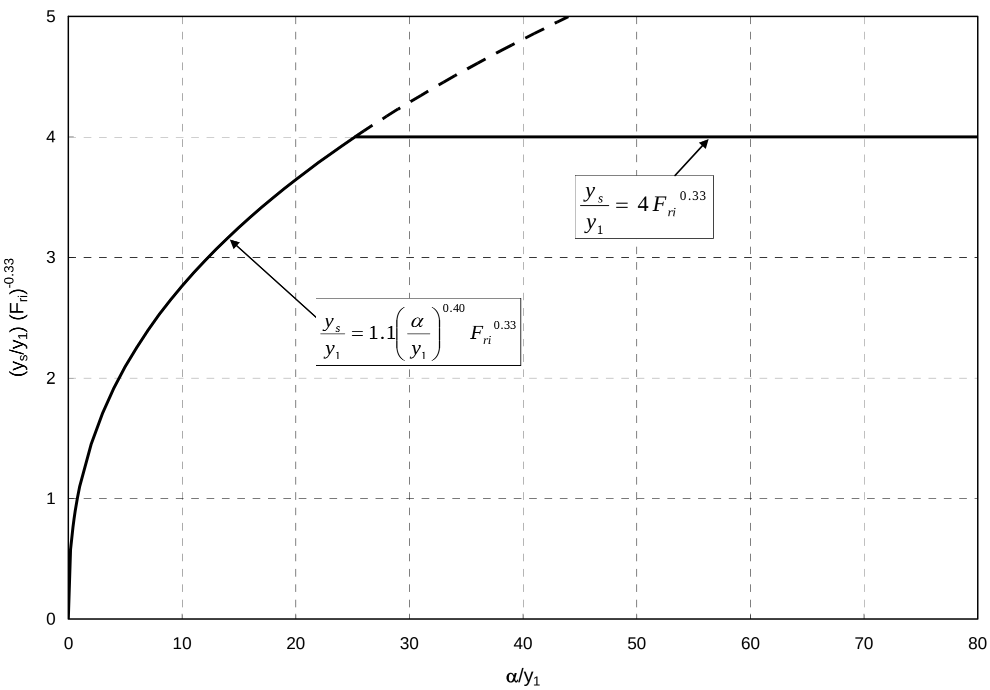


Figure 5.8. Recommended prediction curves for scour at the end of spurs with permeability up to about 35 percent (Lagasse et al., 2001a). See Section 3.5.3 for a definition of terms.

1. <u>Sketch the desired thalweg or flow alignment</u>. Based on the up- and downstream flow conditions, sketch the desired flow lines with smooth transitions to up- and downstream conditions.

2. Sketch the desired bankline and alignment of the spur tips. Based on the desired flow lines and the general guidelines on spur length, sketch the alignment of the toe of the spur tips and desired bankline. Note that typically spur length is measured from the desired bankline.

3. Locate the first spur. Locate the most downstream spur so that the flow expansion angle, as defined in **Figure 5.9** and determined from **Figure 5.10** will intersect the downstream bankline at the desired location. If spurs are designed to provide erosion protection for the areas within the LEE line, the bank end of the first spur should extend to LEE Line .

4. Locate remaining spurs. The remaining spurs are located based on the following:

   a. Use the following equation to determine the minimum flow expansion angle:

$$S = L \quad \cot(\theta) \tag{5.4}$$

   where S = spacing between spurs at the toe, ft
   L = effective length of spur, or the distance between arcs describing the toe of spurs and the desired bankline, ft
   θ = expansion angle downstream of spur tips, degrees.

   b. Use Figure 3.22 and the procedures to compute channel widths for dominant discharge (W_D) to compute the maximum offset from the channel bank, assumed to be at the channel end of the spurs.

Example. An example of spur field design, as well as more detailed discussion of spur design, are provided in FHWA's HEC-20 (Lagasse et al., 2001a). This example does not include the adjustment for meander flow path that is recommended by this Design Guide.

5.9.3. Guidebanks

Guidebanks can be used at bridges or culverts whenever there is overbank flow which must return to a bridge opening or when road-approach embankments encroach into the channel (see **Figure 5.11**). The function of guidebanks (in older references these are referred to as spur dikes) is to provide a more streamlined flow through the bridge or culvert by reducing separation of the flow which must return to the constricted bridge or culvert opening upstream of the crossing. The guidebank will minimize local scour at abutments by transferring the local scour upstream to the nose of the guidebank. Guidebanks can be constructed with rock revetment or embankment material. Since local scour can occur at the upstream nose of the guidebank, protection in the form of revetment needs to be considered. The design and layout of guidebanks are discussed in FHWA's HEC-20 (Lagasse et al., 2001a).

5.10. Drop Structures

Drop structures are used to provide vertical control of the channel bed. For small drops, vertical or sloping drops may be adequate; for larger drop heights the use of gabion drops or baffled drops may be required. Guidance for the type of drop structure to use is given in AMAFCA's *Design Guide for Riprap-Lined Flood Control Channels* (1983). A method to estimate the depth of scour below a vertical drop structure is presented in Section 3.5.2. Care must be exercised in the design to prevent outflanking or undermining by the flow, and to mitigate lateral bank erosion downstream of the drop structure.

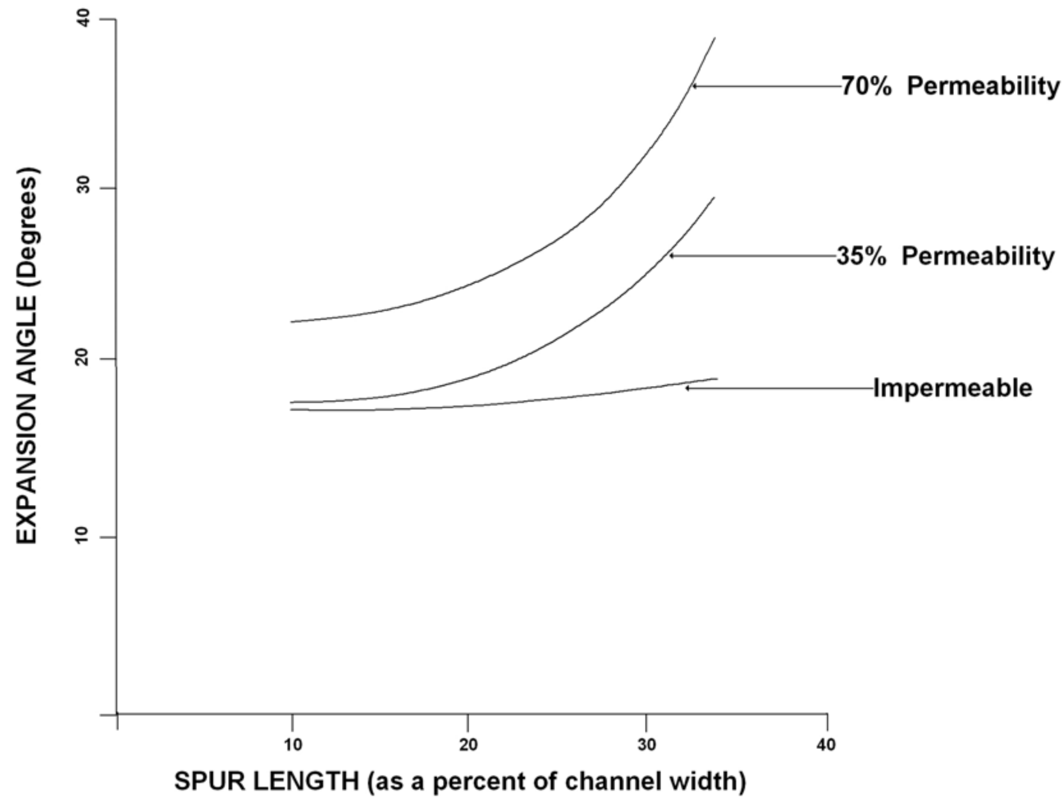


Figure 5.9. Relationship between spur length and expansion angle for several spur permeabilities (after Brown, 1985).

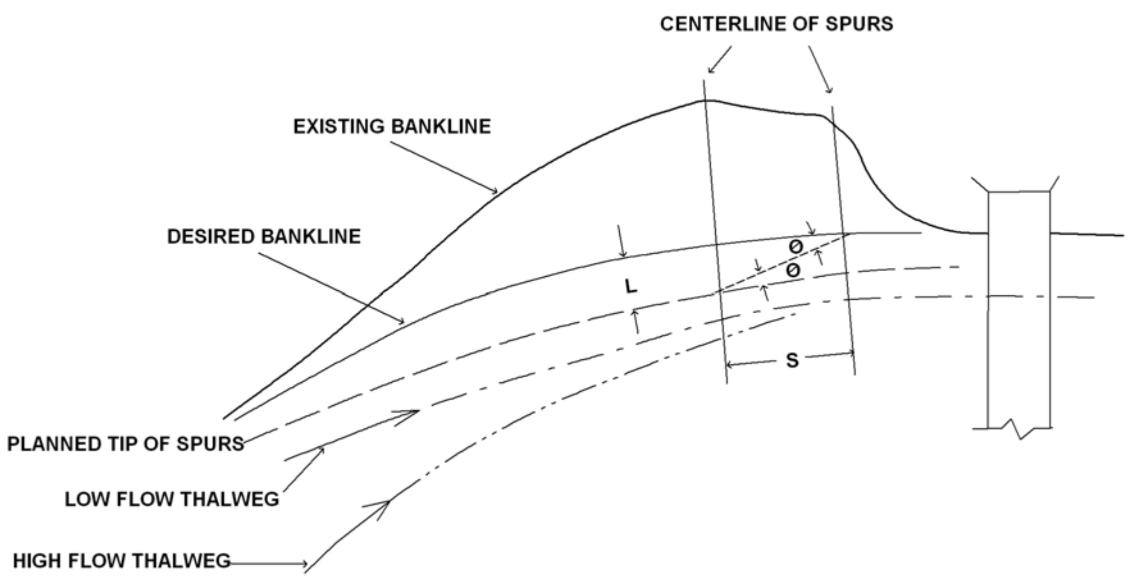


Figure 5.10. Spur spacing in a meander bend (after Brown, 1985).

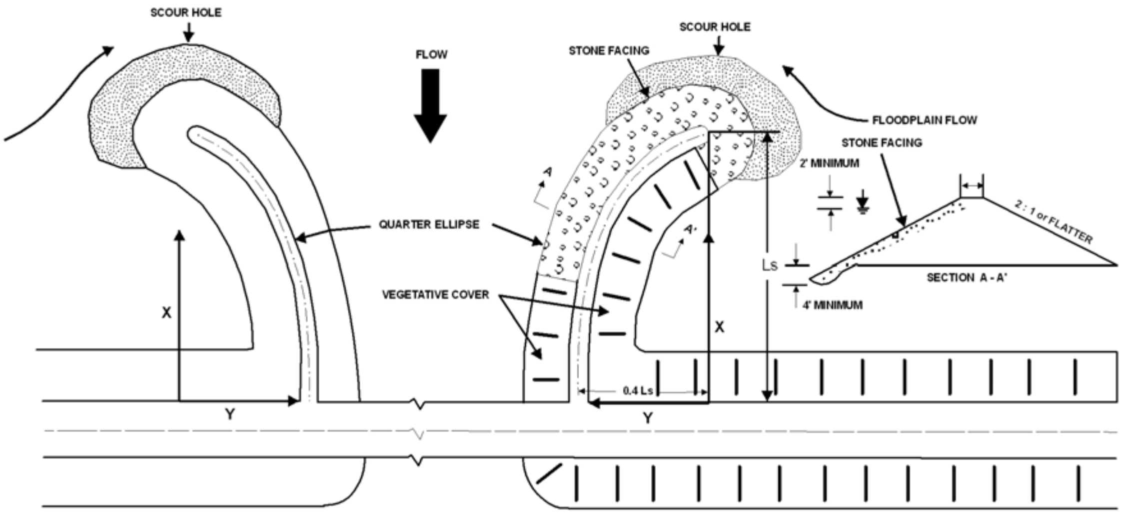


Figure 5.11. Typical guidebank (modified from Bradley, 1978).

5.11. Case Histories of Countermeasure Performance

The Federal Highway Administration has assembled a collection of case histories of hydraulic problems at bridge sites. These case histories provide insight on the performance of a variety of countermeasures and are summarized in this section to provide information on the relative success of the various countermeasures that can be used to stabilize streams. All case histories are taken from Brice and Blodgett (1978a, b); Brice (1984), and Brown et al. (1980). Site data are from Report No. FHWA-RD-78-163 (Brice and Blodgett, 1978a, b). This compilation of case histories at 224 bridge sites in various geographic regions is recommended reference material for those responsible for selecting countermeasures for stream instability. Additional information regarding experience with various countermeasures in the Albuquerque area is provided, where available.

5.11.1. Flexible Revetment

5.11.1.1. Rock Riprap

Dumped rock riprap is the most widely used revetment in the United States. Its effectiveness has been well established where it is of adequate size, of suitable size gradation, and properly installed. Brice and Blodgett (1978a, b) documented the use of rock riprap at 110 sites. They rated the performance at 58 sites and found satisfactory performance at 34 sites, partially satisfactory performance at 12 sites, and failure to perform satisfactorily at 12 sites. Keeley (1971) concluded that riprap used in Oklahoma performed without significant failure and provides basic and efficient bank control on the meandering streams in Oklahoma.

A review of the causes of failure at the sites studied by Brice and Blodgett (1978a, b) is instructive. They found the absence of a filter blanket clearly the cause of the failure at one site. The riprap was placed on a fill of sand and fine gravel which eroded through the interstices of the riprap. Internal slope failure was the cause of failure of riprap at the abutment of bridges at two sites. Inadequate rock size and size gradation were given as the causes of failure at eight sites. All of these sites are complex, and it is difficult to assign failure to one cause, but rock size was definitely a factor.

Channel degradation accounted for failure at three sites in Mississippi. Channel degradation at these sites is due to channel straightening and clearing by the Soil Conservation Service and USACE. Riprap installations on the streambanks, at bridge abutments and in the streambed have failed to stop lateral erosion. At one site, riprap placed on the banks and bed of the stream resulted in severe bed scour and bank erosion downstream of the riprap.

Failure of riprap at one site was attributed to the steep slope on which the riprap was placed. At this site, rock riprap failed to stop slumping of the steep banks downstream of a check dam in a degrading stream.

Successful rock riprap installations at bends were found at five sites. Bank erosion was controlled at these sites by rock riprap alone. Installations rated as failing were damaged at the toe and upstream end, indicating inadequate design and/or construction, and damage to an installation of rounded boulders, indicating inadequate attention to riprap specifications. Other successful rock riprap study sites were sites where bank revetment was used in conjunction with other countermeasures, such as spurs or retards.

The success of these installations was attributed more to the spurs or retards, but the contribution of the bank revetment was not discounted.

5.11.1.2. Broken Concrete

Broken concrete is commonly used in emergencies and where rock is unavailable or very expensive. No specifications were found for its use. Performance was found to be more or less unsatisfactory at three sites. SSCAFCA does not allow the use of broken concrete as riprap for a number of reasons: the appearance is generally unsatisfactory and can attract subsequent dumping, the slab shape of most broken concrete does not provide the interlocking or gradation characteristics of properly designed riprap, and finally, the durability is generally not satisfactory.

5.11.1.3. Rock-and-Wire Mattress and Gabions

The distinction made between rock-and-wire mattress and gabions is in the dimensions of the devices. Rock-and-wire mattress is usually one foot or less in thickness and a gabion is thicker and nearly equi-dimensional. The economic use of rock-and-wire mattress is favored by an arid climate, availability of stones of cobble size, and unavailability of rock for dumped rock riprap. Corrosion of wire mesh is slow in arid climates, and ephemeral streams do not subject the wire to continuous abrasion. Where large rock is not available, the use of rock-and-wire mattress may be advantageous in spite of eventual corrosion or abrasion of the wire.

Rock-and-wire mattress performance was found to be generally satisfactory although local failure of the wire mesh and spilling out of the rock was not uncommon. Mattresses are held in place against the bank by railroad rails at sites in New Mexico and Arizona where good performance was documented. This is known locally as "railbank protection." The steel rail supported rock-and-wire mattress stays in place better than dumped rock riprap on the unstable vertical banks found on the ephemeral streams of this area. Mattress held in place by stakes has been found to be effective in Wyoming.

The use of rock-and-wire mattress has diminished in California because of the questionable service of wire mesh, the high cost of labor for installation, and the efficiency of modern methods of excavating for dumped riprap toe protection. The Los Angeles Flood Control District, however, has had installations in-place for 15 years or more with no evidence of wire corrosion. On the other hand, Montana and Maryland reported abrasion damage of wire. These experiences illustrate that economic use of countermeasures is dependent on the availability of materials, costs, and the stream environment in which the measure is placed.

Several sites were identified where gabions were installed, but the countermeasures had been tested by floods at only one site where gabions placed on the downstream slope of a roadway overflow section performed satisfactorily.

In the Albuquerque area, performance of gabions for bank protection and grade control has generally been satisfactory. Performance problems have related more to the proper design of the filter between the gabion and the in-situ material, rather than with broken or corroded gabion wire. In other urban settings such as Denver, Colorado, vandalism and the use of gabion installations as a source of landscaping materials have been reported as significant problems and the local flood control authority discourages their use.

5.11.1.4. Other Flexible Revetment

Favorable performance of precast-concrete blocks at bridges was reported in Louisiana. Vegetation is reported to grow between blocks and contribute to appearance and stability. Vegetation apparently is seldom used alone at bridges. Iowa relies on sod protection of spur dikes, but Arkansas reported failure of sod as bank protection.

5.11.2. Rigid Revetments

Failure of rigid revetment tends to be progressive; therefore, special precautions to prevent undermining at the toe and termini and failure from unstable soils or hydrostatic pressure are warranted.

5.11.2.1. Concrete Pavement

Well-designed concrete paving is satisfactory as fill slope revetment, as revetment on streams having low gradients, and in other circumstances where it is well protected against undermining at the toe and ends. The case histories include at least one location where riprap launching aprons were successful in preventing undermining at the toe from damaging the concrete pavement revetment. Weep holes for relief of hydrostatic pressure are required for many situations.

Documented causes of failure in the case histories are: undermining at the toe (six sites), erosion at termini (five sites), eddy action at downstream end (two sites), channel degradation (two sites), high water velocities (two sites), overtopping (two sites), and hydrostatic pressure (one site). Good success is reported with concrete slope paving in Florida, Illinois, and Texas.

5.11.2.2. Sacked Concrete

No highway agency reported a general use of sacked concrete as revetment. California was reported to regard this as an expensive revetment almost never used unless satisfactory riprap was not available. Sacked concrete revetment failures were reported from undermining of the toe (two sites), erosion at termini (one site), channel degradation (two sites), and wave action (one site).

5.11.2.3. Concrete-grouted Riprap

Concrete-grouted riprap permits the use of smaller rock, a lesser thickness, and more latitude in gradation of rock than in dumped rock riprap. No failures of grouted riprap were documented in the case histories, but it is subject to the same types of failures as other impermeable, rigid revetments. This is particularly true since grouted riprap has little or no tensile strength and is subject to failure due to the buildup of pore water pressure underneath or by undercutting. In the Albuquerque area, over grouting is a common problem which can produce a much smoother surface than ungrouted riprap. This can adversely affect hydraulic parameters such as roughness and velocity.

5.11.2.4. Concrete-filled Fabric Mat

Concrete-filled fabric mat is a patented product (Fabriform) consisting of porous, pre-assembled nylon fabric forms which are placed on the surface to be protected and then filled with high-strength mortar by injection. Variations of Fabriform and Fabricast consist of nylon bags similarly filled. Successful installations were reported by the manufacturer of Fabriform in Iowa, and North Dakota reported successful installations.

5.11.2.5. Soil Cement

In areas where any type of riprap is scarce, use of in-place soil combined with cement provides a practical alternative. The resulting mixture, soil cement, has been successfully used as bank protection in many areas of the Southwest. Unlike other types of bank revetment, where milder side slopes are desirable, soil cement in a stair-step construction can be used on steeper slopes (i.e., typically one to one), which reduces channel excavation costs. For many applications, soil cement can be more aesthetically pleasing than other types of revetment.

5.11.3. Bulkheads (Erosion Barrier Walls)

A bulkhead is a steep or vertical wall used to support a slope and/or protect it from erosion. Bulkheads usually project above ground, although the distinction between bulkheads and cutoff walls is not always sharp. Most bulkhead applications were found at bridge abutments. They were found to be most useful at the following locations: (1) on braided streams with erodible sandy banks, (2) where banks or abutment fill slopes have failed by slumping, and (3) where stream alignment with the bridge opening was poor, to provide a transition between stream banks and the bridge opening. It was not clear what caused failures at five sites summarized in Brice and Blodgett (1978a, b), but in each case, the probable cause was undermining.

5.11.4. Spurs

Spurs are permeable or impermeable structures which project from the bank into the channel. Spurs may be used to alter flow direction, induce deposition, or reduce flow velocity. A combination of these purposes is generally served. Where spurs project from embankments to decrease flow along the embankment, they are called embankment spurs. These may project into the floodplain rather than the channel, and thus function as spurs only during overbank flow. According to a summary prepared by Richardson and Simons (1984), spurs may protect a stream bank at less cost than riprap revetment. By deflecting current away from the bank and causing deposition, they may more effectively protect banks from erosion than revetment. Uses other than bank protection include the constriction of long reaches of wide, braided streams to establish a stable channel, constriction of short reaches to establish a desired flow path and to increase sediment-transport capacity, and control of flow at a bend. Where used to constrict a braided stream to a narrow flow channel, the structure may be more correctly referred to as a dike or a retard in some locations.

Several factors enter into the performance of spurs, such as permeability, orientation, spacing, height, shape, length, construction materials, and the stream environment in which the spur is placed.

5.11.4.1. Impermeable Spurs

The case histories show good success with well-designed impermeable spurs at bends and at crossings of braided stream channels (eight sites). At one site, hardpoints barely projecting into the stream and spaced at about 100 to 150 feet failed to stop bank erosion at a severe bend. At another site, spurs projecting 40 feet into the channel, spaced at 100 feet, and constructed of rock with a maximum diameter of 1.5 feet experienced erosion between spurs and erosion of the spurs. At a third site, spurs constructed of timber piling filled with rock were destroyed. Failure was attributed to the inability to get enough penetration in the sand-bed channel with timber piles and the unstable wide channel in which the thalweg wanders unpredictably. Spurs (or other countermeasures) are not likely to be effective over the long term in such an unstable channel unless well-designed, well-built, and deployed over a substantial reach of stream.

5.11.4.2. Permeable Spurs

A wide variety of permeable spur designs were also shown to successfully control bank erosion by the case histories. Failures were experienced at a site which is highly unstable with rapid lateral migration, abundant debris, and extreme scour depths. Bank revetments of riprap and car bodies and debris deflectors at bridge piers, as well as bridges, have also failed at this site. At another site, steel H-pile spurs with wire mesh have partially failed on a degrading stream.

5.11.5. Retardance Structures

A retardance structure (retard) is a permeable or impermeable linear structure in a channel, parallel with and usually at the toe of the bank. The purposes of retardance structures are to reduce flow velocity, induce deposition, or to maintain an existing flow alignment. They may be constructed of earth, rock, timber pile, sheet pile, or steel pile, and steel jacks or tetrahedrons (see below) are also used.

Most retardance structures are permeable and most have good performance records. They have proved to be useful in the following situations: (1) for alignment problems very near a bridge or roadway embankment, particularly those involving rather sharp channel bends and direct impingement of flow against a bank (ten sites), and (2) for other bank erosion problems that occur very near a bridge, particularly on streams that have a wandering thalweg or very unstable banks (seven sites).

The case histories include a site where a rock retardance structure similar to a rock toe dike was successful in protecting a bank on a highly unstable channel where spurs had failed. There were, however, deficiencies in the design and construction of the spur installation. At another site, a rock retardance structure similar to a rock toe-dike has reversed bank erosion at a bend in a degrading stream. The USACE reported that longitudinal rock toe dikes were the most effective bank stabilization measure studied for channels having very dynamic and/or actively degrading beds.

5.11.6. Dikes

Dikes are impermeable linear structures for the control or containment of overbank flow. Most are in floodplains, but they may be within channels, as in braided streams or on alluvial fans. Dikes at study sites were used to prevent floodwater from bypassing a bridge at four sites, or to confine channel width and maintain channel alignment at two sites. Performance of dikes at study sites was judged generally satisfactory.

5.11.7. Guidebanks (Spur Dikes)

The major use of guidebanks (or spur dikes) in the United States is to prevent erosion by eddy action at bridge abutments or piers where concentrated flood flow traveling along the upstream side of an approach embankment enters the main flow at the bridge. By establishing smooth parallel streamlines in the approaching flow, guidebanks improve flow conditions in the bridge waterway. Scour, if it occurs, is near the upstream end of the dike away from the bridge. A guidebank differs from dikes described above in that a dike is intended to contain overbank flow while a guidebank only seeks to align overbank flow with flow through the bridge opening. An extension of the usual concept of the purpose for guidebanks, but not in conflict with that concept, is the use of guidebanks and highway fill to constrict braided channels to one channel. At three sites studied Brice and Blodgett (1978a, b), guidebanks only or guidebanks plus revetment on the highway fill were used to constrict wide braided channels rather severely, and the installations have performed well.

5.11.8. Check Dams

A check dam is a low weir or dam across a channel for the control of water stage or velocity, or to stop degradation from progressing upstream. They may be constructed of concrete, rock, sheet pile, rock-and-wire mattress, gabions, or concrete-filled fabric mat. They are usually used to stop degradation in a channel. At one site, however, a check dam was apparently used to inhibit contraction scour in a bridge waterway. The problem with vertical scour was resolved, but lateral scour became a problem and riprap revetment on the streambanks failed (Brice and Blodgett, 1978a, b).

Scour downstream of check dams was found to be a problem at two sites, especially lateral erosion of the channel banks. Riprap placed on the streambanks at the scour holes also failed, at least in part because of the steep slopes on which the riprap was placed. At the time of the study, lateral erosion threatened damage to bridge abutments and highway fills. At another site, a check dam placed at the mouth of a tributary stream failed to stop degradation in the tributary and the delivery of damaging volumes of sediment to the main stream.

No structural failure of check dams was documented. Failures are known to have occurred, however, and the absence of documented failures should not be given undue weight. Failure can occur by bank erosion around the ends of the structure resulting in outflanking; by seepage or piping under or around the structure resulting in undermining and structural or functional failure; by overturning, especially after degradation of the channel downstream of the structure; by bending of sheet pile; by erosion and abrasion of wire fabric in gabions or rock-and-wire mattress; or by any number of structural causes for failure.

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7. GLOSSARY

A

Aggradation—The process of building up a surface by deposition

Alluvial fan—An outspread, gently sloping, fan-shaped alluvial deposit by a stream; especially in an arid or semiarid region where a stream issues from a narrow canyon onto a plain or valley floor.

Alluvial terrace—A step-like bench composed of alluvium.

Alluvium--Deposits of clay, silt, sand, gravel, or other particulate material that has been deposited by a stream or other body of running water in a streambed, on a flood plain, on a delta, or at the base of a mountain.

Antidune—A transient form of ripple on a streambed analogous to a sand dune.

Aquifer--A geologic formation(s) that is water bearing. A geological formation or structure that

Armor layer—A coarse layer of sediment protecting the finer sediment beneath it.

Avulsion—Relatively rapid change of channel location.

B

Bed load—The part of a stream's load that is moved on or immediately above the streambed by sliding, rolling, or saltation.

Bedforms—Sedimentary features formed on the bed of a stream.

C

Contraction scour—Channel scour as a result of narrowing of a channel.

Critical bank height—The bank height at which failure occurs.

D

Degradation—The general lowering of the surface of the land by erosive processes.

Deposition—The laying down of rock-forming material by any natural agent.

Dominant discharge—The channel-forming discharge.

E

Equilibrium slope—The slope at which neither aggradation or degradation occurs.

Erosion--The process in which a material is worn away by a stream of liquid (water) or air, often due to the presence of abrasive particles in the stream.

F

Froude Number—Ratio of inertial forces to gravitational forces in a flowing fluid.

Floodplain--A strip of relatively flat and normally dry land alongside a stream, river, or lake that is covered by water during a flood.

Flood stage--The elevation at which overflow of the natural banks of a stream or body of water begins in the reach or area in which the elevation is measured.

G

Gabions—Rectangular wire boxes filled with stone.

Geomorphology—The science that treats the general configuration of the earth's surface; the study of the classification, description, nature, origin, and development of landforms and their relationships to underlying structures and the history of geologic changes as recorded by these surface features.

H

Hydraulic jump—In fluid flow, a change in flow condition accompanied by a stationary, abrupt turbulent rise in water level in the direction of flow.

I

Incipient motion—The initiation of sediment movement in a stream

infiltration--Flow of water from the land surface into the subsurface.

L

Lateral migration—The lateral shift of a river.

LEE Line—Lateral envelope line representing the outer limits to which a channel is anticipated to shift perpendicular to the downstream axis due to a lateral erosion within an engineering time frame.

Levee--A natural or manmade earthen barrier along the edge of a stream, lake, or river. Land alongside rivers can be protected from flooding by levees.

Local scour—Erosion caused by an abrupt change of flow duration or velocity.

M

Manning's n—The coefficient of roughness accounting for energy loss due to friction in a stream channel used in the Manning uniform flow equation (units are sec/ft$^{1/3}$ in US customary system).

Meander—One of a series of sinuous curves or loops in the course of a mature stream, produced as the stream swings from side to side in flowing across its floodplain or shifts its course laterally toward the convex side of an original curve.

N

Nickpoint—Any interruption of break of slope, especially a point of abrupt change or inflection in the longitudinal profile of a stream or of its valley.

P

Pier scour—Scour occurring at a pier.

Permeability--The ability of a material to allow the passage of a liquid, such as water through rocks. Permeable materials, such as gravel and sand, allow water to move quickly through them, whereas unpermeable material, such as clay, don't allow water to flow freely.

Porosity--A measure of the water-bearing capacity of subsurface rock. With respect to water movement, it is not just the total magnitude of porosity that is important, but the size of the voids and the extent to which they are interconnected, as the pores in a formation may be open, or interconnected, or closed and isolated. For example, clay may have a very high porosity with respect to potential water content, but it constitutes a poor medium as an aquifer because the pores are usually so small.

R

Revetment—A facing of resistant material protecting the bank of a river.

Riprap—Large fragments of broken rock thrown together irregularly (as offshore or on a soft bottom) or fitted together (as on the downstream face of a dam).

S

Scour hole—The depression formed by the removal of bed sediment by the action of moving water.

Sediment--Usually applied to material in suspension in water or recently deposited from suspension. In the plural the word is applied to all kinds of deposits from the waters of streams, lakes, or seas.

Sediment Bulking factor—Multiplier applied to the clear-water discharge to account for the increase in volume due to entrained sediment.

Suspended sediment--Very fine soil particles that remain in suspension in water for a considerable period of time without contact with the bottom. Such material remains in suspension due to the upward components of turbulence and currents and/or by suspension.

Suspended-sediment concentration--The ratio of the mass of dry sediment in a water-sediment mixture to the mass of the water-sediment mixture. Typically expressed in milligrams of dry sediment per liter of water-sediment mixture.

Suspended-sediment discharge--The quantity of suspended sediment passing a point in a stream over a specified period of time. When expressed in tons per day, it is computed by multiplying water discharge (in cubic feet per second) by the suspended-sediment concentration (in milligrams per liter) and by the factor 0.0027.

Sediment continuity—Balance between the sediment supply and sediment transport capacity.

Sediment yield—The total sediment outflow from a drainage basin.

Shear stress—That component of stress which acts tangential to a plane through any given point in a body; any of the tangential components of the stream tensor.

Shields parameter—A number referred to as a dimensionless shear stress.

Sinuosity—The ratio of channel length to valley length.

Spur—A ridge that projects sharply from the crest or side of a mountain; a hill extending from a prominent range of hills or mountains.

Supercritical flow—A system that is at a temperature higher than its critical temperature.

Superelevation—The tendency of the water surface to rse on the outside of a bend due to centrifugal forces.

T

Trap efficiency—Proportion of sediment inflow to a stream reach or reservoir that is retained within that reach or reservoir.

W

Watershed--The land area that drains water to a particular stream, river, or lake. It is a land feature that can be identified by tracing a line along the highest elevations between two areas on a map, often a ridge. Large watersheds, like the Mississippi River basin contain thousands of smaller watersheds.

Wash load—The part of the total stream load that is carried for a considerable period of time in suspension, free from contact with the streambed and consists mainly of clay, silt and sand.

Width-depth ratio—The ratio of channel width to channel depth.

Y

Yield--Mass per unit time per unit area

APPENDIX A
Relationship of Sediment Yield to Other Analysis

APPENDIX A
Relationship of Sediment Yield to Other Analysis

A.1. Introduction

The total sediment yield at a specific location along an arroyo consists of sediment delivered to the channel from overland areas and sediment eroded from the arroyo boundaries by the flowing water. The most accurate method of estimating the quantity of sediment eroded from the channel boundary involves bed-material transport capacity computations. The amount of sediment derived from overland areas can be estimated using empirical relationships involving various parameters that describe the characteristics and condition of the watershed, or from applicable sediment yield data from existing retention structures. **Watershed sediment yield computations are typically performed to estimate the fine sediment (or wash load) component of the total sediment yield.**

Delivery of sediment from overland areas to the arroyo channels can be a significant consideration in planning and designing erosion and flood-protection measures. Urbanization tends to increase runoff and, over the long-term, to decrease sediment yield which increases the tendency of arroyos to incise and erode laterally. In the short- to mid-term, this potentially destabilizing influence can result in a net increase in the amount of sediment delivered to downstream reaches. Conversely, disturbance of the land surface during construction or the absence of natural erosion protection in the watershed area can significantly increase the quantity of sediment reaching the arroyo compared to natural conditions. In either case, the capacity and effectiveness of flood detention structures can be substantially reduced and the lateral erosion potential of the arroyo can increase.

A.2. Description of Sediment Yield Processes

Watershed sediment yield in arid regions such as the SSCAFCA jurisdictional area, results from two primary processes: (1) sheet wash (which includes rilling), and (2) gullying.

Sheet wash is largely a function of raindrop detachment and transport by overland flow. The susceptibility of an area to erosion by these processes is directly related to the type of soil, amount of protection by vegetation or other types of surface cover and the steepness of the land slope. Overland erosion usually results in the delivery of relatively fine sediment to the stream channel. This material is carried mostly in suspension (wash load). Limited quantities of wash load normally do not pose serious problems for the stability of the arroyo channels. However, if the quantity of wash load being carried by the flows is significant (greater than approximately 20,000 ppm by weight), the bed material transport capacity will be higher than under low-concentration conditions, resulting in increased erosion in the arroyo channel and deposition in flood-detention structures.

Gullying results from the concentration of overland flows into small headward channels, which can result in delivery of large quantities of coarser particles to the arroyo. The coarser sand and gravel is carried as bed-material load, and can have significant implications regarding the vertical and lateral stability of the arroyo and sediment deposition in flood detention structures.

Most of the sand and coarser sediment delivered to in-channel detention structures is derived from bed and bank erosion within the arroyo upstream of the structures. The transport rate, and thus sediment yield, varies considerably depending on the stage of adjustment of the arroyo (Figure 1.1), with the highest rates occurring by incision and widening during Class III and IV of the arroyo development process (Schumm et al., 1987; Begin, 1979). Sediment delivery to in-channel structures is related to the hydraulic conditions in the arroyo. In designing the storage capacity of detention and sedimentation basins, the estimated sediment yield should include both the amount of fine sediment delivered from the watershed and the amount of bed material that is controlled by the transport capacity of the arroyo.

A.3. Methods for Estimating Sediment Yield

Accurate quantification of the sediment yield, either on an annual basis or in response to individual storms is, at best, a difficult problem. Evaluation of the watershed sediment yield first requires a qualitative evaluation of the sediment sources in the watershed and the types of erosion that are prevalent. Natural Resource Conservation Service (NRCS) soil surveys are a valuable source of data for quantifying watershed sediment yields (e.g., SCS, 1977 for the SSCAFCA jurisdictional area). Other sources include maps, drilling logs, reservoir records, climate records and, of course, field observations.

Available methods for estimating sediment yield include a watershed rating procedure developed by the Pacific Southwest Interagency Committee (PSIAC, 1968), the Universal Soil Loss Equation (Wischmeier and Smith, 1965; 1978), and the Modified Universal Loss Equation (Williams and Berndt, 1972). These methods have been tested under a variety of conditions with mixed success.

The PSIAC method provides a general guide to estimating total sediment yields based on the climatic and physical characteristics of the watershed. This method is intended for broad planning purposes only. As shown in **Table A.1**, the method predicts a range of annual sediment yields that may be expected based on a watershed rating system. Shown (1970) and Renard (1980) tested the PSIAC method against sediment yields measured in ponds and dams located in the Southwestern U.S. with contributing watersheds of less than approximately 20 square miles. They showed a strong correlation between estimates of the annual sediment yield using the PSIAC method and measured annual yield. The details of the rating system are presented in **Appendix A.1**.

Table A.1. Summary of PSIAC classifications.

Classification	Rating	Sediment Yield (annual)	
		(ac-ft/sq mi)	(tons/ac)*
1	>100	3.0	10.2
2	75-100	1.0-3.0	3.4-10.2
3	50-75	0.5-1.0	1.7-3.4
4	25-50	0.2-0.5	0.7-1.7
5	0-25	<0.2	<0.7

* Assuming bulked unit weight of sediment = 100 pcf (1 ac-ft/mi^2 = 3.4 t/ac)

The Universal Soil Loss Equation (USLE) is a widely used empirical relationship based on runoff and soil loss data from agricultural land. The USLE relates annual soil loss due to sheet-and-rill erosion to the product of six factors describing rainfall energy, soil erodibility, cropping and management, supplemental erosion-control practices such as contouring and terracing, and a topographic factor involving the steepness and length of the overland slope. Applicability of the USLE to the SSCFACA jurisdictional area is limited since the original data upon which it is based are largely from the Central and Eastern U.S. where the precipitation patterns and land characteristics are significantly different. In the arid southwest, runoff-producing precipitation usually occurs in the form of high-intensity, short-duration thunderstorms that cannot be completely incorporated into the equation. In addition, the weathering process caused by the wind and sun between storms is much more severe in arid areas, which can increase the supply of easily erodible material. An additional drawback of the USLE is the need to define the sediment delivery ratio to estimate the amount of sediment eroded from the watershed that actually reaches a given point in the channel.

The Modified Universal Loss Equation (MUSLE) was developed to estimate sediment yields from watersheds based on single storms. The equation, as presented by Williams and Berndt (1972), differs from the USLE by inclusion of a runoff factor in place of the rainfall energy factor. Since it directly considers the runoff associated with individual storms, it is more applicable to the ephemeral streams found in the arid southwest where runoff and sediment delivery to the channel system is primarily the result of high-intensity thunderstorms. The MUSLE relationship is given by:

$$Y_S = \alpha (Vq_p)^{\beta} \ KLSCP \tag{A.1}$$

where Y_s is the sediment yield for the storm in tons, K is the soil erodibility factor, LS is the topographic factor representing the combination of slope length and slope gradient, C is the cover and management factor, P is the erosion-control practice factor, V is the runoff volume for the storm in acre-feet, and q_p is the peak discharge of the storm in cfs. Values for α and β can be derived through calibration when sufficient data are available. Once calibrated, the MUSLE can be used to evaluate the effect of watershed modifications on the sediment yield associated with a given storm event. The most commonly used values for α and β are 95 and 0.56, respectively. These values were derived using data from experimental watersheds in Texas and Nebraska that are very different from those in the SSCAFCA area.

Although sufficient data are not available to re-derive these coefficients for the SSCAFCA area, although results obtained using the above values appear to be low in comparison with observed sediment yields. From analyses of several watersheds in the area and data from existing flood detention dams on tributaries to the middle and lower Rio Grande, it appears that the fine sediment yield is about three times higher than predicted using the standard values. It is, therefore, recommended that α be increased to 285 for use in the SSCAFCA area.

The MUSLE equation was originally developed to represent the total watershed sediment yield. Since the bed material component of the total sediment load (i.e., sand and gravel) can be more accurately estimated using bed material transport capacity calculations, the MUSLE should be used to estimate only the fine sediment (wash load) yield. The total sediment yield is then computed as the sum of the wash load yield estimated from MUSLE and the computed bed material load.

Based on the above discussion, the recommended form of the MUSLE equation for estimating wash load (i.e., fine sediment) yield in the SSCAFCA area is given by:

$$Y_S = 285 \ (Vq_p)^{0.56} \ KLSCPP_f \tag{A.2}$$

where P_f is the percentage of watershed soils in the silt and clay size-range. If additional site-specific data become available in the future, it is recommended that the values of both α and β be re-evaluated and adjusted accordingly. If the watershed contains a substantial amount of impervious area, such as roads, parking lots and building that are not subject to overland erosion, the wash-load yield should also be reduced by the percentage of impervious (noncontributing) area

A detailed description of MUSLE is presented in **Appendix A.2**.

A.4. Available Sediment Yield Data

Several sources of data are available with which to verify the above relationships. These include publications by various government agencies, consultant reports, and AMAFCA records of sediment removal from detention ponds and channels.

Table A.2 contains a summary of the available sediment yield data for the Southwestern U.S. from reports by a variety of government agencies and consultants. A more detailed discussion of these data is presented in "Unser Bridge/Calabacillas Arroyo Detention Basin" prepared for AMAFCA by RCI in 1989. The data from SCS (1936) and USGS (1952) are for relatively small watersheds similar in size to the individual drainages in the AMAFCA area of concern (less than 5 to 10 square miles). The majority of the remaining data are for larger systems with drainage areas of several hundred square miles. Due to the high variability in sediment yield data and the effect of local conditions, the values contained in Table A.2 should only be used as a guide.

Using a combination of bed-material transport calculations and wash-load estimates, RCI (1989) estimated the annual sediment yields for Calabacillas Arroyo to be about 2 tons/acre for undeveloped conditions and about 1.5 tons/acre for fully developed conditions.

AMAFCA silt-haul records were used by Bohannan Huston (1990; 1991) to supplement other available information to estimate sediment yields for Black and Ladera Arroyos. These records indicate that about 3.4 tons/acre/year were removed from Black Arroyo between 1981 to 1989 and about 1 ton/acre/year was removed from the Ladera system between 1977 and 1989. Based on the AMAFCA records and the above data, Bohannan-Huston estimated the total annual sediment yields for the Ladera system to be about 1.7 tons/acre/year.

Table A.2. Summary of regional sediment yield data.

Source	Location	Sediment Yield Range (tons/ac/yr)
ARS, 1964	Central New Mexico	1.4-7.3*
Bondurant, 1951	Conchas Reservoir	0.1-4.3
Curtis, 1976	New Mexico	0.2-31.0
Norman, 1968	Rio Grande, Rio Chama Rivers	0.3-1.0
SCS, 1936	Rio Grande, Pecos, Zuni Rivers	0.3-4.1
USGS, 1952	Navajo Indian Reservation	0.03-4.1
USGS, 1982	San Juan River	1.0-1.2
Resource Consultants & Engineers, Inc. (1993)	North Diversion Channel and Tributaries, Albuquerque (1982-1990)	0.05 to 1.2 (Average=0.7)
Resource Technologies, Inc 1996	Rio Grande Tributaries, Caballo Dam to El Paso	0.2-5.3
Mussetter Engineering, Inc., 2006	Rio Grande Tributaries above Las Cruces Dam	0.3-0.7
Mussetter Engineering, Inc., 2007	Rio Grande Tributaries, El Paso to Ft Quitman	0.5-2.6

* Does not include two outliers which are clearly unrepresentative.

AMAFCA silt haul records were used by RCE (1993) to estimate sediment yields from tributaries to the North Diversion Channel (NDC), as well as the sediment yield at the NDC outfall for the period from 1980 and 1992. These data are derived from maintenance supervisor records, based on the number of truck loads of material hauled from the various locations by either AMAFCA crews or contractors. These data show that approximately 0.7 tons/acre of sediment was removed from the NDC during the 13-year period, of which 0.1 to 0.2 tons/acre were finer than sand-sized material. If the trap efficiency of the overall system is assumed to be greater than 90 percent, which is believed to be reasonable, the average annual yield of sand and coarser material was about 0.6 to 0.7 tons/acre over the approximately 101 square-mile watershed during this period. A significant reduction in sediment delivery to the NDC system appears to have occurred after 1982; the average amount of sediment removed from the system was only about 0.4 tons/acre from 1983 to 1992 (**Figure A.1**).

As discussed in Section 3.1, arroyo evolution follows a predictable pattern from unincised swales through an incision and widening process to a more stable, near equilibrium condition. The above data applies primarily to arroyos that have progressed to the latter, more stable stage of development. During the incision phase, sediment yields can be extremely high due to the unstable condition of the arroyo. Watson et al. (1986) found that approximately 75 percent of the sediment yield from incising channels in the Southeastern U.S. was derived from the channel bed and banks. Nickpoints (i.e., abrupt oversteepening of the channel profile) typically occur in incising channels, and they migrate upstream through erosion of the oversteepened area, lowering the channel bed (**Figure A.2**). This process can cause significant increases in sediment load. The associated deepening of the channel bed may also induce bank instability and erosion, further increasing the sediment load. A similar increase in sediment load can occur through incision caused by release of essentially clear water from a detention basin or culvert crossing with upstream backwater. In either case, the local sediment yield may be many times greater than the regional average until the incising reach adjusts to a more stable form.

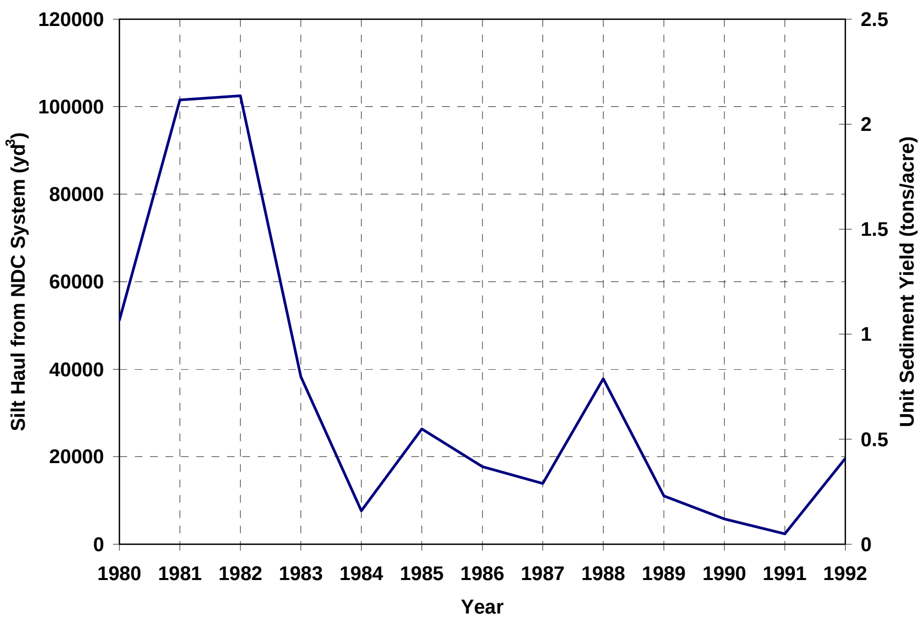

Figure A.1. Quantity of sediment removed from the system by year.

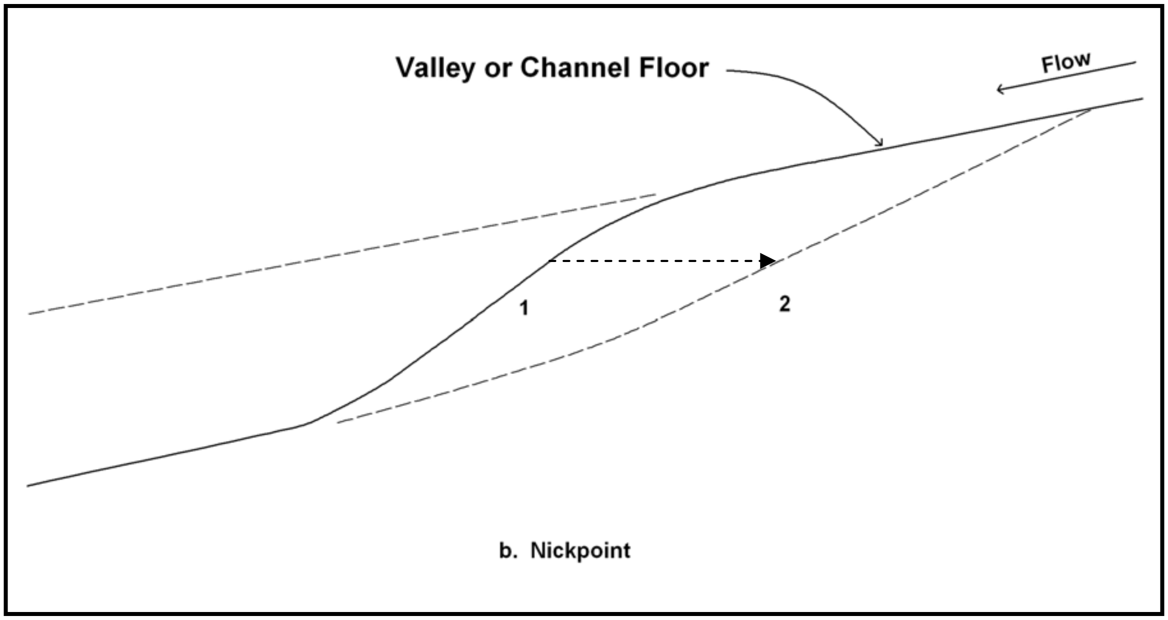

Figure A.2. Schematic diagram of a nickpoint. 1=Initial condition; 2=condition after nickpoint migration.

APPENDIX A.1

Pacific Southwest Inter-Agency Committee (PSIAC) Method For Predicting Watershed Soil Loss

APPENDIX A.1

PACIFIC SOUTHWEST INTER-AGENCY COMMITTEE (PSIAC) METHOD FOR PREDICTING WATERSHED SOIL LOSS

Note: The information presented in this appendix is from the following source:
Pacific Southwest Inter-Agency Committee, 1968. Report of the Water Management Subcommittee on Factors Affecting Sediment Yield in the Pacific Southwest Area and Selection and Evaluation of Measures for Reduction of Erosion and Sediment Yield. October.

Introduction

The material that follows is suggested for use in the evaluation of sediment yield in the Pacific Southwest. It is intended as an aid to the estimation of sediment yield for the variety of conditions encountered in this area.

The classifications and companion guide material are intended for broad planning purposes only, rather than for specific projects where more intensive investigations of sediment yield would be required. For these purposes it is recommended that map delineations be for areas no smaller than 10 square miles.

It is suggested that actual measurements of sediment yield be used to the fullest extent possible. This descriptive material and the related numerical evaluation system would best serve its purpose as a means of delineating boundaries between sediment yield areas and in extrapolation of existing data to areas where none is available.

This may involve a plotting of known sediment yield data on work maps. Prepared materials such as geologic and soil maps, topographic, climatic, vegetative type and other references would be used as aids in delineation of boundaries separating yield classifications. A study of the general relationships between known sediment yield rates and the watershed conditions that produce them would be of substantial benefit in projecting data to areas without information.

Sediment Yield Classification

It is recommended that sediment yields in the Pacific Southwest area be divided into five classes of average annual yield in acre-feet per square mile. These are as follows:

Classification	1	> 3.0	acre-feet/square mile
	2	1.0 - 3.0	" "
	3	0.5 - 1.0	" "
	4	0.2 - 0.5	" "
	5	< 0.2	" "

Nine factors are recommended for consideration in determining the sediment yield classification. These are geology, soils, climate, runoff, topography, ground cover, land use, upland erosion, and channel erosion and sediment transport.

Characteristics of each of the nine factors which give that factor high, moderate, or low sediment yield level are shown on Table A-1. The sediment yield characteristic of each factor is assigned a numerical value representing its relative significance in the yield rating. The yield rating is the sum of values for the appropriate characteristics for each of the nine factors. Conversion to yield classes should be as follows:

Rating	Class
> 100	1
75 - 100	2
50 - 75	3
25 - 50	4
0 - 25	5

Guidelines which accompany the table are an integral part of the procedure. They describe the characteristics of factors which influence sediment yield and these are summarized in the space provided on the table.

The factors are generally described, for purposes of avoiding complexity, as independently influencing the amount of sediment yield. The variable impact of any one factor is the result of influence by the others. To account for this variable influence in any one area would require much more intensive investigational procedures than are available for broad planning purposes.

To briefly indicate the interdependence of the factors discussed separately, ground cover is used as an example. If there is no vegetation, litter or rock fragments protecting the surface, the rock, soil, and topography express their uniqueness on erosion and sediment yield. If the surface is very well protected by cover, the characteristics of the other factors are obscured by this circumstance. In similar vein, an arid region has a high potential for erosion and sediment yield because of little or no ground cover, sensitive soils and rugged topography. Given very low intensity rainfall and rare intervals of runoff, the sediment yield could be quite low.

Each of the 9 factors shown on Table A-1 are paired influences with the exception of topography. That is, geology and soils are directly related as are climate and runoff, ground cover and land use, and upland and channel erosion. Ground cover and land use have a negative influence under average or better conditions. Their impact on sediment yield is therefore indicated as a negative influence when affording better protection than this average.

It is recommended that the observer follow a feedback process whereby he checks the sum of the values on the table from A through G with the sum of H and I. In most instances high values in the former should correspond to high values in the latter. If they do not, either special erosion conditions exist or the A through G factors should be re-evaluated.

Although only the high, moderate and low sediment yield levels are shown on the attached table, interpolation between these levels may be made.

Surface Geology

Over much of the southwest area, the effect of surface geology on erosion is readily apparent. The weaker and softer rocks are more easily eroded and generally yield more sediment than do the harder more resistant types. Sandstones and similar coarse-textured rocks that disintegrate to form permeable soils erode less than shales and related mudstones and siltstones under the same conditions of precipitation. On the other hand, because of the absence of cementing agents in some soils derived from sandstone, large storms may produce some of the highest sediment yields known.

The widely distributed marine shales, such as the Mancos and shale members of the Moenkopi Formation, constitute a group of highly erodible formations. The very large areal extent of the shales and their outwash deposits gives them a rank of special importance in relation to erosion. Few of the shale areas are free from erosion. Occasionally, because of slope or cover conditions, metamorphic rocks and highly fractured and deeply weathered granites and granodiorites produce high sediment yield. Limestone and volcanic outcrop areas are among the most stable found within the western lands. The principal reason for this appears to be the excellent infiltration characteristics, which allow most precipitation to percolate into the underlying rocks.

In some areas, all geologic formations are covered with alluvial or colluvial material which may have no relation to the underlying geology. In such areas the geologic factor would have no influence and should be assigned a value of O in the rating.

Soils

Soil formation in the Pacific Southwest generally has not had climatic conditions conducive to rapid development. Therefore, the soils are in an immature stage of development and consist essentially of physically weathered rock materials. The presence of sodium carbonate (black alkali) in a soil tends to cause the soil particles to disperse and renders such a soil susceptible to erosion.

There are essentially three inorganic properties--sand, silt, and clay-which may in any combination give soil its physical characteristics. Organic substances plus clay provide the binding material which tends to hold the soil separates together and form aggregates. Aggregate formation and stability of these aggregates are the resistant properties of soil against erosion. Unstable aggregates or single grain soil materials can be very erodible.

Climate and living organisms acting on parent material, as conditioned by relief or topography over a period of time, are the essential factors for soil development. Any one of these factors may overshadow or depress another in a given area and cause a difference in soil formation. For instance, climate determines what type of vegetation and animal population will be present in an area, and this will have a definite influence or determine the type of soil that evolves. As an example, soils developing under a forest canopy are much different from soils developing in a grassland community.

The raw, shaley type areas (marine shales) of the Pacific Southwest have very little, if any, solid development. Colluvial-alluvial fan type areas are usually present at the lower extremities of the steeper sloping shale areas. Infiltration and percolation are usually minimal on these areas due to the fine textured nature of the soil material. This material is easily dispersed and probably has a high shrink-swell capacity. Vegetation is generally sparse, and consists of a salt desert shrub type.

There are areas that contain soils with definite profile development, and also, stony soils that contain few fines, which constitutes an improved physical condition for infiltration and plant growth over the fine textured shaley areas. These areas usually occur at higher and more moist elevations where bare, hard crystalline rocks provide the soil parent material. Vegetation and other ground cover, under these circumstances, provide adequate protection against the erosive forces and thus low sediment yield results.

In arid and semi-arid areas, an accumulation of rock fragments (desert pavement) or calcareous material (caliche) is not uncommon. These layers can offer substantial resistance to erosion processes.

The two extreme conditions of sediment yield areas have been described. Intermediate situations would contain some features of the two extremes. One such situation might be an area of predominately good soil development that contains small areas of badlands. This combination would possibly result in an intermediate classification.

Climate and Runoff

Climatic factors are paramount in soil and vegetal development and determine the quantity and discharge rate of runoff. The same factors constitute the forces that cause erosion and the resultant sediment yield. Likewise, temperature, precipitation, and particularly the distribution of precipitation during the growing season, affect the quantity and quality of the ground cover as well as soil development. The quantity and intensity of precipitation determine the amount and discharge rates of runoff and resultant detachment of soil and the transport media for sediment yield.

The intensity of prevailing and seasonal winds affects precipitation pattern, snow accumulation and evaporation rate.

Snow appears to have a minor effect on upland slope erosion since raindrop impact is absent and runoff associated with snow melt is generally in resistant mountain systems.

Frontal storms in which periods of moderate to high intensity precipitation occur can produce the highest sediment yields within the Southwest. In humid and subhumid areas the impact of frontal storms on sediment may be greatest on upland slopes and unstable geologic areas where slides and other downhill soil movement can readily occur.

Convective thunderstorm activity in the Southwest has its greatest influence on eroison (sic) and sedimentation in Arizona and New Mexico and portions of the adjoining states. High rainfall intensities on low density cover or easily dispersed soils produces high sediment yields. The average annual sediment yield is usually kept within moderate bounds by infrequent occurrence of thunderstorms in any one locality.

High runoff of rare frequency may cause an impact on average annual sediment yield for a long period of time in a watershed that is sensitive to erosion, or it may have little effect in an insensitive watershed. For example, sediment that has been collecting in the bottom of a canyon and on side slopes for many years of low and moderate flows may be swept out during the rare event, creating a large change in the indicated sediment yield rate for the period of record.

In some areas the action of freezing and thawing becomes important in the erosion process. Impermeable ice usually forms in areas of fine textured soils where a supply of moisture is available before the advent of cold weather. Under these conditions the ice often persists throughout the winter and is still present when the spring thaw occurs. In some instances water tends to run over the surface of the ice and not detach soil particles, but it is possible for the ice in a surface layer to thaw during a warm period and create a very erodible situation. Spring rains with ice at shallow depth may wash away the loose material on the surface.

In some areas of the Pacific Southwest, particularly those underlain by marine shale, freezing and thawing alters the texture of soil near the surface, and thus changes the infiltration characteristics. These areas generally do not receive enough snow or have cold enough temperatures to build a snow pack for spring melt. Later in the year soil in a loosened condition is able to absorb a large part of the early rainfall. As rains occur during the summer, the soil becomes compacted on the surface, thus allowing more water to run off and affording a greater chance for erosion.

Topography

Watershed slopes, relief, floodplain development, drainage patterns, orientation and size are basic items to consider in connection with topography. However, their influence is closely associated with geology, soils, and cover.

Generally, steep slopes result in rapid runoff. The rimrock and badlands, common in portions of the Pacific Southwest, consist of steep slopes of soft shales usually maintained by the presence of overlying cap rock. As the soft material is eroded, the cap rock is undercut and falls, exposing more soft shales to be carried away in a continuing process. However, high sediment yields from these areas are often modified by the temporary deposition of sediment on the intermediate floodplains.

The high mountain ranges, although having steep slopes, produce varying quantities of sediment depending upon the type of parent materials, soil development, and cover which directly affect the erosion processes.

Southerly exposed slopes generally erode more rapidly than do the northerly exposed slopes due to greater fluctuation of air and soil temperatures, more frequent freezing and thawing cycles, and usually less ground cover.

The size of the watershed may or may not materially affect the sediment yield per unit area. Generally, the sediment yield is inversely related to the watershed size because the larger areas usually have less overall slope, smaller proportions of upland sediment sources, and more opportunity for the deposition of upstream derived sediments on floodplains and fans. In addition, large watersheds are less affected by small convective type storms. However, under other conditions, the sediment yield may not decrease as the watershed size increases. There is little change in mountainous areas of relatively uniform terrain. There may be an increase of sediment yield as the watershed size increases if downstream watersheds or channels are more susceptible to erosion than upstream areas.

Ground Cover

Ground cover is described as anything on or above the surface of the ground which alters the effect of precipitation on the soil surface and profile. Included in this factor are vegetation, litter, and rock fragments. A good ground cover dissipates the energy of rainfall before it strikes the soil surface, delivers water to the soil at a relatively uniform rate, impedes the flow of water, and promotes infiltration by the action of roots within the soil. Conversely, the absence of ground cover, whether through natural growth habits or the effect of overgrazing or fire, leave the land surface open to the worst effects of storms.

In certain areas, small rocks or rock fragments may be so numerous on the surface of the ground that they afford excellent protection for any underlying fine material. These rocks absorb the energy of falling rain and are resistant enough to prevent cutting by flowing water.

The Pacific Southwest is made up of land with all classes of ground cover. The high mountain areas generally have the most vegetation, while many areas in the desert regions have practically none. The abundance of vegetation is related in a large degree to precipitation. If vegetative ground cover is destroyed in areas where precipitation is high, abnormally high erosion rates may be experienced.

Differences in vegetative type have a variable effect on erosion and sediment yield, even though percentages of total ground cover may be the same. For instance, in areas of pinyon-juniper forest having the same percentage of ground cover as an area of grass, the absence of understory in some of the pinyon-juniper stands would allow a higher erosion rate than in the area of grass.

Land Use

The use of land has a widely variable impact on sediment yield, depending largely on the susceptibility of the soil and rock to erosion, the amount of stress exerted by climatic factors and the type and intensity of use. Factors other than the latter have been discussed in appropriate places in this guide.

In almost all instances, use either removes or reduces the amount of natural vegetative cover which reflects the varied relationships within the environment. Activities which remove all vegetation for parts of each year for several years, or permanently, are cultivation, urban development, and road construction. Grazing, logging, mining, and fires artifically (sic) induce permanent or temporary reduction in cover density.

High erosion hazard sites, because of the geology, soils, climate, etc., are also of high hazard from the standpoint of type and intensity of use. For example, any use which reduces cover density on a steep slope with erodible soils and severe climatic conditions will strongly affect sediment yield. The extent of this effect will depend on the area and intensity of use relative to the availability of sediment from other causes. Construction of road or urban development with numerous cut and fill slopes through a large area of widespread sheet or gully erosion will probably not cause a change in sediment yield classification. Similar contruction (sic) and continued disturbance in an area of good vegetative response to a favorable climate can raise yield by one or more classifications.

Use of the land has its greatest potential impact on sediment yield where a delicate balance exists under natural conditions. Alluvial valleys of fine, easily dispersed soils from shales and sandstones are highly vulnerable to erosion where intensive grazing and trailing by livestock have occurred. Valley trenching has developed in many of these valleys and provides a large part of the sediment in high yield classes from these areas.

A decline in vegetative density is not the only effect of livestock on erosion and sediment yield. Studies at Badger Wash, Colorado, which is underlain by Mancos shale, have indicated that sediment yield from ungrazed watersheds is appreciably less than from those that are grazed. This difference is attributed to the absence of soil trampling in the ungrazed areas, since the density of vegetation has not noticeably changed since exclusion began.

Areas in the arid and semi-arid portions of the Southwest that are surfaced by desert pavement are much less sensitive to grazing and other use, since the pavement affords a substitute for vegetative cover.

In certain instances the loss or deterioration of vegetative cover may have little noticeable on-site impact but may increase off-site erosion by acceleration of runoff. This could be particularly evident below urbanized areas where accelerated runoff from pavement and rooftops has increased the stress on downstream channels. Widespread destruction of cover by poor logging practices or by brush and timber fires frequently increases channel erosion as well as that on the directly affected watershed slopes. On the other hand, cover disturbances under favorable conditions, such as a cool, moist climate, frequently result in a healing of erosion sources within a few years.

Upland Slope Erosion

This erosion form occurs on sloping watershed lands beyond the confines of valleys. Sheet erosion, which involves the removal of a thin layer of soil over an extensive area, is usually not visible to the eye. This erosion form is evidenced by the formation of rills. Experience indicates that soil loss from rill erosion can be seen if it amounts to about 5 tons or more per acre. This is equivalent in volume per square mile to approximately 2 acre-feet.

Wind erosion from upland slopes and the deposition of the eroded material in stream channels may be a significant factor. The material so deposited in channels is readily moved by subsequent runoff.

Downslope soil movement due to creep can be an important factor in sediment yield on steep slopes underlain by unstable geologic formations.

Significant gully erosion as a sediment contributor is evidenced by the presence of numerous raw cuts along the hill slopes. Deep soils on moderately steep to steep slopes usually provide an environment for gully development.

Processes of slope erosion must be considered in the light of factors which contribute to its development. These have been discussed in previous sections.

Channel Erosion and Sediment Transport

If a stream is ephemeral, runoff that traverses the dry alluvial bed may be drastically reduced by transmission losses (absorption by channel alluvium). This decrease in the volume of flow results in a decreased potential to move sediment. Sediment may be deposited in the streambed from one or a series of relatively small flows only to be picked up and moved on in a subsequent larger flow. Sediment concentrations, determined from field measurements at consecutive stations, have generally been shown to increase many fold for instances of no tributary inflow. Thus, although water yield per unit area will decrease with increasing drainage area, the sediment yield per unit area may remain nearly constant or may even increase with increasing drainage area.

In instances of convective precipitation in a watershed with perennial flow, the role of transmission losses is not as significant as in watersheds with ephemeral flow, but other channel factors, such as the shape of the channel, may be important.

For frontal storm runoff, the flow durations are generally much longer than for convective storms, and runoff is often generated from the entire basin. In such instances, sediment removed from the land surfaces is generally carried out of the area by the runoff. Stream channel degradation and/or aggradation must be considered in such cases, as well as bank scour. Because many of the stream beds in the Pacific Southwest are composed of fine-grained alluvium in well defined channels, the potential for sediment transport is limited only by the amount and duration of runoff. Large volumes of sediment may thus be moved by these frontal storms because of the longer flow durations.

The combination of frontal storms of long duration with high intensity and limited areal-extent convective activity will generally be in the highest class for sediment movement in the channels. Storms of this type generally produce both the high peak flows and the long durations necessary for maximum sediment transport.

Sediment yield may be substantially affected by the degree of channel development in a watershed. This development can be described by the channel cross sections, as well as by geomorphic parameters such as drainage density, channel gradients and width-depth ratio. The effect of these geomorphic parameters is difficult to evaluate, primarily because of the scarcity of sediment transport data in the Pacific Southwest.

If the cross section of a stream is such as to keep the flow within defined banks, then the sediment from an upstream point is generally transported to a downstream point without significant losses. Confinement of the flow within alluvial banks can result in a high erosional capability of a flood flow, especially the flows with long return periods. In most channels with wide floodplains, deposition on the floodplain during floods is often significant, and the transport is thus less than that for a within bank flow. The effect of this transport capability can be explained in terms of tractive force which signifies the hydraulic stress exerted by the flow on the bed of the stream. This average bed-shear stress is obtained as the product of the specific weight of the fluid, hydraulic radius, and energy gradient slope. Thus, greater depth results in a greater bed shear and a greater potential for moving sediment. By the same reasoning, steep slopes (the energy slope and bed slope are assumed to be equivalent) also result in high bed-shear stress.

The boundary between sediment yield classifications in much of the Pacific Southwest may be at the mountain front, with the highest yield designation on the alluvial plain if there is extensive channel erosion. In contrast, many mountain streams emerge from canyon reaches and then spread over fans or valley flats. Here water depths can decrease from many feet to only a few inches in short distances with a resultant loss of the capacity to transport sediment. Sediment yield of the highest classification can thus drop to the lowest in such a transition from a confined channel to one that has no definition.

Channel bank and bed composition may greatly influence the sediment yield of a watershed. In many areas within the Pacific Southwest, the channels in valleys dissect unconsolidated material which may contribute significantly to the stream sediment load. Bank sloughing during periods of flow, as well as during dry periods, piping, and bank scour generally add greatly to the sediment load of the stream and often change upward the sediment yield classification of the watershed. Field examination for areas of head cutting, aggradation or degradation, and bank cutting are generally necessary prior to classification of the transport expectancy of a stream. Geology plays a significant role in such an evaluation. Geologic controls in channels can greatly affect the stream regimen by limiting degradation and headcuts. Thus, the transport capacity may be present, but the supply of sediment from this source is limited.

Man-made structures can also greatly affect the transport characteristics of the stream. For example, channel straightening can temporarily upset the channel equilibrium and cause an increase in channel gradient and an increase in the stream velocity and the shear stress. Thus, the sediment transport capacity of the stream may be temporarily increased. Structures such as debris darns, lined channels, drop spillways, and detention dams may drastically reduce the sediment transport.

AN EXPLANATION OF THE USE OF THE RATING CHART (TABLE A.1.1) FOR EVALUATING FACTORS AFFECTING SEDIMENT YIELD IN THE PACIFIC SOUTHWEST FOLLOWS

Table A.1.1. Factors Affecting Sediment Yield in the Pacific Southwest.

Sediment Yield Levels	A SURFACE GEOLOGY (10)*	B SOILS (10)	C CLIMATE (10)	D RUNOFF (10)	E TOPOGRAPHY (20)	F GROUND COVER (10)	G LAND USE (10)	H UPLAND EROSION (25)	I CHANNEL EROSION & SEDIMENT TRANSPORT (25)
High	a. Marine shales and related mud-stones and silt-stones.	a. Fine textured easily dispersed; saline-alkaline; high shrink-swell characteristics b. Single grain silts and fine sands	a. Storms of several days' duration with short periods of intense rain-fall. b. Frequent intense convective storms c. Freeze-thaw occurrence	a. High peak flows per unit area b. Large volume of flow per unit area	a. Steep upland slopes (in excess of 30%) High relief; little or no floodplain development	Ground cover does not exceed 20% a. Vegetation sparse; little or no litter b. No rock in surface soil	a. More than 50% cultivated b. Almost all of area intensively grazed c. All of area re-cently burned	a. More than 50% of the area char-acterized by rill and gully or landslide erosion	a. Eroding banks con-tinuously or at frequent intervals with large depths and long flow duration b. Active headcuts and degradation in trib-utary channels
**									
Moderate	(5) a. Rocks of medium hardness b. Moderately weathered c. Moderately fractured	(5) a. Medium textured soil b. Occasional rock fragments c. Caliche layers	(5) a. Storms of moder-ate duration and intensity b. Infrequent con-vective storms	(5) a. Moderate peak flows b. Moderate volume of flow per unit area	(10) a. Moderate upland slopes (less than 20%) b. Moderate fan or floodplain develop-ment	(10) Cover not exceed-ing 40% a. Noticeable litter b. If trees present understory not well developed	(0) a. Less than 25% cultivated b. 50% or less re-cently logged c. Less than 50% in-tensively grazed d. Ordinary road and other construction	(10) a. About 25% of the area character-ized by rill and gully or land-slide erosion b. Wind erosion with deposition in stream channels	(10) a. Moderate flow depths, medium flow duration with occasionally eroding banks or bed
**									
Low	(0) a. Massive, hard formations	(0) a. High percentage of rock fragments b. Aggregated clays c. High in organic matter	(0) a. Humid climate with rainfall of low intensity b. Precipitation in form of snow c. Arid climate, low intensity storms d. Arid climate; rare convective storms	(0) a. Low peak flows per unit area b. Low volume of runoff per unit area c. Rare runoff events	(0) a. Gentle upland slopes (less than 5%) b. Extensive alluvial plains	(-10) a. Area completely protected by veg-etation, rock fragments, litter. Little opportunity for rainfall to reach erodible material	(-10) a. No cultivation b. No recent logging c. Low intensity grazing	(0) a. No apparent signs of erosion	(0) a. Wide shallow channels with flat gradients, short flow duration b. Channels in massive rock, large boulders or well vegetated c. Artificially controlled channels

* THE NUMBERS IN SPECIFIC BOXES INDICATE VALUES TO BE ASSIGNED APPROPRIATE CHARACTERISTICS.
THE SMALL LETTERS, a, b, c, REFER TO INDEPENDENT CHARACTERISTICS TO WHICH FULL VALUE MAY BE ASSIGNED.

** IF EXPERIENCE SO INDICATES, INTERPOLATION BETWEEN THE 3 SEDIMENT YIELD LEVELS MAY BE MADE.

Use of the Rating Chart of Factors Affecting Sediment Yield in the Pacific Southwest

The following is a summary of the sediment yield classification presented for this methodology.

Classification	Rating	Sediment Yield AF/sq. ml.
1	> 100	3.0
2	75 - 100	1.0 - 3.0
3	50 - 75	0.5 - 1.0
4	25 - 50	0.2 - 0.5
5	0 - 25	< 0.2

In most instances, high values for the A through G factors should correspond to high values for the H and/or I factors.

An example of the use of the rating chart is as follows:

A watershed of 15 square miles in western Colorado has the following characteristics and sediment yield levels:

Factors	Sediment Yield Levels	Rating
A Surface geology	Marine Shales	10
B Soils	Easily dispersed, high shrink-swell characteristics	10
C Climate	Infrequent convective storms, freeze-thaw occurrence	7
D Runoff	High peak flows; low volumes	5
E Topography	Moderate slopes	10
F Ground cover	Sparse, little or no litter	10
G Land use	Intensively grazed	10
H Upland erosion	More than 50% rill and gully erosion	25
I Channel erosion	Occasionally eroding banks and bed but short flow duration	5
	TOTAL	92

This total rating of 92 would indicate that the sediment yield is in Classification 2. This compares with a sediment yield of 1.96 acre-feet per square mile as the average of a number of measurements in this area.

APPENDIX A.2

Modified Universal Soil Loss Equation for Predicting Watershed Soil Loss

APPENDIX A.2

MODIFIED UNIVERSAL SOIL LOSS EQUATION FOR PREDICTING WATERSHED SOIL LOSS

The Modified Universal Soil Loss Equation (MUSLE) described by Williams (1975) is an empirically derived methodology for predicting watershed sediment yield on a per-storm basis. The MUSLE is:

$$Y_S = R_W \; K \; LS \; C \; P \tag{A.2.1}$$

where Y_S is the total sediment yield in tons for the storm event, R_W is a storm runoff energy factor, K is the soil erodibility factor, LS is the topographic factor representing the combination of slope length and slope gradient, C is the cover and management factor and P is the erosion control practice factor. Factors K, LS, C and P are as defined for the Universal Soil Loss Equation (USLE) as described in the USDA Soil Conservation Service, Agriculture Handbook Number 537 "Predicting Rainfall Erosion Losses" (1978), and as reviewed in later paragraphs, Smith and Wischmeier, 1975; Wischmeier, 1960; and Wischmeier and Smith, 1978, provide detailed descriptions of the USLE factors and their values.

The storm runoff energy factor R_W in Equation A.2.1 is a function of both the storm volume and the storm peak discharge, given by:

$$R_W = \alpha\left(Vq_p\right)^{\beta} \tag{A.2.2}$$

In Equation A.2.2, V is the storm event runoff volume in acre-feet, q_p is the storm event peak flow rate in cfs, and α and β are coefficients. Use of the storm runoff factor makes the MUSLE applicable to semiarid regions of the West where short-duration, high-intensity storms are dominant. The original derivation of this relationship suggested values of the coefficients α and β of 95 and 0.56, respectively, based on data from experimental watersheds in Texas and Nebraska. **For the Albuquerque area, it is recommended that the formula be used only to establish wash load (silts and clays), and that values of α and β of 285 and 0.56, respectively, be used.**

Recommended values for the soil erodibility factor (K), which are based primarily on USDA soil texture, are given in USDA SCS (1981), New Mexico Conservation Agronomy Technical Note No. 28. These values are shown in **Table A.2.1**. A nomograph to determine the K factor based on percent of silt and coarse sand permeability, soil structure and percent organic matter is found in the SCS (1978), Agriculture Handbook Number 537. **Figure A.2.1** is a guide to determine the textural classification of soil based on the percentage of clay, silt, and sand.

Table A.2.1. Soil Erodibility Factor K Based on USDA Texture.				
	Estimated K Factor[1]			
USDA Texture	Normal[2]	Gravelly[2]	Very Gravelly[2]	Extremely Gravelly[2]
Coarse Sand	0.10	0.05	0.02	0.02
Sand	0.10	0.05	0.02	0.02
Fine Sand	0.17	0.10	0.05	0.02
Very Coarse Sand	0.10	0.05	0.02	0.02
Loamy Coarse Sand	0.15	0.10	0.05	0.02
Loamy Sand	0.17	0.10	0.05	0.02
Loamy Fine Sand	0.20	0.10	0.05	0.02
Loamy Very Fine Sand	0.49	0.28	0.15	0.05
Coarse Sandy Loam	0.20	0.10	0.05	0.02
Sandy Loam	0.24	0.15	0.10	0.05
Fine Sandy Loam	0.28	0.15	0.10	0.05
Very Fine Sandy Loam	0.55	0.28	0.17	0.10
Loam	0.37	0.20	0.10	0.05
Silt Loam	0.43	0.24	0.15	0.05
Silt	0.64	0.37	0.20	0.10
Sandy Clay Loam	0.32	0.15	0.10	0.05
Clay Loam	0.32	0.15	0.10	0.05
Silty Clay Loam	0.37	0.20	0.10	0.05
Sandy Clay	0.32	0.15	0.10	0.05
Silty Clay	0.24	0.15	0.10	0.05
Clay	0.20	0.10	0.05	0.02

[1]Where a Soils Survey Interpretation Sheet, SOILS-5, is available for a soil, the K Factor listed will be more accurate than the factor provided by this table.
[2]Total rock fragments are included in these figures, not just gravel. Normal = 0-15 percent, gravelly = 15-35 percent, very gravelly = 35-60 percent, and extremely gravelly = over 60 percent.

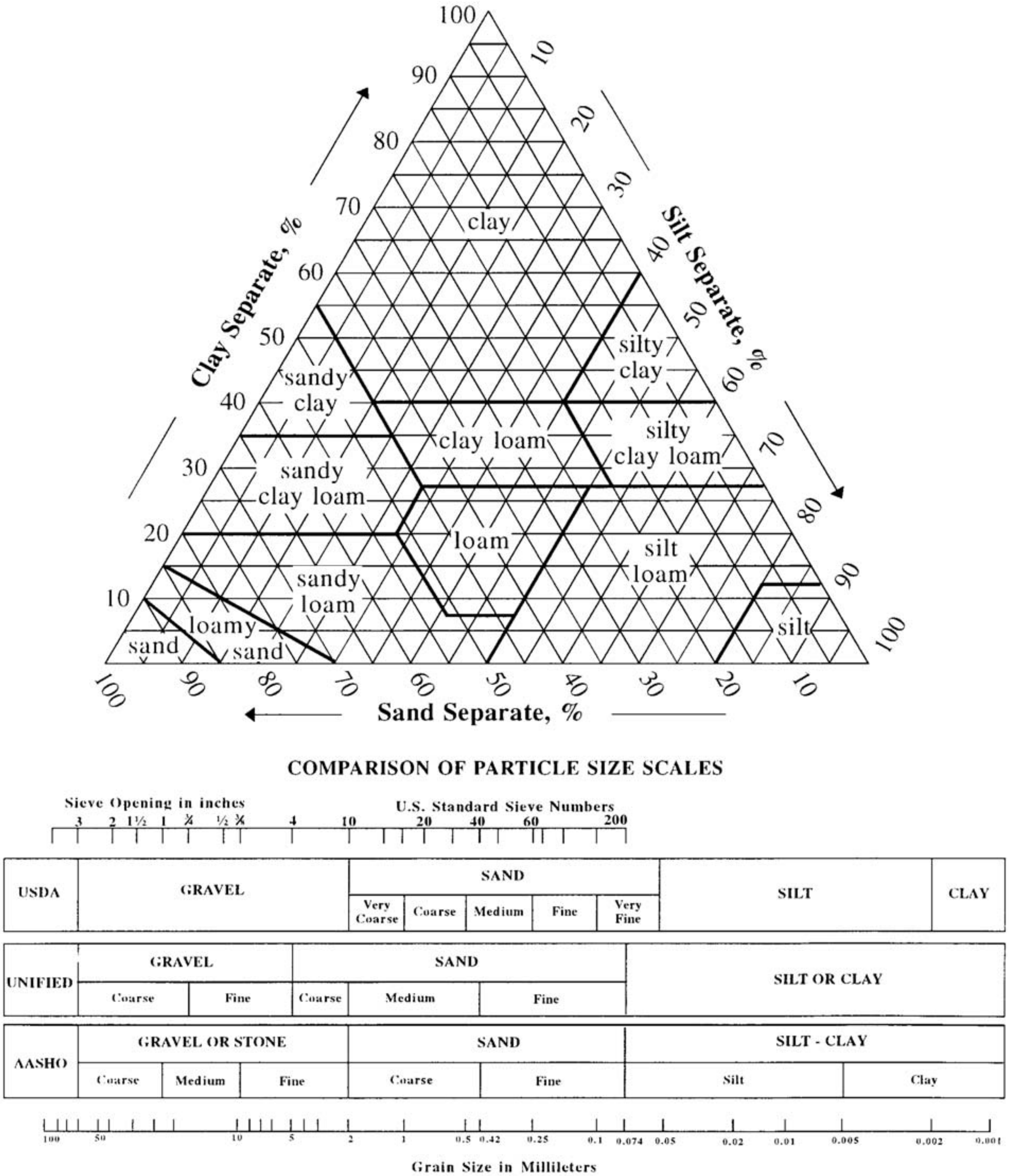

Figure A.2.1. Guide for the textural classification of soil.

SCS Agriculture Handbook Number 537 (1978) also presents a table to determine the Cover and Management Factor (Cropping management factor, C) for pasture, range, idle land, and grazed woodland. This table is reproduced as **Table A.2.2**. A table with identical values is contained in SCS-New Mexico Technical Note 28 (1981). The Cover and Management Factors (L) for cropland and fallow areas not described in Table A.2.2 are continued in SCS Agriculture Handbook Number 537 and SCS-New Mexico Technical Note 28 (1981).

The topographic factor (LS) is defined as the ratio of soil loss from any slope and length to soil loss from a 72.6-foot plot length at a nine percent slope, with all other conditions the same. Slope length is defined as the distance from the point of overland flow origin to the point where either slope decreases to the extent that deposition begins or runoff water enters a well-defined channel (Smith and Wischmeier, 1975). Effect of slope length on soil loss is primarily a result of increased potential due to greater accumulation of runoff on the longer slopes. Based on data for slopes between three and 20 percent and lengths up to 400 feet, Wischmeier and Smith (1965) proposed the following equation to estimate the topographic factor:

$$LS = \left(\frac{\lambda}{72.6}\right)^{n}\left(0.065 + 0.0454\,S + 0.0065\,S^{2}\right) \qquad \text{(A.2.3)}$$

where λ = slope length,
S = percent slope, and
n = an exponent depending upon slope. The exponent n is given by:

n = 0.3 for slope $\leq$ 3 percent
n = 0.4 for slope = 4 percent
n = 0.5 for slope $\geq$ 5 percent

SCS New Mexico Technical Note 28 (1981) has extended Equation A.2.3 to slopes between 0.2 and 60 percent and to lengths up to 2,000 feet. However, for lengths exceeding 400 feet and slopes greater than 24 percent, soil loss estimates are speculative as these values are beyond the range of research data. The LS data from SCS New Mexico Technical Note 28 are included as **Table A.2.3**.

In applying Equation A.2.3, the overland slope lengths (λ) have typically been measured during field surveys or obtained from Soil Conservation Service tables. Recent developments in GIS technology have yielded improved methods for computing the LS factor using digital elevation models (DEM). One of the more popular approaches uses the "upslope drainage contributing area" and the formula developed by Mitasova et al. (1996):

$$LS = \left(\frac{FAC\ x\ Cell\ Size}{22.13}\right)^{0.4}\left(\frac{Sin(Slope)}{0.09}\right)^{1.3} \qquad \text{(5.5)}$$

where FAC = upslope contributing area per unit contour width
Cell Size = width of the DEM element
Slope = slope of the cell

Erosion-control practice factor P accounts for the effect of conservation practices such as contouring, strip cropping, and terracing on erosion. It is defined as the ratio of soil loss using one of these practices to the loss using straight row farming up and down the slope. Terracing is generally the most effective conservation practice for decreasing soil erosion. This factor has no significance for range and wild-land areas and can be set at 1.0.

As noted above, results from the MUSLE represent the total sediment yield, including both the coarse (sand and larger) and fine (silt and clay) fractions. Because the coarse fraction is typically carried as bed material load at the hydraulic capacity of the stream, it is recommended that the bed material transport capacity be computed using an appropriate relationship based on the hydraulic characteristics of the arroyo and the bed material gradation using one of the equations described in Section 3.3. In this case, results from the MUSLE should be adjusted to account for only the fine (wash load) fraction by multiplying the total sediment load by the area-weighted percentage of the watershed soils in the silt and clay size range (i.e., finer than the #200 sieve).

Table A.2.2. Cover and management Factor C for permanent pasture, range, and idle land[1].								
Vegetative Canopy		Cover that contacts the soil surface						
Type and Height[2]	Percent Cover[3]	Type[4]	Percent Ground Cover					
			0	20	40	60	80	95+
No appreciable canopy		G	0.45	0.20	0.10	0.042	0.013	0.003
		W	0.45	0.24	0.15	0.091	0.043	0.011
Tall weeds or short brush with average drop fall height of 20 inches	25	G	0.36	0.17	0.09	0.038	0.013	0.003
		W	0.36	0.20	0.13	0.083	0.041	0.011
	75	G	0.26	0.13	0.07	0.035	0.012	0.003
		W	0.26	0.16	0.11	0.076	0.039	0.011
Appreciable brush or brushes with average drop fall height of 6-1/2 feet	25	G	0.40	0.18	0.09	0.040	0.13	0.003
		W	0.40	0.22	0.14	0.087	0.42	0.011
	50	G	0.34	0.16	0.08	0.038	0.012	0.003
		W	0.34	0.19	0.13	0.082	0.041	0.011
	75	G	0.28	0.14	0.08	0.036	0.012	0.003
		W	0.28	0.17	0.12	0.078	0.040	0.011
Trees, but no appreciable low brush. Average drop fall height of 13 feet	25	G	0.42	0.19	0.10	0.041	0.013	0.003
		W	0.42	0.23	0.14	0.089	0.042	0.011
	50	G	0.39	0.18	0.09	0.040	0.013	0.003
		W	0.39	0.21	0.14	0.087	0.042	0.011
	75	G	0.36	0.17	0.09	0.039	0.012	0.003
		W	0.36	0.20	0.13	0.084	0.041	0.011

[1]The listed C values assumes that the vegetation and mulch are randomly distributed over the entire area.

[2]Canopy height is measured as the average fall height of water drops falling from the canopy to the ground. Canopy effect is inversely proportional to drop fall height and is negligible if fall height exceeds 33 feet.

[3]Portion of total-area surface that would be hidden from view by canopy in a vertical project (a bird's eye view).

[4]G = cover at surface is grass, grass-like plants, decaying compacted duff, or litter 2 inches deep.
W = cover at surface—is mostly broad-leafed plants (as weeds with little lateral-root network near the surface) or undecayed residues or both.

Table A.2.3. Slope-length and slope-steepness Factor LS (topographic factor).

Length of Slope (L)[2]	Percent of Slope (LS)[1]																					
	0.2	0.3	0.4	0.5	1.0	2.0	3.0	4.0	5.0	6.0	8.0	10.0	12.0	14.0	16.0	18.0	20.0	25.0	30.0	40.0	50.0	60.0
20	0.05	0.05	0.06	0.06	0.08	0.12	0.18	0.21	0.24	0.30	0.44	0.61	0.81	1.00	1.20	1.60	1.80	2.60	3.50	5.50	8.00	10.00
40	0.06	0.07	0.07	0.08	0.10	0.15	0.22	0.28	0.34	0.43	0.63	0.87	1.20	1.40	1.80	2.20	2.60	3.50	5.00	8.00	11.00	15.00
60	0.07	0.08	0.08	0.08	0.11	0.17	0.25	0.33	0.41	0.52	0.77	1.00	1.40	1.80	2.00	2.60	3.00	4.50	6.00	10.00	14.00	18.00
80	0.08	0.08	0.09	0.09	0.12	0.19	0.27	0.37	0.48	0.60	0.89	1.20	1.60	2.00	2.60	3.00	3.50	5.50	7.00	11.00	16.00	21.00
100	0.08	0.09	0.09	0.10	0.13	0.20	0.29	0.40	0.54	0.67	0.99	1.40	1.80	2.30	2.80	3.40	4.10	6.00	8.00	13.00	18.00	23.00
110	0.08	0.09	0.10	0.10	0.13	0.21	0.30	0.42	0.56	0.71	1.00	1.40	1.80	2.40	3.00	3.50	4.50	6.00	8.00	13.00	19.00	24.00
120	0.09	0.09	0.10	0.10	0.14	0.21	0.30	0.43	0.59	0.74	1.00	1.60	2.00	2.60	3.00	4.00	4.50	6.00	9.00	14.00	20.00	25.00
130	0.09	0.09	0.10	0.10	0.14	0.22	0.31	0.44	0.61	0.77	1.20	1.60	2.00	2.60	3.00	4.00	4.50	7.00	9.00	14.00	20.00	26.00
140	0.09	0.10	0.10	0.10	0.14	0.22	0.32	0.46	0.63	0.80	1.20	1.60	2.20	2.80	3.50	4.00	5.00	7.00	9.00	15.00	21.00	27.00
150	0.09	0.10	0.10	0.10	0.15	0.23	0.32	0.47	0.66	0.82	1.20	1.70	2.20	2.80	3.50	4.20	5.00	7.00	10.00	15.00	22.00	28.00
160	0.09	0.10	0.11	0.11	0.15	0.23	0.33	0.48	0.68	0.85	1.20	1.80	2.20	3.00	3.50	4.50	5.00	7.00	10.00	16.00	23.00	29.00
180	0.10	0.10	0.11	0.11	0.15	0.24	0.34	0.51	0.72	0.90	1.40	1.80	2.40	3.00	4.00	4.50	5.50	8.00	11.00	17.00	24.00	31.00
200	0.10	0.10	0.11	0.11	0.16	0.25	0.35	0.53	0.76	0.95	1.40	1.90	2.60	3.20	4.00	5.00	5.80	8.00	11.00	18.00	25.00	33.00
300	0.10	0.11	0.12	0.12	0.18	0.28	0.40	0.62	0.93	1.20	1.70	2.40	3.10	4.00	4.90	6.00	7.10	10.00	14.00	22.00	31.00	40.00
400	0.11	0.12	0.13	0.13	0.20	0.31	0.44	0.70	1.10	1.40	2.00	2.70	3.60	4.60	5.70	6.90	8.20	12.00	16.00	25.00	36.00	46.00
500	0.11	0.12	0.13	0.13	0.21	0.33	0.47	0.76	1.20	1.50	2.20	3.10	4.00	5.10	6.40	7.70	9.10	13.00	18.00	28.00	40.00	52.00
600	0.11	0.12	0.13	0.14	0.22	0.34	0.49	0.82	1.30	1.60	2.40	3.40	4.40	5.60	7.00	8.40	10.00	14.00	19.00	31.00	44.00	57.00
700	0.12	0.13	0.14	0.14	0.23	0.36	0.52	0.87	1.40	1.80	2.60	3.50	5.00	6.00	8.00	9.00	11.00	16.00	21.00	33.00	47.00	61.00
800	0.12	0.13	0.14	0.15	0.24	0.38	0.54	0.92	1.50	1.90	2.80	3.90	5.10	6.50	8.00	9.70	11.50	17.00	22.00	36.00	50.00	65.00
900	0.12	0.13	0.14	0.15	0.25	0.39	0.56	0.96	1.60	2.00	3.00	4.00	5.50	7.00	9.00	10.00	12.00	18.00	24.00	38.00	53.00	69.00
1,000	0.13	0.14	0.15	0.15	0.26	0.40	0.57	1.00	1.70	2.10	3.10	4.30	5.70	7.30	9.00	10.90	12.90	19.00	25.00	40.00	56.00	73.00
1,100	0.17	0.18	0.19	0.20	0.27	0.41	0.59	1.00	1.80	2.20	3.50	4.50	6.00	8.00	9.00	11.00	14.00	20.00	26.00	42.00	59.00	77.00
1,200	0.17	0.18	0.20	0.21	0.27	0.42	0.61	1.00	1.80	2.40	3.50	4.50	6.00	8.00	10.00	12.00	14.00	20.00	28.00	44.00	62.00	80.00
1,300	0.18	0.19	0.20	0.21	0.28	0.43	0.62	1.20	2.00	2.40	3.50	5.00	7.00	8.00	10.00	12.00	15.00	21.00	29.00	46.00	64.00	83.00
1,400	0.18	0.19	0.21	0.22	0.29	0.44	0.63	1.20	2.00	2.60	3.50	5.00	7.00	9.00	11.00	13.00	15.00	22.00	30.00	47.00	67.00	87.00
1,500	0.19	0.20	0.21	0.22	0.29	0.45	0.65	1.20	2.00	2.60	4.00	5.50	7.00	9.00	11.00	13.00	16.00	23.00	31.00	49.00	69.00	90.00
1,600	0.19	0.20	0.21	0.23	0.30	0.46	0.66	1.20	2.20	2.60	4.00	5.50	7.00	9.00	11.00	14.00	16.00	24.00	32.00	51.00	71.00	93.00
1,700	0.19	0.21	0.22	0.23	0.30	0.47	0.67	1.20	2.20	2.80	4.00	5.50	7.00	9.00	12.00	14.00	17.00	24.00	33.00	52.00	73.00	95.00
2,000	0.20	0.22	0.23	0.24	0.32	0.49	0.71	1.40	2.40	3.00	4.50	6.00	8.00	10.00	13.00	15.00	18.00	26.00	36.00	57.00	80.00	104.00

[1]From USDA Ag Handbook No. 537, December 1978.
[2]When the length of slope exceeds 400 feet and/or percent of slope exceeds 24 percent; soil loss estimates are speculative as these values are beyond the range of research data.

APPENDIX B
Incipient Motion and Armoring

Appendix B
Incipient Motion and Armoring

B.1. Introduction

Incipient motion refers to the condition where the hydrodynamic forces acting on a grain of sediment are of sufficient magnitude that, if increased even slightly, the grain will move. Under critical conditions (i.e., the point of incipient motion), the hydrodynamic forces acting on the grain are just balanced by the resisting forces of the particle. In cases where there is a significant amount of coarse material (gravel and cobbles) in the bed, the relative susceptibility of the channel bed to degradation can be evaluated by analyzing the magnitude of flows required to produce incipient motion conditions. The bed material in most arroyos in the SSCAFCA area is in the sand size-range; thus, armoring is typically not an important factor in assessing arroyo stability. Methods for evaluating armoring potential in cases where significant gravel is present in the bed material are provided in the following sections.

B.2. Estimation of the Incipient Particle Size

Incipient motion refers to the condition where the hydrodynamic forces acting on a grain of sediment are of sufficient magnitude that, if increased even slightly, the grain will move. Under critical conditions (i.e., the point of incipient motion), the hydrodynamic forces acting on the grain are just balanced by the resisting forces of the particle. In cases where there is a significant amount of coarse material (gravel and cobbles) in the bed, the relative susceptibility of the channel bed to degradation can be evaluated by analyzing the magnitude of flows required to produce incipient motion conditions.

Research on coarse-grained streams, where incipient motion analysis is of most interest, has shown that, in a well-graded mixture of sediment, the large particles tend to protrude farther into the flow than they would in a mixture with uniform sizes and smaller particles tend to be hidden behind the larger particles. As a result, the shear stress required to mobilize the larger particles in a well-graded mixture tends to be less than would be required if all of the particles were of uniform size. Conversely, the shear stress required to move the smaller particles tends to be larger than would occur if the particles were of uniform size due to the hiding effect. As a result, it can be assumed that when the shear stress is lower than the critical value for the median (D_{50}) size, the bed is essentially armored, and when the critical shear stress for the D_{50} is exceeded, the bed is mobilized and all sizes up to about 5 times the median size can be transported by the flow (Parker et al., 1982; Andrews, 1984).

The incipient-motion analysis is performed by evaluating the effective shear stress on the channel bed in relation to the amount of shear stress that is required to move the particles on the bed surface. With the above near-equal mobility concept in mind, the shear stress required for bed mobilization is estimated using the Shields (1936) relation, given by:

$$\tau_C = \tau_{*C}(\gamma_S - \gamma)D_{50} \tag{B.1}$$

where τ_c = critical shear stress for particle motion,
τ_{*c} = dimensionless critical shear stress (often referred to as the Shields parameter),
γ_s = unit weight of sediment (~165 lb/ft^3),
γ = unit weight of water (62.4 lb/ft^3), and
D_{50} = median particle size of the bed material.

Reported values for the Shields parameter range from 0.03 (Neill, 1968; Andrews, 1984) to 0.06 (Shields, 1936). A value of 0.047 is commonly used in engineering practice, based on the point at which the Meyer-Peter, Müller (MPM) bed-load equation indicates no transport (MPM, 1948). Close examination of the data on which the data on which the MPM equation data was developed, as well as a range of other data (Parker et al., 1982; Andrews, 1984) indicates that true incipient motion occurs at a value of about 0.03 in gravel- and cobble-bed streams. Neill (1968) concluded that a dimensionless shear value of 0.03 corresponds to true incipient motion of the bed-material matrix while 0.047 corresponds to a low, but measurable transport rate. A value of 0.03 was used in this analysis.

In performing the incipient-motion analysis, the bed shear stress due to grain resistance (τ') should be used rather than the total shear stress, because it is a better descriptor of the near-bed hydraulic conditions that are responsible for sediment movement. The grain shear stress is computed from the following relation:

$$\tau' = \gamma Y'S \qquad \text{(B.2)}$$

where

- Y' = the portion of the total hydraulic stress associated with grain resistance (Einstein, 1950), and
- S = the energy slope at the cross section.

The value of Y' is computed by iteratively solving the semilogarithmic velocity profile equation:

$$\frac{V}{V_*'} = 6.25 + 5.75 \log \frac{Y'}{k_S} \qquad \text{(B.3)}$$

where

- V = mean velocity at the cross section,
- k_s = characteristic roughness of the bed, and
- V_*' = shear velocity due to grain resistance given by:

$$V_*' = \sqrt{gY'S} \qquad \text{(B.4)}$$

The characteristic roughness height of the bed (k_s) is typically assumed to be 3.5 D_{84} (Hey, 1979).

The normalized grain shear stress (ϕ'), defined as the ratio of the grain shear stress (τ') to the critical shear stress for particle mobilization (τ_c) is a convenient way to evaluate incipient motion results. When ϕ' is less than 1, the bed is effectively armored, and when ϕ' is equal to 1, the bed material begins to mobilize (point of incipient motion). When ϕ' is greater than about 1.5, substantial sediment transport occurs (Mussetter et al., 2001). Values of ϕ' between 1.0 and 1.5 are termed marginal transport conditions, where the amount of bed load transport is very small, and the surface particle sizes tend to be much coarser than the subsurface material due to winnowing of the fines from the bed-material matrix.

Evaluation of the incipient size for various discharges provides information on the magnitude of floods that could potentially disrupt channel stability. The results of such an analysis are generally most useful when applied to gravel- or cobble-bed systems. When applied to sand-bed channels, incipient motion results usually show that all particles in the bed material are capable of being moved by even very small discharges.

B.3. Evaluation of Armoring Potential

Armoring occurs when the finer fraction of the bed material is winnowed (or removed) from the channel bed leaving a layer of coarser, relatively immobile material on the surface. This process occurs because the finer particles in the bed are inherently more transportable than the coarser particles. In degradational streams that have sufficient large particles in their bed material, the surface layer will continue to coarsen during bed mobilizing flows until the transport capacity is in balance with the supply of coarse material from upstream. During this process, a layer of large particles will accumulate to shield, or "armor" the bed surface, arresting further degradation.

An armor layer sufficient to protect the bed against moderate discharges can be disrupted during high flow, but may be restored as flows diminish. The analysis should, therefore, be based on the 100-year peak discharge for fully-developed conditions. If the armor layer is stable for that event, it is reasonable to conclude that little or no degradation will occur under design conditions. However, flows exceeding the design event may disrupt the armor layer, resulting in degradation.

Potential for development of an armor layer can be assessed using incipient motion analysis and a bed-material gradation typical of the material within the depth of anticipated degradation. For given hydraulic conditions, the incipient particle size can be computed using the Shields relation (Equation B.1). If no sediment of the computed size or larger is present in significant quantities in the bed, armoring will not occur. Within practical limits of planning and design, the D_{95} size is considered to be about the maximum size for armor formation (Gessler, 1970). Therefore, armoring is probable when the computed incipient size is equal to or smaller than the D_{95} size of the bed material.

The depth of scour necessary to establish an armor layer can be estimated using the following relation (USBR, 1984):

$$Y_s = y_a\left(\frac{1}{P_c} - 1\right) \tag{B.5}$$

where y_a is the thickness of the armor layer and P_c is the decimal fraction of material coarser than the armoring size. The thickness of the armor layer (y_a) is normally assumed to be in the range from 2 to 3 times the critical particle size (D_c). Equation B.5 is conceptually illustrated in **Figure B.1**. Field observations suggest that an armor layer thickness of approximately twice the diameter of the incipient particle size is usually necessary for stability, thus the minimum recommended value for y_a in Equation B.5 is $2D_c$. To increase the factor of safety for the design purposes, the assumed armor layer thickness can be increased to 2 to 4 times D_c.

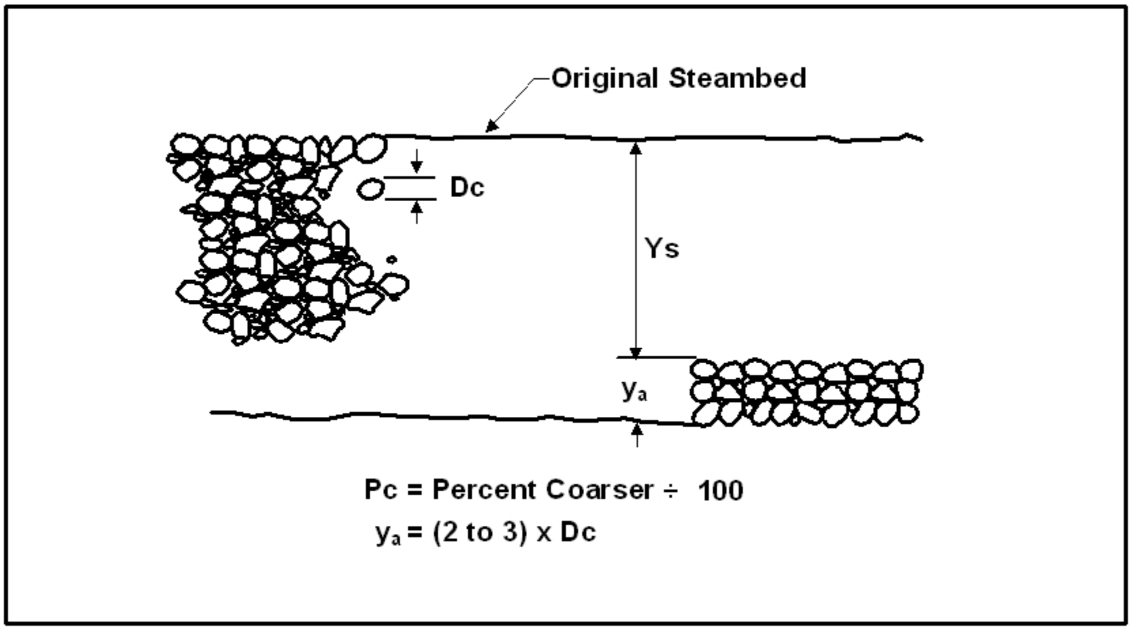

Figure B.1. Conceptual illustration of armor layer development.

APPENDIX C
Sediment Transport Equations

Appendix C
Sediment Transport Equations

Detailed, quantitative analysis of the aggradation/degradation and lateral migration potential requires knowledge of both the sediment-transport capacity of the stream and the sediment supply. Numerous equations are available for estimating the bed-material transport capacity based on the hydraulic conditions (i.e., velocity, depth, width, shear stress, stream power) and the size characteristics of the bed material. These equations are described in a variety of commonly available books, including Vanoni (1977), SLA (1982), and Chang (1988).

Several of the more commonly used equations have been implemented in the SAMwin software (USACE, 2003). This software also includes guidance on the range of conditions for which the various equations are applicable. At the time of this manual, the USACE is also in the process of implementing several of the more frequently used relationships into a new version of the HEC-RAS software. A beta-test version of the software (Version 4.0) is available on the Hydrologic Engineering Center website (http://www.hec.usace.army.mil/). Official release of the software has not been scheduled, but is expected to occur by the end of 2007. While this software provides a convenient means of performing sediment transport computations, it should be used with caution because it has not been fully tested.

The conditions for which most of the available relations were developed are significantly different from those encountered in the arroyos in the SSCAFCA area, due to the combination of steep gradients and relatively small sediment sizes. Two other relatively simple relationships that are not available in the SAMWin software or HEC-RAS 4.0, but that have been used successfully on projects in the greater Albuquerque area are the MPM-Woo (Mussetter, 2000) and Zeller and Fullerton (1983) equations. The MPM-Woo equation is most applicable to steep, sand-bed channels that carry high concentrations of wash load, and it typically predicts conservatively high transport capacities. The Zeller-Fullerton (1983) equation was developed from transport capacities computed using the Meyer-Peter (MPM)-Einstein method (SLA, 1982) for channels that carry relatively low wash-load concentrations. Results from this equation are often in the lower range of realistic values. Because they are simple and have a history of successful use in the greater Albuquerque area, these equations are described in more detail in below.

C.1. Zeller and Fullerton Relation

As noted above, the Zeller and Fullerton (1983) was developed for low fine sediment loads by estimating the bed material transport capacity for a broad range of hydraulic conditions and bed material sizes using the MPM-Einstein method, and fitting a multiple regression equation through the resulting values. This relationship is given by:

$$q_S = 0.0064 \frac{n^{1.77} V^{4.32} G^{0.45}}{Y^{0.30} D_{50}^{0.61}} \quad \text{(C.1)}$$

where G = gradation coefficient of the bed material, given by Equation 3.14.

(Note that D_{50} *in Equation C.1 is in millimeters; all other variables are in the foot-pound-second system.)* **Table C.1** summarizes the range of conditions for which Equation C.1 was developed.

Table C.1. Range of conditions for which Equation C.1 is applicable.	
Parameter	Range
Velocity	3-30 (ft/sec)
Manning's *n*	0.018-0.035
Bed Slope	0.001-0.040
Unit Discharge	10-200 (cfs/ft)
Particle Size	0.05 mm $\leq D_{50} \leq$ 10 mm
Depth	1-20 ft
Gradation Coefficient	2-5

C.2. Effect of Fine Sediment Concentration on Bed-material Transport Capacity

Leopold and Miller (1956) observed suspended-sediment concentrations in arroyos in north central New Mexico approaching 200,000 ppm by weight (9 percent by volume). Nordin (1963) reported concentrations in the Rio Puerco, Paria, and Little Colorado Rivers as high as 650,000 ppm by weight (41 percent by volume), with sand concentrations up to nearly 450,000 ppm by weight (24 percent by volume). Others (Beverage and Culbertson, 1964; Lane, 1974) have reported similar observations. Based on the watershed sediment yield discussions in Chapter 2, similar concentrations may occur in the arroyos in the SSCAFCA jurisdictional area during large storm events.

The presence of high concentrations of fine suspended sediment enhances the capacity of the stream to transport coarser sediments (Vanoni, 1977). As a result, if the Zeller-Fullerton Equation or any of the transport equations in the SAMwin software are used under conditions where the fine sediment concentration exceeds about 10,000 ppm by weight (mg/l), the estimated bed-material loads should be adjusted using the following procedure from Colby (1964):

1. Compute the bed material load (q_{si}) using the applicable relationship,

2. Estimate the fine sediment concentration based on the watershed sediment yield relationships discussed in Chapter 2 and Appendix A, or from field data, and

3. Apply the Colby correction factor using the following equation with the values of k_1, k_2 and k_3 estimated from **Figure C.1**:

$$Q_s = [1+(k_1k_2-1)]\, Q_{s0} \qquad \text{(C.2)}$$

where Q_{s0} and Q_s are the uncorrected and corrected bed-material transport capacities, respectively.

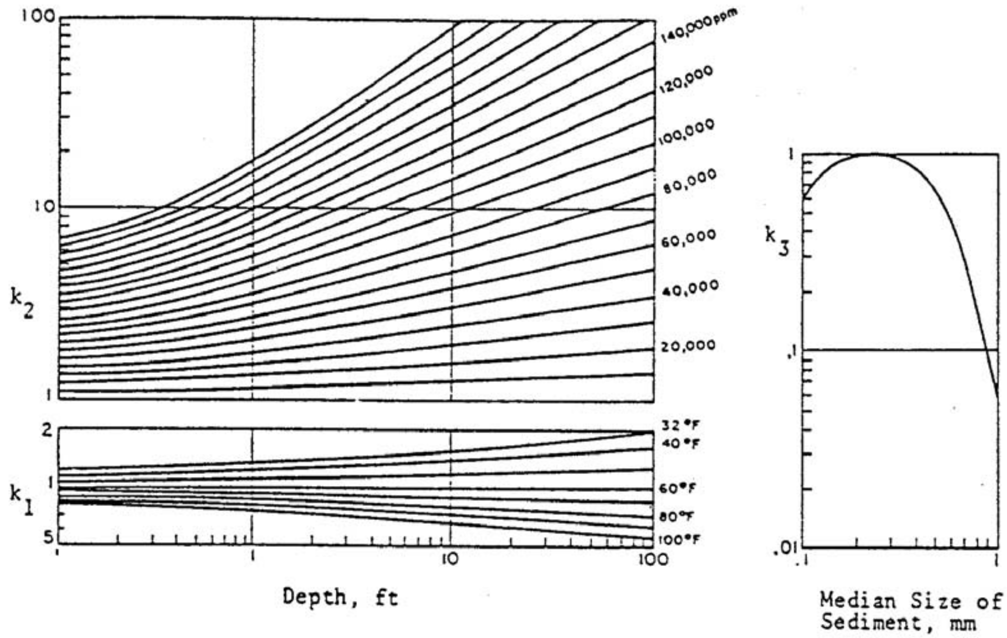

Figure C.1. Colby's corrections factors (Colby, 1964).

B.7. Transport Equation for High Suspended Sediment Concentrations (MPM-Woo Equation)

As an alternative to the above procedure, Mussetter (2000) developed a power function equation specifically for steep, sand-bed channels carrying high fine sediment loads that is believed to provide more realistic estimates of the bed-material transport capacity than the other available relationships under most conditions. As discussed below, this relation overcomes many of the shortcomings of the other relations when applied to conditions that are found in the arroyos in the SSCAFCA jurisdictional area. It is, therefore, recommended that this power function relation (Equation C.3, described below) be used, unless there is analysis and calibration that demonstrates that other methods are more representative of actual conditions.

The Einstein suspended-sediment method that is used in the MPM-Einstein method is based on the Rouse (1937) equation for the suspended-sediment concentration profile. For suspended-sediment concentrations less than about 4 percent by weight, the original derivation provides reasonable results for most conditions because the physical properties of the water/sediment mixture are very similar to the properties of clear water. Einstein and Chien (1955) found that the vertical distribution of sands in the water column deviates significantly from that predicted by the original Einstein method when the suspended-sediment concentration is greater than 4 percent. They also found that the concentration profile becomes progressively more uniform with depth with increasing concentration. This occurs because the presence of the suspended sediment causes the unit weight and viscosity to increase, which in turn, changes turbulence characteristics of the mixture. Based on their experimental results, Einstein and Chien (1955) suggested a correction for the exponent on the concentration profile equation to partially compensate for the differences. Subsequent investigators (Chien and Wan, 1965; Ordonez, 1970; Lavelle and Thacker, 1978, van Rijn, 1984; Woo, 1985 and Woo et al., 1988) considered the effects of fall velocity reduction, fluid viscosity, and fluid density on the suspended sand concentration profile. The work of Woo (1985) and Woo et al. (1988) resulted in a complex differential equation that appears to account for the significant changes in fluid characteristics with increasing sediment concentration, and converges to essentially the same solution as obtained from Rouse's (1937) equation at low concentrations. Results obtained from the application of this relationship match the available measured data well.

Mussetter (2000) linked Woo's (1985) relationship for computing the suspended-sediment concentration with the MPM bed-load equation to obtain a method for computing bed-material load in streams carrying high concentrations of suspended sediment. Based on comparison of the results obtained from this method with the results from other available relations, and, to the extent possible, measured sediment yield data, the resulting equation appears to provide realistic results over the range of flow and sediment-transport conditions encountered in the greater Albuquerque area. Because of the complexity involved in solving the differential equations, the method was applied for a broad range of hydraulic and bed-material conditions typical of the greater Albuquerque area, (see **Table C.2**) and results from these computations were then used to develop the following power function relation using multiple regression:

$$q_s = a V^b Y^c \left(1 - C_f / 10^6\right)^d \tag{C.3}$$

where q_s = unit width **bed-material** load, cfs/ft
V = average velocity, fps
Y = hydraulic depth, ft
C_f = fine sediment (silt and clay) concentration by weight
S_o = channel slope

Values for the coefficient (a) and exponents (b, c, and d) are shown in **Figure C.2** as a function of the median bed-material size.

Table C.2. Range of conditions for which Equation C.3 was developed.

q (cfs)	V (fps)	Y (ft)	S_o	C_f (ppm)	D_{50} (mm)
1	1.9	0.3	0.005	0	0.2
80*	20.8*	7.2*	0.04*	60,000*	4.0*

*Larger values may be used with expected loss of accuracy; the amount of inaccuracy is unknown, but would depend on the specific conditions being analyzed.

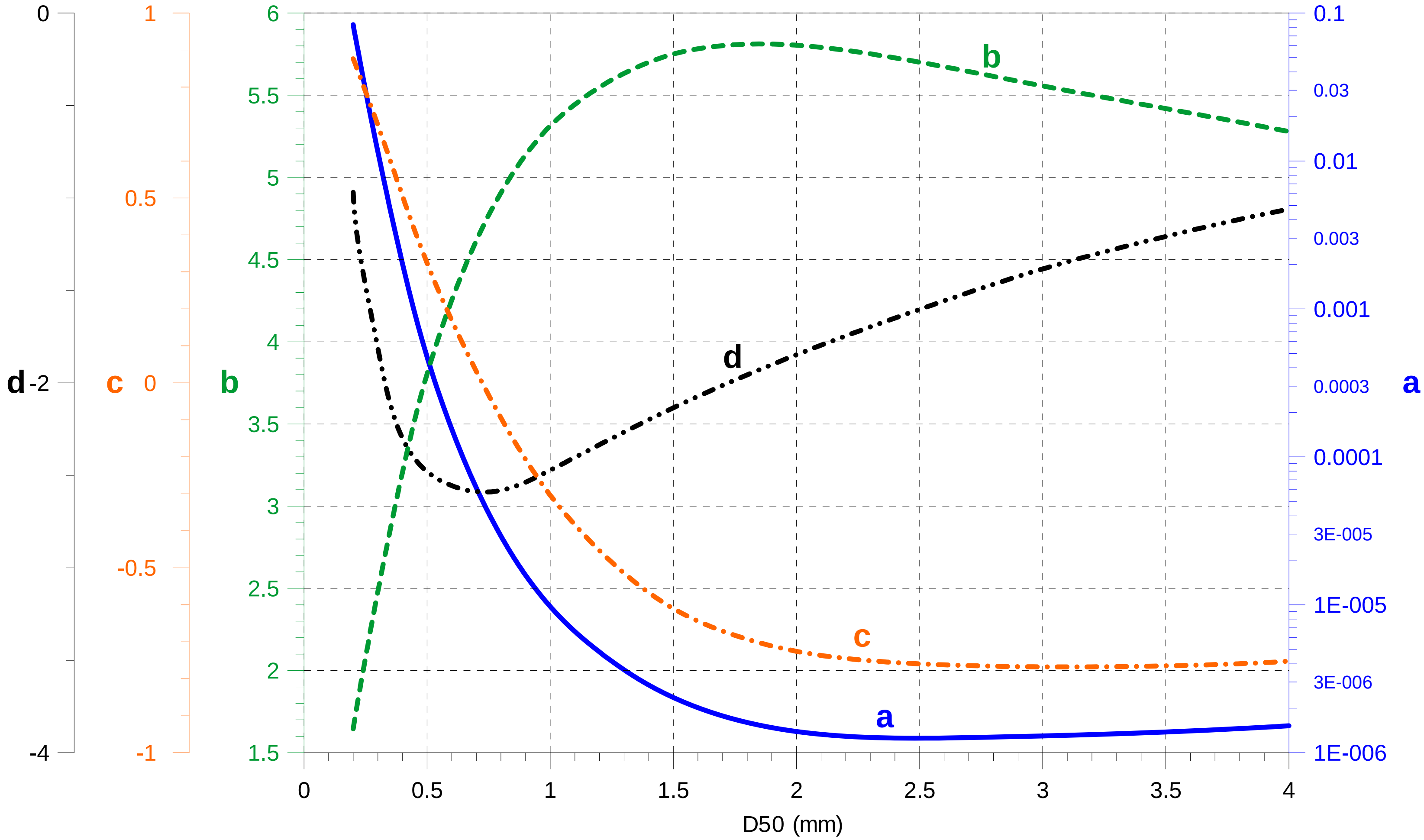

Figure C.2. Coefficient and exponents of Equation C.3 developed using MPM-Woo method.

Previous studies of high concentration water and sediment flows have established limits on the amount of sediment that can physically be carried by the flow (**Table C.3**). For concentrations exceeding approximately 65 percent by weight, the water/sediment mixture is no longer a Newtonian fluid and the basic hydraulic and sediment-transport assumptions no longer apply. Mud and debris flows occur at higher concentrations. Mud and debris flows are probably rare in the SSCAFCA jurisdictional area because the amount of cohesive material in the watershed soils is relatively small and overland slopes are generally too flat to sustain mud-flow conditions.

In applying Equation C.4, the predicted bed material concentration (C_s) should not exceed the maximum concentration given by the equation:

$$C_{s_{max}} = 510{,}000 \quad - \quad 65{,}000\, D_{50} \tag{C.4}$$

where D_{50} = medium bed-material size, mm
C_s = computed from the relation:

Table C.3. Fluid matrix characteristics (O'Brien, 1986).			
Type of Flow	Percent of Solids Concentration by Volume C_v	Percent of Solids Concentration by Weight C_w	Flow Characteristics
Landslide	>64%	>88%	Will not flow; failure by block sliding or tumbling. Unsaturated soil conditions.
Landslide	50%-64%	73%-88%	Will not flow; block sliding failure with some internal deformation, slow creep prior to failure; saturated
Mudflow	45%-50%	69%-73%	Flow initiates; plastic deformation with slow sustained creep. Begins spreading; moves subject to repeated vibration.
Mudflow	40%-45%	65%-69%	Mixes easily; shows some fluid properties. Surface may be inclined at rest. Waves dissipate rapidly.
Mud flood	35%-40%	59%-65%	Spreads on horizontal surface; marked particle settling, liquid horizontal surface, two-phase separation in quiescent condition;' waves travel easily.
Mud flood	30%-35%	54%-59%	Sand and gravel settle; distinct wave action.
Mud flood	20%-30%	41%-54%	Particles rest on bottom in wave motion.
Water flood	<20%	<41%	Water flood with bed and suspended load.

$$C_s = \frac{2.65 \times 10^6 Q_s}{(Q + 2.65\ Q_s)} \tag{C.5}$$

and

$$Q_s = q_s W \tag{C.6}$$

Concentrations exceeding C_{smax} result in non-Newtonian flow conditions, and the assumptions on which Equation C.3 (and the other bed-material transport equations that were previously discussed) were developed are no longer valid. For these conditions, procedures applicable for non-Newtonian fluids (e.g., mudflows) should be considered.

Note that Equation C.3 predicts bed-material load only; the total sediment load is computed by adding the bed-material and fine sediment loads, where the fine sediment load is computed from:

$$Q_{s_f} = \frac{C_f\ Q}{2.65 \times 10^6} \tag{C.7}$$

and the total sediment load is given by:

$$Q_{s_{total}} = Q_s + Q_{s_f} \tag{C.8}$$

The data used to develop Equation C.3 were derived using the following procedures and assumptions:

1. The bed load was computed by size fraction using the MPM bed-load equation.

2. The suspended-sediment load is computed for the median size only due to the mathematical formulation of the differential equation used in the computation.

3. The bed layer thickness used to estimate the reference concentration for the suspended- sediment computations was assumed to vary as a function of the ratio of the shear velocity (V_*) to the critical shear velocity (V_{*cr}) for the median particle size (Karim and Kennedy, 1983), limited to values between $2D_{50}$ and $2D_{84}$. The reference bed layer concentration was limited to values not exceeding 650,000 ppm by weight.

4. The viscosity of the fine sediment/water mixture was estimated using the following empirical relationship suggested by O'Brien and Julien (1989) and Julien and Lan (1991):

$$\mu = \alpha e^{\beta C_v} \tag{C.9}$$

where μ = dynamic viscosity in lb-s/ft^2

The values of α and β of 2.38×10^{-5} and 9.21, respectively, were developed from Figure 2 in Julien and Lan (1991) for the data sets excluding O'Brien and Julien (1989). These data were excluded because they were based on highly bentonitic clays in suspension which have a significant effect on the viscosity of the fluid that would not be expected to occur for materials in the SSCAFCA jurisdictional area (Julien, 1992, personal communication). The kinematic viscosity used in the suspended-load computation was computed from:

$$\nu = \frac{\mu}{\rho_f} \tag{C.10}$$

where ρ_f = bulk density of the water sediment mixture computed from:

$$\rho_f = \rho \quad [1-(1-S)C_v / 10^6] \tag{C.11}$$

where C_v = sediment concentration by volume

C_v is related to the fine sediment concentration by weight (C_f) by the relation:

$$C_v = \frac{C_f / 10^6}{SG-(C_f / 10^6)(SG-1)} \tag{C.12}$$

5. The fall velocity of the sediment was estimated using Ruby's equation (Simons and Sentürk, 1976) for clear water adjusted for the presence of fine sediment using the relation suggested by Maude and Whitmore (1958) given by:

$$\omega_s = \omega \left(1 - C_v / 10^6\right)^{3.5} \tag{C.13}$$

where ω = fall velocity in clear water
ω_s = fall velocity in the water/sediment mixture

APPENDIX D
Hydraulic Calculations

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Hydraulic Calculations

D.1. Normal Depth Calculations

Manning's equation can be written for discharge as:

$$Q = \frac{1.486}{n} A R^{\frac{2}{3}} S^{\frac{1}{2}} \tag{D.1}$$

Area and wetted perimeter for a trapezoidal channel may be expressed as a function of depth as follows:

$$A = zy^2 + by \tag{D.2}$$

where z = sideslope
b = bottom width
y = depth

Wetted perimeter is given by:

$$P = b + 2y(1 + z)\frac{1}{2} \tag{D.3}$$

Therefore, the discharge for normal depth (y_o) in a trapezoidal channel, is:

$$Q = \frac{1.486}{n} \frac{\left(z y_o^2 + b y_o\right)^{\frac{5}{3}} S^{\frac{1}{2}}}{\left[b + 2y_o (1 + z)^{\frac{1}{2}}\right]^{\frac{2}{3}}} \tag{D.4}$$

For a known discharge, this equation may be solved for y_o in terms of the other known parameters using an iterative technique such as Newton's method.

D.2. Normal Depth Calculations for Natural Channels

Using cross section profile data at a given cross section, it is often convenient to express the wetter perimeter (P) as a power function of the cross-sectional flow area (A) by regression:

$$P = a_1 A^{b_1} \tag{D.5}$$

Similarly, flow area may be related to flow depth as:

$$A = a_2 y^{b_2} \tag{D.6}$$

Here, a_1, a_2, b_1, and b_2 are statistically fitted coefficients and exponents. By using these expressions, hydraulic radius R in Equation D.1 may be expressed as a function of y as follows:

$$R = \frac{A}{P} = \frac{a_2 y^{b_2}}{a_1 \left(a_2 y^{b_2}\right)^{b_1}} = \frac{a_2}{a_1 a_2{}^{b_1}} y^{\left(b_2 - b_2 b_1\right)} \tag{D.7}$$

Equation D.1 can then be rewritten in terms of depth of flow as:

$$Q = \frac{1.486}{n} a_2 y_c{}^{b_2} \left[\frac{a_2}{a_1 a_2{}^{b_1}} y_o{}^{\left(b_2 - b_2 b_1\right)} \right]^{\frac{2}{3}} S_o{}^{\frac{1}{2}} \tag{D.8}$$

This equation can be directly solved for y_o, resulting in:

$$y_o = \left[\left(\frac{Qn}{1.486\, S_o{}^{\frac{1}{2}}} \right)^{\frac{3}{2}} \left(\frac{a_1}{a_2 \left(\frac{5}{2} - b^2 \right)} \right) \right]^{\left(\frac{2}{5b_2 - 2b_1 b_2} \right)} \tag{D.9}$$

D.3. Critical and Supercritical Flow Calculations

Froude Number $\left(F_r\right) = \dfrac{V}{\sqrt{gy}} = \dfrac{Q}{A\sqrt{gA/w}}$ (D.10)

where V = velocity
g = acceleration of gravity (32.2 ft/sec)
y = depth
A = area of section
w = width
Q = flow rate

Critical depth (Rectangular Section) = $y_c = 3\sqrt{\dfrac{Q^2}{gb^2}} = 3\sqrt{\dfrac{q^2}{g}}$ (D.11)

Critical velocity = $V_c = \sqrt{g\, y_c}$ (D.12)

Critical discharge (Q_c):

Channel Section	Equation	
Rectangular	$Q_c = \sqrt{g}\ y^{1.5}$	(D.13)
Trapezoidal	$Q_c = \dfrac{\sqrt{g}\ [(b + Zy)\,y]^{1.5}}{(b + 2Zy)^{0.5}}$	(D.14)
Circular	$Q_c = \dfrac{0.251(\theta - \sin\theta)^{1.5}}{(\sin 1/2\ \theta)^{0.5}} d_o^{2.5}$	(D.15)

y = depth
b = bottom width
z = sideslope
d_o = diameter

$$\theta = 2\,COS^{-1}\left(\frac{d_o / 2 - y}{d_o}\right) \tag{D.16}$$

Sequent depth (at hydraulic jump)

$$\frac{y_2}{y_1} = 1/2\left(\sqrt{1 + 8\ F_r^2} - 1\right) \tag{D.17}$$

where Y_1 = depth before hydraulic jump
Y_2 = depth after hydraulic jump

or

$$Y_2 = Y_1 + 0.5 * \left(\frac{A}{w}\right)\left(\sqrt{1 + 8F_r^2} - 3\right) \tag{D.18}$$

APPENDIX E
Example Problems

APPENDIX E

EXAMPLE PROBLEMS

Given an arroyo with the following characteristics:

- Relatively flat bottom with minor cross section irregularity, gradual variation in cross section shape,
- Negligible obstructions (small vegetation),
- Vertical banks – Bank Height (H_0)=3.0,
- Sparse vegetation within the active channel,
- Minor sinuosity,
- 100-year Peak discharge (Q_{100}) = 1,045 cfs,
- Channel width (W) = 39 feet,
- Bed slope (S_0) = 4%,
- Bed material size D_{50} = 1.9 mm, D_{84} = 4.2 mm, D_{16} = 0.48 mm, and
- Channel thalweg (minimum bed) elevation (Z) = 6,000 ft MSL.

I. HYDRAULICS

1. Compute normal depth, velocity and Froude Number for the peak of the 100-year storm.

   Use Manning's equation assuming wide rectangular channel (i.e., R~y) to compute normal depth:

$$q = V_y = \frac{1.49}{n} y^{5/3} \sqrt{S_f} \tag{3.3}$$

From continuity, q = Q/W_d = 1045/39 = 26.8 cfs/ft

Estimate Manning's n using Brownlie's equation for the base *n*-value (n_b) and Equation 3.11 to adjust n_b for channel irregularities, etc. Due to the steepness of the arroyo, upper regime flow is expected; therefore, use Equation 3.13 for n_b:

$$n_b = [1.0213\,(R/D_{50})^{0.0662}\,S_0^{0.0395}\,G^{0.1282}]\;0.034\,D_{50}^{0.167} \tag{3.13}$$

$$G = \frac{1}{2}\left(\frac{D_{84}}{D_{50}} + \frac{D_{50}}{D_{16}}\right) = \frac{1}{2}\left(\frac{4.2}{1.9} + \frac{1.9}{.48}\right) = 3.1$$

Assume R = 2 feet

$$n_b = \left[1.0213 \left\{ \frac{2}{(1.9/304.8)} \right\}^{0.0662} (0.04)^{.0395} (3.1)^{.1282} \right] 0.034 \left(\frac{1.9}{304.8} \right)^{0.167} = 0.022$$

From Equation 3.11 and Table 3.2:

$$n = (n_b + n_1 + n_2 + n_3 + n_4)\, m \qquad (3.11)$$

$$n = (0.022 + .004 + 0 + .002 + .007) 1.0 = 0.035$$

Rearrange Equation 3.3:

$$y = \left[\frac{qn}{1.486 \sqrt{S_o}} \right]^{\frac{3}{5}} = \left[\frac{(26.8)(.035)}{1.486 \sqrt{.04}} \right]^{\frac{3}{5}} = 2.0 \text{ ft}$$

By continuity:

$$v = \frac{q}{y} = \frac{26.8}{2.0} = 13.4 \text{ fps}$$

$$A = \frac{Q}{v} = \frac{1045}{13.4} = 78.0 \text{ ft}^2$$

2. Compute normal depth water-surface elevation.

$$WSEL_n = Z + Y_n = 6000 + 2.0 = 6002.0 \text{ ft MSL}$$

3. Compute energy gradeline elevation.

$$EGL = Z + Y_n + \frac{V^2}{2g} = 6000 + 2.0 + \frac{13.4^2}{2g} = 6004.8 \text{ ft MSL}$$

4. Compute critical depth (y_c) and critical water surface elevation ($CWSEL_{crit}$).

By continuity and Equation 3.23:

$$y_c = 3\sqrt{\frac{q^2}{g}}$$

$$y_c = 3\sqrt{\frac{26.8^2}{g}} = 2.8 \text{ ft}$$

$$CWSEL_{crit} = Z + Y_c = 6002.8$$

5. Compute conjugate (or sequent) depth and water surface elevation. (This is the expected high-water condition for natural channels. Bank protection and protection of structures will require appropriate freeboard.)

$$CWSEL_{seq} = CWSEL + 0.5\,(A/W)\left(\sqrt{1 + 8\,F_r^2} - 3\right) \tag{3.22}$$

$$CWSEL_{seq} = 6002 + 0.5\left(\frac{78}{39}\right)\left(\sqrt{1 + 8(1.67)^2}\right) = 6002 + 1.83$$

$$= 6003.83 \text{ ft MSL}$$

II. SUPERELEVATION ON A CURVE

At one location, the arroyo in the previous problem makes a bend. From available mapping, the radius of curvature of the bend (r_c) is estimated to be 195 feet.

1. Compute the superelevation on the outside of the curve for normal depth, at the peak of the 100-year storm.

$$\Delta Z = C\frac{v^2 W}{g r_c} \qquad (3.21)$$

From Table 3.3, with rapid flow, rectangular channel, and circular curve, C = 1.0.

$$\Delta Z = 1.0\frac{(13.4)^2\,(39)}{g(195)} = 1.1 \text{ feet}$$

2. Compute the expected water surface elevation on the outside of the bend.

$$\begin{aligned} CWSEL_{Bend} &= Z + y_n + \Delta Z \\ &= 6000 + 2.0 + 1.1 \\ &= 6003.1 \text{ ft MSL} \end{aligned}$$

III. SHEAR STRESS

Determine if the bed material is mobile at the peak of the 100-year flood.

1. Compute total average shear stress in the channel.

$$\tau_o = \gamma RS = \gamma y_n S = (62.4)(2.0)(0.04) = 5.0 \text{ psf} \tag{3.8}$$

2. Compute grain shear stress (τ'_o).

Combined Equations 3.9, 3.10, and B.3, where $k_s = 3.5\ D_{84}$

$$\tau_o = \frac{\rho V^2}{\left[5.75 \log\left(12.27 \frac{y_n}{k_s}\right)\right]^2} = \frac{(1.94)(13.4)^2}{\left\{5.75 \log\left[12.27 \frac{2}{(3.5)(4.2/304.8)}\right]\right\}^2} = 1.44 \text{ psf}$$

This result represents the grain roughness rather than the total roughness. The grain roughness is more realistic for determining bed material transport than the total shear.

3. Using the grain shear from the previous calculation, determine the critical grain size.

$$D_c = \frac{\tau_0}{0.047\,(\gamma_s - \gamma)} \tag{B.1}$$

$$= \frac{1.44}{.047[(2.65)(62.4) - 62.4]} = 0.30 \text{ feet} = 90.7 \text{ mm} \succ\succ 4.2 \text{ mm}$$

Is the bed mobile at the given discharge? Yes.

Is there potential for armoring? No, since $D_c >> D_{84}$ (and probably D_{100}).

IV. FINE SEDIMENT YIELD

The watershed upstream of the location analyzed in the previous problems has the following characteristics:

- Drainage area = 370 acres
- Watershed soil type: 52% Rock outcrop, Orthids Complex (ROF)
  48% Tesajo-Millett (Te)
- Percent impervious (roads, roofs, etc.) = 9.5%
- Average overland slope = 25%
- Average slope length = 100 feet
- Rangeland, grass-like plants, 10% ground cover, no canopy
- 100-year storm runoff volume = 40.2 ac-ft

1. Compute fine sediment yield from the watershed using MUSLE:

$$Y_s = C\ 95\ \left(V_w\ Q_p\right)^{0.56}\quad K\ \ LS\ \ C\ \ P \tag{A.1 \& A.2}$$

where C is a calibration factor (for the SSCAFCA jurisdictional area, use 3.0, unless data are available indicating a more appropriate value).

Estimate K from Table A.2.1 (see also SCS, 1992):

for Te use 0.1 - very gravelly, sandy loam and loamy sand
for ROF use 0.0 (Rock outcrop 40%, Orthids 30%)

Compute weighted K:

$$K' = \frac{\left(A_{ROF}\ K_{ROF} + A_{T\theta}\ K_{T\theta}\right)}{A_{total}} = \frac{0.52(0) + 0.48(0.1)}{1.0} = 0.048$$

Estimate C value from Table A.2.2:

C = 0.32

P = 1.0 (no terracing)

Estimate LS using Equation A.3:

$$LS = \left(\frac{\lambda}{72.6}\right)^n \left(0.065 + 0.0454S + 0.0065S^2\right) \tag{A.3}$$

$$= \left(\frac{100}{72.6}\right)^{0.5} \left[0.065 + 0.0454(9.45) + 0.0065(9.45)^2\right]$$

$$= 1.26$$

As discussed above, use a calibration factor (C) of 3.

$$Y_S = (3) \quad (95) \quad [(40.5)(1045)]^{0.56} \; (0.048)(1.26)(0.32)(1.0)$$

$= 2150$ tons fine sediment

This result assumes 100% of the watershed is pervious. Adjust for given percent impervious:

Y_s' = (1 - % impervious) Y_s = (1 - 0.095) (2150) = 1946 tons

2. Compute average fine sediment concentration from watershed for the 100-year storm:

$$C_f\ (ppm) = 10^6 \quad \frac{W_S}{W_W + W_S}$$

$$= 10^6 \, \frac{1946\ (2000)}{[(40.5)(43560)(62.4) + (1946)(2000)]}$$

$$= 34{,}147 \ ppm - w$$

V. BED MATERIAL AND TOTAL SEDIMENT LOAD

Compute the bed material transport capacity, total sediment load and bulking factors for the peak of the 100-year storm, for the arroyo in the previous problems.

1. Compute bed material transport capacity at Q_{100} = 1,045 cfs using Equation C.3.

From Figure C.2:

a' = 1.5×10^{-6}
b = 5.8
c = -0.7
d = -1.9

From hydraulics example problem:

V_{100} = 13.4 fps
y_{100} = 2.0 feet
W = 39 feet

Assume constant fine sediment yield throughout the storm:

C_f = 34,147 ppm-w

Apply Equation C.3:

$$q_s = a' V^b y^c \left(1 - C_f / 10^6\right)^d \tag{C.3}$$

$$= 1.5 \times 10^{-6} (13.4)^{5.8} (2.0)^{-0.7} \left(1 - \frac{34{,}147}{10^6}\right)^{-1.9} = 3.40\ cfs/ft$$

$$Q_s = q_s W$$

$$= (3.40)(39) = 132.6\ cfs$$

2. Compute the bed material concentration.

From Equation C.5:

$$C_s = \frac{2.65 \times 10^6\ Q_s}{(Q + 2.65\ Q_s)} \tag{C.5}$$

$$= \frac{2.65 \times 10^6\ (132.6)}{1045 + 2.65(132.6)}\ 251{,}642\ ppm - w$$

3. Compute the total sediment load.

Compute the wash load discharge (Q_f):

$$Q_f = \left(\frac{Q}{2.65}\right)\left(\frac{C_f}{10^6 - C_f}\right) \qquad \text{(rearranging C.7)}$$

$$= \left(\frac{1045}{2.65}\right)\left(\frac{34{,}147}{10^6 - 34{,}147}\right) = 13.9 \text{ cfs}$$

$$Q_{sTotal} = Q_s + Q_f$$

$$= 132.6 + 13.9 = 146.5 \text{ cfs}$$

4. Compute the total sediment concentration, bulking factor, and bulked peak discharge.

$$C_{sTotal} = \frac{2.65 \times 10^6\, Q_{sTotal}}{\left(Q + 2.65\, Q_{sTotal}\right)}$$

$$= \frac{2.65 \times 10^6\,(146.5)}{1045 + 2.65\,(146.5)}$$

$$= 270{,}875 \text{ ppm} - w$$

$$BF = \frac{1}{1 - \dfrac{C_{sTotal}/10^6}{2.65 - \left(C_{sTotal}/10^6\right)(S - 1)}} \qquad (3.41)$$

$$= \frac{1}{1 - \dfrac{270{,}875/10^6}{2.65 - \dfrac{270{,}875}{10^6}(2.65 - 1)}} = 1.14$$

$$Q_{P_{bulked}} = BF\, Q_P$$

VI. ANNUAL SEDIMENT YIELD

The following total sediment yield results were obtained by integrating the bed-material transport capacity and adding the fine sediment for each storm.

Return Period (years)	Water Yield (ac-ft)	Total Sediment Yield (tons)	Unit Sediment Yield (tons/acre)
100	40.2	6,142	16.6
50	33.8	4,863	13.1
25	27.5	3,774	10.2
10	19.4	2,413	6.5
5	13.7	1,559	4.2
2	7.1	668	1.8

Compute the mean annual water and sediment yields.

From Equation 3.26:

$$Y_{x\,Annual} = 0.015\,Y_{x\,100} + 0.015\,Y_{x\,50} + 0.04\,Y_{x\,25} + 0.08\,Y_{x\,10} + 0.20\,Y_{x\,5} + 0.40\,Y_{x\,2} \qquad (3.26)$$

where x = Either the total sediment or water yield.

(1) Water yield:

$$Y_{w\,a} = 0.015\,(40.2) + 0.015\,(33.8) + 0.04\,(27.5) + 0.08\,(19.4) + 0.2\,(13.7) + 0.4\,(7.1)$$
$$= 9.34\ ac - ft$$

Sediment yield:

$$Y_{s\,a} = .015\,(6142) + 0.015\,(4863) + 0.04\,(3774) + 0.08\,(2413) + 0.2\,(1559) + 0.4\,(668)$$

$$=1088\ tons$$

Unit sediment yield:

$$Y_{s\,a} = \frac{1088}{370} = 2.94\ tons/acre$$
$$= 2.94/3.4^{*} = 0.86\ ac\text{ - }ft/\ mi^{2}$$

(*assuming bulked unit weight of 100 pcf, see Constants and Conversions)

VII. EQUILIBRIUM SLOPE

For the given arroyo, estimate the equilibrium slope for the dominant discharge.

1. Estimate the dominant discharge (Q_D) from Equation 3.46:

$$Q_D = 0.2\, Q_{100} = (.2)(1045) = 209\ cfs$$

2. Estimate the hydraulic conditions for Q_D. Using procedures in the hydraulic example problem, the following results are obtained:

Velocity =7.1 fps
Hydraulic depth =0.76 feet

3. If the bed material supply at this discharge is 3.2 cfs (from similar analysis of the supply reach) estimate the equilibrium slope.

Method 1 - Use Equation 3.30:

$$S_{eq} = \left(\frac{a}{q_{s\,supply}}\right)^{\frac{10}{3(c-b)}} q^{\frac{2(2b+3c)}{3(c-b)}} \left(\frac{n}{1.486}\right)^2 \tag{3.30}$$

where a, b, c are given by Equation C.3:

$$Q_s = a' V^b Y^c (1 - (C_f)^d$$

a' = 1.5 x 10^{-6}
b = 5.8
c = -0.7
d = -1.9

$$q = Q/W = 210/39 = 5.38\ cfs/ft$$

$$q_{ssupply} = Q_{ssupply}/W = \frac{3.2}{39} = 0.082\ cfs/ft$$

If the fine sediment concentration for (C_f) Q_d is 10,000 ppm

$$a = a'\left(1-\left(C_f/10^6\right)\right)^d = 1.5\ x\,10^6 \left(1 - \frac{10{,}000}{10^6}\right)^{-1.9} = 1.53\ x\,10^{-6}$$

$$S_{eq} = \left(\frac{1.53\ x\,10^{-6}}{0.082}\right)^{\frac{10}{3(-0.7-5.8)}} (5.38)^{\frac{[2(2(5.8)+3(-0.7))]}{3(-0.7-5.8)}} \left(\frac{0.035}{1.486}\right)^2$$

$$= 0.029$$

Method 2 - Use Equation 3.31

$$S_{eq} = S_{existing} \left(\frac{Q_{s\ Supply}}{Q_{s\ Existing}} \right)^{\left(\frac{2}{(b-x)} \right)} \tag{3.31}$$

where

$$X = \left(\frac{3}{5} \right) \left(\frac{2b}{3} + c \right) = \left(\frac{3}{5} \right) \left[\frac{2(5.8)}{3} + (-0.7) \right] = 1.9$$

$$Q_{s\ Existing} = a' V^b Y^c \left(1 - \left(C_f / 10^6 \right) \right)^d W$$

$$= \left(1.5 \ x \ 10^{-6} \right) (7.1)^{2.8} (.76)^{-0.7} \left(1 - \frac{10{,}000}{10^6} \right)^{-1.9} (39) \quad = \quad 6.25 \ cfs$$

$$S_{eq} = 0.04 \left(\frac{3.2}{6.25} \right)^{\left(\frac{2}{(5.8 - 1.9)} \right)} = 0.028$$

What is the required spacing (L) of a series of grade control structures if the maximum drop height over one structure is 3 feet?

$$L = \frac{H_{max}}{\Delta S}$$

$$= \frac{3}{(0.04 - 0.028)} = 250 \ feet$$

Method 3 – Use Equation 3.33.

An alternative method for estimating the equilibrium slope is suggested in Chapter 3 that is based on the anticipated Froude number in the channel after the degradation has occurred. The Froude number for the existing channel in the above example is:

$$F_r = \frac{V}{\sqrt{gY}} = \frac{7.1}{\sqrt{32.2 * 0.76}} = 1.4$$

and the Froude number for the equilibrium channel would be about 1.2, assuming rigid boundary conditions. Supercritical flow typically occurs only over relatively short distances and short timeframes in erodible natural channels. Based on this factor alone, the bed slope should eventually adjust so that the Froude Number does not exceed 1. Use Equations 3.33 and 3.34 to estimate the slope for this condition:

$C = 18.28n^2F^{0.133}F_r^{2.133} = 18.28(0.035)^2(40)^{0.133}(1)^{2.133} = 3.66$

$S_s = CQ_D^{-0.133} = 3.66(209)^{-0.133} = 0.018$

VIII. CONTINUITY ANALYSIS

For the given arroyo, the bed material supply to the study reach during the 100-year storm is 4,685 tons and the existing bed material transport capacity is 4,107 tons [of the total yield of 6,142 (see Part VI)].

1. If the study reach is 470 feet long, estimate the average change in bed elevation that would be expected during the 100-year storm:

   Convert bed material load to solid volume:

   $$V_s = \frac{2000}{2.65\gamma} Y_s$$

   Supply:

   $$V_s = \frac{2000}{2.65\gamma}(4685) = 56{,}664\ ft^3$$

   Transport capacity:

   $$V_s = \frac{2000}{2.65\gamma}(4107) = 49{,}673\ ft^3$$

   Apply continuity equation

   $$\Delta V_s = V_{s\ Inflow} - V_{s\ Outflow} \tag{3.28}$$

   $$= 56{,}664 - 49{,}673 = 6{,}991\ ft^3$$

   Estimate average change in bed elevation from Equation 3.29, assuming porosity (η) = 0.4 (equivalent to in-situ unit weight of ~100 pcf)

   $$\Delta Z = \frac{\Delta V_s}{WL}(1-\eta) = \frac{6991}{(39)(470)(1-.4)} = 0.64\ feet\ (i.e., \sim 0.6\ of\ aggradation) \tag{3.29}$$

2. Assume that a culvert upstream of the study reach is partially blocked such that half of the bed material supply is trapped with no significant change in the hydrograph in the study reach. Recompute the average change in bed elevation.

   $$\Delta V_s = \frac{1}{2}(56{,}664) - 49{,}673$$

$$= -21{,}341\ ft^3$$

$$\Delta Z = \frac{-21{,}341}{(39)(470)(1-0.4)} = -1.94\ ft\ (i.e., \sim 1.9\ feet\ of\ degradation)$$

X. LATERAL EROSION

In the equilibrium slope example, it was determined that approximately 1.2 feet of degradation per 100 feet of channel length will occur under the given conditions (i.e., ΔZ/100 = (0.4 - .028) 100 = 1.2). Assume the arroyo in the study reach has an average bank height of approximately 3 feet and the in-situ overbank soils have cohesion of –200 psf and internal friction angle of –30E.

1. Compute the required spacing of a series of grade control structures to avoid exceeding the critical bank height.

   Estimate the critical bank height from Figure 3.15:

   $$H_c = 9.2\ ft$$

   Compute the maximum allowable degradation.

   $$\Delta Z_{max} = H_c - H_0 = 9.2 - 3.0 = 6.2\ ft$$

   Compute the maximum spacing of the structures.

   $$L = \Delta Z_{max} / \Delta S = 6.2/(0.04 - 0.028) = 517\ ft$$

2. Estimate the maximum lateral erosion distance (Δ_{max}) that can be expected for the given arroyo based on an idealized bend shape.

   Estimate the meander wavelength and unconstrained bend length from Equation 3.43b:

   $$\lambda / W_D = 0.8 + 4 \log Q_D \qquad (3.43b)$$

   $$= 0.8 + 4 \log (290) = 10.1$$

   Compute the dominant channel width.

   Estimate the critical slope from Equation 3.48:

   $$S_c = 0.037\ Q_d^{-0.133} = 0.018 \prec S_o = 0.04 \qquad (3.48)$$

   Since $S_0 > S_c$ use Equation 3.36 to compute W_D:

   $$W_D = 4.6\ Q_D^{0.4} = 39\ feet \qquad (3.36)$$

   $$\lambda = 10.1(39) = 394\ feet$$

   $$L_{BU} = \lambda/2 = 197\ feet$$

From Equation 3.44b:

$$\Delta_{max} = [0.2 + \log Q_D]\, W_D \tag{3.44b}$$

$$= [0.2 + \log(209)](39) = 98.4 \text{ feet}$$

or, alternatively, from Equation 3.49b:

$$\Delta_{max} = [0.92 + 4.6 \log Q_D]\, Q_D^{0.4}$$

$$= [0.92 + 4.6 \log(209)]\,(209)^{0.4} = 98.4 \text{ feet} \tag{3.49b}$$

For this example, Δ_{max} represents the bankline setback (BSB). The centerline setback is BSB + $W_{D/2}$ = 117.9.

3. The limits of the erosion envelope can be reduced by constructing lateral controls at intervals less than the unconstrained bend length. Assuming that the desired distance of the erosion envelope from the centerline of the downvalley direction is 50 feet, what is the required spacing of the lateral controls (L_s)?

Compute the ratio of maximum offset to unconstrained offset.

$$\Delta_{max} / \Delta_{max\,u} = 50/98.4 = 0.51$$

From Figure 3.19:

$L_s / (\lambda/2) = 0.72$

$L_s = L_{BU}\,(0.72) = (197)\,(0.72) = 142$ feet

X. CHECK DAMS (DROPS)

A. Check dams are used to mitigate vertical instability problems.

- Used to maintain a stable bed elevation.
- Arrest head cuts.

B. Design considerations.

- Scour depth downstream of drop;
- Number of drop structures;
- Bank erosion.

C. Design problem.

Given:

Design Discharge	Q	= 1000 ft^3/s
Channel Width	B	= 40 feet
Mean Water Depth*	d_m	= 2.7 feet
Unit Discharge	q	= 25 $ft^3/s/ft$
Mean Velocity*	V	= 9.26 ft/s
Total Drop Height	h	= 5 feet
Maximum Allowable Scour Depth	d_s	= 7 feet

*For this example, the depth and velocity is the same up- and downstream.

From Energy Equation: $Z_u - Z_d = H_T$ = drop height

See Figure 3.22 for definition of variables.

1. Determine the maximum scour depth (single check dam).

Use the Veronese Equation (Equation 3.57)

$$d_s = 1.32\, H_t^{0.225}\, q^{0.54} - d_m$$

$$= 1.32\,(5.0)^{.225}\,(25)^{.54} - 2.7$$

$$= 8.08\ ft$$

2. Use multiple drops if scour is too deep.

Decrease drop height and use more drops.

Estimate drop for scour depth of 7 feet:

$$7 = 1.32\, H_{max}^{.225} (25)^{.54} - 2.7$$

$$H_{max} = 3.12\ ft$$

Re-compute scour depth.

Number of equally spaced drops required = 2

Drop height per drop = 2.50 feet

$$d_s = (1.32)(2.5)^{.225} (25)^{.54} - 2.7 = 6.5\ ft \prec 7'$$

Scour depth per drop, d_s = 6.5 feet < 7' (given)

3. Adequately protect banks downstream of check dam.

   a. Briefly discuss extent of riprap protection.

      5 to 10 times d_s in downstream direction

   b. Briefly discuss any other considerations which must be specified in the design.

      Use equilibrium slope to determine spacing of drops and the total drop height (see previous example).

XI. LOCAL SCOUR AT FLOOD WALL

A flood wall will be constructed along one side of the given arroyo to limit the lateral erosion potential. The unconstrained valley width after construction of the wall will be 82 feet. Estimate the scour depth along the wall at the peak of the 100-year flood.

Estimate the dominant arroyo width (W_D):

$$Q_D = 0.2 * Q_{100} = (0.2)(500) = 100 \text{ cfs}$$

Check critical slope (S_c):

$$S_c = 0.037 * Q_D^{-0.133}$$

$$S_c = 0.037(100)^{-0.133} = 0.02 \prec S_0 = 0.022$$

Since $S_0 > S_c$,

$$W_D = 4.6\,Q^{0.4}$$

$$W_D = (4.6)(100)^{0.4} = 29.0 \text{ feet}$$

Estimate the ratio of unconstrained valley width to channel width:

$$W_v / W_D = 82/29 = 2.83$$

Determine the Froude number for the 100-year flood peak:

Find the normal depth from Manning's equation:

$$y = \frac{(\frac{Q}{W})n}{1.486\sqrt{S_0}}$$

$$y = [(500/29) * 0.035/1.486/\sqrt{0.022}\,]^{3/5} = 1.83 \text{ feet}$$

$$V\ Q / W / y$$

$$V = 500/29/1.83 = 9.42 \text{ fps}$$

$$F_r = (9.42)/\sqrt{(g)(1.83)} = 1.23$$

From Figure 3.24:

$D_{SC}/y = 4.4$

$D_{SC} = (4.4)(1.83) = 8.0\ feet$

APPENDIX F

Mussetter, R.A. and Harvey, M.D., 2005. Design Discharges for Arroyos in an Urban Setting. Proceedings of the EWRI 2005 World Water and Environmental Resources Congress, Anchorage, ASCE, Alaska, May 15-19.

Design Discharges for Arroyos in an Urban Setting

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Abstract

Perennial, mobile-boundary streams tend to adjust their shape, planform, and gradient to the range of commonly occurring flows, and the hydrologic and sediment transport characteristics of these streams can be reasonably predicted through the use of flow duration curves, standard flood frequency analyses and sediment transport computations. As a result, design discharges for channel restoration might logically include the effective discharge (i.e., the range of discharges that carries the most sediment over the long term) for establishing the cross sections and planform of the primary channel, and consideration of larger flood flows in the 10- to 100-year recurrence-interval range to insure stability of critical project features and the required level of flood protection. Because of their ephemeral flow conditions and highly erodible boundary materials, a similar approach to identifying design discharges for restoration or habitat enhancement in arroyos may not be successful because the condition of the channel is often largely the product of the last major flood. In addition, arroyos tend to respond rapidly and dramatically to urbanization and other land-use changes due to their effects on both runoff and sediment supply; thus, the long-term conditions that will occur after implementation of a project may be very different from those leading up to the project. This paper discusses important issues associated with establishing appropriate design discharges for restoration or habitat enhancement in arroyos, with particular focus on the urban setting where the watershed discharge and sediment loads will undergo progressive change as urbanization continues.

Introduction

Selection of the appropriate design discharge (or design discharges) for a stream restoration project requires a clear understanding of the objectives that are to be met by each component of the project. For public safety purposes, the design discharge is primarily determined by the acceptable level of risk of flooding or erosion damage to adjacent properties and infrastructure, as well as the acceptable level of the risk that certain project features could be damaged or destroyed during high flows. As a result, the design discharge for project components associated with public safety issues tend to be infrequent, high flows in the range of the 100-year flood. From an ecological perspective, the primary objectives are typically related to insuring that the restored channel functions, to the extent possible, like a natural stream. Specific objectives include achieving a quasi-equilibrium state between the sediment supply and transport capacity, bed mobilization at sufficient frequency to protect and rejuvenate instream habitat, and connectivity between the floodplain and channel on a regular basis. As a result, design discharges for these project features include flows in the range of the mean annual flood, but they must also consider the full range of frequently occurring flows that are capable of mobilizing and transporting the bed material.

Because of the highly discontinuous nature (both temporally and spatially) of hydrologic and sediment transport processes in arid-land arroyos, the possibility of achieving even approximate equilibrium is unlikely in most cases (Graf, 1988). As a result, it may not be possible to identify an equilibrium channel form that can serve as the basis for establishing the dimensions of the reconstructed channel, and the design discharges must be identified by other criteria.

Current Practices in Selecting Design Discharges for Channel Function

As described in several other papers in this and the accompanying sessions, the criteria for selecting the design discharge to meet ecological and stream function objectives typically center on the concept of the channel-forming discharge. This discharge is also referred to as the effective or dominant discharge, and the concept is based on the concept that, for a particular geomorphic setting, there exists a single, steady discharge that, if it occurred for a sufficient period of time, would produce channel dimensions equivalent to those produced by the natural long-term hydrograph (Inglis, 1941). When such a discharge can reasonably be defined, this provides a basis for determining the appropriate size and shape of the restored channel.

The effective discharge can be quantified by determining the relative amount of the sediment load that is transported during each increment of flow based on a combination of the instantaneous transport rate and the duration over which each increment of flow occurs (Biedenharn, et al, 2000). Plots of the resulting incremental transport volumes with discharge for perennial streams typically have a bell-shaped curve, and the effective discharge is taken as the peak of the curve (**Figure 1**). Numerous researchers have demonstrated that the effective, or channel-forming, discharge is approximately equivalent to the bankfull discharge (Inglis, 1941; Simons and Albertson, 1960; Kellerhals, 1967; Andrews, 1980). One explanation for this relationship is that, in self-adjusted streams that are bounded by an alluvial floodplain, the in-channel shear stress, and thus sediment transport capacity, increases relatively rapidly with increasing discharge, up to the point at which flow begins to spill onto the floodplain, but then levels off at higher flows. Several researchers have also suggested that the bankfull discharge has a recurrence interval in the range of the mean annual flood peak (Wolman and Leopold, 1957; Dury, 1973; Andrews, 1980), although others have shown that the recurrence interval of the bankfull discharge can vary dramatically (Williams, 1978). One of the fundamental assumptions behind using the effective discharge concept as the basis for establishing the design discharge for stream restoration is that streams tend toward a state of dynamic equilibrium with the upstream water and sediment supply, and the bankfull channel corresponds to the desired equilibrium form (Graf, 1988).

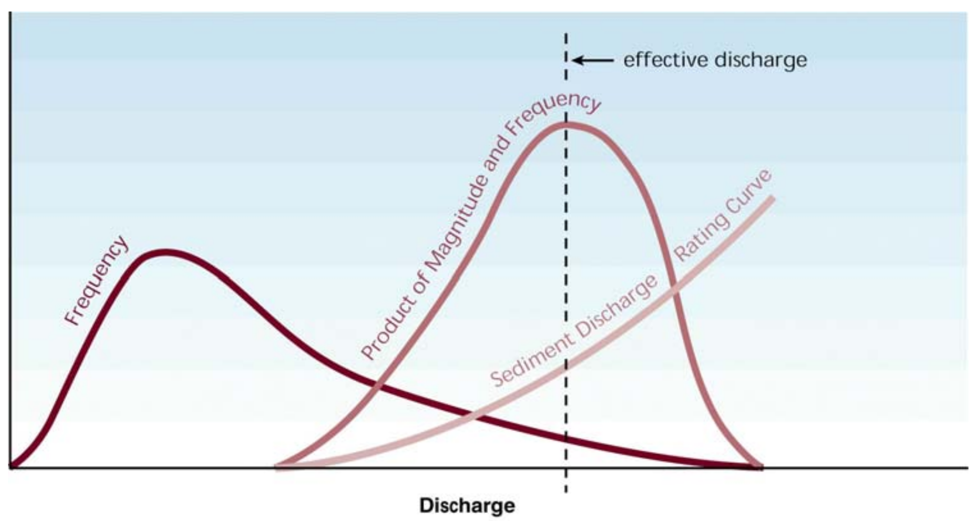

Figure 1. Determination of effective discharge from sediment-rating and flow-duration curves. (FISRWG, 1998; after Wolman and Miller, 1960).

Challenges in Selecting Design Discharges for Urban Arroyos

Identification of a self-adjusted, bankfull cross section that corresponds to a frequently occurring flow is difficult in many environments (Woodyer, 1976), but it is especially problematic in arroyos that are frequently incised, excessively broad and braided (Graf, 1988). Because the precipitation and runoff inputs to arroyos are so strongly discontinuous, the relative difference between high and low flows is much greater than in humid region streams, and *these wide fluctuations prevent the development of an intimate linkage between a particular discharge magnitude and channel geometry related to "bankfull" conditions* (Graf, 1988). Large floods have a strong influence on channel geometry, and the resulting geometry confines subsequent, lower magnitude flows. In this sense, then, *process controls form in high magnitude events, while form controls process in low magnitude events* (Graf, 1988). Discharges in the range of the mean annual (1.5- to 2.33-year recurrence interval) flood are, therefore, probably not the most geomorphically significant flows in arroyos in either undeveloped or urbanized settings. As a result, it is tempting to suggest that urbanized arroyos should be designed primarily to meet public safety objectives.

In recent decades, however, there has been increasing emphasis on preserving or restoring habitat in urban channels (Lagasse et al., 1991; Mussetter et al., 1994). The desire to meet this objective can lead to conflict with public safety objectives because traditional engineering approaches to flood- and erosion-control that provide high factors of safety are often incompatible with good habitat characteristics. Bioengineering and other "soft" treatments provide opportunities to achieve both safety and environmental objectives, but their factor of safety are often much lower than can be achieved with traditional, "hard" engineering approaches, and many of the more commonly used bioengineering approaches are incompatible with the climate and soil conditions associated with arroyos. In addition, arroyos that drain urbanized areas in the arid southwestern U.S. present a particularly challenging problem in this regard because they tend to be very sensitive to changes in water and sediment supply, and advocates for increased open space and environmental protection often have a mistaken view of the ultimate stability and appearance of the drainageway after it responds to the effects of urbanization.

The dramatic effects of urbanization on water and sediment yields, and the resulting channel responses are well documented (Wolman, 1967; Graf, 1975; Morisawa and LaFlure, 1979; Harvey et al., 1983; von Guerard, 1989; Urbonas and Benik, 1995; Mussetter et al., 1994; Stogner, 2000; Nanson and Young, 1981; Park, 1977). Hydrologic effects associated with urbanization include reduction in infiltration and depression storage, which increases runoff volume and decreases the time of concentration, resulting in higher flood peaks (FISRWG, 1998). Hollis (1975), for example, found that the annual direct runoff after urbanization can increase by up to 2.5 times and flood peaks can increase by up to 10 times their pre-urbanization levels. Wilson (1967) showed that the size of the mean annual flood can increase by a factor of 4 following urbanization. In addition, significant runoff can occur in response to frequent rainstorm events that did not produce runoff under pre-urbanization conditions. Rainfall/runoff modeling of the West Branch of Calabacillas Arroyo in Albuquerque, for example, indicates that the 2-year rainstorm produced no runoff in the downstream portion of the reach under undeveloped conditions, but under urbanized conditions, the same storm would produce a peak flow of about 28 cms with 195,000 m^3 of runoff (**Figure 2**; Mussetter Engineering, Inc., 1999). Similarly, the 10-year storm would produce a peak discharge of about 5.9 cms and 54,000 m^3 of runoff under undeveloped conditions, and these would increase to about 68 cms and 408,000 m^3, respectively, under urbanized conditions.

While sediment yields can increase during the construction phase of urbanization, they tend to decline to pre-urbanization levels or less after construction is complete (Wolman, 1967; Wolman and Schick, 1967). In the West Branch of Calabacillas Arroyo, for example, the contribution of the frequently occurring (2- to 5-year) storms to the long-term, average annual sediment load is quite small under undeveloped conditions because these storms typically produce little, if any, runoff (**Figure 3**[1]). Although the larger events transport large quantities of sediment, their annual

[1] The long-term, average annual sediment load can be estimated by considering the combination of the individual storm volume and the probability of occurrence of each storm during any given year (Chang, 1988). Applying the trapezoidal rule to the sediment yield frequency curve results in the following equation:

$$Y_s=0.015Y_{s\_100}+ 0.015Y_{s\_50}+ 0.04Y_{s\_25}\ 0.08Y_{s\_10}+ 0.2Y_{s\_5}+ 0.4Y_{s\_2}$$

where Y_s is the long-term, average annual sediment yield, Y_{s_i} is the sediment yield associated with i-year recurrence interval event, and $A*Y_{s\_i}$ is the incremental contribution of i-year recurrence interval event to the annual sediment yield (Mussetter et al., 1994).

contribution is relatively small because they occur infrequently. The resulting channel has a shallow, essentially flat-bottomed cross section that varies from 5 to 10 feet wide in the upstream part of the basin to 25 to 30 feet wide in the downstream portion of the basin (**Figure 4**). Under developed conditions, even the frequent storms will produce significant runoff. During the early periods when the arroyo is adjusting to the increased flow, these storms will contribute a large proportion of the annual sediment load because each time they occur, they transport substantial sediment, and they occur relatively frequently.

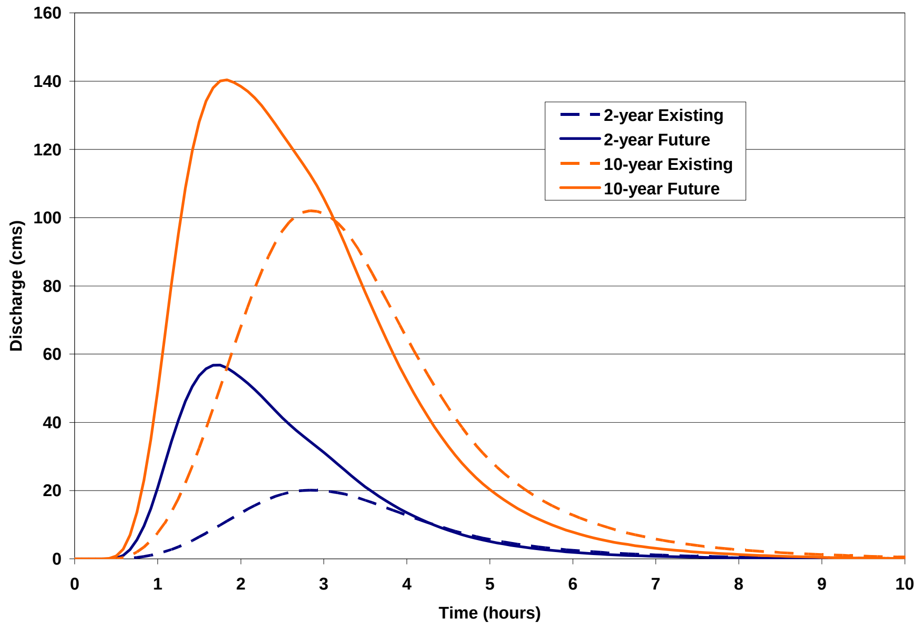

Figure 2. Undeveloped- and developed-conditions hydrographs, West Branch Calabacillas Arroyo, New Mexico.

The combination of increased runoff and decreased sediment supply typically causes channel incision which leads to bank failure and channel widening (Richards, 1982; Wolman and Schick, 1967; Hammer, 1972; Park, 1977; von Guerard, 1989; Richards and Wood, 1977; Harvey et al., 1983; Schumm et al., 1984) (**Figure 5**). When stability thresholds are exceeded, systematic disequilibrium occurs and recovery to a new state of equilibrium may be complex and gradual, following a predictable sequence of adjustments that are described by the Incised Channel Evolution Model (ICEM) (**Figure 6;** Schumm, 1977; Schumm et al., 1984). The most significant hazard to public safety along the incised reach is typically related to lateral erosion into property improvements adjacent to the channel rather than flooding, because the capacity of the incised channel is typically quite large. Of course, the sediment that is eroded during the incision process is carried downstream, where it can deposit in low energy zones, decreasing channel capacity, which can lead to increased flooding problems, and causing channel widening and lateral instability that can also be a hazard to adjacent property improvements.

The Albuquerque Metropolitan Arroyo Flood Control Authority (AMAFCA) and other municipalities have implemented the Prudent Line concept in an effort to balance the often competing public safety and environmental objectives associated with planning and designing features in urban arroyos. The Prudent Line concept is based on the idea that a corridor can be defined along an arroyo within which it would not be *prudent* to construct property improvements due to the potential for damage associated with erosion, flooding or a combination of the two. The perceived benefit of the resulting undeveloped corridor is that is maintains open space and a naturalistic environment while providing the required level of public safety. In the absence of structural protection, the width of the buffer zone within the Prudent Line corridor is often sufficiently large that it creates political and economic conflict between planning and resource agencies, property owners and environmental interests. Unfortunately, many advocates for open space have a mistaken view of the ultimate stability; appearance and habitat quality in *naturalistic* arroyos after their watersheds have been urbanized.

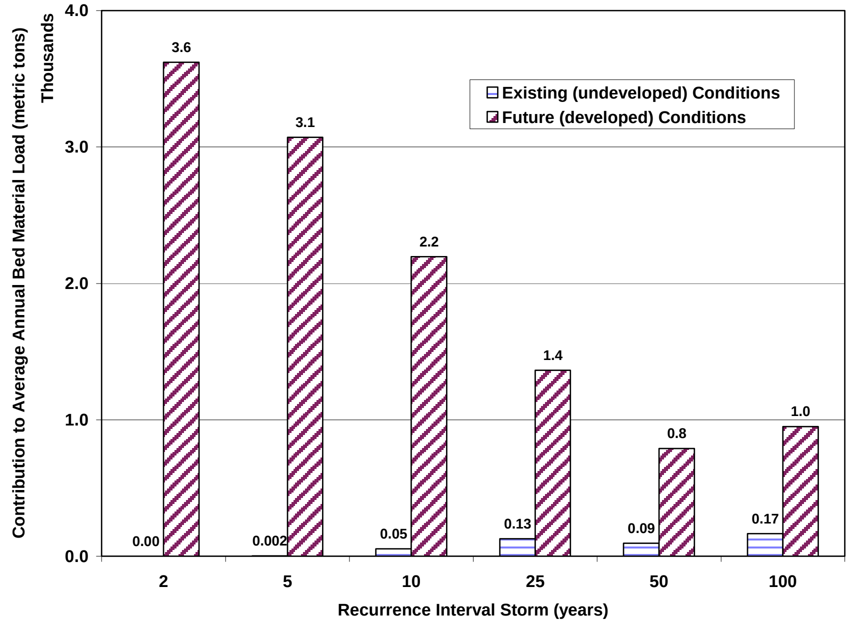

Figure 3. Contribution of the 2-, 5-, 10-, 25-, 50- and 100-year storms to the long-term, average annual bed-material load, West Branch of Calabacillas Arroyo, New Mexico.

Figure 4. Unincised reach of the West Branch of Calabacillas Arroyo, Albuquerque, New Mexico.

Figure 5. Incised channel typical illustrating typical effects of urbanization (Black Squirrel Creek, Colorado).

Considering the importance of public infrastructure and the political pressure to preserve existing, high value real estate, locations will exist along most Prudent Line corridors where unconstrained erosion will not be acceptable, and structural measures will be required. The outstanding questions are then: (1) what are the appropriate design discharges for estimating the ultimate flooding and erosion corridor along the arroyos where structural protection is not provided, and (2) what are the appropriate design discharges for designing protection measures where they are necessary?

Recommendations for Selecting Design Discharges for Arroyos

The infrequent, high magnitude flows in the range of the 100-year flood must be included among the design discharges for certain aspects of the project because this is the well-accepted level of protection that is required by planning and regulatory agencies, including the Federal Emergency Management Agency (FEMA). In general, the flooding and erosion corridor (i.e., the Prudent Line) must be capable of passing the 100-year flood without damage to adjacent property improvements. To provide a *safe* mechanism for allowing urbanized arroyos to remain in a *naturalistic* condition, AMAFCA has defined the Prudent Line as the line along the arroyos that has a low probability of being disturbed by erosion, scour or meandering of the arroyo by any storm up to and including the 100-year storm occurring at any time during a 30-year period. Under this definition, protection measures that are required to preserve the corridor in areas where the erosion or flooding limits would be unacceptably wide if left unprotect must clearly be designed to withstand the hydraulic forces and scour potential associated with the 100-year storm.

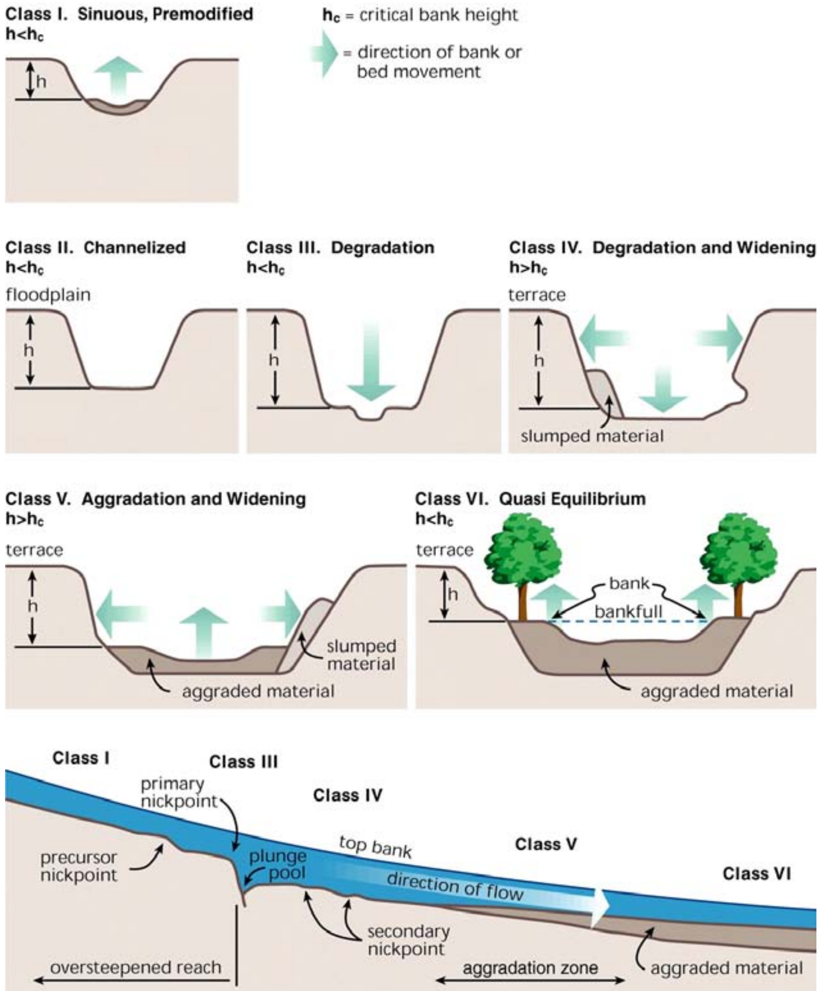

Figure 6. Incised Channel Evolution Model (from FISWRG, 1988; after Schumm et al., 1984).

The channel size and planform that will develop over the long-term as a result of the full range of flows including frequent smaller storms is less certain. In spite of the lack of a strong relationship between any particular discharge and the channel geometry related to the bankfull condition, natural arroyos typically have well-defined channels in which at least the width tends to scale to the size of the watershed. Based on observations from a large number of arroyos in the Albuquerque, New Mexico area, Mussetter et al. (1994) suggested that the active channel will adjust its width so that it has a width-to-depth ratio of about 40 at the peak discharge associated with the equivalent storm event that would transport the long-term, average annual sediment load. By interpolation of the sediment yield frequency curve that is discussed in Footnote[1], this storm typically has a recurrence interval in the range of 5 to 10 years for watersheds that are mostly undeveloped. The recurrence interval of the storm that produces the average annual sediment yield decreases to the 2- to 5-year range in urbanized watersheds, due primarily to the increase in runoff for the more frequent storms compared to undeveloped conditions.

Conclusions

Design discharges for restoration of perennial streams that are in quasi-equilibrium with the upstream water and sediment supply are typically selected based on the dual objectives of protecting public safety and, to the extent possible, insuring natural stream function which might include periodic mobilization of the bed material, energy to transport the upstream sediment supply, and connectivity between the channel and floodplain on a relatively frequent basis. The basis for designing the channel often centers on a bankfull discharge that is typically in the range of the mean annual (1.5- to 2.33-year flood peak) based on the idea that the channel-forming discharge (a.k.a., the dominant or effective discharge) is about the same as the bankfull discharge, and this represent the long-term condition to which the channel will adjust.

Because the precipitation and runoff response of arroyos is highly discontinuous and frequently occurring storms tend to produce little or no runoff under undeveloped conditions, arroyos represent a nonequilibrium form that is the product of less frequent, higher magnitude flows. As the watershed urbanizes, the relative contribution of more frequent storm events to the annual sediment load and the long-term condition of the arroyo tends to increase due to reduced infiltration and depression storage and increases in the hydraulic efficiency of the local drainage channels that to feed the arroyo. Based on data and observations throughout the Albuquerque area, as well as other arid-land streams, it is suggested that the peak discharge associated with the individual storm event that would produce the long-term, average annual sediment load provides a reasonable basis for estimating appropriate channel size and planform for reconstructing or rehabilitating arroyos that are subjected to the effects of urbanization.

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APPENDIX G
Bed-material Particle Size Gradations for Collected Samples

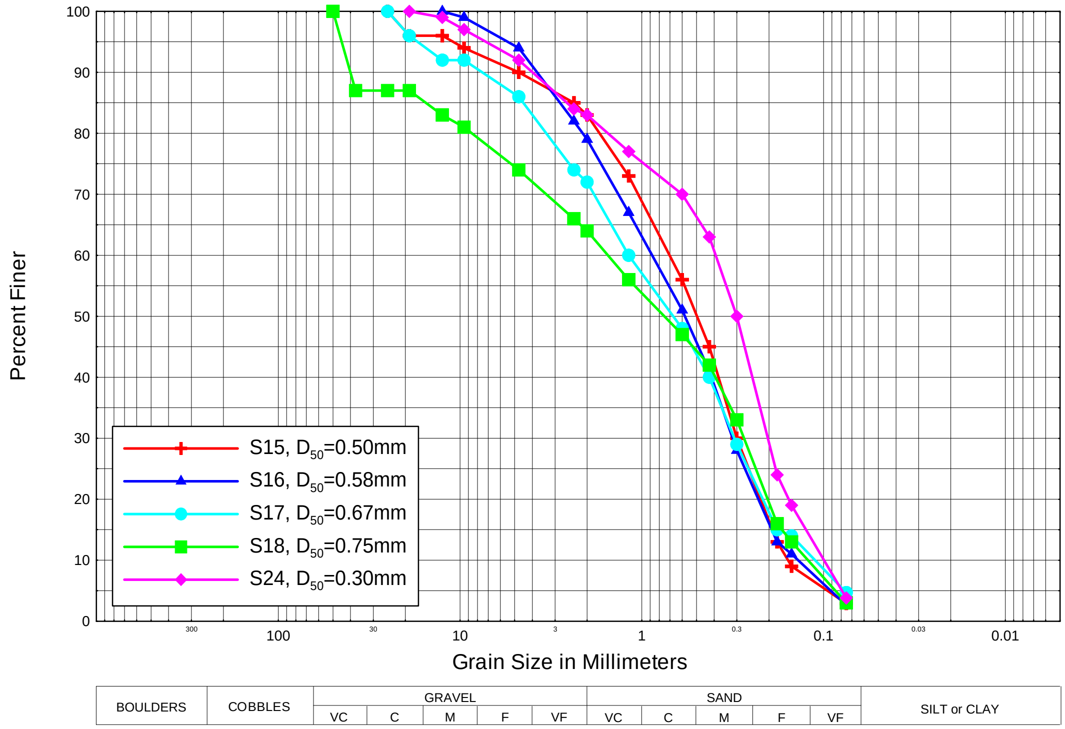

Figure G.1. Bed material particle size gradations for samples collected in La Barranca Arroyo by MEI in January 2008.

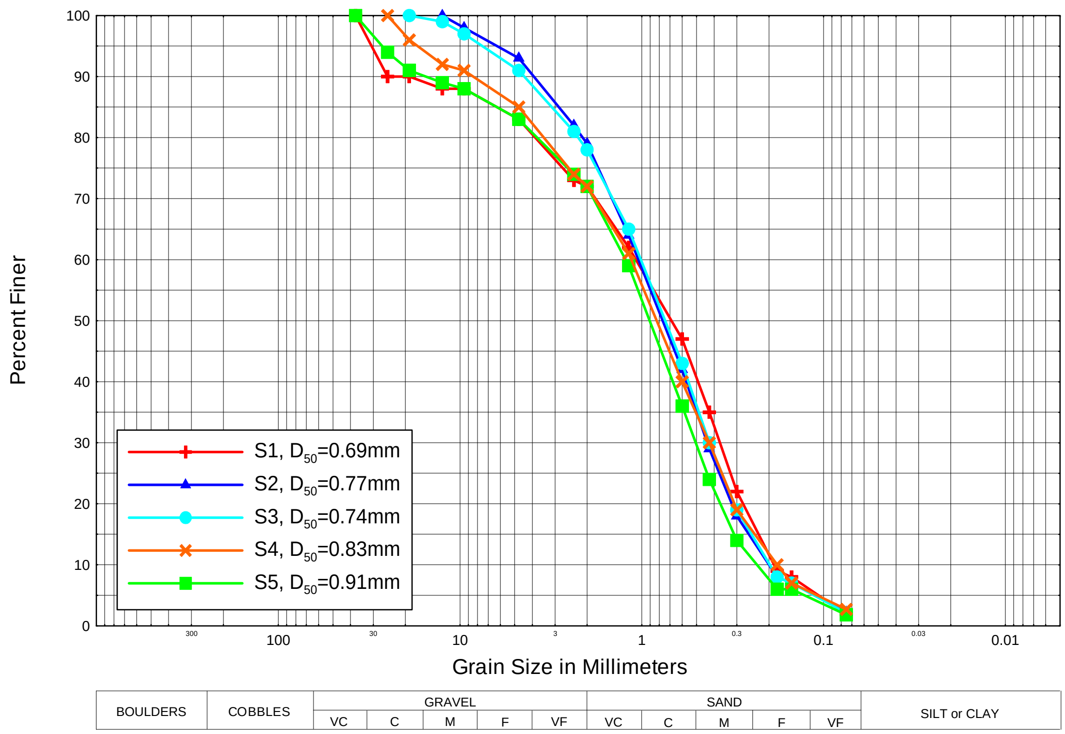

Figure G.2. Bed material particle size gradations for samples collected in Calabacillas Arroyo by MEI in January 2008.

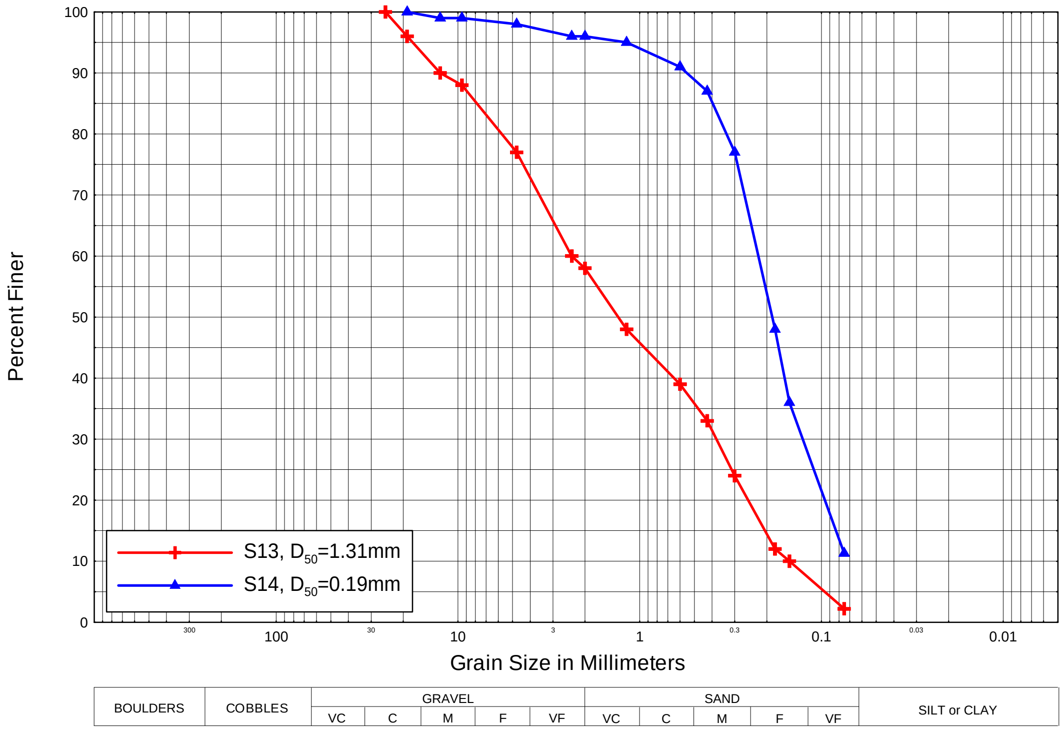

Figure G.3. Bed material particle size gradations for samples collected in Lomitas Negras Arroyo by MEI in January 2008.

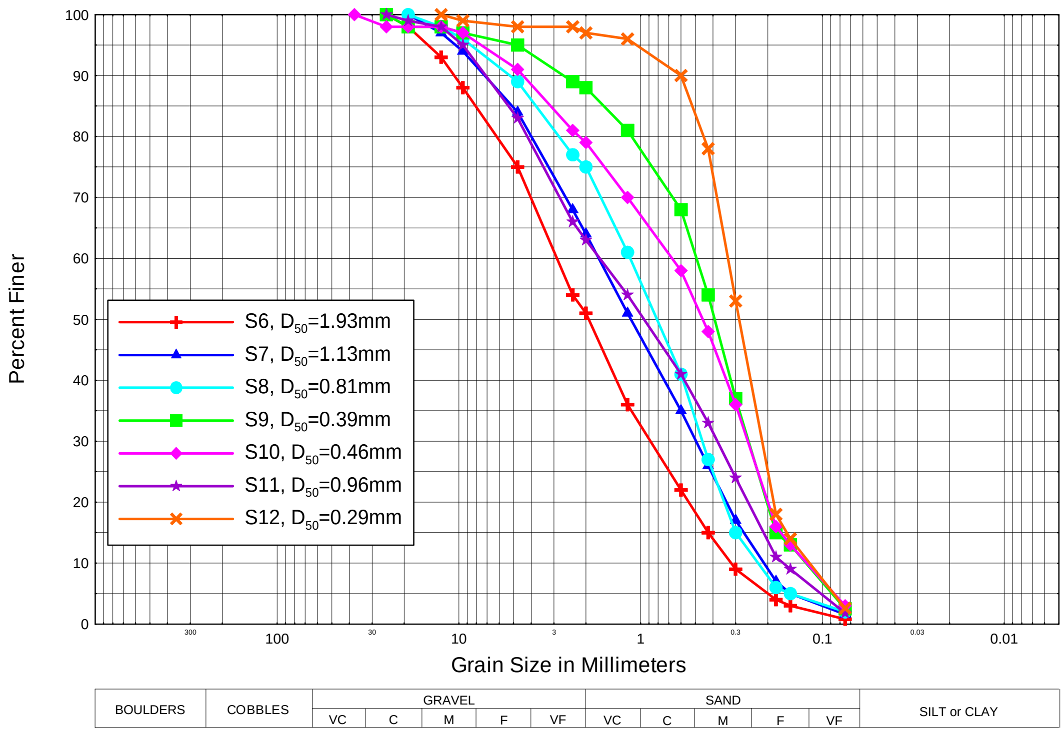

Figure G.4. Bed material particle size gradations for samples collected in Montoyas Arroyo by MEI in January 2008.

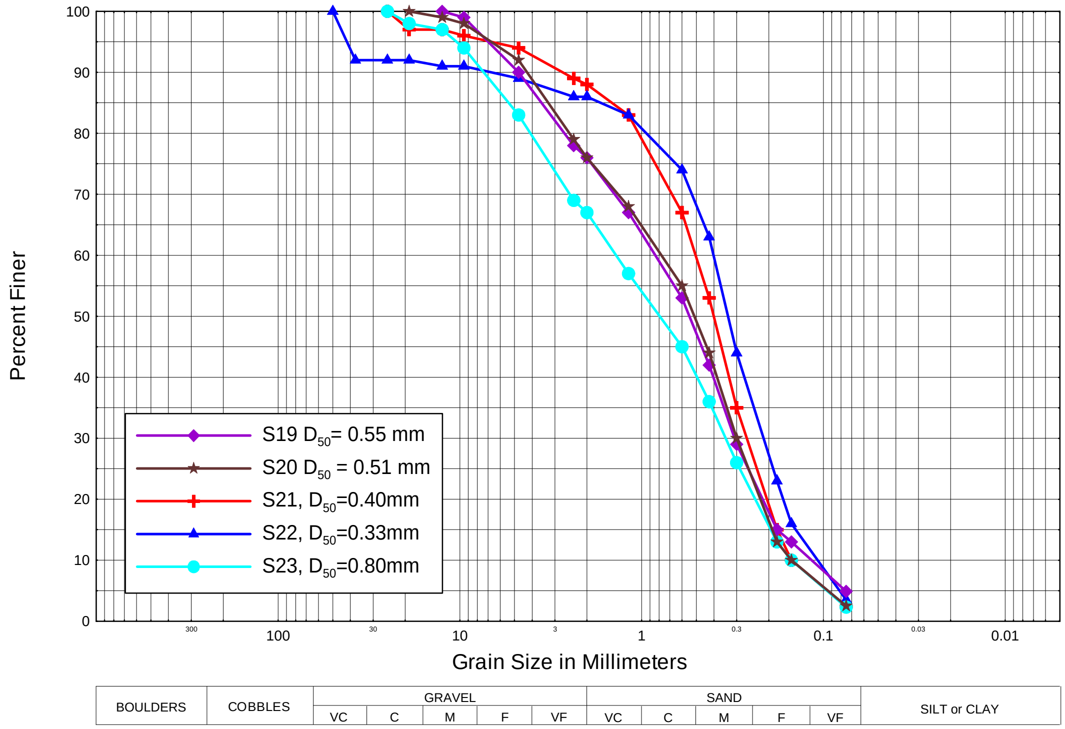

Figure G.4. Bed material particle size gradations for samples collected in Venada Arroyo by MEI in January 2008.