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Hydraulic Engineering Circular No. 18

Evaluating Scour at Bridges

Office of Research and Development
Turner-Fairbank Highway Research Center
6300 Georgetown Pike
McLean, Virginia 22101-2296

U.S. Department of Transportation
Federal Highway Administration

Memorandum

Washington, DC 20590

Subject: Implementation Packages:
Evaluating Scour at Bridges, FHWA-IP-90-017
Stream Stability at Highway Structures, FHWA-IP-90-014

Date: MAR 29 1991

From: Director, Office of Technology Applications
Director, Office of Engineering

Reply to Attn of: HTA-22

To: Regional Federal Highway Administrators
Federal Lands Highway Program Administrator

These manuals provide guidelines for the hydraulic evaluation of the stability of highway bridges and streams. Both manuals are included in the Office of Engineering design guideline series titled, *Hydraulic Engineering Circulars* (HEC). The HEC's are intended to provide state-of-the-art procedures for highway hydraulic design. Both are covered in NHI training course No. 13046, *Stream Stability and Scour at Highway Structures*.

Evaluating Scour at Bridges is referred to as HEC 18. This manual provides procedures for designing new bridges to resist scour damage and for evaluating the scour vulnerability of existing bridges. HEC 18 replaces *Interim Procedures for Evaluating Scour at Bridges*, which was distributed with FHWA Technical Advisory T 5140.20, dated September 16, 1988 (revised November 7, 1988). The Technical Advisory is referenced in Item 113 of the FHWA *Recording and Coding Guide for the Structural Inventory and Appraisal of the Nation's Bridges* as the recommended procedures for making scour critical bridge determinations.

Stream Stability at Highway Structures is referred to as HEC 20. This manual provides procedures for classifying streams, assessing the lateral and vertical stability of the stream boundary, identifying appropriate countermeasures, and designing selected countermeasures. HEC 20 is in part a concise implementation of the more comprehensive river mechanics text, *Highways in the River Environment*, FHWA-HI-90-106.

Direct distribution of these manuals is being made to Region and Division Offices. If you have any questions concerning these manuals or require additional copies, please contact Mr. Thomas Krylowski at FTS 285-2365.

Thomas O. Willett

for Dennis C. Judycki

Attachments

Technical Report Documentation Page

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16. Abstract

This document contains the state-of-knowledge and practice for dealing with scour at highway bridges. The procedures for designing new, replacement and rehabilitated bridges to resist scour are presented. Procedures are presented for evaluating the scour vulnerability of existing bridges as well as inspecting bridges for scour. The use of countermeasures to protect bridges evaluated as failure prone due to scour is also presented. This document replaces the Federal Highway Administration (FHWA) publication "Interim Procedures for Evaluating Scour at Bridges," which was issued with FHWA Technical Advisory 5140.20, "Scour at Bridges," in September 1988.

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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

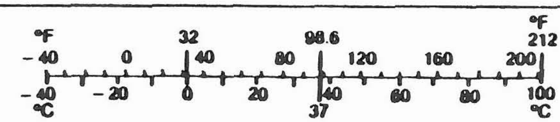
Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimetres	mm
ft	feet	0.305	metres	m
yd	yards	0.914	metres	m
mi	miles	1.61	kilometres	km
		AREA		
in^2	square inches	645.2	millimetres squared	mm^2
ft^2	square feet	0.093	metres squared	m^2
yd^2	square yards	0.836	metres squared	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	kilometres squared	km^2
		VOLUME		
fl oz	fluid ounces	29.57	millilitres	mL
gal	gallons	3.785	litres	L
ft^3	cubic feet	0.028	metres cubed	m^3
yd^3	cubic yards	0.765	metres cubed	m^3

NOTE: Volumes greater than 1000 L shall be shown in m^3.

Symbol	When You Know	Multiply By	To Find	Symbol
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams	Mg
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5(F-32)/9	Celcius temperature	°C

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimetres	0.039	inches	in
m	metres	3.28	feet	ft
m	metres	1.09	yards	yd
km	kilometres	0.621	miles	mi
		AREA		
mm^2	millimetres squared	0.0016	square inches	in^2
m^2	metres squared	10.764	square feet	ft^2
ha	hectares	2.47	acres	ac
km^2	kilometres squared	0.386	square miles	mi^2
		VOLUME		
mL	millilitres	0.034	fluid ounces	fl oz
L	litres	0.264	gallons	gal
m^3	metres cubed	35.315	cubic feet	ft^3
m^3	metres cubed	1.308	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.205	pounds	lb
Mg	megagrams	1.102	short tons (2000 lb)	T
		TEMPERATURE (exact)		
°C	Celcius temperature	1.8C + 32	Fahrenheit temperature	°F



* SI is the symbol for the International System of Measurement

(Revised April 1989)

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PREFACE

This Federal Highway Administration (FHWA) publication, Hydraulic Engineering Circular 18 (HEC 18), "Evaluating Scour at Bridges," provides procedures for the design, evaluation and inspection of bridges for scour. It is a revision of the publication, "Interim Procedures for Evaluating Scour at Bridges," which was issued in September 1988 as part of the FHWA Technical Advisory 5140.20, "Scour at Bridges."(5) It contains revisions as the result of the use of the Technical Advisory by the highway community.

The principal changes are 1) the inclusion of Niell's equation for beginning of motion for coarse bed material in Chapter 2; 2) a statement in Chapter 2 that while the document pertains to scour in the riverine context, judicious use of the document for tidal scour purposes is necessary due to the lack of technology for tidal scour; 3) only one analysis method is given in Step 3 of Chapter 4 with the second method presented in the Appendix A; 4) the removal of all but one abutment scour equation to Appendix B; 5) the recommendation to use guide banks (spur dikes) and/or rock riprap to protect abutments from scour, thereby minimizing the need to compute abutment scour; 6) the addition of procedures to calculate local pier scour when footings or pile caps are exposed, when multiple columns are at an angle to the flow and when pile groups are exposed; 7) the addition of a discussion of local pier scour when pressure flow occurs; i.e., the bridge deck is at least partially submerged; 8) the inclusion of an equation to calculate the width of the pier scour hole; 9) the elimination of the equation to calculate the worse case (deepest) local pier scour from Chapter 5; 10) a slight modification in the equation to determine rock riprap size for pier protection given in Chapter 7 to include recent research; 11) inclusion of recent unpublished research by FHWA for abutment rock riprap protection in Chapter 7; and 12) extensive editorial changes. Also, some changes were made in the appendices. This principally involves the inclusion of the North Carolina Department of Transportation's scour evaluation procedure in place of the Minnesota Department of Transportation's procedure.

EVALUATING SCOUR AT BRIDGES

CHAPTER 1

INTRODUCTION

A. PURPOSE

The purpose of this manual is to provide guidance in:

1) designing new and replacement bridges to resist scour,
2) evaluating existing bridges for vulnerability to scour,
3) inspecting bridges for scour,
4) providing scour countermeasures, and
5) improving the state-of-practice of estimating scour at bridges.

B. ORGANIZATION OF THIS CIRCULAR

The procedures presented in this document contain the state-of-knowledge and practice for dealing with scour at highway bridges. Chapter 1 gives the background of the problem and the general state-of-knowledge of scour. Basic concepts and definitions are presented in Chapter 2. Chapter 3 gives recommendations for designing bridges to resist scour. Chapter 4 gives equations for calculating scour depths at piers and abutments. Chapter 5 provides procedures for conducting scour evaluation and analysis at existing bridges. Chapter 6 presents guidelines for inspecting bridges for scour. Chapter 7 gives a plan of action for installing countermeasures to strengthen bridges that are considered vulnerable to scour.

In the appendices additional information on abutment scour and examples of what several states are doing to assess and evaluate their scour problems is given.

C. BACKGROUND

The most common cause of bridge failures stems from floods. The scouring of bridge foundations is the most common cause of flood damage to bridges. **The hydraulic design of bridge waterways has and is typically based on flood frequencies somewhat less than those recommended for scour analysis in this publication.** During the Spring floods of 1987, 17 bridges in New York and New England were damaged or destroyed by scour. In 1985, 73 bridges were destroyed by floods in Pennsylvania, Virginia, and West Virginia. A 1973 national study for the FHWA of 383 bridge failures caused by catastrophic floods showed that 25 percent involved pier damage and 72 percent involved abutment damage (1). A second more extensive study done in 1978 (2) indicated local scour at bridge piers to be a problem about equal to abutment scour problems. A number of case histories on the causes and consequences of scour at major bridges are presented in Transportation Research Number 950 (3).

D. OBJECTIVES OF A BRIDGE SCOUR EVALUATION PROGRAM

The need to minimize future flood damage to the nation's bridges requires that additional attention be devoted to developing and implementing improved procedures for designing and inspecting bridges for scour. (See National Bridge Inspection Standards, 23 CFR 650 Subpart C.) Approximately 86 percent of the 577,000 bridges in the National Bridge Inventory are built over waterways. Statistically, we can expect thousands of these bridges to experience floods on the order of magnitude of a 100-year flood or greater each year. Because it is not economically feasible to construct all bridges to resist all conceivable floods or to install scour countermeasures at all existing bridges to ensure absolute invulnerability from scour damage, some risks of failure may have to be accepted from future floods. However, every bridge over a stream, whether existing or under design, should be assessed as to its vulnerability to floods in order to determine the prudent measures to be taken. The added cost of making a bridge less vulnerable to scour is small when compared to the total cost of a failure which can easily be two or three times the cost of the bridge itself. Moreover, the need to ensure public safety and to minimize the adverse effects resulting from bridge closures requires our best efforts to improve the state-of-practice for designing and maintaining bridge foundations to resist the effects of scour.

The procedures presented in this manual serve as guidance for implementing the recommendations contained in the FHWA Technical Advisory entitled "Scour at Bridges." The recommendations have been developed to summarize the essential elements which should be addressed in developing a comprehensive scour evaluation program. A key element of the program will be the identification of scour-critical bridges which will be entered into the National Bridge Inventory using the revised Recording and Coding Guide for the Structure inventory and Appraisal of the Nation's Bridges (4).

E. IMPROVING THE STATE-OF-PRACTICE OF ESTIMATING SCOUR AT BRIDGES

The problems associated with estimating scour and providing cost-effective and safe designs need to be addressed further in research and development programs of the FHWA and the States. In the following sections some of the most pressing research needs will be described.

1. **Field Measurements of Scour.** The current equations and methods for estimating scour at bridges are based mainly on laboratory research. Very little field data has been collected to verify the applicability and accuracy of the various design procedures for the range of soil conditions, stream flow conditions, and bridge designs encountered throughout the United States. In particular, States are encouraged to initiate studies for the purpose of obtaining field measurements of scour and related hydraulic conditions at bridges for evaluating, verifying and improving existing scour prediction methods. Several States have already initiated cooperative studies with the Water Resources Division of the U. S. Geological Survey to collect scour data at existing bridges. A model cooperative agreement with the U. S. Geological Survey for purposes of conducting a scour study was included in the FHWA report "Interim Procedures for Evaluating Scour at Bridges," which accompanied the September 1988 FHWA Technical Advisory (5).

2. **Scour Monitoring and Measurement Equipment.** There is a need for the development of instrumention and equipment to indicate when a bridge is in danger of collapsing due to scour. Many bridges in the United States were constructed prior to the development of scour estimation procedures. Some of these bridges have scour vulnerable foundations. It is not economically feasible to repair or replace these bridges at once. Therefore, these bridges need to be monitored during floods and closed before they fail. At this time there are a few devices to monitor bridge scour, but such devices cannot be used on all bridge geometries. Furthermore, the reliability of these devices has not been fully determined.

   There is also the need to develop instrumentation to measure scour depths during and after a flood event. As well, instrumentation is needed to determine unknown bridge foundations.

   The FHWA in cooperation with State highway agencies and the Transportation Research Board has initiated several research projects to develop scour monitoring and measuring instruments.

3. **Scour Analysis Software.** There is a continued need for the development and maintenance of computer software for the analysis of all aspects of scour at bridges. The FHWA has developed computer software for the analysis of flow through bridges and of scour. There currently is a contract for the development of software to determine total scour at a bridge

crossing. This effort should continue. In addition, the maintenance, support and improvement of existing and future software should be provided on a continual basis.

4. **Laboratory Studies of Scour.** There is a need for laboratory studies to determine specific scour processes and to develop scour countermeasures. Only through controlled experiments can the effect of the variables and parameters associated with scour be determined. Scour prediction equations can then be improved and design methods for additional countermeasures can be developed.

   Some examples of needed laboratory research are:

   a. improved prediction of the effect of flow angle of attack against a pier or abutment on scour depth,

   b. improved knowledge of the effect of flow depth and velocity on scour depths,

   c. determine the effect of the pile cap or footing on depth of scour,

   d. determine the magnitude of decrease in scour depth likely to occur if there are large sediment particles in the bed material (armoring of the scour hole),

   e. determine coefficients for the abutment scour equations to replace the simplistic use of abutment length,

   f. determine the width of scour hole as a function of scour depth and bed material size,

   g. determine how to estimate contraction scour when abutments are set back from the channel and there is overbank flow,

   h. fundamental research on the mechanics of scour,

   i. determine the mechanics of tidal scour,

   j. determine the size and placement of riprap (elevation, width and location) in the scour hole needed to protect piers and abutments,

   k. determine methods to predict scour depths associated with pressure flow,

   l. determine methods to predict scour depths when there is ice or debris buildup at a pier or abutment, and

m. determine a rational scour failure mechanism that combines the various scour components (pier, abutment, contraction, lateral migration, degradation) into an estimate of the scoured cross section under the bridge.

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CHAPTER 2

BASIC CONCEPTS AND DEFINITIONS OF SCOUR

A. GENERAL

Scour is the result of the erosive action of flowing water, excavating and carrying away material from the bed and banks of streams. Different materials scour at different rates. Loose granular soils are rapidly eroded by flowing water, while cohesive or cemented soils are more scour resistant. However, ultimate scour in cohesive or cemented soils can be as deep as scour in sandbed streams. Scour will reach its maximum depth in sand and gravel bed materials in hours; cohesive bed materials in days; glacial tills, sand stones and shales in months; limestones in years and dense granites in centuries. Massive rock formations with few discontinuities are highly resistant to scour during the lifetime of a typical bridge.

Designers and inspectors need to carefully study site specific subsurface information in evaluating scour potential at bridges, giving particular attention to foundations on rock.

This entire document relates to scour in the riverine context. That is, scour resulting from flow in one direction, downstream. In coastal areas of the Nation, highway associated transverse and/or longitudinal stream encroachments are subject to tidal flow. The determination of scour in tidal situations has not been studied sufficiently to permit its inclusion in this document. The best guidance for determination of tidal scour until research and operational experience give direction is judicious use of the material developed for the riverine situation in this publication.

B. TOTAL SCOUR

Total scour at a highway crossing is comprised of three components:

1. <u>Aggradation and Degradation</u>. These are long-term stream bed elevation changes due to natural or man induced causes within the reach of the river on which the bridge is located. Aggradation involves the deposition of material eroded from other sections of a stream reach, whereas degradation involves the lowering or scouring of the bed of a stream.

2. <u>Contraction Scour</u>. Contraction scour in a natural channel involves the removal of material from the bed and banks across all or most of the channel width. This component of

scour can result from a contraction of the flow, change in downstream control of the water surface elevation or flow around a bend. The scour is caused by increased velocities and a resulting increase in bed shear stresses.

Contraction of the flow by bridge approach embankments encroaching onto the floodplain and/or into the main channel is the most common cause of contraction scour.

3. **Local Scour**. Local scour involves removal of material from around piers, abutments, spurs, and embankments. It is caused by an acceleration of flow and resulting vortices induced by the flow obstructions.

   In addition to the types of scour mentioned above, naturally occurring **lateral migration of a stream** may erode abutments, the approach roadway or change the total scour by changing the flow angle of attack. Factors that affect lateral movement also affect the stability of a bridge. These factors are the geomorphology of the stream, location of the crossing on the stream, flood characteristics, and the characteristics of the bed and bank materials (see HEC-20 (6) and HIRE (7)).

   The following paragraphs contain additional information on the types of scour discussed above.

C. AGGRADATION AND DEGRADATION, LONG-TERM STREAM BED ELEVATION CHANGES

Long-term bed elevation changes may be the natural trend of the stream or may be the result of some modification to the stream or watershed condition. The stream bed may be aggrading, degrading or in relative equilibrium in the bridge crossing reach. In this section long-term trends are considered. This does not include the cutting and filling of the bed of the stream that might occur during a runoff event. A stream may cut and fill during a runoff event and also have a long-term trend of an increase or decrease in bed elevation. The problem for the engineer is to determine what the long-term bed elevation changes will be during the life of the structure. What is the current rate of change in the stream bed elevation? Is the stream bed elevation in relative equilibrium? Is the stream bed degrading? Is it aggrading? What is the future trend in the stream bed elevation?

During the life of the bridge the present trend may change. These long-term changes are the result of modifications to the stream or watershed. Such changes may be the result of natural processes or man's activities. The engineer must assess the present state of the stream and watershed and then evaluate potential future changes in the river system. From this assessment the engineer must estimate the long-term stream bed changes.

Factors that affect long-term bed elevation changes are: dams and reservoirs (upstream or downstream of the bridge), changes in watershed land use (urbanization, deforestation, etc.), channelization, cutoffs of meander bends (natural or man made), changes in the downstream channel base level (control), gravel mining from the stream bed, diversion of water into or out of the stream, natural lowering of the total system, movement of a bend, bridge location with respect to stream planform, and stream movement in relation to the crossing.

An assessment of long-term stream bed elevation changes should be made using the principles of river mechanics. Such an assessment requires the consideration of all influences upon the bridge crossing; i.e., runoff from the watershed to a stream (hydrology), the sediment delivery to the channel (erosion), the sediment transport capacity of a stream (hydraulics) and the response of a stream to these factors (geomorphology and river mechanics). Many of the largest impacts are from man's activities. This assessment requires a study of the history of the river and man's activities on it as well as a study of present water and land use and stream control activities. All agencies involved with the river should be contacted to determine possible future changes in the river.

To organize such an assessment, this three-level fluvial system approach can be used: 1) a qualitative determination based on general geomorphic and river mechanics relationships; 2) an engineering geomorphic analysis using established qualitative and quantitative relationships to estimate the probable behavior of the stream system to various scenarios of future conditions; and 3) physical process computer modeling using mathematical models such as BRI-STARS and the U. S. Corps of Engineers' HEC 6 to make predictions of quantitative changes in stream bed elevation due to changes in the stream and watershed. Methods to be used in stages 1 and 2 are presented in Hydraulic Engineering Circular No. 20, "Stream Stability at Highway Structures," (6) and "Highways in the River Environment" (7). Additional discussion of this subject is presented in Chapter 4 of this document.

In coastal areas highway crossings (bridge) and/or longitudinal stream encroachments are subject to tidal influences. The impact of the ebb and flow of tides on long-term stream bed elevation changes is relatively indeterminant at this time.

D. CONTRACTION SCOUR

Contraction scour occurs when the flow area of a stream at flood stage is decreased from the normal, either by a natural contraction or by a bridge. With a decrease in flow area, there is an increase in average velocity and bed shear stress through the contraction. Hence, there is an increase in erosive forces in the contraction and more bed material is removed from the contracted reach than is transported into the reach. This increase in transport of bed material from the reach lowers the natural bed elevation. As the bed elevation is lowered, the flow area increases and the velocity and shear stress decrease until relative equilibrium is reached; i.e., the quantity of bed material that is transported into the reach is equal to that removed from the reach.

Contraction scour can also be caused by short-term (daily, weekly, yearly or seasonally) changes in the downstream water surface elevation that controls the backwater and hence the velocity through the bridge opening. Because this scour is reversible, it is included in contraction scour rather than in long-term scour. Contraction scour can also result from a bridge located in a channel bend. If a bridge is located on or close to a bend, the concentration of the flow in the outer part of the channel can erode the bed.

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 contraction of flow due to a bridge can be caused by a decrease in flow area of the stream channel either naturally or by the abutments projecting into the channel and/or the piers taking up a large portion of the flow area. Also, the contraction can be caused by the approaches to a bridge cutting off the flood plain flow. This causes clear-water scour at the bridge section because the flood plain flow normally does not transport significant concentrations of bed material sediments. This clear water picks up additional sediment from the bed upon reaching the bridge opening. In addition, local scour at abutments may well be greater due to the clear-water floodplain flow entering the main channel at that point. A guide bank at an abutment decreases the risk from scour at the abutment by its realignment of the stream lines of the flood plain flow to parallel the main channel flow. However, clear-water scour will occur at the upstream end of the guide bank. Another method to decrease abutment scour is to install relief bridges. They decrease the scour problem at the bridge cross section by decreasing the quantity of clear-water returning to the main channel.

Other factors that can cause contraction scour are: 1) a natural stream constriction, 2) long highway approaches over the flood plain to the bridge, 3) ice formation or jams, 4) a natural berm forming along the banks due to sediment deposits, 5) island or bar formations upstream or downstream of the bridge opening, 6) debris, and 7) the growth of vegetation in the channel or flood plain.

In a natural channel, the depth of flow is always greater on the outside of a bend. In fact there may well be deposition on the inner portion of the bend. If a bridge is located on or close to a bend, the contraction scour will be concentrated on the outer part of the bend. Also, in bends the thalweg (the part of the stream where the flow is deepest and, typically, the velocity is the greatest) may shift toward the center of the stream as the flow increases. This can increase scour and the nonuniform distribution of the scour in the bridge opening.

1. **Contraction Scour Equations.** Contraction scour equations are based on a single principle of conservation of sediment transport. It simply means that the fully developed scour in the bridge cross-section reaches equilibrium when sediment transported into the contracted section equals sediment transported out in the case of live-bed scour or the shear stress in the contracted section has been decreased by scour increasing the area so that it is equal to the critical shear stress of the sediment at the bottom of the contracted cross section.

   There are two forms of contraction scour depending upon the competence of the uncontracted approach flow to transport bed material into the contraction. Live-bed scour occurs when there is sediment being transported into the scour hole. Clear-water scour is the case when the sediment transport in the uncontracted approach flow is zero. In this case the scour hole reaches equilibrium when the average bed shear stress is the critical required for incipient motion of the bed material. Clear-water and live-bed scour are discussed further in another section in this chapter.

   Laursen (8) derived the following live-bed contraction scour equation based on his simplified transport function and several other simplifying assumptions:

$$\frac{y_2}{y_1} = \left(\frac{Q_{mc2}}{Q_{mc1}}\right)^{\frac{6}{7}} \left(\frac{W_{c1}}{W_{c2}}\right)^{K_1} \left(\frac{n_2}{n_1}\right)^{K_2} \qquad (1)$$

$y_s = y_2 - y_1$ (Average scour depth)

Where:

y_1 = average depth in the main channel

y_2 = average depth in the contracted section

W_{c1} = bottom width of the main channel

W_{c2} = bottom width of the contracted section

Q_{mc1} = flow in the approach channel transporting sediment

Q_{mc2} = flow in the contracted channel. Often this is Q_{total} but not always.
n_2 = Manning's n for contracted section
n_1 = Manning's n for main channel
K_1 & K_2 = exponents determined below

V_{*c}/w	e	K_1	K_2	Mode of Bed Material Transport
<0.50	0.25	0.59	0.066	mostly contact bed material discharge
0.50 to 2.0	1.0	0.64	0.21	some suspended bed material discharge
>2.0	2.25	0.69	0.37	mostly suspended bed material discharge

e = transport factor

V_{*c} = $(gy_1S_1)^{0.5}$, shear velocity

w = bed material, D_{50}, fall velocity (see Figure 4.2)

g = gravity constant

S_1 = slope of energy grade line of main channel

$K_1 = \dfrac{6(2+e)}{7(3+e)}$

$K_2 = \dfrac{6e}{7(3+e)}$

Laursen's (9) clear-water contraction scour equation has a much simpler derivation because it does not involve any transport function. It simply recognizes that:

$$\tau_2 = \tau_c$$

Where:

τ_2 = *average bed shear stress, contracted section.*

τ_c = *critical bed shear stress, incipient motion.*

At equilibrium for noncohesive bed materials and for fully developed clear-water scour, Laursen used the following equation:

$$\tau_c = 4\ D_{50}$$

Also:

$$\tau_2 = \gamma y_2 S_f = \frac{\gamma V_2^2 n^2}{1.49^2\ y_2^{\frac{1}{3}}}$$

Using Strickler's approximation for Manning's n:

$$n = 0.034\ D_{50}^{\frac{1}{6}}$$

Then at incipient motion:

$$\frac{\tau_2}{\tau_c} = 1.0$$

Therefore:

$$y_2 = \left[\frac{V_2^2}{120\ D_{50}^{\frac{2}{3}}}\right]^3$$

or:

$$y_2 = \left[\frac{Q_2^2}{120\ D_{50}^{\frac{2}{3}}\ W_2^2}\right]^{\frac{3}{7}} \qquad (2a)$$

A dimensionless form of equation 2a can be written if flow continuity can be assumed for the approach and contracted segments of the flood plain being analyzed. That is:

$$Q_2 = Q_1 = V_1\ W_1\ y_1$$

then:

$$\frac{y_2}{y_1} = \left(\frac{W_1}{W_2}\right)^{\frac{6}{7}} \left[\frac{V_1^2}{120\ y_1^{\frac{1}{3}}\ D_{50}^{\frac{2}{3}}}\right]^{\frac{3}{7}} \qquad (2b)$$

The above contraction scour equations were developed for hand computations and are based on rather limiting assumptions. For example they are based on homogeneous bed materials and would not apply for stratified layers of different bed materials. However, with clear-water scour in stratified materials, using the finest D_{50} would give the worse case scour depths. Also, the equations could, in the clear-water case, be used sequentially for stratified bed materials. These equations are the best that are available and should be regarded as a first level of analysis. If a more precise analysis is warranted, a sediment transport model like BRI-STARS could be used.

Calculation of contraction scour is presented in Chapter 4.

E. LOCAL SCOUR

The basic mechanism causing local scour at a pier or abutment is the formation of vortices at their base. The formation of these vortices results from the pileup of water on the upstream surface and subsequent acceleration of the flow around the nose of the pier or embankment. The action of the vortex removes bed materials from the base region. With the transport rate of sediment away from the base region greater than the transport rate into the region, a scour hole develops. As the depth of scour increases, the strength of the vortices is reduced, thereby reducing the transport rate from the base region, and eventually equilibrium is reestablished and scouring ceases.

In addition to a horseshoe vortex around the base of a pier, there is a vertical vortex downstream of the pier called the wake vortex, Figure 2.1. Both vortices remove material from the pier base region. However, the intensity of these wake vortices diminishes rapidly as the distance downstream of the pier increases. Therefore, immediately downstream of a long pier there is often deposition of material.

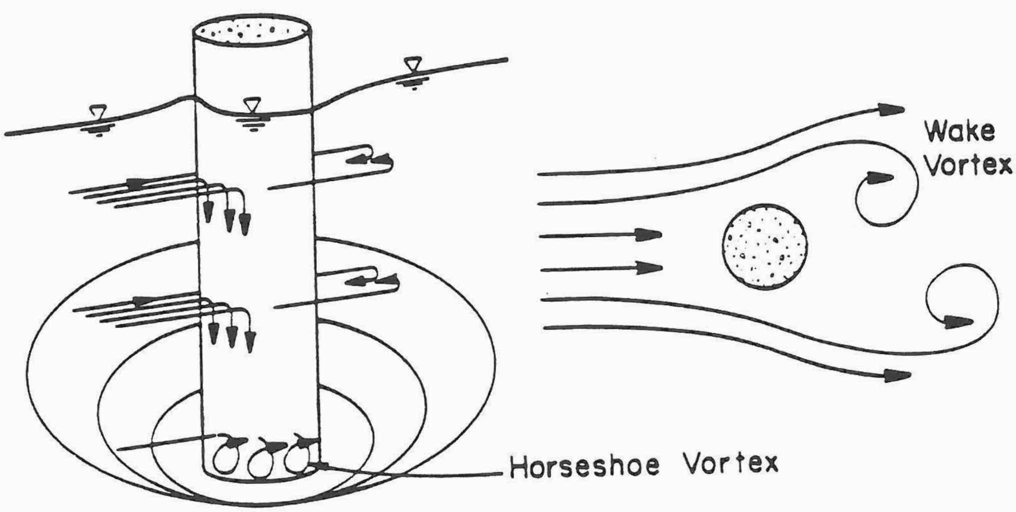

Figure 2.1 Schematic Representation of Scour at a Cylindrical Pier.

Factors affecting local scour are: 1) width of the pier, 2) projected length of an abutment into the flow, 3) length of the pier if skewed to flow, 4) depth of flow, 5) velocity of the approach flow, 6) size and gradation of bed material, 7) angle of attack of the approach flow to a pier or abutment, 8) shape of a pier or abutment, 9) bed configuration, 10) ice formation or jams, and 11) debris.

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

2. Projected length of an abutment into the stream affects the depth of local scour. An increase in the projected length of an abutment into the flow increases scour. However, there is a limit on the increase in scour depth with an increase in length. This limit is reached when the ratio of projected length into the flow to the depth of the approach flow is 25.

3. Pier length has no appreciable affect 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 affect; i.e., with the same angle of attack, doubling the length of the pier increases scour depth by 33 percent.

4. Flow depth has an affect on the depth of local scour. An increase in flow depth can increase scour depth by a factor of 2 or greater for piers. With abutments the increase is from 1.1 to 2.15 depending on the shape of the abutment.

5. The approach flow velocity affects scour depth. The greater the velocity, the deeper the scour. There is a high probability that scour is affected by whether the flow is subcritical or supercritical. However, most research and data are for subcritical flow; i.e., flow with a Froude Number much less than one ($Fr < 1$).

6. Bed material characteristics such as size, gradation, and cohesion can affect local scour. Bed material in the sand size range has no affect 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 or abutment 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. Research at the University of Aukland, New Zealand, and by the Washington State Department of Transportation (10) (11) (12) (13) developed an equation that takes into account the decrease in scour due to the armoring of the scour hole. Richardson and Richardson (14) combined the work of Raudkivi, Ettema, Melville, Sutherland, Cope, Johnson and MacIntosh into a simplified equation. However, field data are inadequate to support these equations at this time. The extent that large particles will decrease scour is not clearly understood.

   The size of the bed material also determines whether the scour at a pier or abutment is clear-water or live-bed scour. This topic is discussed later in this chapter.

Fine bed material (silts and clays) will have scour depths as deep as sandbed streams. This is true even if bonded together by cohesion. The affect 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.

7. Angle of attack of the flow to the pier or abutment has a significant affect on local scour, as was pointed out in the discussion of pier length. Abutment scour is reduced when embankments are angled downstream and increased when embankments are angled upstream. According to the work of Ahmad, the maximum depth of scour at an embankment inclined 45 degrees downstream is reduced by 20 percent, whereas, the maximum scour at an embankment inclined 45 degrees upstream is increased about 10 percent.

8. Shape of the nose of a pier or an abutment has a significant affect 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.

   Full retaining abutments with vertical walls on the streamside (parallel to the flow) will produce scour depths about double that of spill-through abutments.

9. Bed configuration effects the magnitude of local scour. In streams with sand bed material, the shape of the bed (bed configuration) as determined by Richardson et al (15) may be ripples, dunes, plane bed and antidunes. The bed configuration depends on the size distribution of the sand bed material, flow conditions, and fluid viscosity. The bed configuration may change from dunes to plane bed or antidunes during an increase in flow for a single flood event. It may change back with a decrease in flow. The bed configuration may also change with a change in water temperature or change in suspended sediment concentration of silts and clays. The type of bed configuration and change in bed configuration will effect flow velocity, sediment transport, and scour. "Highways in the River Environment" (7) discusses bed configuration in detail.

10. Ice and debris potentially increase the width of the piers, change the shape of piers and abutments, increase the projected

length of an abutment 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 or length of an abutment. Debris and ice affects on contraction scour can also be accounted for by estimating the amount of flow blockage (decrease in width of the bridge opening) in the equations for contraction scour. Limited field measurements of scour at ice jams indicate the scour can be as much as 10 or 20 feet.

F. CLEAR-WATER AND LIVE-BED SCOUR

There are two conditions for contraction and local scour. These are 1) clear-water scour and 2) live-bed scour. Clear-water scour occurs when there is no movement of the bed material of the stream upstream of the crossing, but the acceleration of the flow and vortices created by the piers or abutments causes the material in the crossing to move. Live-bed scour occurs when the bed material upstream of the crossing is moving.

Typical clear-water scour situations include: 1) course bed material streams, 2) flat gradient streams during low flow, 3) local deposits of larger bed materials that are larger than the biggest fraction being transported by the flow (rock riprap is a special case of this situation), 4) armored stream beds where the only locations that tractive forces are adequate to penetrate the armor layer are at piers and/or abutments and 5) vegetated channels where, again, the only locations the cover is penetrated is at piers and/or abutments.

During a flood event, bridges over streams with coarse bed material are often subjected to clear-water scour at low discharges, live-bed scour at the higher discharges and then clear-water scour on the falling stages. Clear-water scour reaches its maximum over a longer period of time than live-bed scour (See Figure 2.2). This is because clear-water scour occurs mainly in coarse bed material streams. In fact clear-water scour may not reach a maximum until after several floods. Maximum clear-water scour is about 10 percent greater than the maximum live-bed scour.

The following equation suggested by Neill (16) for determining the velocity associated with initiation of motion is an indicator for clear-water or live-bed scour.

$$V_c = 1.58 \, [(S_s - 1) g D_{50}]^{1/2} \, (y/D_{50})^{1/6}$$

Where: V_c = critical velocity above which bed

materials of size D_{50} and smaller
will be transported.

S_s = specific gravity of bed materials.

y = depth of flow

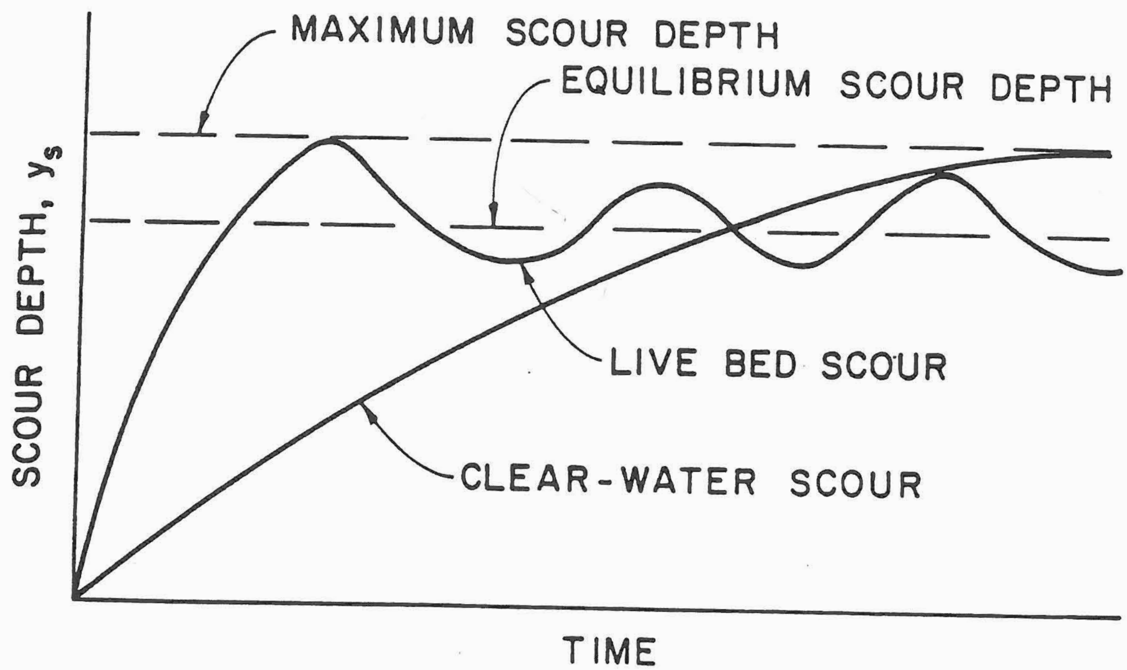

Figure 2.2 Scour Depth as a Function of Time

Live-bed scour in sand bed streams with a dune bed configuration fluctuates about the equilibrium scour depth. The reason for this is the variability of the bed material sediment transport in the approach flow when the bed configuration of the stream is dunes. In this case (dune bed configuration in the channel upstream of the bridge), maximum depth of scour is about 30 percent larger than equilibrium depth of scour.

The maximum depth of scour is the same as the equilibrium depth of scour for live-bed scour with a plain bed configuration. With antidunes occurring upstream and in the bridge crossing the maximum depth of scour from the limited research of Jain and Fisher (17) is about 10 percent greater than the equilibrium depth of scour.

For a discussion of bed forms in alluvial channel flow the reader is referred to Chapter 3 of "Highways in the River Environment" (7). Equations for estimating local scour at abutments or piers are given in Chapter 4 of this publication. These equations were developed from laboratory experiments and limited field data for both clear-water and live-bed scour.

G. LATERAL SHIFTING OF A STREAM

Streams are dynamic. Areas of flow concentration continually shift bank lines. A meandering stream has its "S" shaped plan form continually moving laterally and downstream. A braided stream has its various channels continually changing. Incidentally, the deepest natural scour occurs when two channels of a braided stream come together or when the flow comes together downstream of an island or bar. This has been observed to be 5 times the downstream flow depth.

A bridge is static. It fixes the stream at one place in time and space. A meandering stream continues to move laterally and downstream, eroding the approach embankment and affecting contraction and local scour because of changes in flow direction. A braided stream can shift its channels under a bridge, and have two channels come together at a pier or abutment, thus increasing scour. Descriptions of stream morphology are given in "Highways in the River Environment" (7) and in Hydraulic Engineering Circular 20 (6).

Factors that affect lateral shifting and the stability of a bridge are the geomorphology of the stream, location of the crossing on the stream, bed and bank materials, flood characteristics, the characteristics of the bed material and washload discharge.

It is difficult to anticipate when a change in plan form may occur. It may be gradual with time or the result of a major flood event. Also, the direction and magnitude of the movement of the stream is not easily determined. **ALTHOUGH IT IS DIFFICULT TO PROPERLY EVALUATE THE VULNERABILITY OF A BRIDGE DUE TO CHANGES IN PLAN FORM, IT IS ESSENTIAL TO DO SO AND TO CONSIDER COUNTERMEASURES.**

Countermeasures may be changes in the bridge design, construction of river control works, protection of piers and/or abutments with riprap or even just careful monitoring of the river in a bridge inspection program. **SERIOUS CONSIDERATION SHOULD BE GIVEN TO PLACING FOOTINGS/FOUNDATIONS LOCATED ON FLOOD PLAINS AT ELEVATIONS APPROXIMATING THOSE LOCATED IN THE MAIN CHANNEL.**

To control lateral shifting requires river training works, bank stabilizing by riprap and/or guide banks. The design of these works is beyond the scope of this circular. Design methods are given by FHWA (18), U.S. Corps of Engineers (19, 20) and AASHTO (21) publications. Of particular importance are "Hydraulic Analyses for the Location and Design of Bridges," Volume VII-Highway Drainage Guidelines, 1982 (21); "Highways in the River Environment" (7); "Spur and Guide Banks" (22) and "Stream Stability" Hydraulic Engineering Circular 20 (6).

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CHAPTER 3

DESIGNING BRIDGES TO RESIST SCOUR

A. <u>DESIGN PHILOSOPHY AND CONCEPTS</u>

Bridges should be designed to withstand the effects of scour from a superflood (a flood exceeding the 100-year flood) with little risk of failing. This requires careful evaluation of the hydraulic, structural, and geotechnical aspects of bridge foundation design.

The guidance in this chapter is based on the following concepts.

- The foundation should be designed by an interdisciplinary team of engineers with expertise in hydraulic, geotechnical and structural design.

- Hydraulic studies of bridge sites are a necessary part of a bridge design. These studies should address both the sizing of the bridge waterway opening and the designing of the foundations to resist scour. The scope and depth of the analysis should be commensurate with the importance of the highway and the consequences of failure.

- Adequate consideration must be given to the limitations and gaps in existing knowledge when using currently available formulas for estimating scour. The designer needs to apply engineering judgment in comparing results obtained from scour computations with available hydrologic and hydraulic data to achieve a reasonable and prudent design. Such data should include:

    a. Performance of existing structures during past floods,

    b. Effects of regulation and control of flood discharges,

    c. Hydrologic characteristics and flood history of the stream and similar streams, and

    d. Whether the bridge is structurally continuous.

- The principles of economic analysis and experience with actual flood damage indicates that it is almost always cost-effective to provide a foundation that will not fail, even from a very large flood event or superflood. Occasional damage to highway approaches from rare floods can be repaired rather quickly to restore traffic service.

On the other hand, a bridge which collapses or suffers major structural damage from scour can create safety hazards to motorists as well as large social impacts and economic losses over a long period of time. Aside from the costs to the highway agency of replacing/repairing the bridge and constructing and maintaining detours, there can be significant costs to communities or entire regions due to additional detour travel time, inconveniences, and lost business opportunities. Therefore, a higher hydraulic standard is warranted for the design of bridge foundations as a protection against scour than is usually required for sizing of the bridge waterway. This concept is reflected in the following design procedure which is to be applied to the bridge design sized to accommodate the design discharge.

B. DESIGN PROCEDURE

The design procedure for scour outlined in the following steps is recommended for the proposed bridge type, size, and location (TS&L) of substructure units:

1. Select the flood event(s) with return periods of 100 years or less that are expected to produce the most severe scour conditions. Experience indicates that this is likely to be the overtopping flood which may or may not be equal to the 100-year flood. Check the 100-year flood, the overtopping flood (if less than the 100-year flood) and other flood events if there is evidence that such events would create deeper scour than the 100-year or overtopping floods.

2. Develop water surface profiles for the flood flows in Step 1, taking care to evaluate the range of potential tailwater conditions below the bridge which could occur during these floods. The FHWA microcomputer software WSPRO, "Bridge Waterways Analysis Model" (23), or the Corps of Engineers HEC 2, are recommended for this task.

3. Using the design procedures in Chapter 4, estimate total scour for the worst condition from Steps 1 and 2 above.

4. Plot the total scour depths obtained in Step 3 on a cross section of the stream channel and flood plain at the bridge site.

5. Evaluate the answers obtained in Steps 3 and 4. Are they reasonable, considering the limitations in current scour estimating procedures? The scour depth(s) adopted may differ from the equation value(s) based on engineering judgement.

6. Evaluate the bridge TS&L on the basis of the scour analysis

performed in Steps 3-5. Modify the TS&L as necessary.

- o Visualize the overall flood flow pattern at the bridge site for the design conditions. Use this mental picture to identify those bridge elements most vulnerable to flood flows and resulting scour.

- o The extent of protection to be provided should be determined by:

  a. The degree of uncertainty in the scour prediction method.
  b. The potential for and consequences of failure.
  c. The added cost of making the bridge less vulnerable to scour. Design measures incorporated in the original construction are almost always less costly ~~than costly~~ than retrofitting scour countermeasures.

7. Perform the bridge foundation analysis on the basis that all stream bed material in the scour prism above the total scour line (Step 4) has been removed and is not available for bearing or lateral support. All foundations should be designed in accordance with the AASHTO Standard Specifications for Highway Bridges. In the case of a pile foundation, the piling should be designed for additional lateral restraint and column action because of the increase in unsupported pile length after scour. In areas where the local scour is confined to the proximity of the footing, the lateral ground stresses on the pile length which remains embedded may not be significantly reduced from the pre-local scour conditions. The depth of local scour and volume of soil removed from above the pile group should be considered by geotechnical engineers when computing pile embedment to sustain vertical load.

   a. <u>Spread Footings On Soil</u>.

   - o Place the top of the footing below the design scour line from Step 4.

   - o Make sure that the bottom of the footing is at least 6.0 feet below the stream bed as per AASHTO standards.

   b. <u>Spread Footings On Rock Highly Resistant To Scour</u>.

   Place the bottom of the footing directly on the cleaned rock surface for massive rock formations (such as granite) that are highly resistant to scour. Small embedments (keying) should be avoided since blasting to achieve keying frequently damages the sub-footing rock structure and makes it more susceptible to scour. If footings on smooth massive rock surfaces require lateral constraint, steel dowels should be drilled and grouted into the rock below the footing level.

c. Spread Footings On Erodible Rock.

Weathered or other potentially erodible rock formations need to be carefully assessed for scour. An engineering geologist familiar with the area geology should be consulted to determine if rock or soil or other criteria should be used to calculate the support for the spread footing foundation. The decision should be based on an analysis of intact rock cores including rock quality designations and local geology, as well as hydraulic data and anticipated structure life. An important consideration may be the existence of a high quality rock formation below a thin weathered zone. For deep deposits of weathered rock, the potential scour depth should be estimated (Steps 4 and 5) and the footing base placed below that depth. Excavation into weathered rock should be made with care. If blasting is required, light, closely spaced charges should be used to minimize overbreak beneath the footing level. Loose rock pieces should be removed and the zone filled with lean concrete. In any event, the final footing should be poured in contact with the sides of the excavation for the full designed footing thickness to minimize water intrusion below footing level. The excavation above the top of the spread footing should be filled with rock riprap sized to withstand flood flow velocities.

d. Spread Footings Placed On Tremie Seals And Supported On Soil.

- Place the tremie base three feet below the scour line (Step 4) if the tremie is structurally capable of sustaining the imposed structural load without lateral soil support.

- Check the design for the superflood to insure a safety factor of not less than 1.0.

e. For Deep Foundations (Drilled Shaft And Driven Piling) With Footings Or Caps.

Placing the top of the footing or pile cap below streambed a depth equal to the estimated contraction scour depth will minimize obstruction to flood flows and resulting local scour. Even lower footing elevations may be desirable for pile supported footings when the piles could be damaged by erosion and corrosion from exposure to river currents.

f. <u>Stub Abutments on Piling</u>

Stub abutments positioned in the embankment should be founded on piling driven below the elevation of the thalweg in the bridge waterway to assure structural integrity in the event the thalweg shifts and the piling scour to the thalweg elevation.

8. Repeat the procedure in Steps 2 - 6 above and calculate the scour for a superflood. It is recommended that this superflood or check flood be on the order of a 500-year event or a flood 1.7 times the magnitude of the 100-year flood if the magnitude of the 500-year flood can not be estimated. However, flows greater or less than these suggested floods may be appropriate depending upon hydrologic considerations and the consequences associated with damage to the bridge. An overtopping flood within the range of the 100-year to 500-year flood may produce the worst-case situation for checking the foundation design. The foundation design determined under Step 7 should be reevaluated for the superflood condition and design modifications made where required.

- Check to make sure that the bottom of spread footings on soil or weathered rock is below the scour depth for the superflood.

- **All foundations should have a minimum factor of safety of 1.0 (ultimate load) under the superflood conditions.** Note that in actual practice, the calculations for Step 8 would be performed concurrently with Steps 1 through 7 for efficiency of operation.

C. CHECKLIST OF DESIGN CONSIDERATIONS

TABLE 3.1 CHECKLIST OF DESIGN CONSIDERATIONS

General

1. Raise the bridge superstructure elevation above the general elevation of the approach roadways wherever practicable. This provides for overtopping of approach embankments and relief from the hydraulic forces acting at the bridge. This is particularly important for streams carrying large amounts of debris which could clog the waterway of the bridge.

2. Superstructures should be securely anchored to the substructure if buoyant, debris, and ice forces are probable. Further, the superstructure should be shallow and open to minimize resistance to the flow where overtopping is likely.

3. Continuous span bridges withstand forces due to scour and resultant foundation movement better than simple span bridges. Continuous spans provide alternate load paths (redundancy) for unbalanced forces caused by settlement and/or rotation of the foundations. This type of structural design is especially recommended for bridges where there is a significant scour potential.

4. Local scour holes at piers and abutments may overlap one another in some instances. If local scour holes do overlap, the scour can be deeper. The top width of a local scour hole ranges from 1.0 to 2.75 times the depth of scour.

5. For pile and drilled shaft designs subject to scour, consideration should be given to using a lesser number of longer piles or shafts as compared with a greater number of shorter piles or shafts to develop bearing loads. This approach will provide a greater factor of safety against pile failure due to scour at little or no increase in cost.

6. At some bridge sites, hydraulics and traffic conditions may necessitate consideration of a bridge that will be partially or even totally inundated during high flows. This consideration results in pressure flow through the bridge waterway. Since this consideration has received no attention relative to estimation of bridge scour, there is no recommendation for determination of scour pending future research.

TABLE 3.1 CHECKLIST OF DESIGN CONSIDERATIONS

Piers

1. Pier foundations on floodplains should be designed to the same elevation as the pier foundations in the stream channel if there is a likelihood that the channel will shift its location on the floodplain over the life of the bridge.

2. Align piers with the direction of flood flows. Assess the hydraulic advantages of round piers, particularly where there are complex flow patterns during flood events.

3. Streamline pier shapes to decrease scour and minimize potential for buildup of ice and debris. Use ice and debris deflectors where appropriate.

4. Evaluate the hazards of ice and debris buildup when considering use of multiple pile bents in stream channels. Where ice and debris buildup is a problem, design the bent as though it were a solid pier for purposes of estimating scour. Consider use of other pier types where clogging of the waterway area could be a major problem.

Abutments

1. Recognizing that abutment scour solutions lack definition, it is recommended that rock riprap and/or guide banks be seriously considered for abutment protection. Properly designed, these two protective measures negate the need to compute abutment scour.

2. Relief openings, guide banks (spur dikes), and river training works should be used where needed to minimize the effects of adverse flow conditions at abutments.

3. Utilize rock riprap where needed to protect abutments. Design rock riprap to resist the hydraulic forces associated with design conditions using Hydraulic Engineering Circular No. 11, "Design of Riprap Revetment" (24) with rock riprap design guidance given in Chapter 7.

4. Where ice build-up is likely to be a problem, set the toe of spill-through slopes or vertical abutment walls some distance from the edge of the channel bank to facilitate passage of the ice.

5. Scour at spill-through abutments is about 50% of that of vertical wall abutments.

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CHAPTER 4

ESTIMATING SCOUR AT BRIDGES

I. INTRODUCTION

This chapter presents the methods and equations for determining total scour at a bridge; i.e., long-term aggradation or degradation, contraction scour and local scour. Example problems are given at the end of the chapter.

Prior to applying the various scour forecasting methods for contraction and local scour, it is necessary to 1) obtain the fixed-bed channel hydraulics, 2) estimate the long-term profile degradation or aggradation, 3) adjust the fixed-bed hydraulics to reflect these changes, and 4) compute the bridge hydraulics.

II. DESIGN APPROACH

The seven steps recommended for estimating scour at bridges are:

STEP 1. Determine scour analysis variables.

STEP 2. Analyze long-term bed elevation change.

STEP 3. Evaluate the scour analysis method.

STEP 4. Compute the magnitude of contraction scour.

STEP 5. Compute the magnitude of local scour at abutments.

STEP 6. Compute the magnitude of local scour at piers.

STEP 7. Plot the total scour depths

The procedures for each of the steps, including recommended scour equations, are discussed in detail in the following sections.

III. DETAILED PROCEDURES

A. STEP 1. DETERMINING SCOUR ANALYSIS VARIABLES

1. Determine the magnitude of the discharges for the floods in Step 1 of the Design Procedure, Chapter III, including the overtopping flood when applicable. If the magnitude of the 500-year flood is not available, use a discharge equal to 1.7 X Q_{100}. Experience has shown that the incipient overtopping discharge often puts the most stress on a bridge. However, special conditions (angle of attack, pressure flow, decrease in velocity or discharge resulting from high flows overtopping

approaches or going through relief bridges, ice jams, etc.) may cause a more severe condition for scour with a flow smaller than the overtopping or 100-year flood.

2. Determine the water-surface profiles for the discharges judged to produce the most scour from Step 1, using WSPRO or HEC 2. In some instances the designer may wish to use BRI-STARS. The engineer should anticipate future conditions at the bridge, in the stream's watershed, and at downstream water-surface elevation controls.

3. Determine if there are existing or potential future factors that will produce a combination of high discharge and low tailwater control. Are there bedrock or other controls (old diversion structures, erosion control checks, other bridges, etc.) that might be lowered or removed? Are there dams or locks downstream that would control the tailwater elevation seasonally? Are there dams upstream or downstream that could control the elevation of the water surface at the bridge? Select the lowest reasonable downstream water-surface elevation and the largest discharge to estimate the greatest scour potential. Assess the distribution of the velocity and discharge per foot of width for the design flow and other flows through the bridge opening. Consider also the approach flow and the flow distribution downstream (the contraction and expansion of the flow). This should take into consideration present conditions and anticipated future changes in the river.

4. From computer analysis and from other hydraulic studies, determine the discharge velocity and depth input variables needed for the scour calculations.

5. Collect and summarize the following information as appropriate.

   a. Boring logs to define geologic substrata at the bridge site.

   b. Bed material size and gradation distribution in the bridge reach.

   c. Existing stream and flood plain cross-section through the reach.

   d. Stream geomorphic plan form.

   e. Watershed characteristics.

   f. Scour data on other bridges in the area.

   g. Slope of energy grade line upstream and downstream of the bridge.

h. Bed material sediment discharge estimates for flood discharges (flood discharges are mean annual, and 5, 10, 25, 50, 100 and 500 year frequencies). Use Colby's method for sand-bed streams and the Meyer-Peter, Muller equation for coarse bed streams (7).

i. History of flooding.

j. Location of bridge site with respect to other bridges in the area, confluence with tributaries close to the site, bed rock controls, man-made controls (dams, old check structures, river training works, etc.), and downstream confluences with another stream.

k. Character of the stream (perennial, flashy, intermittent, gradual peaks, etc.).

l. Geomorphology of the site (flood plain stream; crossing of a delta, youthful, mature or old age stream; crossing of an alluvial fan; meandering, straight or braided stream; etc.).

m. Erosion history of the stream.

n. Development history (past, present and future) of the stream and watershed. Collect maps, ground photographs, aerial photographs; interview local residents; check for water research projects planned or contemplated.

o. Sand and gravel mining from streambed up and downstream from site.

p. Other factors that could affect the bridge.

q. Make a qualitative evaluation of the site with an estimate of the potential for stream movement and its effect on the bridge.

B. STEP 2. ANALYSIS OF LONG-TERM BED ELEVATION CHANGE

1. Using the information collected in Step 1 above, determine qualitatively the long-term trend in the stream elevation. Where conditions indicate that significant aggradation or degradation is likely, estimate the change in bed elevation over the next 100 years using one or more of the following:

a. Available computer programs such as BRI-STARS and the Corps of Engineers HEC 6,

b. Straight line extrapolation of present trends,

c. Engineering judgment,

d. The worse-case scenarios; i.e., in the case of a confluence with another stream just downstream of the bridge, assume the design flood would occur with a low downstream water-surface elevation through a qualitative assessment of the joint probability of flood magnitudes and river conditions on the main stream and its tributary.

2. If the stream is aggrading and this condition can be expected to affect the crossing, taking into account contraction scour, consider relocation of the bridge or raising the lower cord of the bridge.

3. If the stream is degrading, use the change in elevation in the calculations of total scour.

C. STEP 3. <u>EVALUATE THE SCOUR ANALYSIS METHOD</u>

The method is based on the assumption that the scour components develop independently. Thus, the potential local scour is added to the contraction scour without considering the effects of contraction scour on the channel and bridge hydraulics. If contraction scour is significant, an alternate method presented in Appendix A may be used.

- o Estimate the natural channel hydraulics for a fixed-bed condition based on existing conditions,
- o Assess the expected profile and plan form changes,
- o Adjust the fixed-bed hydraulics to reflect any expected long-term profile or plan form changes,
- o Estimate contraction scour using the empirical contraction formula and the adjusted fixed-bed hydraulics,
- o Estimate local scour using the adjusted channel and bridge hydraulics, and
- o Add the local scour to the contraction scour to obtain the total scour. Chapter 3, Design procedure, Step 5.

D. STEP 4. CONTRACTION SCOUR

1. General

Contraction scour can be caused by different bridge site conditions. There are four (4) conditions (cases) which are:

Case 1. Involves overbank flow on a flood plain being forced back to the main channel by the approaches to the bridge.

a. The river channel width becomes narrower either due to the bridge abutments projecting into the channel or the bridge being located at a narrowing reach of the river ($W_{c1} > W_{c2}$).

b. Does not involve any contraction of the main channel, but the overbank flow area is completely obstructed by the embankment ($W_{c1} = W_{c2}$).

c. Abutments set back from the stream channel (($W_{c1} < (W_{c2} + W_{setback})$)).

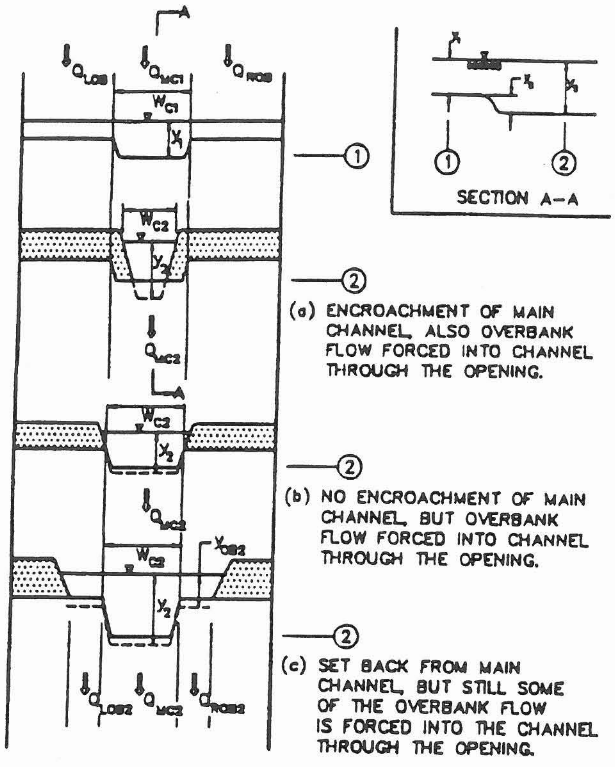
Case 2. Flow is confined to the main channel; i.e., there is no overbank flow. The normal river channel width becomes narrower due to the bridge itself or the bridge site being located at a narrower reach of the river.

Case 3. A relief bridge in the overbank area with little or no bed material transport in the overbank area; i.e., clear-water scour.
($W_1 > W_{c2}$)

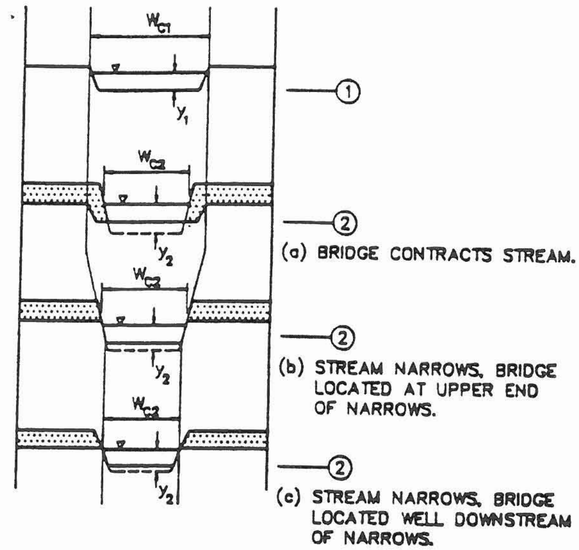
Case 4. A relief bridge over a secondary stream in the overbank area. (Similar to Case 1).

W_{c1} = bottom width of the main channel
W_{c2} = bottom width of the contracted section
W_1 = width of upstream overbank area

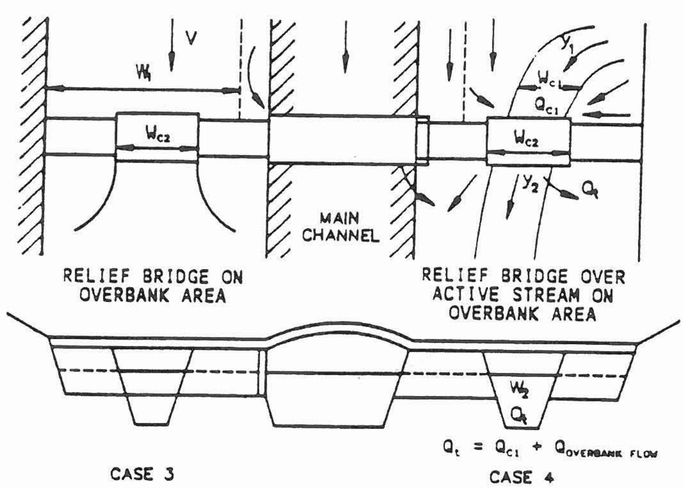
These 4 cases are illustrated in Figure 4.1. The equations for solving each case are presented in the following sections.



Case 1: Contraction w/ Overbank Flow.



Case 2: Contraction w/ no Overbank Flow.



Case 3 and 4: Relief Bridge.

Figure 4.1 The Four Main Cases of Contraction Scour

2. Estimating Contraction Scour.

a. **CASE 1.** **CONTRACTION SCOUR, OVERBANK FLOW BEING FORCED BACK INTO THE MAIN CHANNEL. (Live-bed scour)**

For Cases 1a and 1b use Laursen's 1960 Equation (8) for a long contraction to predict the depth of scour in the contracted section. This equation was given in Chapter 2. It assumes that bed material is being transported in the main channel, but not in the overbank zones.

$$\frac{y_2}{y_1} = \left(\frac{Q_{mc2}}{Q_{mc1}}\right)^{\frac{6}{7}} \left(\frac{W_{c1}}{W_{c2}}\right)^{K_1} \left(\frac{n_2}{n_1}\right)^{K_2} \qquad (1)$$

$y_s = y_2 - y_1$ (Average scour depth)

Where:

y_1 = average depth in the main channel
y_2 = average depth in the contracted section
W_{c1} = bottom width of the main channel
W_{c2} = bottom width of the bridge opening
Q_{mc1} = flow in the approach channel that is transporting sediment
Q_{mc2} = flow in the contracted channel which is often Q_{total}, but not always
n_2 = Manning's n for contracted section
n_1 = Manning's n for main channel
K_1 & K_2 = exponents determined below

V_{*c}/w	K_1	K_2	Mode of Bed Material Transport
<0.50	0.59	0.066	mostly contact bed material discharge
0.50 to 2.0	0.64	0.21	some suspended bed material discharge
>2.0	0.69	0.37	mostly suspended bed material discharge

V_{*c} = $(gy_1S_1)^{0.5}$, shear velocity

w = fall velocity of D_{50} of bed material. (See Figure 4.2)

g = gravity constant

S_1 = slope of energy grade line of main channel

Notes.

1. Q_{mc2} may be the total flow going through the bridge opening as in Cases 1a and 1b. It is <u>not</u> the total for Case 1c.

2. Q_{mc1} is the flow in the main channel upstream of the bridge.

3. The Manning's n ratio can be significant for a condition of dune bed in the main channel and a corresponding plain bed, washed out dunes or antidunes in the contracted channel (7). HOWEVER LAURSEN'S EQUATION DOES NOT CORRECTLY ACCOUNT FOR THE INCREASE IN TRANSPORT THAT WILL OCCUR AS THE RESULT OF THE BED PLANING OUT WHICH DECREASES RESISTANCE TO FLOW AND INCREASES VELOCITY AND THE TRANSPORT OF BED MATERIAL AT THE BRIDGE. THAT IS, LAURSEN'S EQUATION INDICATES A DECREASE IN SCOUR FOR THIS CASE WHEREAS IN REALITY THERE IS AN INCREASE IN SCOUR DEPTH. THEREFORE SET THE TWO n VALUES EQUAL.

4. The average width of the bridge opening (W_{c2}) is normally taken as the bottom width, with the width of the piers subtracted.

5. Laursen's equation will overestimate the depth of scour at the bridge if the bridge is located at the upstream end of the contraction or if the contraction is the result of the bridge abutments and piers. At this time, however, it is the best equation available.

<u>CASE 1a.</u>

Case 1a involves contraction of the channel and overbank flow. In this case:

$Q_{mc1} < Q_{mc2}$

Q_{mc2} = total flow going through the bridge. It equals Q_{mc1} plus $Q_{overbank}$ (Q_{ob}) less any flow going over the roadway, through a relief bridge or otherwise bypassing the main bridge.

$W_{c1} > W_{c2}$

W_{c2} = bottom width of the channel at the bridge less the width of piers.

$n_1 = n_2$

Equation 1 reduces to:

$$\frac{y_2}{y_1} = \left(\frac{Q_{mc2}}{Q_{mc1}}\right)^{\frac{6}{7}} \left(\frac{W_{c1}}{W_{c2}}\right)^{K_1} \qquad (2)$$

A typical application of Case 1a would be to evaluate the effect of piers in the main channel when there is overbank flow.

CASE 1b.

Case 1b involves overbank flow with out any contraction of the main channel (even by piers). In this case:

$Q_{mc1} < Q_{mc2}$

Q_{mc2} = total flow going through the bridge. It equals Q_{mc1} plus $Q_{overbank}$ less any flow going over the roadway, through a relief bridge or otherwise bypassing the main bridge. $(Q_t - Q_{bypass})$

$W_{c1} = W_{c2}$

$n_1 = n_2$

Then Equation 1 reduces to:

$$\frac{y_2}{y_1} = \left(\frac{Q_{mc2}}{Q_{mc1}}\right)^{\frac{6}{7}} \qquad (3)$$

CASE 1c.

Case 1c is very complex. The depth of contraction scour depends on factors such as 1) how far back from the bank line the abutment is set, 2) the condition of the bank (is it easily eroded, are there trees on the bank, is it a high bank, etc.), 3) whether the stream is narrower or wider at the bridge than at the upstream section, 4) the magnitude of the overbank flow that is returned to the bridge opening, 5) the distribution of the flow in the bridge section, etc.

Case 1c is a general situation that can be analyzed using the contraction scour equations given in Chapter 2. The contraction scour in the main channel portion is an application of Equation 1. The only difference in this portion of the cross section at the bridge and case 1a is that the magnitude of Q_{mc2} is not intuitively obvious.

Equation 1 for the main channel portion becomes:

$$\frac{y_2}{y_1} = \left(\frac{Q_{mc2}}{Q_{mc1}}\right)^{\frac{6}{7}} \left(\frac{W_{c1}}{W_{c2}}\right)^{K_1} \qquad (4)$$

Q_{mc1} = flow in upstream main channel.

Q_{mc2} = flow in the main channel portion of the bridge cross section.

W_{c1} = bottom width of the upstream main channel.

W_{c2} = bottom width of the channel at the bridge less the width of piers.

A water surface model like WSPRO (23) can be used to determine the distribution of flow between the main channel and the set-back overbank areas in the contracted bridge opening.

The **set-back overbank area for Case 1c can be analyzed by using the clear water scour Equations 2a or 2b described in Chapter 2.** Again, the problem is in determining the discharge that will be in the overbank area. Each overbank area could be treated as a separate channel, but this case represents a situation for which flow continuity may not be appropriate because some of the approach overbank flow will probably end up in the main channel in the contracted section.

For the set-back portion, apply Equation 2a given in Chapter 2 with:

$Q_2 \quad = \quad Q_{ob2}$

$W_2 \quad = \quad W_{setback}$

Where:

Q_{ob2} = overbank flow through the contracted section for the left or right overbank area.

$W_{setback}$ = distance the abutment is set back from the main channel.

$$y_2 = \left[\frac{Q_{ob2}^2}{120\ D_{50}^{\frac{2}{3}}\ W_{setback}^2}\right]^{\frac{3}{7}} \qquad (5)$$

The quantity and depth of flow in the overbank area (left or right) can be determined using a water surface model like WSPRO (23). A conservative assumption for determining contraction scour on the setback overbank area would be that all of the overbank flow (left or right) at the upstream section must pass through the setback area as it moves through the contraction. The value of y_1 can best be approximated by the depth of flow on the overbank area (left or right).

Then:

$$Q_{ob1} = Q_{ob2}$$

If the abutment is set back only a small distance from the bank (less than 3 to 5 times the depth of flow through the bridge), there is the possibility that the combination of contraction scour and abutment scour may destroy the bank. Also, the two scour mechanisms are not independent. Then consideration should be given of using a guide bank or of rock riprapping the bank and bed under the bridge in the overflow area, using HEC 11 (24) to determine the rock riprap size.

Also, Laursen's abutment scour equations given in Appendix B will estimate both contraction and local scour at abutments, but will not give contraction scour for the channel.

b. **<u>CASE 2</u>. CONTRACTION SCOUR, NO OVERBANK FLOW. (LIVE-BED SCOUR)**

Case 2 is a special case where there is no overbank flow and the main channel narrows either naturally or due to the bridge piers or the abutment and embankment occupying part of the main channel. Assuming that the main channel is transporting bed material (live-bed) then Equation 1 applies and reduces to:

$$\frac{y_2}{y_1} = \left(\frac{W_{c1}}{W_{c2}}\right)^{k_1} \qquad (6)$$

Although the computations are the same for Cases 2a, 2b, and 2c, the latter two cases represent situations where contraction scour is not bridge related. Nevertheless this contraction scour is flood related and needs to be considered in the design or evaluation of a foundation. In Case 2b, Laursen's long contraction scour given in Equation 1 is conservative.

c. **<u>CASE 3</u>. CONTRACTION SCOUR, RELIEF BRIDGE WITH NO BED MATERIAL TRANSPORT. (CLEAR-WATER SCOUR)**

Case 3 applies to a relief bridge on a floodplain where there is no bed material transport. Use Laursen's 1963 equation (9) given in Chapter 2.

With some algebraic manipulation:

$$\frac{y_s}{y_1} = 0.13 \left[\frac{Q}{D_m^{\frac{1}{3}}\, y_1^{\frac{7}{6}}\, W_2} \right]^{\frac{6}{7}} - 1 \qquad (7)$$

y_s = Depth of scour.

y_1 = Depth of flow on the flood plain upstream of the relief bridge.

Q = Discharge through the relief bridge.

D_m = Effective mean diameter (feet) of the bed material (1.25 D_{50}) in the bridge opening.

D_{50} = Median diameter (feet) of bed material at relief bridge. Use a weighted average of the material in the scour zone.

W_2 = Bottom width of the relief bridge less pier widths.

All above dimensions are in feet.

Note. The depth y_1 is the depth upstream of the relief bridge that has active flow.

d. **<u>CASE 4</u>. CONTRACTION SCOUR, RELIEF BRIDGE WITH BED MATERIAL TRANSPORT. (LIVE-BED SCOUR)**

Case 4 is similar to Case 3, but there is sediment transport into the relief opening (live-bed scour). This case can occur when a relief bridge is over a secondary channel on the flood plain (See Figure 4.1). Hydraulically this is no different from Case 1, but analysis is required to determine the flood plain width associated with the relief opening and the flow distribution going to and through the relief bridge. This information could be obtained from WSPRO (23).

Use the equation given for Case 1 with appropriate adjustments of the variables.

3. <u>Other Contraction Scour Conditions.</u>

Contraction scour resulting from variable water surfaces downstream of the bridge is analyzed by determining the lowest potential water-surface elevation downstream of the bridge in so far as scour processes are concerned. Use the WSPRO (23) computer program to determine the flow variables, such as velocity and depths, through the bridge. With these variables, determine contraction and local scour depths.

Contraction scour in a channel bendway resulting from the flow through the bridge being concentrated in one area is analyzed by determining the superelevation of the water surface on the outside of the bend and estimating the resulting velocities and depths through the bridge. The maximum velocity in the outer part of the bend can be 1.5 to 2 times the mean velocity. A physical model study can also be used to determine the velocity and scour depth distribution through the bridge for this case.

Estimating contraction scour for unusual situations involves particular skills in the application of principles of river mechanics to the specific site conditions and such studies should be undertaken by engineers experienced in the fields of hydraulics and river mechanics. Highways in the River Environment (7) will be of great assistance.

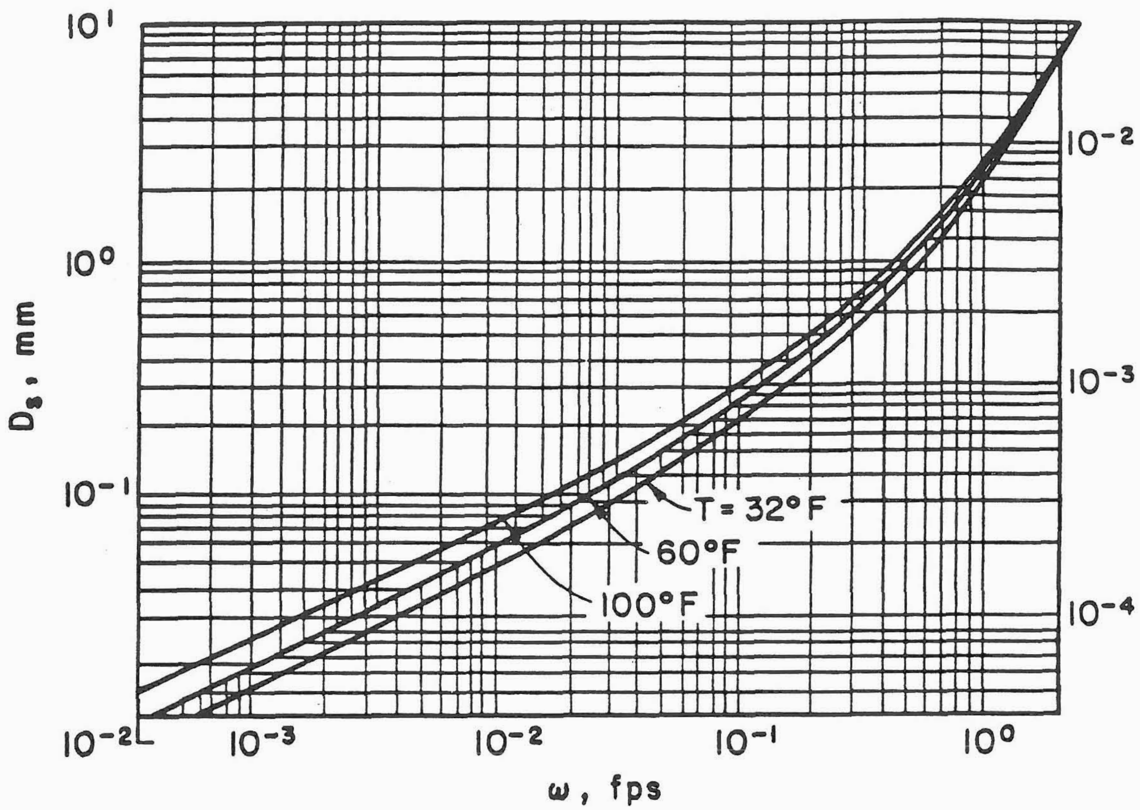


Figure 4.2 Fall Velocity of Sand Size Particles

E. STEP 5. LOCAL SCOUR AT ABUTMENTS

1. General.

Equations for predicting scour depths are based almost entirely on laboratory data. For example, Liu, et al's (1961) (25), Laursen's (1980) (26) and Froehlich's (1989) (27) equations are based entirely on laboratory data. The problem is that little field data on abutment scour exists. Liu, et al's equations were developed by dimensional analysis of the variables and a best-fit line was drawn through the laboratory data. Laursen's equations are based on inductive reasoning of the change in transport relations due to the acceleration of the flow caused by the abutment. Froehlich's equations are derived from a regression analysis of the available laboratory data.

EQUATIONS FOR ABUTMENT SCOUR ARE FOR THE WORSE-CASE CONDITIONS. THEY WILL PREDICT THE MAXIMUM SCOUR THAT COULD OCCUR FOR AN ABUTMENT PROJECTING INTO A STREAM WITH VELOCITIES AND DEPTHS UPSTREAM OF THE ABUTMENT SIMILAR TO THOSE IN THE MAIN CHANNEL. The reason for this is the way the experiments were conducted which do not represent many of the conditions in the field. For example, Liu's experiments were made in a rectangular laboratory flume with a sand bed. The abutments projected out various lengths from one wall or occasionally both walls of the flume. When they projected out from one flume wall then the other wall was taken as the centerline of the bridge. Other research was conducted similarly. Thus, the velocity, depth and sediment transport upstream of the abutment were about the same as in the main channel. Field conditions may have tree lined or vegetated banks, low velocities and shallow depths upstream of the abutment. If there is overland flow it often is at a shallower depth and lower velocity, with little bed material transport. **THEREFORE, ENGINEERING JUDGEMENT IS REQUIRED IN DESIGNING FOUNDATIONS FOR ABUTMENTS. IN MANY CASES FOUNDATIONS CAN BE DESIGNED WITH SHALLOWER DEPTHS THAN PREDICTED BY THE EQUATIONS AND THE FOUNDATIONS PROTECTED WITH ROCK RIPRAP PLACED BELOW THE STREAM BED OR A GUIDE BANK (SPUR DIKE) PLACED UPSTREAM OF THE ABUTMENT. COST WILL BE THE DECIDING FACTOR. A METHOD TO DETERMINE THE LENGTH OF A GUIDE BANK IS GIVEN IN APPENDIX C.**

2. Abutment Site Conditions.

Abutments can be set back from the natural stream bank or can project into the channel. They can have various shapes (vertical walls, spill through slopes) and can be set at varying angles to the flow. Scour at abutments can be live-bed or clear-water scour. Finally, there can be varying amounts of overbank flow intercepted by the approaches to the bridge and returned to the stream at the abutment.

3. Abutment Shape.

There are two general shapes for abutments; i.e., vertical-wall abutments with wing walls and spill-through abutments, Figure 4.3. Depth of scour is about double for vertical-wall abutments as compared with spill-through abutments.

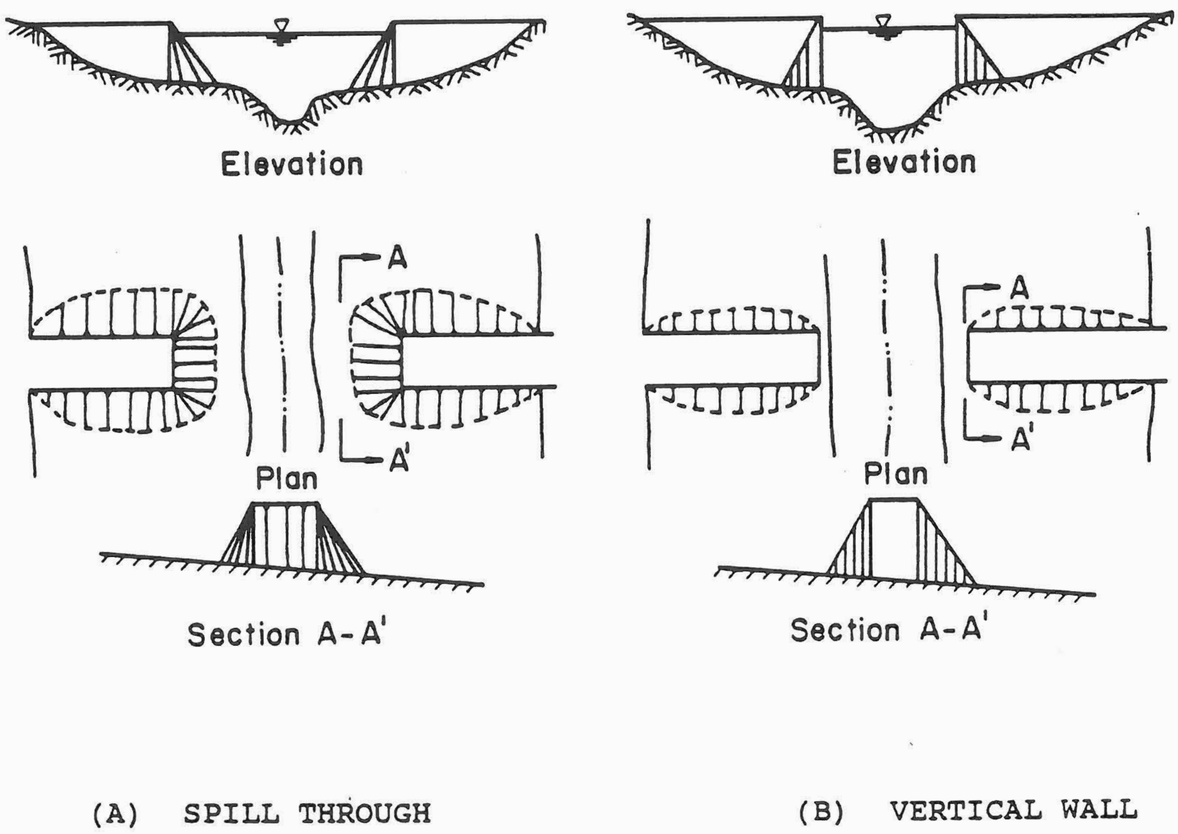


Figure 4.3 Abutment Shape

4. <u>Design for Scour at Abutments.</u>

It is recommended that foundation depths for abutments be set by AASHTO standards. Protection can be provided using rock riprap with the guidance from Chapter 7 and the design procedures of HEC 11 (24), and/or guide banks (spur dikes), designed per Appendix C.

LIVE-BED SCOUR AT ABUTMENTS

As a check on the potential depth of scour to aid in the design of the foundation and placement of rock riprap or guide banks, Froehlich's (27) LIVE-BED SCOUR equation given below can be used. Appendix B presents an alternate design approach, using material contained in the original FHWA Interim Procedures for Evaluating Scour at Bridges.

Froehlich (27) analyzed 170 live-bed scour measurements in laboratory flumes to obtain the following equation:

$$y_s/y_a = 2.27\, K_1\, K_2\, (a'/y_a)^{0.43}\, Fr_e^{0.61} + 1 \qquad (8)$$

Where:

K_1 = coefficient for abutment shape

K_2 = coefficient for angle of embankment to flow

$K_2 = (\theta/90)^{0.13}$

$\theta<90^0$ if embankment points downstream

$\theta>90^0$ if embankment points upstream

a' = the length of abutment projected normal to flow

$a' = A_e/y_a$

A_e = the flow area of the approach cross-section obstructed by the embankment.

Fr_e = Froude number of approach flow upstream of the abutment.

$= V_e/(gy_a)^{0.5}$

$V_e = Q_e/A_e$

Q_e = the flow obstructed by the abutment and approach embankment.

y_a = depth of flood plain flow at the abutment

y_s = scour depth

Description	K_1
VERTICAL-WALL ABUTMENT	1.0
VERTICAL-WALL ABUTMENT WITH WING WALLS	0.82
SPILL-THROUGH ABUTMENT	0.55

TABLE 4.1 ABUTMENT SLOPE COEFFICIENTS

Froehlich (28) suggested that scour depth be increased by $y_1/6$ if there are dunes in the main channel upstream of the abutment.

CLEAR-WATER SCOUR AT AN ABUTMENT

Use Equation 8 for live-bed scour since Froehlich's clear-water scour equation presented in Appendix B potentially decreases scour at abutments due to the presence of coarser material. This decrease is unsubstantiated by field data, however. Froehlich's clear-water scour equation is not recommended.

F. STEP 6. COMPUTE LOCAL SCOUR AT PIERS

1. General.

Local scour at piers is a function of bed material size, flow characteristics, fluid properties and the geometry of the pier. The subject has been studied extensively in the laboratory, but there is limited field data. As a result of the many studies, there are many equations. In general, the equations are for live-bed scour in cohesionless sand bed streams, which give similar results.

The FHWA (29) compared many of the more common equations in 1983. Comparison of these equations is given in Figures 4.4 and 4.5. Some of the equations have velocity as a variable (normally in the form of a Froude number). However some equations, such as Laursen's do not include velocity. A Froude number of 0.3 was used in Figure 4.4 for purposes of comparing commonly used scour equations. In Figure 4.5 the equations are compared with some field data measurements. As can be seen from Figure 4.5, the Colorado State University (CSU) equation encloses all the points, but gives lower values of scour than Jain's, Laursen's and Neill's equations. The CSU equation includes the velocity of the flow just upstream of the pier by including the Froude Number in the equation. Chang (30) pointed out that Laursen's (8) 1960 equation is essentially a special case of the CSU equation with the Fr = 0.4 (See Figure 4.6).

The equations illustrated in Figures 4.4, 4.5 and 4.6 do not take into account the possibility that larger sizes in the bed material could armor the scour hole. That is, the large sizes in the bed material will at some depth of scour limit the scour depth. Raudkivi and others (10,11,12,13) developed equations which take into consideration large particles in the bed. The significance of armoring the scour hole over a long time frame and over many floods is not known. THEREFORE, THESE EQUATIONS ARE NOT RECOMMENDED FOR USE AT THIS TIME.

TO DETERMINE PIER SCOUR, THE CSU EQUATION IS RECOMMENDED FOR BOTH LIVE-BED AND CLEAR-WATER SCOUR. The equation predicts equilibrium scour depths. In the <u>unusual</u> situation where a dune bed configuration exists at a site during flood flow, the maximum scour will be 30 percent greater than the predicted equation value. For the plane bed configuration, 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 CSU's equation.

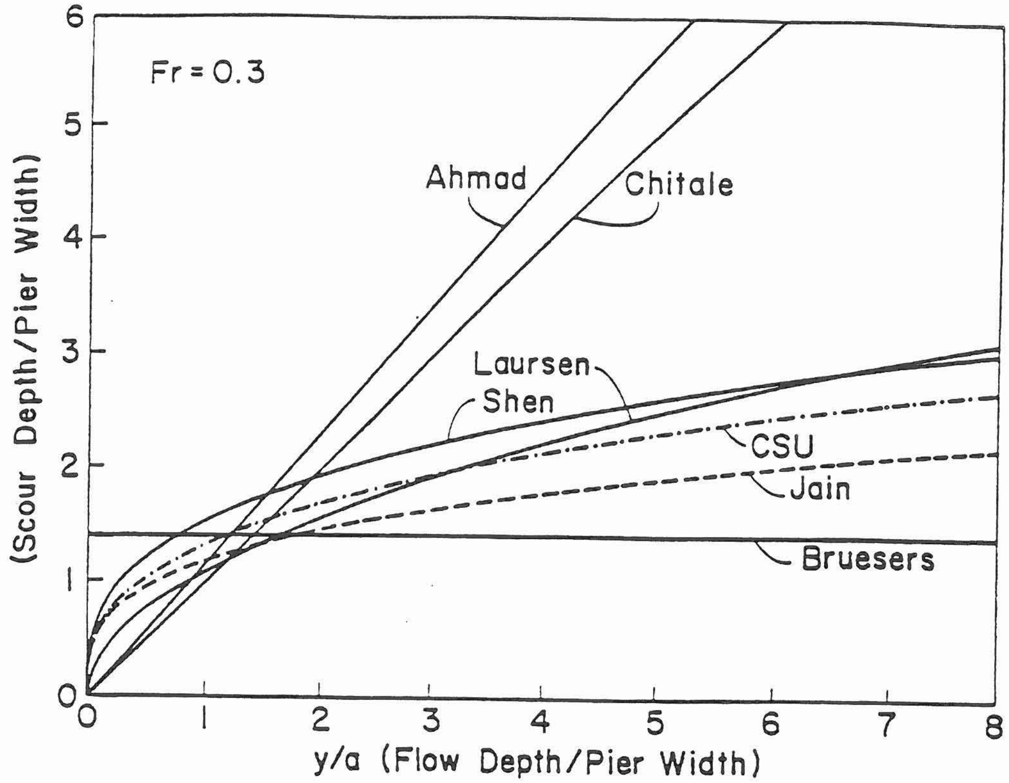


Figure 4.4 Comparison of Scour Formulas for Variable Depth Ratios (y/a) after Jones (29)

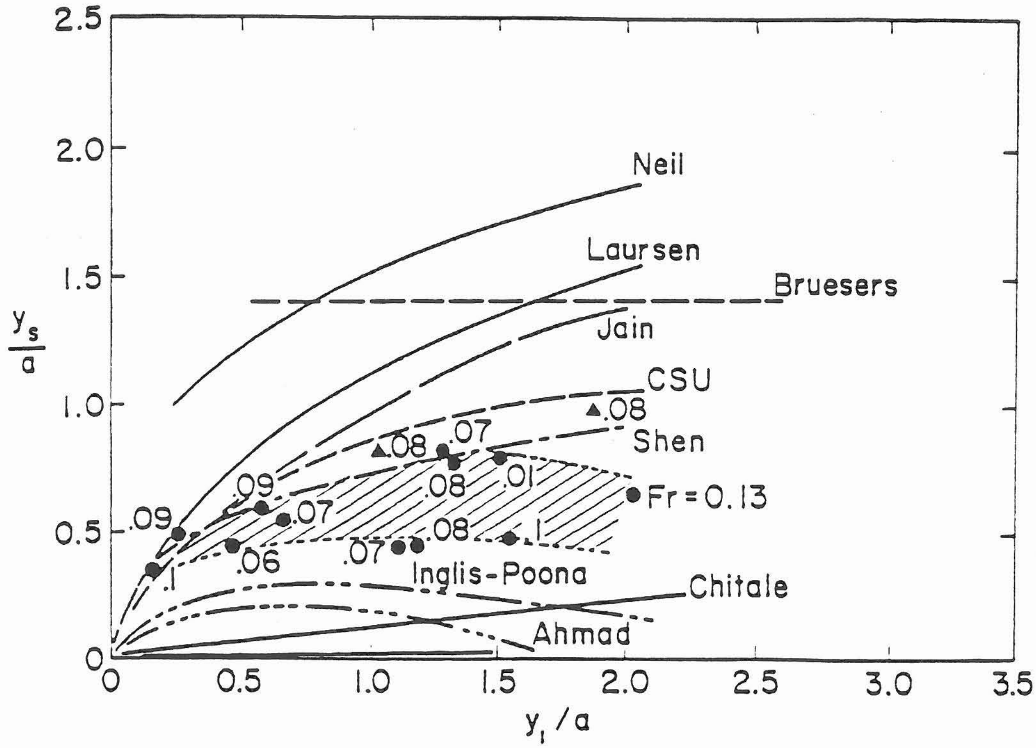


Figure 4.5 Comparison of Scour Formulas with Field Scour Measurements after Jones (29)

In Figure 4.6 the CSU equation relationship between y_s/a and y_1/a is given as a function of the Froude number. This relation was developed by Dr. Fred Chang (30). Note that Laursen's pier scour equation is a special case of the CSU equation when the Froude number is 0.4. Values of y_s/a values around 3.0 were obtained by Jain and Fisher (17) for chute and pool flows with Froude numbers as high as 1.5. The largest value of y_s/a for antidune flow was 2.5 with a Froude number of 1.2. Thus, the CSU equation will correctly predict scour depths for upper regime flows (plain bed, antidunes and chutes and pools).

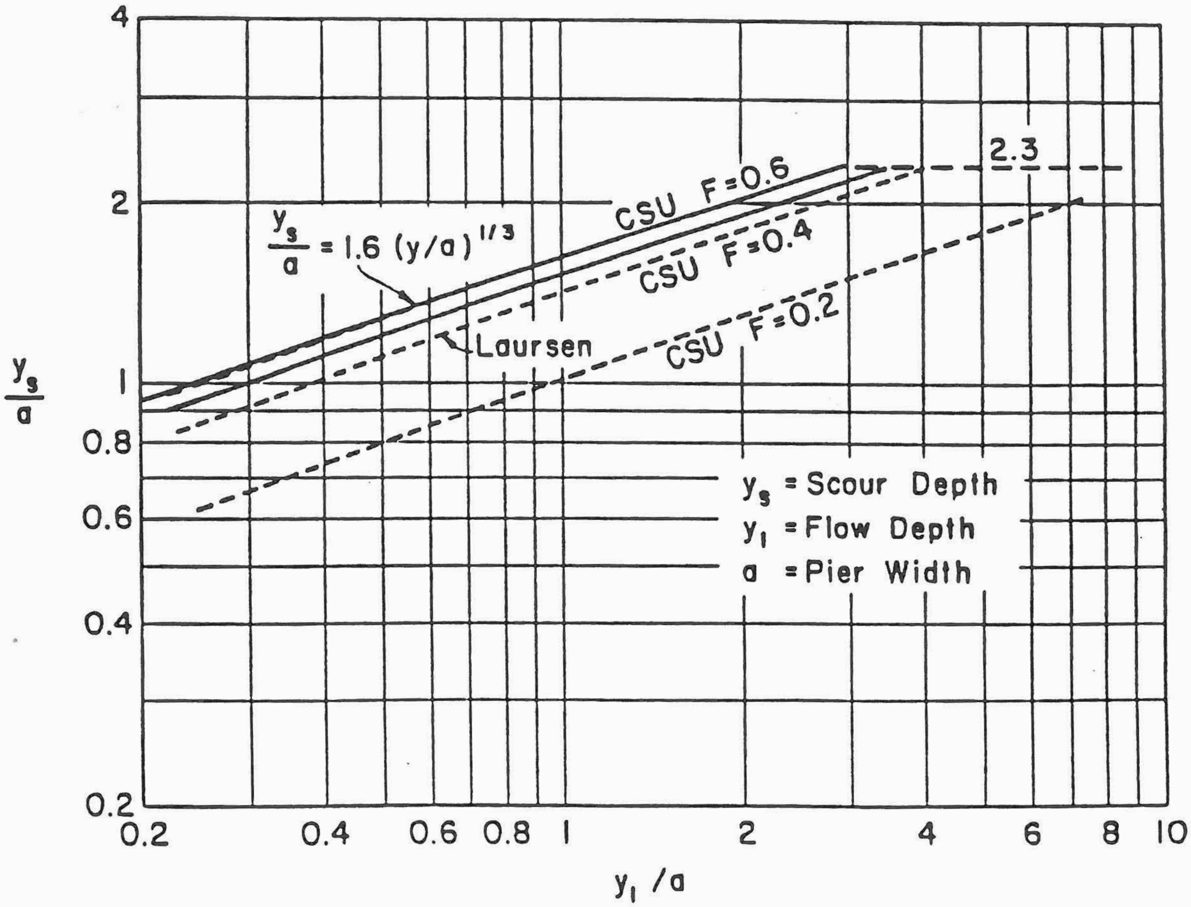


Figure 4.6 Values of y_s/a vs y_1/a for CSU'S Equation (30)

2. Computing Pier Scour.

The Colorado State University equation (7) is as follows:

$$\frac{y_s}{y_1} = 2.0\ K_1\ K_2\ \left(\frac{a}{y_1}\right)^{0.65} Fr_1^{0.43} \qquad (9)$$

Where:

y_s = scour depth
y_1 = flow depth just upstream of the pier
K_1 = correction for pier nose shape from Figure 4.7 and Table 4.3
K_2 = correction for angle of attack of flow from Table 4.4
a = pier width
Fr_1 = Froude number = $V_1/(gy_1)^{0.5}$

TABLE 4.2 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 4.3 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	2.5	3.5
45	2.3	3.3	4.3
90	2.5	3.9	5.0

Angle = skew angle of flow
L = length of pier

Note. The correction factor k_1 for pier nose shape should be determined using Table 4.2 for flow angle of attack up to 5 degrees. For greater angles, pier nose shape loses its affect and k_1 should be considered as 1.0.

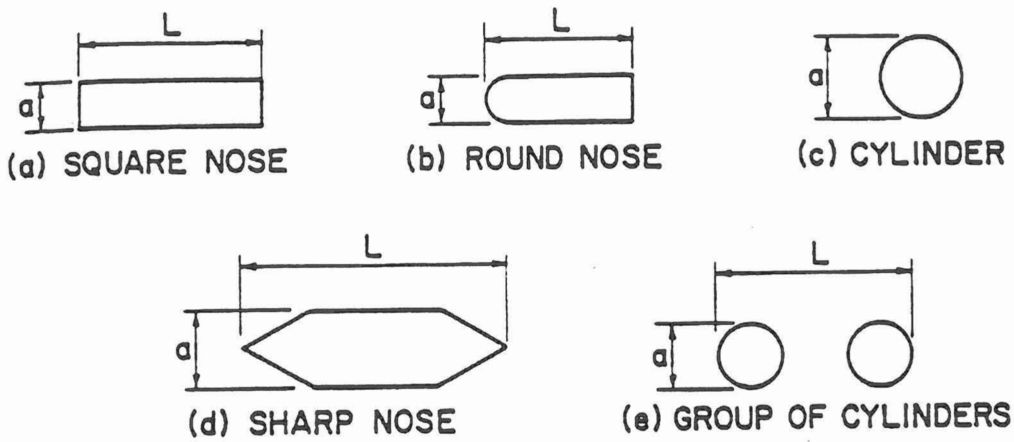


Figure 4.7 Common Pier Shapes

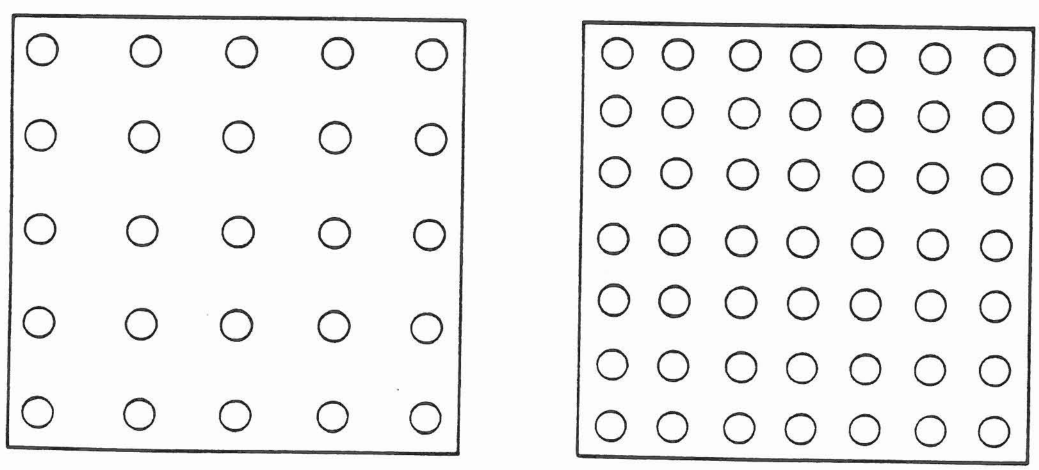


Figure 4.8 Pile Groups

3. <u>Pier Scour for Exposed Footings</u>

Often the pier footings and/or pile groups become exposed to the flow by scour. This may occur either from long term degradation, contraction scour, local scour or lateral shifting of the stream. Computations of local pier scour depths for footings or pile caps exposed to the flow based on footing or pile cap width appear to be too conservative. For example, calculations of scour depths for the Schoharie Creek bridge failure were closer to the measured model and prototype scour depths when pier width was used rather than footing width. Even in this case where the footing top was at the elevation of the bed surface the calculated depths were 47 percent larger than the measured (22 ft vs. 14 and 15 ft)(31). It appeared that the footing decreased the potential scour depth.

A recent model study of scour at the Acosta bridge at Jacksonville, Florida by FHWA (32) found that when the top of the footing was flush with the stream bed local scour was 20 percent less than for other conditions tested. The other conditions were bottom of the footing at the bed surface, the top of the footing at the water surface with pile group exposed and top of footing at mid depth. In a generalized study it was found that a footing with a lip extending upstream of the pier reduced pier scour when the top of the footing was located flush or below the bed but scour holes became deeper and larger in proportion to the extent that the footing projected into the flow field.

Based on this study, the following recommendation was made for calculating pier scour if the footing is or may be exposed to the flow (32).

> "It is recommended that the pier width be used for the value of 'a' in the pier scour equations if the top of the footing is at or below the streambed (taking into account contraction scour). If the pier footing extends above the stream bed, make a second computation using the width of the footing for the value of "a" and **<u>the depth and average velocity in the flow zone obstructed by the footing for the 'y' and 'V' respectively in the scour equation. Use the larger of the two scour computations"</u>**

Determine V_f obstructed by the footing using the following equation:

$$\frac{V_f}{V_1} = \ln(10.93\frac{y_f}{k_s}+1)/\ln(10.93\frac{y_1}{k_s}+1) \qquad (10)$$

Where:

V_f = average velocity in the flow zone below the top of the footing
y_f = distance from the bed to the top of the footing
k_s = the grain roughness of the bed. Normally taken as the D_{84} of the bed material.

The values of V_f and y_f would be used in the CSU equation given above.

4. Pier Scour for Exposed Pile Groups

FHWA (32) also conducted experiments to determine guidelines for specifying the characteristic width of a pile group (Figure 4.8) that are or may be exposed to the flow when the cylinders are spaced laterally as well as longitudinally in the stream flow. The following was concluded:

> "Pile groups that project above the stream bed can be analyzed conservatively by representing them as a single pier width equal to the projected area of the piles ignoring the clear space between piles. Good judgement needs to be used in accounting for debris because pile groups tend to collect debris that could effectively clog the clear spaces between pile and cause the pile group to act as a much larger mass."

For example, five 16-inch cylindrical piles spaced at 6 feet (Figure 4.8) would have an "a" value of 6.67 feet. This composite pier width would be used in Equation 9 to determine depth of pier scour. The correction factor "k_1" in Equation 9 for the multiple piles would be 1.0 regardless of shape. The depth of scour for exposed pile groups will be analyzed in this manner except when addressing the affect of debris lodged between piles. If debris is evaluated, it would be logical to consider the multiple columns and debris as a solid elongated pier. The appropriate L/a value and flow angle of attack would then be used to determine k_2 in Table 4.3.

5. Multiple columns

For multiple columns (as illustrated as a group of cylinders in Figure 4.8) skewed to the flow, the scour depth depends on the spacing between the piers. The correction factor for angle of attack would be smaller than for a solid pier. How much smaller is not known. Raudkivi (11) in discussing effects of alignment states "..the use of cylindrical columns would produce a shallower scour; for example, with five-diameter spacing the local scour can be limited to about 1.2 times the local scour at a single cylinder."

In application of the CSU equation with multiple columns, the pier width "a" is the total projected width of all the columns in a single bent, normal to the flow angle of attack. For example, three 24-inch cylindrical columns spaced at 10 feet would have an "a" value ranging between 2 and 6 feet, depending upon the flow angle of attack. This composite pier width would be used in Equation 9 to determine depth of pier scour. The correction factor "k_1" in Equation 9 for the multiple column would be 1.0 regardless of column shape. The depth of scour for a multiple column bent will be analyzed in this manner except when addressing the affect of debris lodged between columns. If debris is evaluated, it would be logical to consider the multiple columns and debris as a solid elongated pier. The appropriate L/a value and flow angle of attack would then be used to determine k_2 in Table 4.3.

Additional laboratory studies are necessary to provide guidance on the limiting flow angles of attack for given distance between multiple columns beyond which multiple columns can be expected to function as solitary members with minimal influence from adjacent columns.

6. Pressure Flow Scour

Pressure flow at a bridge occurs when bridge decks intersects the flow or are submerged. Limited flume studies at Colorado State University were conducted in the spring of 1990 with a bridge deck partly submerged, with a single pier in the flume, with different distances from the stream bed to the deck and with different flow velocities. There was no sediment transport upstream of the bridge (clear-water scour)(33). Without the deck submerged, there was no contraction scour and local scour occurred. With the deck submerged, there was contraction scour and pier scour depths increased by a factor of two to three. The magnitude of the contraction and local scour, as was to be expected, depended on the velocity of the approach flow and the distance from the deck to the bed. For the same approach velocity, contraction scour and pier scour increased as the distance from the bed to the deck decreased. Further analysis of the results of these experiments and additional laboratory study will be necessary to define the impact of bridge submergence on contraction and local scour.

7. Width of Scour Holes

The top width of a scour hole in cohesionless bed material from one side of a pier or footing can be estimated from the following equation:

$$W = y_s\ (K + \cot\theta) \qquad (11)$$

Where:

- W = top width of the scour hole from the side of the pier or footing
- y_s = scour depth
- K = bottom width of the scour hole as a fraction of scour depth
- θ = Angle of repose of the bed material (it ranges from about 30 to 44 degrees)(7)

If the bottom width of the scour hole is equal to the depth of scour "y_s" (K = 1), an unlikely condition, then the top width in cohesionless sand would vary from 2.07 to 2.80 y_s. At the other extreme if K = 0, the top width would vary from 1.07 to 1.8 y_s. Thus, the range in top width would probably be from 1.0 to 2.8 y_s.

G. STEP 7. PLOT TOTAL SCOUR DEPTHS AND EVALUATE DESIGN

1. Plot the Total Scour Depths.

On the cross-section of the stream channel and floodplain at the bridge crossing, plot the estimate of 1) long-term bed elevation change, 2) contraction scour, and 3) local scour at the piers and abutments. Use a distorted scale so that the scour determinations will be easy to evaluate. Make a sketch of any plan form changes (lateral stream channel movement due to meander migration, etc.) that might be reasonably expected to occur.

- Long-term elevation changes may be either aggradation or degradation.

- Contraction scour is then plotted from and below the long-term aggradation or degradation lines.

- Local scour is then plotted from and below the contraction scour line.

- Plot not only the depth of scour at each pier and abutment, but also the scour hole width. The width can be determined by assuming the bottom of the scour hole is 5 feet wider than the pier or footing and using the angle of repose of the bed material commonly assumed to be 30° for sand bed stream for the side slope of the hole. Or use 2.75 y_s.

2. <u>Evaluate the Total Scour Depths.</u>

- o Are the scour depths reasonable and consistent with the design engineer's previous experience, with his/her engineering judgement? If not, modify the depths to reflect the engineer's engineering judgement.

- o Do the local scour holes from the piers or abutments intersect between spans? If so, local scour depths are larger and indeterminate. Therefore, the length of the bridge opening should be reevaluated and the opening increased or the number of piers decreased as necessary.

- o Are there other factors (lateral movement of the stream, scour hole armoring, stream flow hydrograph, velocity and discharge distribution, moving of the thalweg, shifting of the flow direction, channel changes, type of stream, etc.) to be considered?

- o Do the calculated scour depths appear too deep for the conditions in the field, relative to the laboratory conditions (**Abutment scour equations are for the worse case conditions**). Would rock riprap or spur dikes (guide bank) be a more cost effective solution.

- o Evaluate cost, safety etc. Also, account for debris affects.

- o In the design of bridge foundations, the foundation elevation(s) should be at or below the total scour elevation(s).

3. <u>Reevaluate the Bridge Design.</u>

Reevaluate the bridge design on the basis of the foregoing scour analysis. REVISE THE DESIGN AS NECESSARY. This evaluation should consider:

- o Is the waterway area large enough; i.e., is contraction scour too large?

- o Are the piers too close to each other or to the abutments; i.e., do the scour holes overlap? The top width of a scour hole is about 2.75 times the depth of scour. If scour holes overlap, local scour can be deeper.

- o Is there a need for relief bridges? Should they or the main bridge be larger?

- o Are bridge abutments properly aligned with the flow and located properly in regard to the stream channel and flood plain?

- o Is the bridge crossing of the stream and the floodplain in a desirable location? If the location presents problems:

  a. Can it be changed?

  b. Can river training works, guide banks or relief bridges serve to provide for an acceptable flow pattern at the bridges?

- o Is the hydraulic study adequate to provide the necessary information for foundation design?

  a. Are flow patterns complex?

  b. Should a two-dimensional, water-surface profile model be used for analysis?

  c. Is the foundation design safe and cost effective?

  d. Is a physical model study needed/warranted?

IV. SCOUR EXAMPLE PROBLEMS

A. Example Problems.

STEP 1. DETERMINE SCOUR ANALYSIS VARIABLES

Q_{100} = 43,600 cfs. Q_{500} = 1.7 X 43,600 = 74,120 cfs.

- Main Channel:
  Dimensions
    Bank height = 7 ft
    Bottom width = 398 ft
    Top width at bank elevation 400 ft
  Q_{100}
    Average width = 400 ft, Average depth = 9.00 ft
    Hydraulic radius = A/WP = 3591/416 = 8.63 ft
    Slope = 0.00076, Manning n = 0.024
    Average velocity = 7.21 ft/s, Discharge = 25,890 cfs.
  Boring Results
    Bed material:
      D_{16} 0.18 mm, D_{50} 0.30 mm, D_{84} 2.8 mm.
      D_{50} of 0.30 mm. = 0.00098 ft with
      Fall velocity (w) = 0.13 ft/s (Figure 4.2)
    Description: Bed material is sand.
    Foundation material is sand similar to the bed material with some fine gravel lenses below 43 ft. Bed rock, which is shale, is 1,760 ft below stream bed.
  Bed Forms
    Low flow = Dunes, Max. height 2.4 ft. Q = 2,400 or less
    High flow = plane bed and antidunes.
  $Q_{bankfull}$
    Average width = 399 ft, Average depth = 7.00 ft
    Hydraulic radius = A/WP = 2793/412 = 6.78 ft
    Slope = 0.00076, Manning n = 0.020
    Average velocity = 7.36 ft/s, Discharge = 20,560 cfs.

- Right Overbank:
  Dimensions
    Top of bank above channel bed = 7 ft
    Length of overbank area = 52 ft
  Q_{100}
    Discharge = 70 +/- cfs, neglect.
    Average depth = 2.0 ft
    Average velocity = 0.67 ft/s

  Bed Material
    D_{16} ________, D_{50} 0.014 mm, D_{84} ________.
    Description: Sandy loam first 2.8 ft of depth. Then same material as in the stream bed.

Overbank Area Condition

Trees, brush and grass back to a gravel terrace that is 50 ft high. The conditions continue for about a mile downstream from the bridge site.

Bank Condition

Stable, no signs of erosion, sandy loam with grass above the washline which is at about a height of 3 ft above the bed. The brush and trees grow right to the bank. The bank, if disturbed, will need to be riprapped above, through and below the bridge.

- <u>Left Overbank:</u>

Dimensions

Top of bank above channel bed = 7 ft
Length of overbank area = 1,870 ft

Q_{100}

Discharge = 17,700 cfs
Average depth = 2.8 ft
Average velocity = 3.38 ft/s
Depth at abutment= 4.8 ft

Bed Material

D_{16} ________, D_{50} 0.014 mm, D_{84} ________.

Description: Sandy loam first 2.7 feet. Then same as material under stream channel.

Overbank Area Condition

Natural levee with trees, brush and grass back from the channel for about 30+/- ft. Then there is a field that is fairly level. The field is lower than the natural levee. The left side of the field ends at a gravel terrace over 100 ft high. The conditions continue for about a mile downstream from the bridge site.

Bank Condition

Same as the right bank. Stable, no signs of erosion, sandy loam with grass above the washline which is at about a height of 3 ft above the bed. The brush and trees grow right to the bank. The bank, if disturbed, will need to be riprapped above, through and below the bridge.

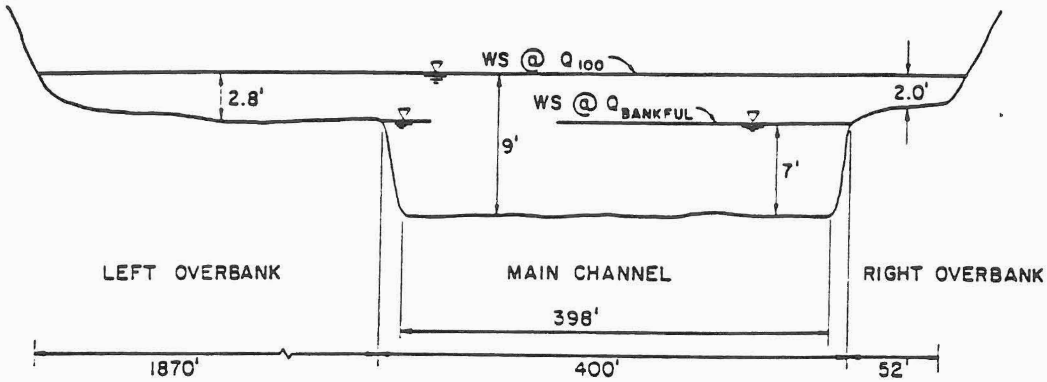


Figure 4.9 Conditions upstream of bridge

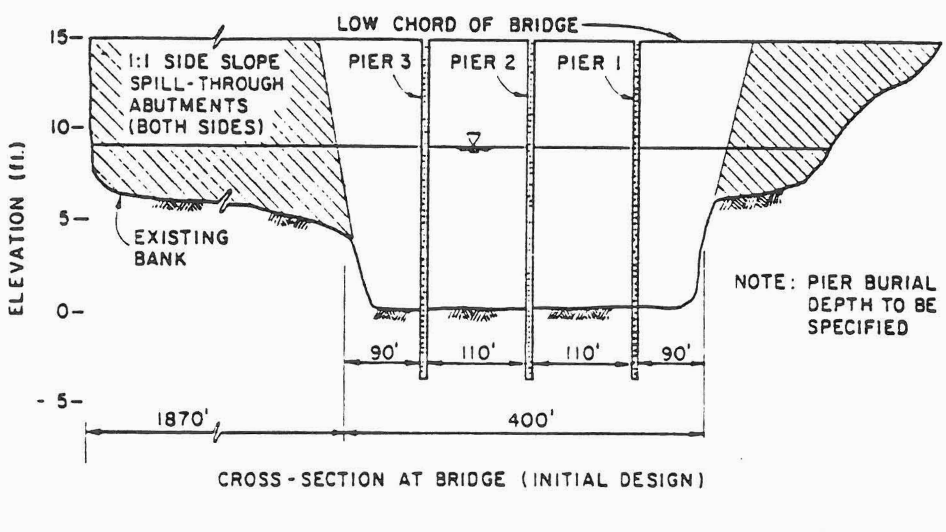


Figure 4.10 Location of piers and abutments

Bridge:

Dimensions

Bottom width first design = 398 ft

Abutments will start at or slightly inside the natural bank.

Number of piers. 3.

One pier is in center of the channel.

Spans between piers = 90, 110, 110, 90 ft

Distance from top of bank to bottom cord = 8 ft

Abutment is spill through with 1 to 1 slope.

Piers are numbered from right to left from the right bank (looking downstream)

Pier and footing geometry.

Pier width	= 3 ft
Pier length	= 36 ft
Pier shape	Round nose
Footing width	= 7 ft
Footing length	= 41 ft

Footing Elev. 2 ft below average stream bed elevation after contraction scour.

Q_{100} at Bridge

Discharge = 43,600 cfs, Manning n = 0.024

Right abutment

Angle with channel = 80^0

Left abutment

Angle with channel = 100^0

Pier 1

Angle of attack = 0^0

Pier 2

Angle of attack = 0^0

Pier 3

Angle of attack = 5^0

Channel Conditions:

Channel is straight for 3,000 ft upstream and for 4,600 ft downstream of the bridge site. The bends upstream and down are very mild so the flow through the bridge is fairly uniform, except for the flow moving to the bridge from the left overbank area.

STEP 2. ANALYZE LONG TERM BED ELEVATION CHANGE.

Analysis of the U. S. Geological Survey stage discharge relation at a gaging station five miles downstream of the bridge site indicates that there is a long term decrease in bed elevation. This decrease is gradual and averages about 0.02 ft per year. It results from erosion of a bed rock control located downstream of the gage. Because this is a sand bed stream this shift will be reflected in a long term bed elevation decrease at the bridge site of 2 feet in 100 years. The decrease does not appear to affect the stream hydraulics, but the main channel is getting deeper with respect to the banks.

Even though this will add 2 feet of long-term bed elevation change to the contraction and local scour, it will not be considered that the deeper main channel results in an increase in main channel flow and a decrease in the overbank flow over time. That is, the hydraulics at the site will not be considered to change. This is a conservative approach.

STEP 3. <u>EVALUATE SCOUR ANALYSIS METHOD</u>

Contraction scour will be limited to around 6 feet by sizing the bridge opening and/or the use of relief bridge if necessary.

Scour components will develop independently so analysis method given in Chapter 4, Step 3 will be used.

The velocity in the pier and abutment scour equations will be adjusted by coefficients times the mean velocity to account for the increase or decrease in velocity resulting from their location in the flow.

STEP 4. <u>CONTRACTION SCOUR</u>

<u>Problem 1</u>

Contraction scour with abutments at the edge of the channel (Case 1b).

$$\frac{y_2}{y_1} = \left(\frac{Q_{mc2}}{Q_{mc1}}\right)^{\frac{6}{7}} \left(\frac{W_{c1}}{W_{c2}}\right)^{K_1} \qquad (2)$$

$y_s = y_2 - y_1$ Average scour depth

Coefficients for Laursen's equation:

Bed material is sand with D_{50} =0.30 mm. = 0.00098 ft

Fall velocity (w) = 0.13 ft/s

Average Shear stress = 62.4 X 8.64 X 0.00076 = 0.41 lb/ft^2

$V_{*c} = (0.41/1.94)^{0.5} = 0.46$ ft/s

$V_{*c}/w = 0.46/0.13 = 3.5$ The mode of bed material transport is mostly by suspension.

Therefore:

$K_1 = 0.69$ and $K_2 = 0.37$

$$y_2/9 = \left(\frac{43,600}{25,890}\right)^{6/7} \left(\frac{398}{389}\right)^{0.69} = 1.59$$

$y_s = 14.3 - 9 = 5.3$ ft

Comments

This amount of contraction scour may be unacceptable because:

1. **This amount of contraction scour plus the local scour could place the foundations (footings or pile caps and piles) too deep.**

2. **The bed material that would be scoured out will deposit downstream, either in the channel or on the floodplain. If deposited in the channel, it could increase flooding.**

Solutions would be to set the abutments back from the channel. Another possibility would be a relief bridge.

A relief bridge to decrease the flow through the bridge would decrease the contraction scour further. However a relief bridge would be very costly.

Will accept this amount of contraction scour. In Problem 2, will calculate the discharge needed through the bridge opening to reduce the contraction scour to 2 feet.

<u>Problem 2</u>

What decrease in the discharge through the bridge is needed to reduce the contraction scour to 2 feet?

$y_s = y_2 - y_1, \quad 2 = y_2 - 9, \quad y_2 = 11, \quad y_2/9 = 1.22$

$$\frac{y_2}{9} = \left(\frac{X}{25{,}900}\right)^{6/7} \left(\frac{398}{389}\right)^{0.69} \left(\frac{0.024}{0.024}\right)^{0.37} = 1.22$$

$$\left(\frac{X}{25{,}900}\right)^{6/7} \left(\frac{398}{389}\right)^{0.69} = 1.22, \quad \frac{X}{25{,}900} = \left(\frac{1.22}{1.016}\right)^{7/6}$$

$$\left(\frac{X}{25{,}900}\right) = 1.20^{\ 7/6}$$

$$X = 32{,}000 \text{ cfs}$$

Decrease is 43,600 - 32,000 = 11,600 cfs

Problem 3

Contraction scour for relief bridge in left approach.

Estimate scour using Laursen's Case 3 equation:

$$\frac{y_s}{y_1} = 0.13 \left[\frac{Q}{D_m^{\frac{1}{3}}\ y_1^{\frac{7}{6}}\ W_2} \right]^{\frac{6}{7}} - 1 \qquad (7)$$

$Q_{relief\ bridge} = 43{,}600 - 32{,000} = 11{,}600$ cfs

y_1 = 2.8 ft
W_2 = 200 ft Assumed initial width within bridge waterway.
D_{50} = .00098 ft Use material under the soil layer at the relief bridge.
D_m = 1.25 X D_{50} = 0.00123 ft

$$\frac{y_s}{2.8} = 0.13 \left[\frac{11{,}600}{0.00123^{\frac{1}{3}}\ 2.8^{\frac{7}{6}}\ 200} \right]^{\frac{6}{7}} - 1 \qquad (7)$$

y_s = 9.20 X 2.8 ft = 25.6 ft

STEP 5. LOCAL SCOUR AT ABUTMENTS

Scour at abutments set at edge of main channel.

Use Froehlich's equation to calculate scour depths.

$$\frac{y_s}{y_a} = 2.27\ K_1\ K_2\ \left(\frac{a'}{y_a}\right)^{0.43} Fr_e^{0.61} + 1$$

K_1 = coefficient for abutment type

K_2 = $(\theta/90)^{0.13}$

a' = length of abutment intercepting overbank flow

$a' = A_e/y_a$

A_e = flow area of the approach cross-section obstructed by the embankment

Fr_e = Froude number of approach flow upstream of the abutment

$Fr_e = V_e/(gy_a)^{0.5}$

$V_e = Q_e/A_e$

Q_e = flow obstructed by the abutment and approach embankment

y_a = depth of flow at the abutment

Problem 1

Scour at right bank abutment.

Assume flow conditions in channel; use depth of flow in the main channel in the initial trial even though this may extent the imagination

$K_1 = 0.55$ (Table 4.1, Chapter 4)

$K_2 = (80/90)^{0.13} = 0.98$

$a' = (52 \times 2.0)/ 9.0 = 11.6$ ft

$V_e = 70/(52 \times 2.0) = 0.67$ ft/s

$Fr_e = 0.67 / (32.2 \times 9.0)^{0.5} = 0.04$

$y_s/9.0 = 2.27 \times 0.55 \times 0.98 \times (11.6/9.0)^{0.43} \times 0.04^{0.61} + 1$

$y_s/9.0 = 1.19, \quad y_s = 10.7$ ft

Does this appear reasonable? No? Why not? Based on this solution, the total depth of scour would be 18.0 feet below the present stream bed (10.7+5.3+2.0). The last two terms are the contraction scour and the long-term degradation.

Comments

This would seem to be much deeper scour than will occur! The limited flow coming around the abutment (70 cfs) intersects the flow in the channel, causing minor vortices, but will probably not produce 10.7 feet of abutment scour. The equations for abutment scour give worse case results. Also, this depth is caused by using the depth of flow of 9.0 feet at the toe of the abutment.

What to do?

1. **The scour depth would be between that calculated using the overbank flow depth at the abutment (2.0 ft) and the channel flow depth (9.0 ft) at the abutment.**
2. **To help in making a decision, calculate abutment scour using the overbank depth at the abutment.**

The depth (y_1) in overbank area near the channel upstream of the abutment is 2.0 feet.

$$K_1 = 0.55$$

$$K_2 = (80/90)^{0.13} = 0.98$$

$$a' = (52 \text{ X } 2.0)/\ 2.0 = 52 \text{ ft}$$

$$V_e = 70/(52 \text{ X } 2.0) = 0.67 \text{ ft/s}$$

$$Fr_e = 0.67\ /\ (32.2 \text{ X } 2.0)^{0.5} = 0.08$$

$$y_s/2.0 = 2.27 \text{ X } 0.55 \text{ X } 0.98 \text{ X } (52.0/2.0)^{0.43} \text{ X } 0.08^{0.61} + 1.0$$

$$y_s/2.0 = 2.1$$

$$y_s = 4.2 \text{ ft}$$

Does this appear reasonable? If not, why not? Based on this solution, the total depth of scour would be 11.5 feet below the present stream bed (4.2+5.3+2.0). The last two terms are the contraction scour and the long-term degradation.

Comments

Again, this may be deeper abutment scour than will occur. However, if the abutment was set back from the stream bank and the original bank was not disturbed, y_a would be based on the 2.0 feet of overbank flow depth used in the calculations. In that case, the scour would be the 4.2 feet from the toe of the abutment.

What to do?

Keep in mind that the abutment will, in all likelihood, be riprapped. This is the normal design practice within State highway agencies. From this perspective, should we be concerned what abutment scour depths are? Not really. That is precisely the reason why FHWA recommends in the text that abutment scour need not be calculated if appropriate protection (riprap and/or guide banks) is provided.

Problem 2

Scour at left bank abutment.

The depth (y_a) at the abutment is given as 4.8 ft. This is the flow depth at the toe of the abutment where it meets the top of main channel bank.

$$K_1 = 0.55$$

$$K_2 = (100/90)^{0.13} = 1.01$$

$$a' = (1,870 \times 2.8)/\ 4.8 = 1091 \text{ ft}$$

$$V_e = 17,700/(1,870 \times 2.8) = 3.38 \text{ ft/s}$$

$$Fr_e = 3.38\ /\ (32.2 \times 4.8)^{0.5} = 0.27$$

$$y_s/4.8 = 2.27 \times 0.55 \times 1.01 \times (1091/4.8)^{0.43} \times 0.27^{0.61} + 1$$

$$y_s/4.8 = 6.85$$

$$y_s = 32.9 \text{ ft}$$

Calculations of scour depth using the depth of flow in the channel (9.0 ft) give a scour depth of 34.6 feet.

In appendix B an equation is given for a/y_1 greater than 25. In this problem $a/y_1 = 1,870/\ 6.0 = 312$.

Therefore, try the equation for Case 6 given in Appendix B:

$y_s/y_1 \quad = \quad 4\ Fr_1^{0.33}$

$V_1 = \{43{,}600/(391 \times 9.0)\} \times 1.1 = 13.6$

$Fr_1 = 13.6/(32.2 \times 9.0)^{0.50} = 0.80$

$y_s/9.0 = 4\ (0.80)^{0.33} = 3.72$

$y_s = 9.0 \times 3.72 = 33.4$ ft

Does an abutment scour depth of about 33 feet sound reasonable? This would result in a total scour depth of in excess of 40.0 feet below the present stream bed (33+5.3+2.0).

Comments **All of these solutions are very deep! Even though these depths are judged to be very conservative, the scour potential is large what with the overbank flow of 17,700 cfs moving to and around the abutment.**

What to do?

<u>Keep in mind that the abutment will, in all likelihood, be riprapped. This is the normal design practice within State highway agencies. From this perspective, should we be concerned what abutment scour depths are? Not really. That is precisely the reason why FHWA recommends in the text that abutment scour need not be calculated if appropriate protection (riprap and/or guide banks) is provided.</u>

STEP 6. <u>LOCAL SCOUR AT PIERS</u>

Pier 1 and 2

$V_1 = 12.4$ ft/s (12.4 X 1.0)
$y_1 = 9.0$ ft
Angle of attack = θ^0

Pier 3

$V_1 = 14.9$ ft/s (12.4 X 1.2)
$y_1 = 9.0$ ft.
Angle of attack = 5^0

Problem 1.

Scour depth at Pier 1 and 2.

$y_s/y_1 = 2.0 \; K_1 \; K_2 \; (a/y_1)^{0.65} \; Fr_1^{0.43}$

$Fr_1 = 12.4 / (32.2 \times 9.0)^{0.5} = 0.73$

$y_s / 9.0 = 2.0 \times 1.0 \times 1.0 \times (3.0 / 9.0)^{0.65} \times (0.73)^{0.43}$

$y_s / 9.0 = 0.86$

$y_s = 7.7$ ft

Use $y_s = 7.7 \times 1.10 = 8.5$ ft (possible antidune flow)

Problem 2.

Pier 3 Scour depth.

$Fr_1 = 14.9 / (32.2 \times 9.0)^{0.5} = 0.88$

Angle of attack Coefficient TABLE 4.3

$L/a = 36 / 3 = 12$, $\theta = 5^0$, Coefficient = 1.5

$y_s / 9.0 = 2.0 \times 1.0 \times 1.5 \times (3 / 9.0)^{0.65} \times (0.88)^{0.43}$

$y_s / 9.0 = 1.4$

$y_s = 12.5$ ft

Use $y_s = 12.5 \times 1.10 = 13.8$ ft (possible antidune flow)

Comments

Would the same depth of scour occur at each pier? NO! Could the pier foundations be set at different depths if there was a substantial saving in cost? Yes. Why? Because it is in a long straight reach, has stable banks upstream and downstream and the channel flow is uniformly distributed across the width . It only has the deep scour at pier three when there is overbank flow.

STEP 7. <u>**PLOT AND EVALUATE TOTAL SCOUR**</u>

The plot of the scour for this problem is given in figure 4.11. Note that the scour holes for the left abutment and pier 3 overlap if the abutment scour is 33 ft.

Evaluation of scour

1. The abutment scour solutions are questionable even though the left overbank flow is very large, the bed material is sand and construction will disturb the area at the bridge. Use a guide bank with riprap on the left abutment and riprap the right.

2. Were there indications of stream instability, abutment foundations should be designed to at least the existing stream bed elevation with consideration given to an elevation dictated by long-term degradation plus contraction scour. Even though the stream is stable, abutment foundations will be evaluated to a depth of 7.3 feet (2 ft long term plus 5.3 ft contraction) below the stream bed.

3. When the left abutment is protected with a guide bank and riprap, the scour holes at the left abutment and pier 3 will not overlap.

4. Scour depths to be given geotechnical engineers are 15.8 feet (8.5+5.3+2.0) for pier 1 and 2 and 21.1 feet (13.8+5.3+2.0) for pier 3. Due to the channel being straight and the lack of overbank flow on the right side, it is possible to set piers 1 and 2 at shallower depths.

5. An interdisciplinary team consisting of hydraulic, geotechnical and structural engineer should review this bridge configuration and the scour depths. It might be advantageous to widen the bridge opening. Even a wider bridge would require a guide bank on the left side.

6. The structure should also be evaluated for the 500-year flood.

B. Other Example Problem.

Appendix F presents the scour analysis for the Great Pee Dee River in South Carolina.

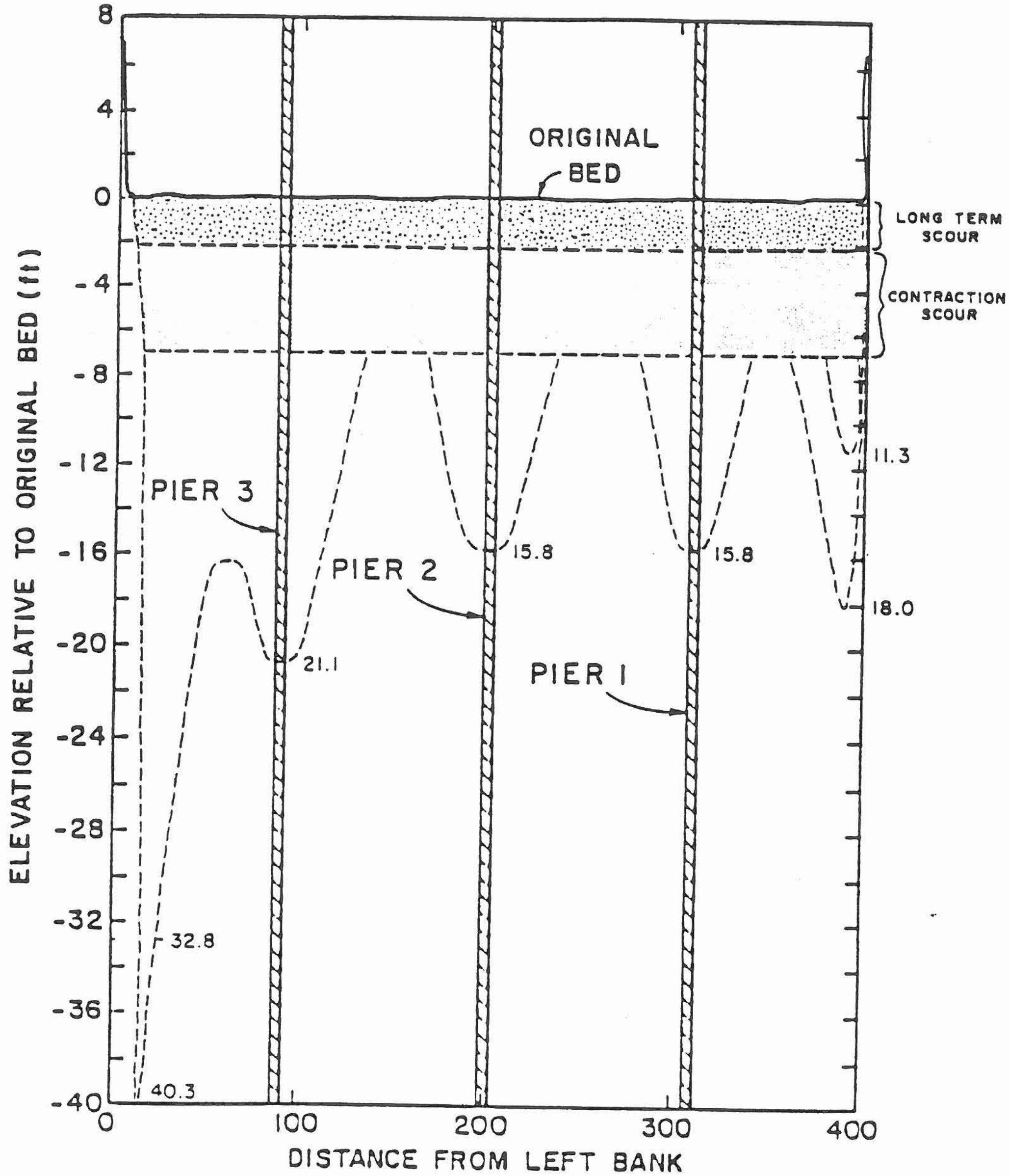


Figure 4.11 Plot of scour for example problem

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CHAPTER 5

EVALUATING THE VULNERABILITY OF EXISTING BRIDGES TO SCOUR

A. INTRODUCTION

Existing bridges over streams subject to scour should be evaluated to determine their vulnerability to floods and whether they are scour vulnerable. This assessment or evaluation should be conducted by an interdisciplinary team of professional, experienced engineers who can make the necessary engineering judgments to decide:

- priorities for making bridge scour evaluations;
- the scope of the scour evaluations to be performed in the office and in the field;
- whether or not a bridge is vulnerable to scour damage; i.e., whether the bridge is a scour-critical bridge;
- which alternative scour countermeasures may serve to make a bridge less vulnerable;
- which countermeasure is most suitable and cost-effective for a given bridge;
- priorities for installing scour countermeasures;
- monitoring and inspection schedules for scour-critical bridges; and
- interim procedures to protect the bridge and the public until the bridge is repaired, replaced or until suitable long-term countermeasures are in place.

The factors to be considered in a scour evaluation require a broader scope of study and effort than those considered in a bridge inspection. The major purpose of the bridge inspection is to identify changed conditions which may reflect an existing or potential problem. The scour evaluation is an engineering assessment of the risk of what might possibly happen in the future and what steps can be taken now to eliminate or minimize the risk.

B. THE EVALUATION PROCESS

The following approach is recommended regarding the development and implementation of a program to assess the vulnerability of existing bridges to scour:

STEP 1 Compile a list of those bridges with actual or potential problems due to scour. Structures that are candidates for this scour susceptible category include:

(a) Bridges currently experiencing scour or that have a history of scour problems during past floods as identified from maintenance records and experience, bridge inspection records, lane, etc.

(b) Bridges over erodible bed streams with design features that make them vulnerable to scour, including:

(1) piers and abutments designed with spread footings or short pile foundations;

(2) superstructures with simple spans or non-redundant support systems that render them vulnerable to collapse in the event of foundation movement; and

(3) bridges with inadequate waterway openings or with designs that collect ice and debris. Particular attention should be given to structures where there are no relief bridges or embankments for overtopping, and where all water must pass through or over the structure.

(c) Bridges on aggressive streams and waterways, including those with:

(1) active degradation or aggregation of the stream bed;

(2) significant lateral movement or erosion of stream banks;

(3) steep slopes or high velocities;

(4) in-stream materials mining operations in the vicinity of the bridge; and

(5) histories of flood damaged highways and bridges.

(d) Bridges located on stream reaches with adverse flow characteristics, including:

(1) crossings near stream confluences, especially bridge crossings of tributary streams near their confluence with larger streams;

(2) crossings on sharp bends in a stream; and

(3) locations on alluvial fans.

STEP 2 Prioritize the scour susceptible bridges, by conducting a preliminary office and field examination of the list of structures compiled in Step 1, using the following factors as a guide:

(a) The potential for bridge collapse or for damage to the bridge in the event of a major flood.

(b) The functional classification of the highway on which the bridge is located, and the effect of a bridge collapse on the safety of the traveling public and on the operation of the overall transportation system for the area or region.

See Appendix D which contains the North Carolina DOT's procedure for conducting office and field examinations for the prioritization of bridges.

STEP 3 Conduct field and office scour evaluations of the bridges on the prioritized list (Step 2) using an interdisciplinary team of hydraulic, geotechnical and structural engineers:

(a) The recommended evaluation procedure is to estimate scour for a superflood, a flood exceeding the 100-year flood, and then analyze the foundations for vertical and lateral stability for this condition of scour. This evaluation approach is similar in concept to the check procedure set forth in paragraph 6, Step 8 of the design procedure in Chapter III. FHWA recommends using the 500-year flood or a flow 1.7 times the 100-year flood for this purpose where the 500-year flood is unknown. The difference between designing a new bridge and assessing an old bridge is simply that the location and geometry of a new bridge and its foundation are not fixed as they are for an old bridge. Thus, the same steps for predicting scour at the piers and abutments should be carried out for an existing bridge as for a new bridge. Just as with the design of a new bridge, engineering judgement must be exercised in establishing the total scour depth for an existing bridge. The maximum scour depths that can be withstood by the existing foundation are compared with the greater scour. An engineering assessment must then be made as to whether the bridge should be classified as a scour-critical bridge; that is, whether the bridge foundations can not withstand the greater scour without failing.

(b) Enter the results of the scour evaluation study in the bridge inventory in accordance with the instructions in the FHWA "Bridge Recording and Coding Guide." (See Reference 4 and Appendix E.) Update the list of the scour-critical bridges.

<u>STEP 4</u> For bridges identified as scour critical from the office and field review in Step 2, determine a plan of action (See Chapter 7) for correcting the scour problem, including:

(a) Interim plan of action to protect the public until the bridge can be replaced or scour countermeasures installed. This could include:

i. Timely installation of temporary scour countermeasures such as riprap.

ii. Plans for monitoring scour-critical bridges during, and inspection after flood events, and for blocking traffic, if needed, until scour countermeasures are installed.

iii. Immediate bridge replacement or the installation of permanent scour countermeasures depending upon the risk involved.

(b) Establishing a time table for Step 5.

<u>STEP 5</u> After completing the scour evaluations for the list of potential problems compiled in Step 1, the remaining waterway bridges included in the State's bridge inventory should be evaluated. In order to provide a logical sequence for accomplishing the remaining bridge scour evaluations, another bridge list should be established, giving priority status to the following:

(a) The functional classification of the highway on which the bridge is located with highest priorities assigned to arterial highways and lowest priorities to local roads and streets.

(b) Bridges that serve as vital links in the transportation network and whose failure could adversely affect area or regional traffic operations.

The ultimate objectives of this scour evaluation program are 1) to review all bridges over streams in the National Bridge Inventory; 2) to determine those foundations which are stable for estimated scour conditions and those which are not; and 3) to provide interim scour protection for scour-critical bridges until adequate scour countermeasures are installed. This may include interim scour protection such as riprap, closing the bridge during high water, monitoring of scour-critical bridges during and inspection after flood events. The final objective 4) would be to replace the bridge or install scour countermeasures in a timely manner, depending upon the perceived risk involved.

C. CONDUCTING SCOUR EVALUATION STUDIES

An overall plan should be developed for conducting engineering bridge scour evaluation studies. An example of this type of a plan, prepared by the North Carolina Department of Transportation, is provided in Appendix D. It is recommended that each State develop its own plan for making engineering scour evaluations based on its own particular needs. The FHWA offers the following recommendations in regard to conducting these studies:

1. The first step of the scour evaluation study should be an office review of available information for purposes of assessing the stability of the stream and the adequacy of the bridge foundations to withstand a superflood (a Q500 flood or a flow 1.7 times Q100 flood, as recommended by the FHWA).

2. The use of worksheets is encouraged since they provide a consistent frame of reference for making field and office reviews and for documenting the results of the investigations.

3. To develop an efficient process for properly evaluating a large number of bridges, a logical sequence needs to be established for conducting the evaluations. This sequence should serve to screen out those bridges where scour is clearly not a problem. For example, sufficient information may be available in the office to indicate that the bridge foundations have been set well below maximum expected scour, and that a field inspection is not necessary for determining

that the bridge is not at risk from scour damage. However, a field inspection is generally recommended for bridges over streams that have one or more of the characteristics listed under Step 1 of the evaluation process, Section B of this chapter.

Where adequate hydraulic studies have been prepared and kept for the original bridge design, the scour estimates can be checked or recalculated from this information. Where hydraulic data is not available, it may have to be recalculated. For such instances, a "worse-case analysis" is suggested. If the bridge foundations are adequate for worse-case conditions, the bridge can be judged satisfactory. Where the worse-case analysis indicates that a scour problem may exist, further field and office analyses should be made.

THE FOLLOWING GUIDE IS OFFERED FOR CONDUCTING A WORSE-CASE ANALYSIS:

<u>Water-Surface Elevations</u>

Information may not be available on the water-surface elevations of the stream at some bridges. This can be compensated for by using procedures developed by the U.S. Geological Survey for many states. These procedures provide for estimating depths of flow by using hydrologic area, drainage area, flood frequency and error of estimate. Using these procedures, a conservative depth-discharge relationship can be determined. This relationship can then be used to develop rough estimates of scour.

<u>Long-Term Aggradation and Degradation</u>

Long-term stream bed profile changes will usually be difficult to assess. The main information sources are the records and knowledge of bridge inspectors, maintenance personnel, or others familiar with the bridge site and the behavior of the stream and other streams in the general area. If aggradation or degradation is a problem, there will usually be some knowledge of its occurrence in the area. Cross sections of the stream at the bridge site, for example, when taken by bridge inspectors over a period of time, may indicate a long-term trend in the elevation of the stream bed. Field inspections should be made at locations where the streams are known to be active and where significant aggradation/degradation or lateral channel movement is occurring. Further discussion on long-term stream bed elevation changes is included in Chapters 2, 3, and 4. Particular attention should be given to bridges at problem sites, as noted earlier in this section. Such bridges should be reviewed in the field. Additional information on conducting field reviews is included in Chapter 6.

Plan Form Changes

Assessing the significance of plan form changes, such as the shifting location of meanders, the formation of islands, and the overall pattern of streams, cannot usually be accomplished in the office. Records and photographs of bridge inspectors and maintenance personnel may provide some insight into the nature of the stream for the initial office assessments. Historical aerial photographs of the stream can be extremely valuable in this analysis. Ultimately, an engineering judgement must be made as to whether possible future or existing plan form changes represent a hazard to the bridge, and the extent of field work required to evaluate this condition.

Contraction Scour

Contraction scour may be calculated using the equations in Chapter 4 where the amount of overbank and main channel flow is known or can be estimated. The worst-case approach would involve estimating the largest reasonable amount of overbank flow on the floodplain beyond the bridge abutments and then calculating contraction scour on this basis. More detailed analyses are recommended for bridges at problem sites, especially where a large difference in the water-surface elevations may exist upstream and downstream of the bridge.

Local Pier Scour

To determine local pier scour use the equations given in Chapter 4.

Local Abutment Scour

Determination of local abutment scour using the equations in Chapter 4 requires an understanding of flow depths and velocities, and the flow distribution on the floodplain upstream of the bridge. However, some preliminary judgments may be developed as to the expected scour potential through an assessment of the abutment location, the amount of flow in the floodplain beyond the abutment and the extent of protection provided (riprap, guide banks, etc.).

D. DOCUMENTING BRIDGE SCOUR ASSESSMENTS

A record should be made of the results of field and office reviews of bridge scour assessments, and Item 113, Scour Critical Bridges, of the FHWA document "Recording and Coding Guide for the Structural Inventory and Appraisal of the Nation's Bridges" should be marked for inclusion in the national bridge inventory. The States have conducted field and office bridge scour assessments. An example of the North Carolina DOT's procedure is given in Appendix D.

CHAPTER 6

INSPECTION OF BRIDGES FOR SCOUR

A. INTRODUCTION

There are two main objectives to be accomplished in inspecting bridges for scour:

- o to accurately record the present condition of the bridge and the stream; and
- o to identify conditions that are indicative of potential problems with scour and stream stability for further review and evaluation by others.

In order to accomplish these objectives, the inspector needs to recognize and understand the inter-relationship between the bridge, the stream, and the flood plain. Typically, a bridge spans the main channel of a stream and perhaps a portion of the flood plain. The road approaches to the bridge are typically on embankments which obstruct flow on the flood plain. This overbank or floodplain flow must, therefore, return to the stream at the bridge and/or overtop the approach roadways. Where overbank flow is forced to return to the main channel at the bridge, zones of turbulence are established and scour is likely to occur at the bridge abutments. Further, piers and abutments may present obstacles to flood flows in the main channel, creating conditions for local scour because of the turbulence around the foundations. After flowing through the bridge, the flood water will expand back to the flood plain, creating additional zones of turbulence and scour.

The following sections in this Chapter present guidance for the bridge inspector's use in developing a comprehension of the overall flood flow patterns at each bridge inspected; and to use this information for rating the present condition of the bridge and the potential for damage from scour. When an actual or potential scour problem is identified by a bridge inspector, the bridge should be further evaluated by an interdisciplinary team using the approach discussed in Chapter 5. The results of this evaluation should be recorded under Item 113 of the "Bridge Recording and Coding Guide", Appendix E (4).

If the bridge is determined to be scour critical, a plan of action (Chapter 7) should be developed for installing scour countermeasures. In this case, the rating of the bridge substructure (Item 60 of the "Bridge Recording and Coding Guide") should be revised to reflect the effect of the scour on the substructure.

B. OFFICE REVIEW

It is desirable to make an office review of bridge plans and previous inspection reports prior to making the bridge inspection. Information obtained from the office review provides a better basis for inspecting the bridge and the stream. Items for consideration in the office review include:

- Has an engineering scour evaluation study been made? If so, is the bridge scour critical?
- If the bridge is scour critical, has a plan of action been made for monitoring the bridge and/or installing scour countermeasures?
- What do comparisons of stream bed cross-sections taken during successive inspections reveal about the stream bed? Is it stable? Degrading? Aggrading? Moving laterally? Are there scour holes around piers and abutments?
- What equipment is needed to obtain stream bed cross-sections? (rods, poles, sounding lines, etc.)
- Are there sketches and aerial photographs to indicate the plan form location of the stream and whether the main channel is changing direction at the bridge?
- What type of bridge foundation was constructed? (Spread footings, piles, drilled shafts, etc.) Do the foundations appear to be vulnerable to scour?
- Do special conditions exist requiring particular methods and equipment for underwater inspections? (divers, boats, electronic gear for measuring stream bottom, etc.)
- Are there special items that should be looked at? (Examples might include damaged riprap, stream channel at adverse angle of flow, problems with debris, etc.)

C. BRIDGE INSPECTION

During the bridge inspection, the condition of the bridge waterway opening, substructure, channel protection, and scour countermeasures should be evaluated, along with the condition of the stream.

The 1988 FHWA "Bridge Recording and Coding Guide" (4) (Appendix E) contains material for the following three items:

- Item 60: Substructure,

- o Item 61: Channel and Channel Protection, and
- o Item 71: Waterway Adequacy.

The guidance in the "Bridge Recording and Coding Guide" for rating the present condition of Items 61 and 71 is set forth in detail. Guidance for rating the present condition of Item 60, Substructure, is general and does not include specific details for scour. The following sections present approaches to evaluating the present condition of the bridge foundation for scour and the overall scour potential at the bridge.

1. <u>Substructure</u>. Item 60, Substructure, is the key item for rating the bridge foundations for vulnerability to scour damage. When a bridge inspector finds that a scour problem has already occurred, it should be considered in the rating of Item 60. Both existing and potential problems with scour should be reported so that a scour evaluation can be made by others. The scour evaluation is reported on Item 113 in the revised "Bridge Recording and Coding Guide." If the bridge is determined to be scour critical, the rating of Item 60 should be evaluated to ensure that existing scour problems have been considered. The following items are recommended for consideration in inspecting the present condition of bridge foundations:

- o Evidence of movement of piers and abutment;

  -rotational movement (check with plumb line),

  -settlement (check lines of substructure and superstructure, bridge rail, etc., for discontinuities; check for structural cracking or spalling),

  -check bridge seats for excessive movement.

- o Damage to scour countermeasures protecting the foundations (riprap, guide banks, sheet piling, sills, etc.),
- o Changes in streambed elevation at foundations (undermining of footings, exposure of piles), and
- o Changes in streambed cross-section at the bridge, including location and depth of scour holes.

In order to note the conditions of the foundations, the inspector should take cross sections of the stream, noting location and condition of stream banks. Careful measurements should be made of scour holes at piers and abutments, probing soft material in scour holes to determine the location of a firm bottom. If equipment or conditions do not permit measurement of the stream bottom, this condition should be noted for further action.

2. <u>Assessing Scour Potential at Bridges</u>. The items listed in Table 6.1 are provided for bridge inspectors' consideration in assessing the adequacy of the bridge to resist scour. In making this assessment, inspectors need to understand and recognize the interrelationships between Item 60 (Substructure), Item 61 (Channel and Channel Protection), and Item 71 (Waterway Adequacy). As noted earlier, additional follow-up by others should be made utilizing Item 113 (Scour Critical Bridges) when the bridge inspection reveals a potential problem with scour.

Table 6.1 ASSESSING THE SCOUR POTENTIAL AT BRIDGES

1. UPSTREAM CONDITIONS

a. Banks

STABLE: Natural vegetation, trees, bank stabilization measures such as riprap, paving, gabions, channel stabilization measures such as dikes and groins.

UNSTABLE: Bank sloughing, undermining, evidence of lateral movement, damage to stream stabilization installation's, etc.

b. Main Channel

- Clear and open with good approach flow conditions, or meandering or braided with main channel at an angle to the orientation of the bridge.
- Existence of islands, bars, debris, cattle guards, fences that may affect flow.
- Aggrading or degrading stream bed.
- Evidence of movement of channel with respect to bridge (make sketches, take pictures).

c. Floodplain

- Evidence of significant flow on flood plain.
- Flood plain flow patterns - does flow overtop road and/or return to main channel?
- Existence and hydraulic adequacy of relief bridges (if relief bridges are obstructed, they will affect flow patterns at the main channel bridge).
- Extent of flood plain development and any obstruction to flows approaching the bridge and its approaches.
- Evidence of overtopping approach roads (debris, erosion of embankment slopes, damage to riprap or pavement, etc.).

d. Debris

- Extent of debris in upstream channel.

TABLE 6.1 CONTINUED

e. Other Features

- Existence of upstream tributaries, bridges, dams, or other features, that may affect flow conditions at bridges.

2. CONDITIONS AT BRIDGE

a. Substructure

b. Superstructure

- Evidence of overtopping by floodwater (Is superstructure tied down to substructure to prevent displacement during floods?)

- Obstruction to flood flows (Does it collect debris or present a large surface to the flow?)

- Design (Is superstructure vulnerable to collapse in the event of foundation movement as are simple spans and non-redundant design for load transfer?)

c. Channel Protection and Scour Countermeasures

- Riprap (Is riprap adequately toed into the stream bed or is it being undermined and washed away? Is riprap pier protection intact, or has riprap been removed and replaced by bed load material? Can displaced riprap be seen in streambed below bridge?)

- Guide banks (Spur dikes) (Are guide banks in place? Have they been damaged by scour and erosion?)

- Stream and streambed (Is main current impinging upon piers and abutments at an angle? Is there evidence of scour and erosion of streambed and banks, especially adjacent to piers and abutments? Has stream cross section changed since last measurement? In what way?)

d. Waterway Area (Does waterway area appear small in relation to stream and its flood plain? Is there evidence of scour across a large portion of the stream bed at the bridge? Do bars, islands, vegetation, and debris constrict flow and concentrate it in one section of the bridge or cause it to attack piers and abutments? Do the superstructure, piers, abutments, and fences, etc., collect debris and constrict flow? Are approach roads regularly overtopped? If waterway opening is inadequate, does this increase the scour potential at bridge foundations?)

3. DOWNSTREAM CONDITIONS

a. Banks

STABLE: Natural vegetation, trees, bank stabilization measures such as riprap, paving, gabions, channel stabilization measures such as dikes and groins.

UNSTABLE: Bank sloughing, undermining, evidence of lateral movement, damage to stream stabilization installations, etc.

b. Main Channel

- Clear and open with good "getaway" conditions, or meandering or braided with bends, islands, bars, cattle guards, and fences that retard and obstruct flow.

- Aggrading or degrading stream bed.

- Evidence of downstream movement of channel with respect to the bridge (make sketches and take pictures).

c. Flood plain

- Clear and open so that contracted flow at bridge will return smoothly to flood plain, or restricted and blocked by dikes, developments, trees, debris, or other obstructions.

- Evidence of scour and erosion due to downstream turbulence.

d. Other Features

- Downstream dams or confluence with larger stream which may cause variable tailwater depths. (This may create conditions for high velocity flow through bridge).

D. UNDERWATER INSPECTIONS

Perhaps the single most important aspect of inspecting the bridge for actual or potential damage from scour is the taking and plotting of measurements of stream bottom elevations in relation to the bridge foundations. Where conditions are such that the stream bottom cannot be accurately measured by rods, poles, sounding lines or other means, other arrangements need to be made to determine the condition of the foundations. Other approaches to determining the cross-section of the streambed at the bridge include:

- use of divers; and
- use of electronic scour and radar equipment (Appendix G).

For the purpose of evaluating resistance to scour of the substructure under Item 60 of the "Bridge Recording and Coding Guide," the questions remain essentially the same for foundations in deep water as for foundations in shallow water:

- What does the stream cross-section look like at the bridge?
- Have there been any changes as compared to previous cross-section measurements? If so, does this indicate that (1) the stream is aggrading or degrading; or (2) local or contraction scour is occurring around piers and abutments?
- What are the shape and depths of scour holes?
- Is the foundation footing (or the piling) exposed to the stream flow; and if so, what is the extent and probable consequences of this condition?
- Has riprap around a pier been moved or removed?

E. NOTIFICATION PROCEDURES

A bridge inspector's site evaluation of the effect of water at the bridge is an important part of a bridge inspection. A positive means of promptly communicating inspection findings to proper agency personnel must be established. Any condition that a bridge inspector considers to be of an emergency or potentially hazardous nature should be reported immediately. That information as well as other conditions which do not pose an immediate hazard, but still warrant further action should be conveyed to the hydraulic/foundation engineers for review.

A report form is, therefore, needed to communicate pertinent problem information to the hydraulic/geotechnical engineers. An existing report form may currently be used by bridge inspectors within a State highway agency to advise maintenance personnel of specific needs. Regardless of whether an existing report is used or a new one is developed, a bridge inspector should be provided the means of advising hydraulics and geotechnical engineers of problems in a timely manner.

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CHAPTER 7

PLAN OF ACTION FOR INSTALLING SCOUR COUNTERMEASURES

A. INTRODUCTION

Scour Countermeasures are those features incorporated at a later date to make a bridge less vulnerable to damage or failure from scour.

New bridges

For new bridges, recommended scour countermeasures have been addressed in Chapters 3 and 4. In summary, the best solutions for minimizing scour damage include:

- locating the bridge to avoid adverse flood flow patterns,
- streamlining bridge elements to minimize obstructions to the flow, and
- deepening the foundations to accommodate scour.

Existing Bridges

For existing bridges, the alternatives available for protecting the bridge from scour are listed below in a rough order of cost:

- providing riprap at piers and abutments,
- constructing guide banks (spur dikes),
- constructing channel improvements,
- strengthening the bridge foundations,
- constructing sills or drop structures, and
- constructing relief bridges or lengthening existing bridges.

These alternatives should be evaluated using sound hydraulic engineering practice.

In developing a plan of action for protecting an existing scour-critical bridge, the four aspects that need to be considered are:

- monitoring, inspecting and potentially closing a bridge until the countermeasures are installed,
- installing temporary scour countermeasures, such as riprap around a pier, along with monitoring a bridge during high

flow,

- o selecting and designing scour countermeasures, and
- o scheduling construction of scour countermeasures.

These considerations are discussed in the following sections.

B. MONITORING, INSPECTING, AND POTENTIALLY CLOSING SCOUR-CRITICAL BRIDGES

As noted in Chapter 5, special attention should be given to monitoring scour-critical bridges during and after flood events. The plan-of-action for a bridge should include special instructions to the bridge inspector, including guidance as to when a bridge should be closed to traffic. Guidance should be given to other DOT officials on bridge closure. The intensity of the monitoring effort is related to the risk of scour hazard, as determined from the scour evaluation study. The following items are recommended for consideration when developing the plan-of-action monitoring effort.

1. Information on any existing rotational movement of abutments and piers or settlement of foundations.

2. Information on rates of stream bed degradation, aggradation, or lateral movement based on analysis of changes in stream cross sections taken during successive bridge inspections, sketches of the stream plan form, aerial photographs, etc.

3. Recommended procedures and equipment for taking measurements of stream bed elevations (use of rods, probes, weights, etc.) during and after floods.

4. Guidance on maximum permissible scour depths, flood flows, water surface elevations, etc., beyond which the bridge should be closed to traffic.

5. Reporting procedures for handling excess scour, larger than normal velocities and water surface elevation or discharge that may warrant bridge closure. Who makes closure decisions and how are they implemented?

6. Instructions regarding the checking of stream bed levels in deep channels where accurate measurements cannot be made from the bridge (use of divers, electronic instruments such as sonar, radar, etc.).

7. Instructions for inspecting existing countermeasures such as riprap, dikes, sills, etc.

8. Forms and procedures for documenting inspection results and instructions regarding follow-up actions when necessary.

9. Information on installation of scour depth warning devices.

C. TEMPORARY COUNTERMEASURES.

Monitoring of bridges during high flow may indicate that collapse from scour is imminent. It may be disadvantageous, however, to close the bridge during high flow because of traffic volume, poor alternate routes, the need for emergency vehicles to use the bridge, etc. Temporary scour countermeasures such as riprap could be installed, allaying the need for immediate closure. Temporary countermeasure installed at a bridge along with monitoring during and inspection after high flows could provide for the safety of the public without closing the bridge.

D. SCHEDULING CONSTRUCTION OF SCOUR COUNTERMEASURES

The engineering scour evaluation study should address the risk of failure at scour-critical bridges so that priorities and schedules can be prepared for installation of scour countermeasures at differing bridge sites. In some cases, the risk may be obvious, as where an inspection reveals that a spread footing for a pier has been partially undermined. Immediate action is warranted. In other cases, the need for immediate action is not so apparent, and considerable judgement must be exercised. An example of the latter case is where a stream meander is gradually encroaching upon a bridge abutment. A judgment must be made on the risk associated with the rate of change of the meander and its probable effect on the abutment and associated foundation.

Problems are common with such gradual river changes. As a consequence, the engineer may wait too long to take action. As the degree of encroachment and scour hazard increases, the number of alternative countermeasures is decreased and costs of correction are corresponding increased. In addition, monitoring a bridge during high flows and inspection after high flow may not determine that a bridge is about to collapse from scour.

E. TYPES OF COUNTERMEASURES

An overview of commonly used scour countermeasures is provided below, along with references for obtaining design procedures and criteria for their application to a specific site. Selection of the appropriate countermeasure is best accomplished through a field and office evaluation of the conditions at the stream crossing.

1. <u>Rock Riprap at Piers and Abutments</u>. The FHWA continues to

evaluate how best to design rock riprap at bridge abutments and piers.

Present knowledge is based on research conducted under laboratory conditions with little field verification, particularly for piers. Flow turbulence and velocities around a pier are of sufficient magnitude that large rocks move over time. Bridges have been lost (Schoharie Creek bridge for example) due to the removal of riprap at piers resulting from turbulence and high velocity flow. Usually this doesn't happen during one storm, but is the result of a sequence of high flows. **Therefore, if rock riprap is placed as scour protection around a pier, the bridge should be monitored and inspected after each high flow event until it is determined that the riprap is stable.**

Sizing Rock Riprap at Abutments

The FHWA conducted an as-yet-unpublished 1990 research study for transverse encroachments of up to about 20 percent of a flood plain width. This study indicates a multiplier of 1.8 times the average constricted or bridge waterway velocity for sizing rock riprap with the design approach of HEC 11 (24) is adequate. Because research must yet consider abutment conditions when contiguous to the main channel, these current recommendations are for abutments on the flood plain, set back from the main channel.

The FHWA study consistently indicated that rock riprap failed at the toe rather than on the slope of the abutment. **It is, therefore, recommended for encroachments not exceeding 20 percent of the flood plain width and abutments removed from the main channel that HEC 11 be used with the 1.8 velocity multiplier.**

The rock apron should extend along the entire length of the abutment toe, around the curved portions of the abutment to the point of tangency with the plane of the embankment slope, both upstream and downstream. The apron should extend away from the toe of the abutment into the bridge waterway a dimension equal to 15 percent of the distance from the edge of the flood plain, for the discharge under consideration, to the top of the main channel bank within the bridge waterway. Because the distance form the edge of flood plain to the main channel bank may well differ on the left and right sides of the main channel, the riprap apron extensions from the toe of abutments into the bridge waterway will differ as well. The designer must use judgement in limiting the apron extension into the waterway for wider flood plains. A maximum dimension of 25 feet would seem reasonable.

The face of the abutment should be protected by the same size rock riprap. The rock riprap on the slope should be carried around the curved potions of an abutment, to terminate at the same point of tangency with the embankment slope discussed above for the apron. FHWA will give further guidance in 1992 on sizing abutment rock riprap for greater flood plain encroachments, pending completion of further research.

Sizing Riprap at Piers

Determine the D_{50} size of the riprap using the rearranged Ishbash equation (34) to solve for stone diameter (in feet, for fresh water):

$$D_{50} = \frac{0.692\ (K\ V)^2}{(s-1)\ 2g}$$

where:
D_{50} = median stone diameter (ft)
K = coefficient for pier shape
V = average velocity approaching pier (ft/sec)
s = specific gravity of riprap (normally 2.65)
g = 32.2 ft/sec^2

K = 1.5 for round-nose pier
K = 1.7 for rectangular pier

To determine V, multiply the average channel velocity (Q/A) by a coefficient that ranges from 0.9 for a pier near the bank in a straight uniform reach of the stream to 1.7 for a pier in the main current of flow around a bend.

- Provide a riprap mat width that extends horizontally at least two times the pier width, measured from the pier face.

- Place the top of a riprap mat at the same elevation as the stream bed. The deeper the riprap is placed into the stream bed, of course, the less likely it will be moved. Placing the bottom of a riprap mat on top of the stream bed is discouraged. **In all cases where riprap is used for scour control, the bridge must be monitored and inspected after high flows.**

  Note. A disadvantage to burying riprap so that the top of the mat is somewhat below the stream bed is that inspectors have difficulty determining if some or all of the riprap has been removed. Therefore, it is wiser to place the top of a riprap mat at the same elevation as the stream bed.

- The thickness of the riprap should be three stone diameters or more.

- In some conditions, place the riprap on filter cloth or a gravel filter. However, if a well-graded riprap is used, a filter may not be needed. In some flow conditions it may not be possible to place a filter or if the riprap is buried in the bed a filter may not be needed.

- The maximum size rock should be no greater than twice the D_{50} size.

2. Guide Banks. Methods for designing guide banks are contained in the FHWA publication Hydraulic Design Series No. 1, "Hydraulics of Bridge Waterways" and HEC 20 (6). A one page summary of the design is in Appendix C. The hydraulic effect of guide banks can be modeled through the use of the FHWA software, WSPRO (23). The purpose of the guide bank is to provide a smooth transition for flows on the flood plain returning to the main channel at the bridge. The guide bank serves to move the point of maximum scour upstream, away from the abutment. Guide banks should be considered for protecting bridge abutments whenever there is a significant amount of flow on the flood plain that must return to the main channel at the bridge.

3. Channel Improvements. A wide variety of countermeasures are available for stabilizing and controlling flow patterns in streams. References 6, 7, 18, 19, 20, 21, 22, 35 and 36 contain methods for designing channel improvements.

   a. Countermeasures for aggrading streams include:

      - contracting the waterway upstream and through the bridge to cause it to scour,

      - construction of upstream dams to create sedimentation basins,

      - periodic cleaning of the channel, and

      - raising the grade of the bridge and its approaches.

   b. Countermeasures for degrading streams include the construction of sills and the strengthening of foundations as discussed below.

   c. Countermeasures for controlling lateral movement of a stream due to stream meanders include placement of dikes along the stream banks to redirect the flow through the bridge along a favorable path that minimizes the angle of attack of the current on the bridge foundations.

   HEC No. 20 (6) addresses this type of countermeasure in detail. Another useful reference is Transportation Research Record 950 (36).

4. Structural Scour Countermeasures. The use of structural designs to underpin existing foundations is discussed in the AASHTO Manual for Bridge Maintenance (35). While structural measures may be more costly, they generally provide more positive protection against scour than countermeasures such as riprap.

5. Constructing Sills or Drop Structures. The use of sills and drop structures at bridges to stabilize the stream bed and counteract the affects of degradation is discussed in FHWA publications (6) and (7).

6. Constructing Relief Bridges or Extra Spans on the Main Bridge. Providing additional waterway to relieve existing flow conditions is essentially a design problem and the guidance in Chapters 3 and 4 are applicable to its implementation. In some locations with very unstable banks, the addition of spans may be more cost effective than attempting to stabilize the channel slopes in the vicinity of the bridge.

SUMMARY - The foregoing discussion of countermeasures presents a wide variety of concepts and approaches for addressing scour problems at bridges. The Interdisciplinary Scour Team needs to collect and evaluate information about the behavior of streams and flood flow patterns through bridges so that the most appropriate countermeasures are selected for the particular set of site conditions under study. The FHWA publication "Countermeasures for Hydraulic Problems at Bridges (Volume 2, Case Histories)," is recommended as a guide for reviewing the performance of the countermeasures discussed above.

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APPENDICES

A. Alternate Scour Analysis Method

B. Equations for Abutment Scour

C. Computation of Guide Bank (Spur Dike) Length

D. North Carolina Scour Evaluation Procedures

E. FHWA 1988 Recording Guide for Structural Inventory

F. Scour Analysis for Great Pee Dee River, South Carolina

G. Scour Detection Equipment

APPENDIX A

ALTERNATE SCOUR ANALYSIS METHOD

This method has merit when contraction scour, discussed in Step 3 of Chapter 3 is significant. It is based on the premise that the contraction and local scour components are inter-dependent. As such, the local scour estimated with this method is determined based on the expected changes in the hydraulic variables and parameters due to contraction scour. Through an interactive process, the contraction scour and channel hydraulics are brought into balance before local scour is computed. The general approach for this method is:

- o estimate the natural channel's hydraulics for a fixed bed condition based on existing site conditions;
- o estimate the expected profile and plan form changes based on the procedures in this manual and any historic data;
- o adjust the natural channel's hydraulics based on the expected profile and plan form changes;
- o select a trial bridge opening and compute the bridge hydraulics;
- o estimate contraction scour;
- o revise the natural channel's geometry to reflect the contraction scour and then again revise the channel's hydraulics. Repeat this iteration until there is no significant change in either the revised channel hydraulics or bed elevation changes (a significant change would be 5 percent or greater variation in velocity, flow depth, or bed elevation);
- o using the foregoing revised bridge and channel hydraulic variables and parameters obtained considering the contraction scour, calculate the local scour; and
- o extend the local scour depths below the predicted contraction scour depths in order to obtain the total scour.

APPENDIX B

EQUATIONS FOR ABUTMENT SCOUR

In this appendix, scour at abutments is divided into its various cases and equations are given for each case (See Table B.1 and Figures B.1 to B.3). These equations are given for the designer who may want to calculate the potential scour depths using additional equations than the one recommended in the report. No single equation is supplied for a given situation when more than one equation is applicable, because with the lack of field data for verification, it is not known which equation is best. It is **suggested that the designer determine what case fits the design situation and then use all equations that apply to the case.**

COMMENTS ON THE SEVEN ABUTMENT SCOUR CASES.

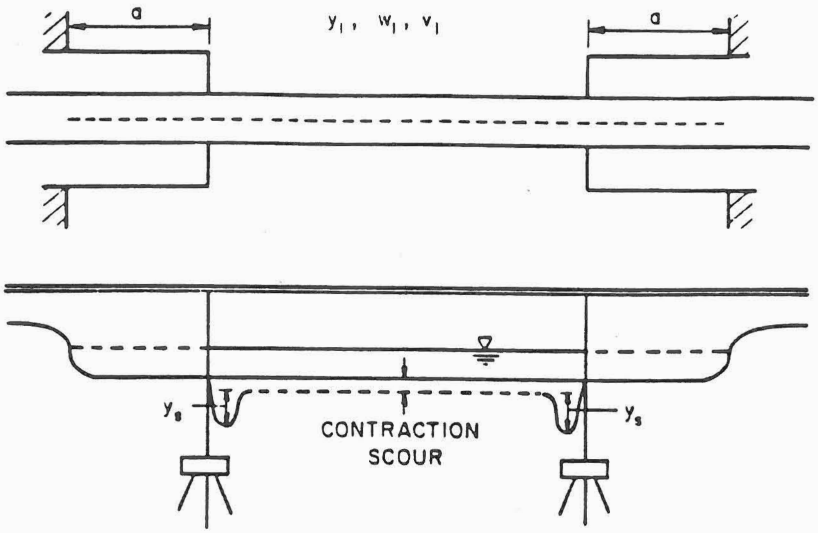
1. Equations for these cases (except for Case 6) are based on laboratory studies with little or no field data.
2. The factor $a/y_1 = 25$ as a limit for Cases 1-5 is rather arbitrary, but it is not practical to assume that scour depth, y_s, would continue to increase with an increase in abutment length "a".
3. There are two general shapes for abutments. These are vertical wall abutments with wing walls and spill-through abutments. Depth of scour is about double for vertical wall abutments as compared with spill-through abutments.
4. Maximum Depth of Scour.
   For live-bed scour with a dune bed configuration, the maximum depth of scour is about 30 percent greater than equilibrium scour depth given by Liu, et al's (1) equations (Equations 1 and 2). Therefore, the values of scour that are calculated for these equations should be increased by 30 percent when the bed form is dunes upstream of the bridge. The reason for this is that the research that was used for determining scour depth for the live-bed scour case was run with a dune bed and equilibrium scour was measured.

   For clear-water scour the maximum depth of scour is about10 percent greater than live-bed scour. However, there is no need to increase the scour depths because the equations predict the maximum scour.

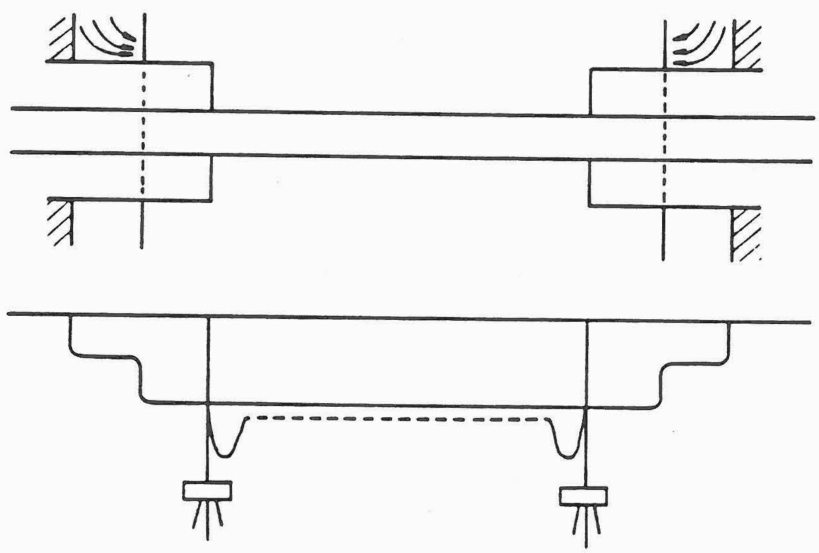
<u>IT IS MOST IMPORTANT THAT THE COMMENTARY ON EACH OF THE EQUATIONS BE READ AND UNDERSTOOD PRIOR TO ATTEMPTING TO USE THE EQUATIONS FOR DESIGN PURPOSES.</u> Engineering judgment must be used to select the depth of foundations. The designer should take into consideration the potential cost of repairs to an abutment and danger to the travelling public in selecting scour depths or in using design measures such as spur dikes and rock riprap.

CASE	ABUTMENT LOCATION	OVERBANK FLOW	VALUE OF a/y_1	BED LOAD CONDITION	ABUTMENT TYPE	EQUATION NUMBER
1	Projects into Channel	No	$a/y_1 < 25$	Live Bed	Vertical Wall	2, 3
					Spill-Through	1, 3
				Clear Water	Vertical Wall	4, 5
					Spill-Through	4, 5
2	Projects into Channel	Yes	$a/y_1 < 25$	Live Bed	Vertical Wall	3, 7
				Clear Water	Vertical Wall	4, 7
3	Set Back from Main Channel	Yes	$a/y_1 < 25$	Clear Water	Vertical Wall	4
4	Relief on Bridge Floodplain	Yes	$a/y_1 < 25$	Clear Water	Vertical Wall	4
5	Set at Edge of Main Channel	Yes	$a/y_1 < 25$	Live Bed	Vertical Wall	7
6	Not Designated	Yes	$a/y_1 > 25$	Not Designated	Spill-Through	8
7	Skewed to Stream	--	--	--	--	--

TABLE B.1 ABUTMENT SCOUR CASES

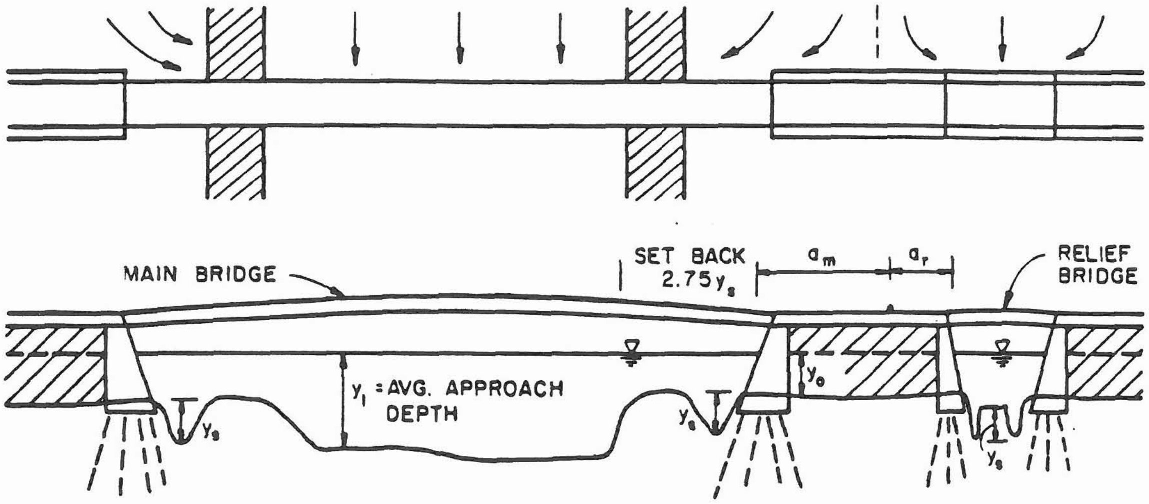


CASE 1 ABUTMENTS PROJECT INTO CHANNEL, NO OVERBANK FLOW

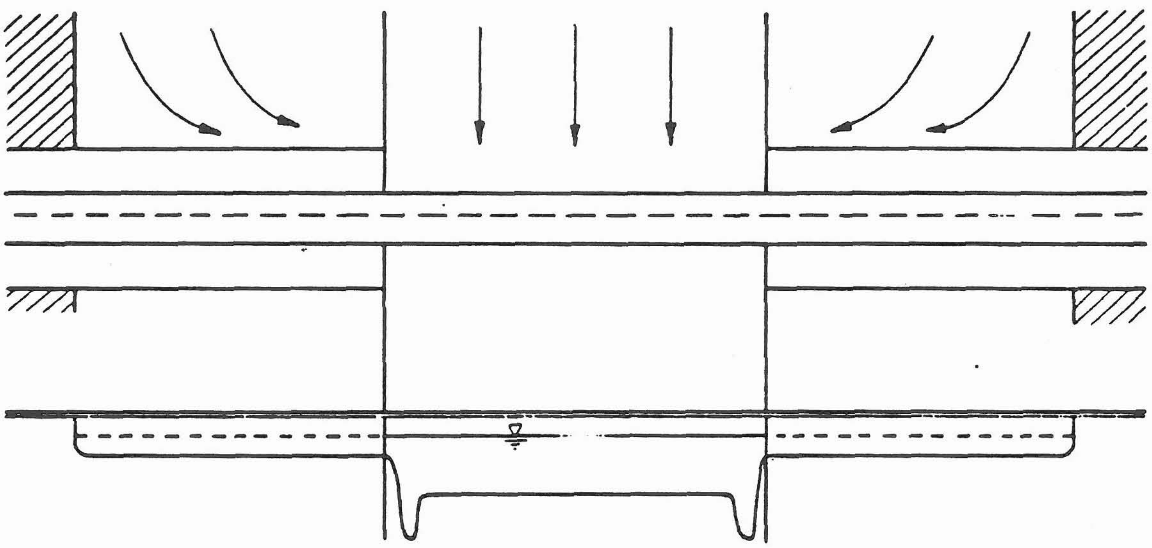


CASE 2 ABUTMENTS PROJECT INTO CHANNEL, OVERBANK FLOW

FIGURE B.1 ABUTMENT SCOUR CASES 1 AND 2.

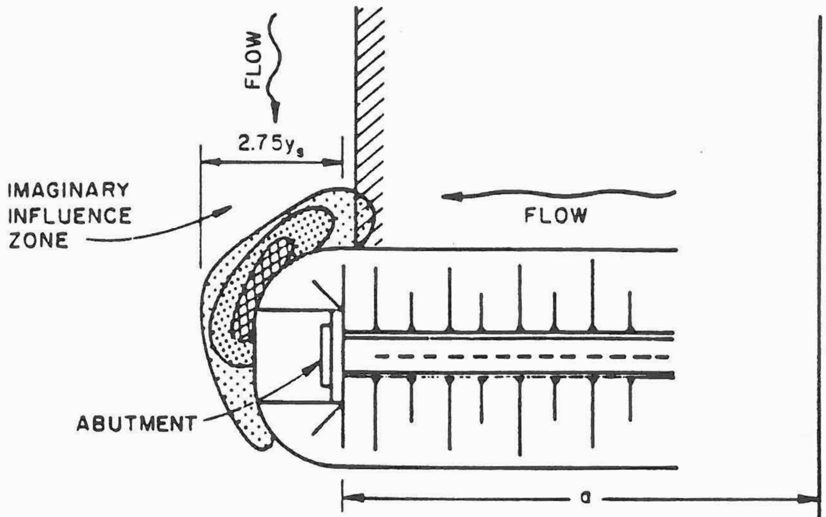


CASE 3 ABUTMENT SETBACK FROM THE CHANNEL MORE THAN 2.75 y_s.
CASE 4 RELIEF BRIDGE

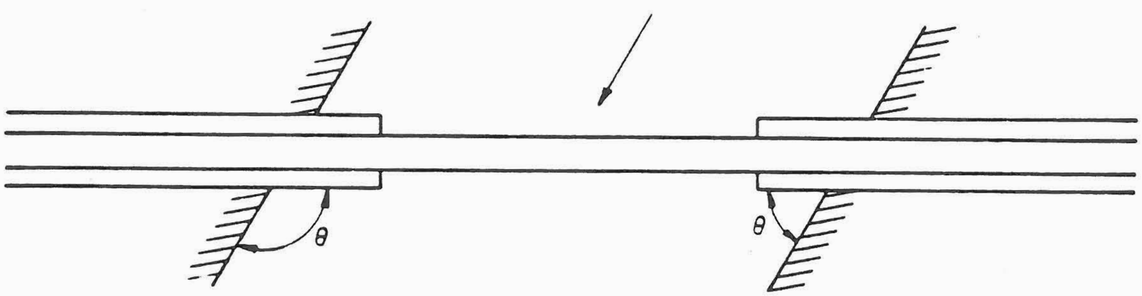


CASE 5 ABUTMENT SET AT EDGE OF CHANNEL, OVERBANK FLOW

FIGURE B.2 ABUTMENT SCOUR CASES 3, 4 AND 5.



CASE 6 RATIO OF ABUTMENT LENGTH, a, TO FLOW DEPTH, y_1, > 25



CASE 7 ABUTMENT SET AT AN ANGLE "θ" TO THE FLOW

FIGURE B.3 ABUTMENT SCOUR CASES 6 AND 7.

SCOUR AT ABUTMENTS

CASE 1 ABUTMENTS PROJECT INTO CHANNEL, NO OVERBANK FLOW

This Case is illustrated in Figure B.4.

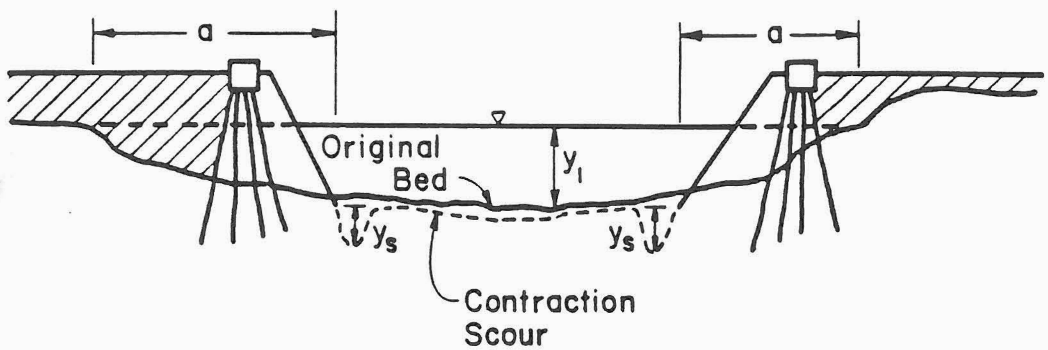


FIGURE B.4 DEFINITION SKETCH FOR CASE 1 ABUTMENT SCOUR

Six equations are given for this case. Two by Liu, et al (1), two by Laursen (2) and two by Froehlich (3).

LIU, ET AL'S CASE 1 EQUATIONS

Equation 1: Liu et al's (1) equation for live-bed scour at a spill through abutment.

According to the 1961 studies of Liu, et al., (1) the equilibrium scour depth for local live-bed scour in sand at a stable spill through slope with no overbank flow when the flow is subcritical is determined by Equation 1.

$$\frac{y_s}{y_1} = 1.1 \left(\frac{a}{y_1}\right)^{0.40} Fr_1^{0.33} \qquad (1)$$

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 length (measured at the top of the water surface and normal to the side of the channel from where the top of the design flood hits the bank to the outer edge of the abutment)

Fr_1 = upstream Froude number

$$Fr_1 = \frac{V_1}{(g\, y_1)^{0.5}}$$

Equation 2: Lui, et al's (1) equation for live bed scour at a vertical wall abutment.

If the abutment terminates at a vertical wall and the wall on the upstream side is also vertical, then the scour hole in sand calculated by equation 1 nearly doubles (Liu, et al, (1) and Gill, (4).

Liu, et al's, (1) equation for the equilibrium scour depth for local live-bed scour in sand at a vertical wall abutment with no overbank flow when the flow is subcritical is determined by Equation 2.

$$\frac{y_s}{y_1} = 2.15 \left(\frac{a}{y_1}\right)^{0.40} Fr_1^{0.33} \qquad (2)$$

LAURSEN'S CASE 1 EQUATIONS

Equation 3: Laursen's (2) equation for live bed scour at a vertical wall abutment.

More recently, Laursen (1980) suggested two relationships for scour at vertical wall abutments for Case 1. One for live-bed scour and another for clear-water scour depending on the relative magnitude of the bed shear stresses to the critical shear stress for the bed material of the stream. For live-bed scour ($\tau_1 > \tau_c$), use equation 3. For other abutment types, see note 2 below.

$$\frac{a}{y_1} = 2.75 \frac{y_s}{y_1} \left[\left(\frac{y_s}{11.5\, y_1} + 1 \right)^{1.7} - 1 \right] \qquad (3)$$

Simplified form:

$$\frac{y_s}{y_1} = 1.5 \left(\frac{a}{y_1} \right)^{0.48}$$

Equation 4: Laursen's (2) equation for clear water scour ($\tau_1 < \tau_c$) at a vertical wall abutment.

$$\frac{a}{y_1} = 2.75 \frac{y_s}{y_1} \left[\frac{\left(\frac{y_s}{11.5\, y_1} + 1 \right)^{\frac{7}{6}}}{\left(\frac{\tau_1}{\tau_c} \right)^{0.5}} - 1 \right] \qquad (4)$$

τ_1 = shear stress on the bed upstream
τ_c = critical shear stress of the D_{50} of the upstream bed material. The value of τ_c can be obtained from Figure A.5.

Laursen's (1) scour depths for other abutment shapes,

Scour values given by Laursen's equations are for vertical wall abutments. He suggests the following multiplying factors for other abutment types for small encroachment lengths:

Abutment Type	Multiplying Factor
45 degree Wing Wall	0.90
Spill-Through	0.80

FROEHLICH'S CASE 1 EQUATIONS

1. Live bed scour at an abutment.

Froehlich's (3) equation for this case is given in Chapter 4 of the report. It is the recommended equation for all seven cases.

2. Clear-water scour at an abutment.

Froehlich (3) using dimensional analysis and multiple regression analysis of 164 clear-water scour measurements in laboratory flumes developed an equation for clear water scour. It is as follows:

$$\frac{y_s}{y_1} = 0.78\, k_1\, k_2\, \left(\frac{a'}{y_1}\right)^{0.63} Fr_e^{1.16} \left(\frac{y_1}{D_{50}}\right)^{0.43} G^{-1.87} + 1 \qquad (5)$$

Where:

K_1 = coefficient for abutment shape

DESCRIPTION	k_1
VERTICAL ABUTMENT	1.00
VERTICAL ABUTMENT WITH WING WALLS	0.82
SPILL THROUGH ABUTMENT	0.55

K_2 = coefficient for angle of embankment to flow

$$K_2 = (\theta/90)^{0.13}$$

$\theta < 90°$ if embankment points downstream
$\theta > 90°$ if embankment points upstream

a' = length of abutment projected normal to flow

$$a' = A_e/y_1$$

A_e = is the flow area of the approach cross-section obstructed by the embankment.

Fr_e = Froude number of approach flow upstream of the abutment.
= $V_e/(gy_1)^{0.5}$

V_e = Q_e/A_e

Q_e = flow obstructed by the abutment and approach embankment.

y_1 = depth of flow at the abutment

G = geometric standard deviation of bed material
$G = (D_{84}/D_{16})^{0.5}$

D_{84}, D_{16} = grain sizes of the bed material. The subscript indicates the percent finer at which the grain size is determined.

The constant term unity (+1) in Froehlich's equations is a safety factor that makes the equation predict a scour depth larger than any of the measured scour depths in the experiments. This safety factor should be used in design.

In using Froehlich's clear water scour equation the D_{50} of the bed and foundation material should be equal to or larger than 0.25 ft and G should be equal to or larger than 1.5.

COMMENTS ON CASE 1 EQUATIONS

1. These equations are limited to cases where $a/y_1 < 25$. For $a/y_1 > 25$ go to Case 6.

2. Laursen's (2) equations are based on sediment transport relations. **THEY GIVE MAXIMUM SCOUR AND INCLUDE CONTRACTION SCOUR. FOR THESE EQUATIONS, DO NOT ADD CONTRACTION SCOUR TO OBTAIN TOTAL SCOUR AT THE ABUTMENT. FOR METHOD 1 ANALYSES LOCAL ABUTMENT SCOUR BELOW THE CONTRACTION SCOUR LINE IS EQUAL TO LOCAL ABUTMENT SCOUR -CONTRACTION SCOUR.**

4. Liu, et al's (1) equations are for a dune bed configuration. Therefore, for a dune bed configuration in the natural stream the scour given by their equations are for equilibrium scour and for maximum scour the values must be increased by 30 percent. For plane bed and antidune flow there are no equations given, but it is suggested that Liu, et al's equations could be used as given unless the antidunes would be occurring at the abutment. If antidunes exist or there is the possibility that they might break at the abutment then the scour depth given by their equation be increased by 20 percent.

5. **IT IS RECOMMENDED THAT THE MAXIMUM VALUE OF THE y_s/Y_1 RATIO IN LAURSEN'S EQUATION BE TAKEN AS 4 BECAUSE HIS EQUATIONS ARE OPEN ENDED AND FIELD DATA FOR CASE 6 DID NOT EXCEED 4 y_1.**

6. Laursen's equations require trial and error solution. Nomographs developed by Chang (5) are given in Figure A.5. Note that the equations have been truncated at a value of y_s/y equal to 4.

7. These equations were developed from laboratory and theoretical studies with very little field data. The values obtained should be evaluated very carefully.

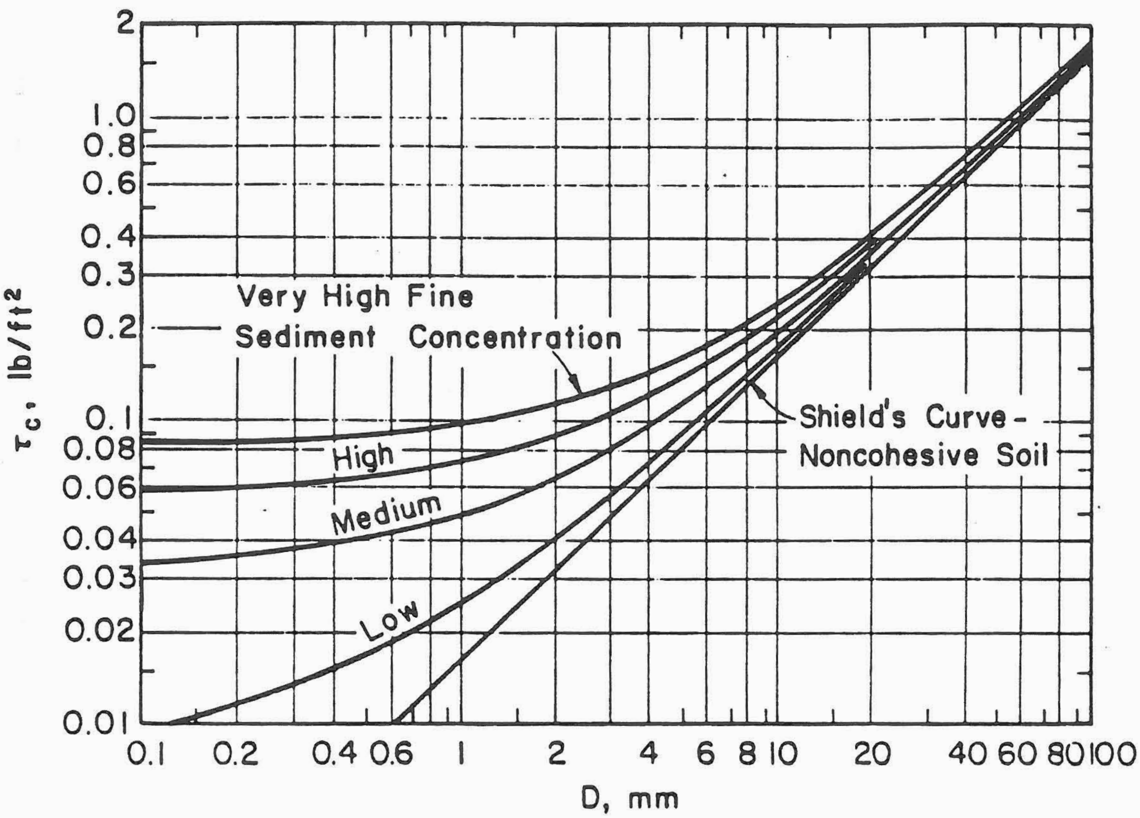
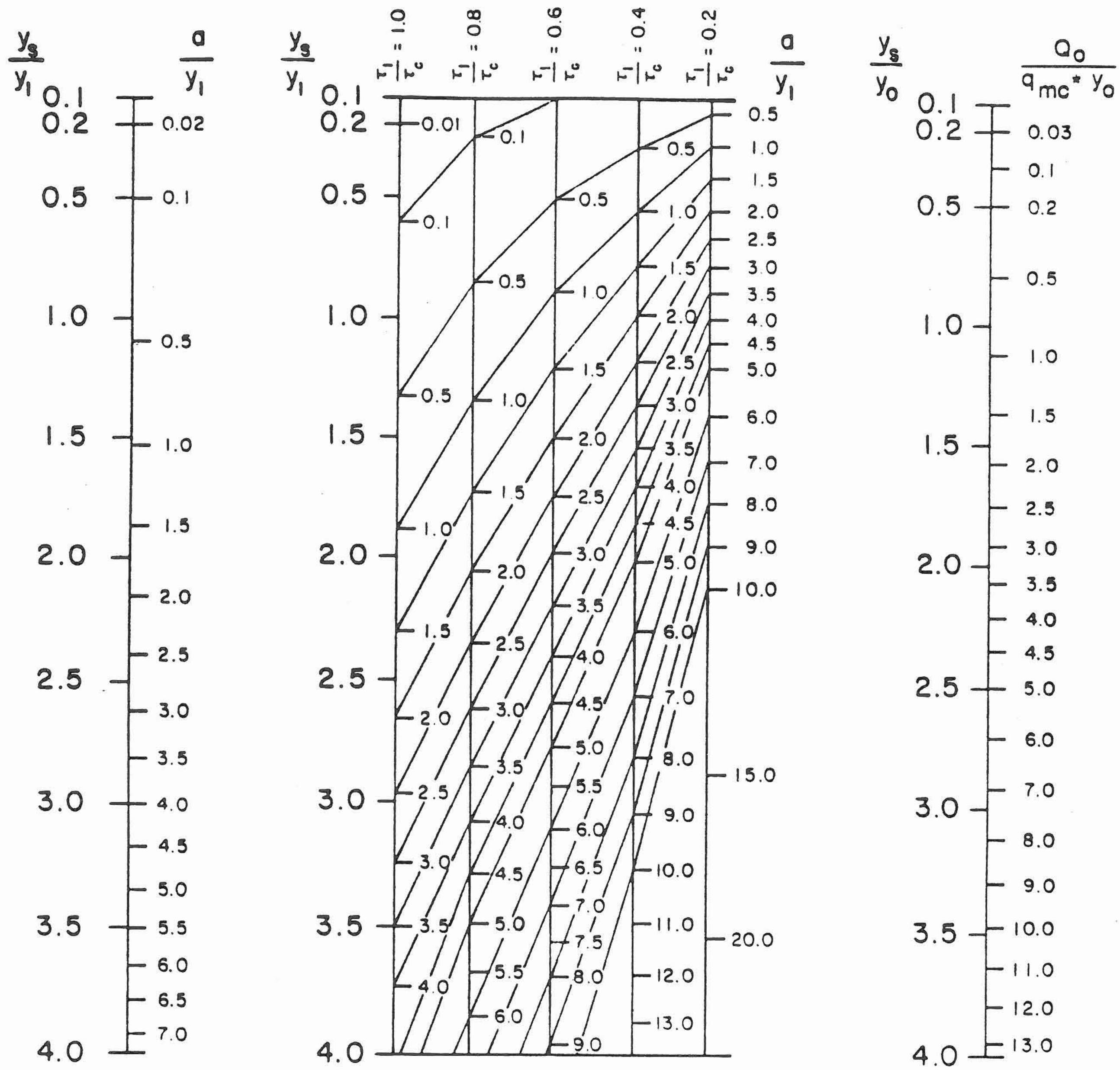


FIGURE B.5 CRITICAL SHEAR STRESS AS A FUNCTION OF BED MATERIAL SIZE AND SUSPENDED FINE SEDIMENT.



Equation 3 Equation 4 Equation 7

FIGURE B.6 NOMOGRAPHS FOR LAURSEN'S ABUTMENT SCOUR EQUATIONS

CASE 2 ABUTMENT PROJECTS INTO THE CHANNEL, OVERBANK FLOW

No bed material is transported in the overbank area and $a/y_1 < 25$. This case is illustrated in Figure B.7.

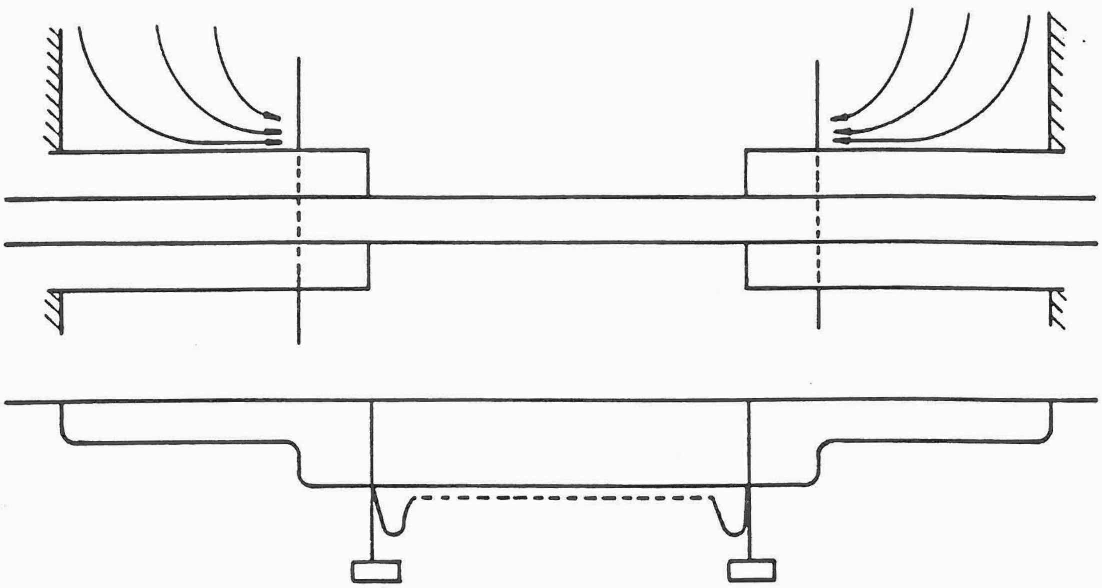


FIGURE B.7 BRIDGE ABUTMENT IN MAIN CHANNEL AND OVERBANK FLOW

Laursen's equation 3 or 4 should be used to calculate the scour depth with abutment length a determined by equation 6.

Laursen's equation 7 can also be used for this case with the appropriate selection of variables.

Live bed scour ($\tau_1 > \tau_c$) use equations 3 and 7.

Clear water scour ($\tau_1 < \tau_c$) use equations 4 and 7.

$$a = \frac{Q_o}{V_1 y_1} \tag{6}$$

τ_1 = The shear stress in the main channel.
τ_c = The critical shear stress for D_{50} of the bed material in the main channel. The value can be determined from Figure A.5.
Q_o = Flow obstructed by abutment and bridge approach.
y_1 = Average upstream flow depth in the main channel.
V_1 = Average velocity in the main channel.

It is assumed that there is no bed material transported by the overbank flow or that the transport is so small that it will not decrease abutment scour.

CASE 3 ABUTMENT IS SET BACK FROM MAIN CHANNEL MORE THAN 2.75 y_s

There is overbank flow with no bed material transport (clear water scour). Figure B.8 illustrates this case.

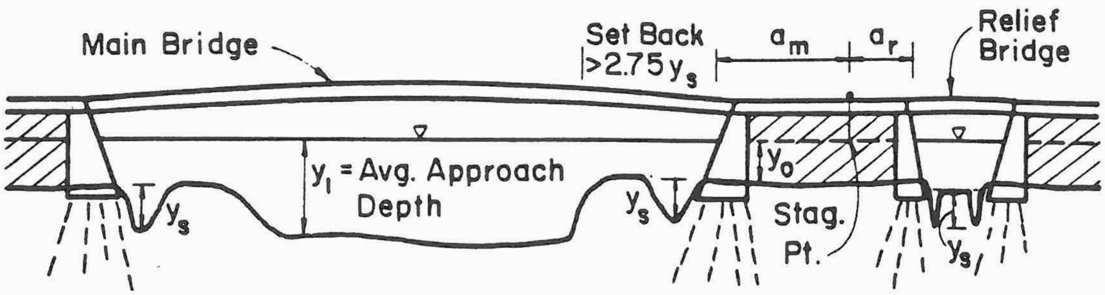


FIGURE B.8 BRIDGE ABUTMENT SET BACK FROM MAIN CHANNEL BANK AND RELIEF BRIDGE

With no bed material transport in overbank flow, scour at a bridge abutment, set back more than 2.75 times the scour depth from the main channel bank line, can be calculated using equation 4 from Laursen (2) with:

τ_o = Shear stress on the overbank area upstream of the abutment.

τ_c = Critical shear stress of material in overbank area. Can be determined from Figure B.5.

<u>Notes.</u>

1. Values of the critical shear stress, τ_c, can be determined from Figure A.5 using the D_{50} of the bed material of the cross-section under consideration. Alternately, they can be calculated using the Shield's relation for beginning of motion given in Highways in the River Environment by Richardson et al (6).
2. When there are relief bridges the a in equation 4 is taken as a_m.
3. The lateral extent of the scour hole is nearly always determinable from the depth of scour and the natural angle of repose of the bed material. Laursen (2) suggested that the width of the scour hole is $2.75y_s$.
4. With no bed material transported in the overbank flow, but the shear stress in the overbank area larger than the critical shear stress ($\tau_o < \tau_c$) then use equation 4 with the shear stress ratio set equal to 1. This can occur if the overland flow is over grass covered land.
5. If there is substantial bed material transport in the overland flow (transport of enough material that in your judgment it could change the scour) then equation 3 can be used. But again engineering judgment is

requires. The equation to be answered is " will the sediment being transported in the overland flow be sufficient to change the scour depth?"

CASE 4 ABUTMENT SCOUR AT RELIEF BRIDGE

Scour depth for a relief bridge on the overbank flow area having no bed material transport is calculated using equation 4 where y_1 is average flow depth on the flood plain. If on the flood plain $\tau_o > \tau_c$, but there is no sediment transport or the sediment transported in the judgement of the engineer will not effect the scour, use equation 4 with the shear ratio set to 1.

Use a_r for a in the equation. Draw stream lines or field observations to delineate where the separation point is for the flow going to the main channel and to the relief bridge. (See Figure B.8)

CASE 5 ABUTMENT SET AT EDGE OF CHANNEL

The case of scour around a vertical wall abutment set right at the edge of the main channel as sketched in Figure B.9 can be calculated with equation 7 proposed by Laursen (2) when $\tau_o < \tau_c$ on the flood plain or there is no appreciable bed material transport by the overbank flow..

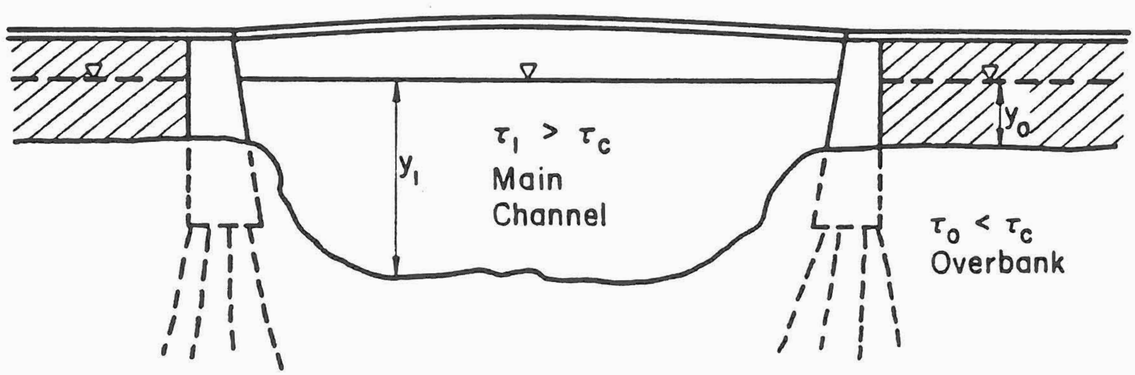


FIGURE B.9 ABUTMENT SET AT EDGE OF MAIN CHANNEL

$$\frac{Q_o}{q_{mc}\, y_0} = 2.75\, \frac{y_s}{y_0}\left[\left(\frac{y_s}{4.1 y_0} + 1\right)^{\frac{7}{6}} - 1\right] \qquad (7)$$

Where:

Q_o = overbank flow discharge
q_{mc} = the unit discharge in the main channel, Q_w/W
  Q_w = discharge in main channel
  W = width of the main channel
y_o = overbank flow depth

If there is no overbank flow for this case then there is no appreciable scour.

COMPARISON OF SCOUR DEPTHS CALCULATED BY EQUATIONS 3, 4 AND 7.

Values of calculated scour depth by equations 3, 4 an 7 are given in Figure B.10.

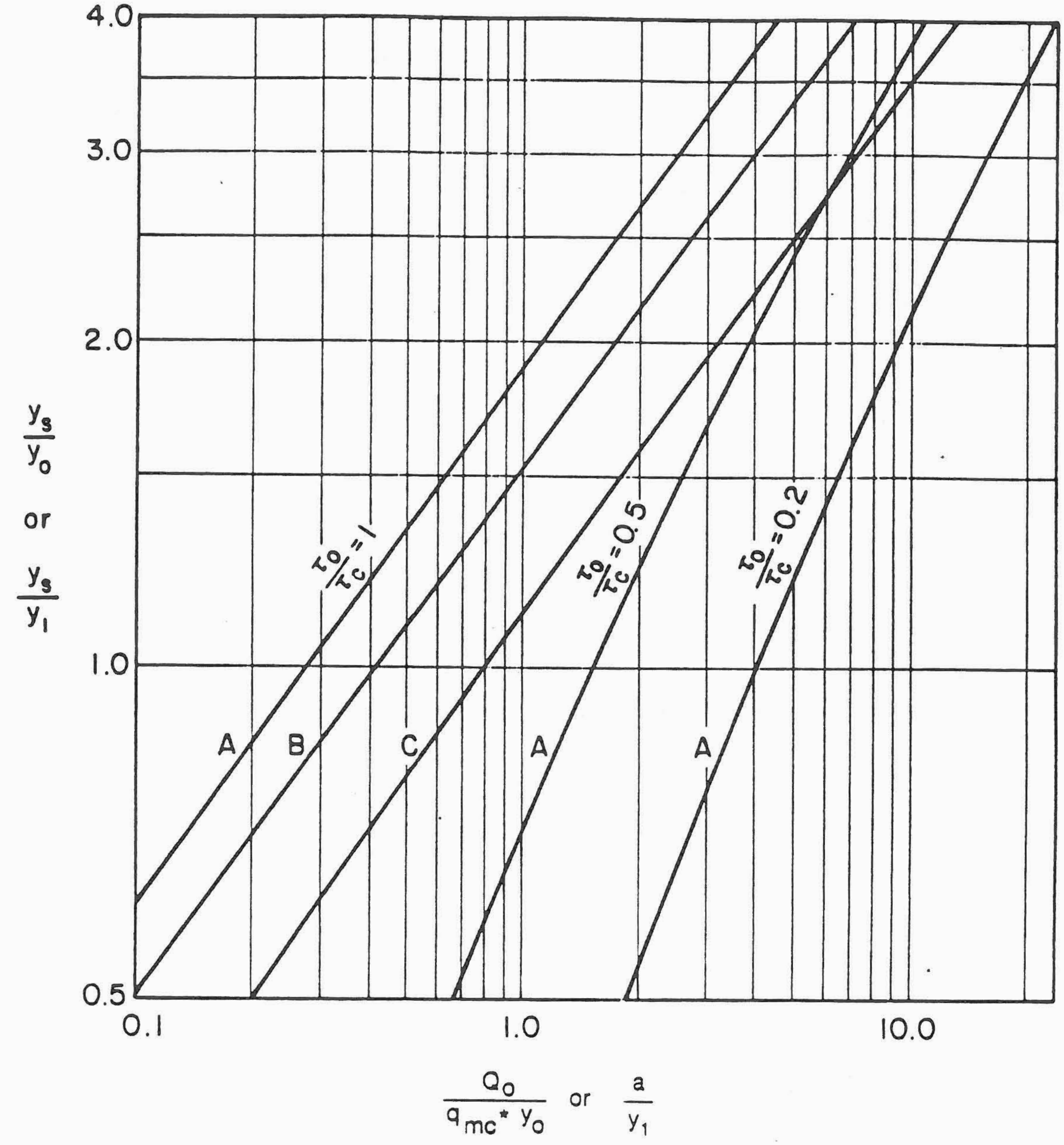


FIGURE B.10 VALUES OF CALCULATED SCOUR DEPTH FROM EQUATIONS 3, 4 and 7. (A is Eq. 4, B is Eq. 3 and C is Eq. 7)

CASE 6 SCOUR AT ABUTMENTS WHEN $a/y_1 > 25$

Field data for scour at abutments for various size streams are scarce, but data collected at rock dikes on the Mississippi indicate the equilibrium scour depth for large a/y_1 values can be estimated by equation 8:

$$\frac{y_s}{y_1} \doteq 4\, Fr_1^{0.33} \qquad (8)$$

The data are scattered, primarily because equilibrium depths were not measured. Dunes as large as 20 to 60 feet high move down the Mississippi and associated time for dune movement is very large in comparison to time required to form live-bed local scour holes. Nevertheless, it is believed that these data represent the limit in scale for scour depths as compared to laboratory data and enables useful extrapolation of laboratory studies to field installations.

Accordingly, it is recommended that equations 1 through 7 be applied for abutments with $0 < a/y_1 < 25$ and equation 8 be used for $a/y_1 > 25$.

CASE 7 ABUTMENTS SKEWED TO THE STREAM

With skewed crossings, the approach embankment that is angled downstream has the depth of scour reduced because of the streamlining effect. Conversely, the approach embankment which is angled upstream will have a deeper scour hole. The calculated scour depth should be adjusted in accordance with the curve of Figure A.11 which is patterned after Ahmad (7).

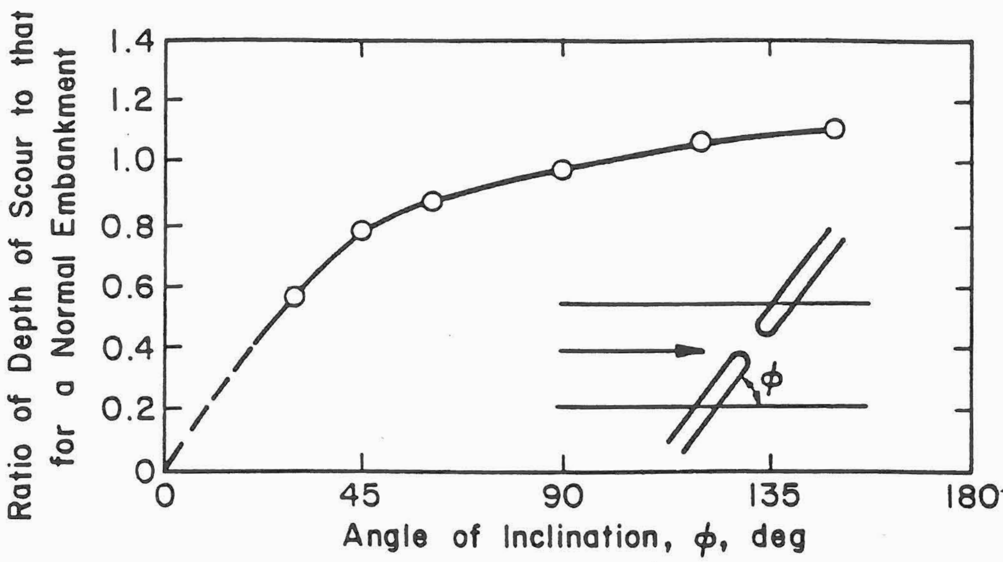


FIGURE B.11. SCOUR ESTIMATE ADJUSTMENT FOR SKEW.

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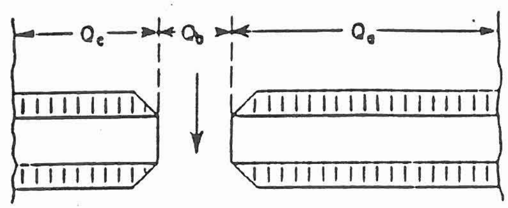
APPENDIX C
SCOUR AT ABUTMENTS
(Computation of Length of Spur Dike)

1. Determine Discharge Upstream of Bridge for Approach Section



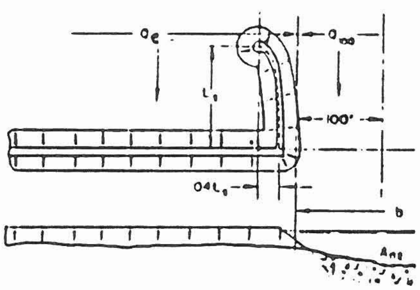
abutment c abutment a

1a. Discharge near abutment a (Q_a) ________

1b. Discharge thru bridge (Q_b) ________

1c. Discharge near abutment c (Q_c) ________

2. Calculate the discharge in the 100 ft. next to the abutment. (This is a portion of Q_b.)



$(Q_{100})_a$ ________

$(Q_{100})_c$ ________

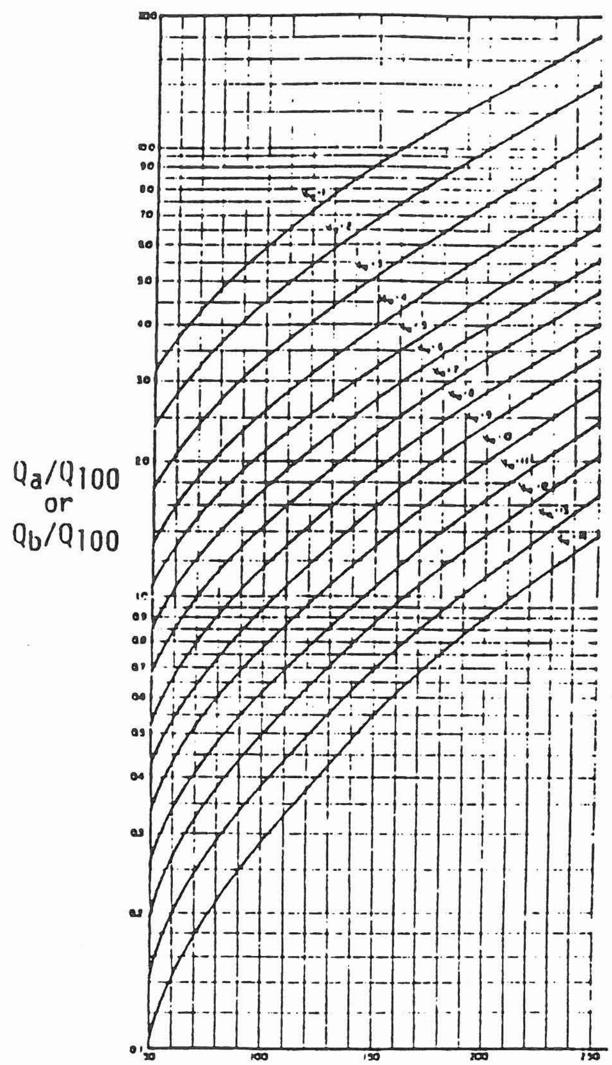
3. Calculate ratio

$Q_a/(Q_{100})_a$ ________

$Q_c/(Q_{100})_c$ ________

4. Calculate Average Velocity in bridge opening (V_{n2}) ________

5. Find Length of Spur Dike for both abutments



Length of Spur Dike needed for:

abutment a ________

abutment c ________

APPENDIX D

NORTH CAROLINA

DEPARTMENT OF TRANSPORTATION

DIVISION OF HIGHWAYS

STRUCTURE SCOUR EVALUATION PLAN

FOR EXISTING STRUCTURES

JUNE 1990

INTERDISCIPLINARY SCOUR WORK GROUP

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Section 1: INTRODUCTION

This "Structure Scour Evaluation Plan For Existing Structures" sets forth North Carolina's Policy for evaluating existing structures for vulnerability to scour and implementing appropriate scour countermeasures. Procedures for evaluating scour at existing structures will be based on FHWA Technical Advisory T 5140.20 entitled "Interim Procedures for Evaluating Scour at Bridges" dated November 7, 1988.

The Scour Evaluation Program Select Committee was formed by the State Highway Administrator to develop and implement a Scour Evaluation Program For Existing Structures. The Interdisciplinary Scour Work Group is advisory to the Scour Evaluation Program Select Committee and received the task to develop an approach to evaluate scour at existing structures in North Carolina.

Scour evaluation is an engineering assessment and prediction of bed form changes at a structure due to flooding and long term flow affects. This evaluation includes identification and assessment of steps that can be taken to eliminate or minimize potential damage to the structure.

A Scour Evaluation Process has been developed by an Interdisciplinary Scour Work Group of engineers representing Bridge Maintenance, Hydraulics, Foundations, Geotechnical, Structure Design, and FHWA. The Interdisciplinary Scour Work Group has developed a Structure Evaluation Plar which includes the following:

1. Initial Screening.
2. Priorities for making structure scour evaluations.
3. The Scope of the scour evaluations to be performed in the office and/or in the field.
4. Identify scour critical structures.
5. Identify alternative scour countermeasures which may serve to make a bridge less vulnerable.
6. Identify which countermeasure is most suitable and cost effective for a given situation.
7. Priorities for installing scour countermeasures.
8. Monitoring and inspection schedules for scour critical structures.

New bridges designed in accordance with Chapter 3 of FHWA Technical Advisory T 5140.20 will not require a Scour Evaluation by the interdisciplinary team. The Structure Design Unit will place a note on the Plans indicating that the bridge has been designed in accordance with FHWA Technical Advisory T 5140.20.

Section 2: SCOUR EVALUATION PROCESS

The following approach has been developed regarding implementation of a program to assess the vulnerability of existing structures to scour:

1. Initial Screening.
2. Prioritization for scour evaluation.
3. Office data collection.
4. Field data collection.
5. Scour calculation/evaluation.
6. Foundation stability analysis.
7. Scour Critical.
8. Structure/Scour monitoring and inspection schedule.
9. Countermeasure design.
10. Structure countermeasure prioritization.
11. Countermeasure implementation.

Figure 1 shows a flow chart of the Scour Evaluation Process.

SCOUR EVALUATION PROCESS

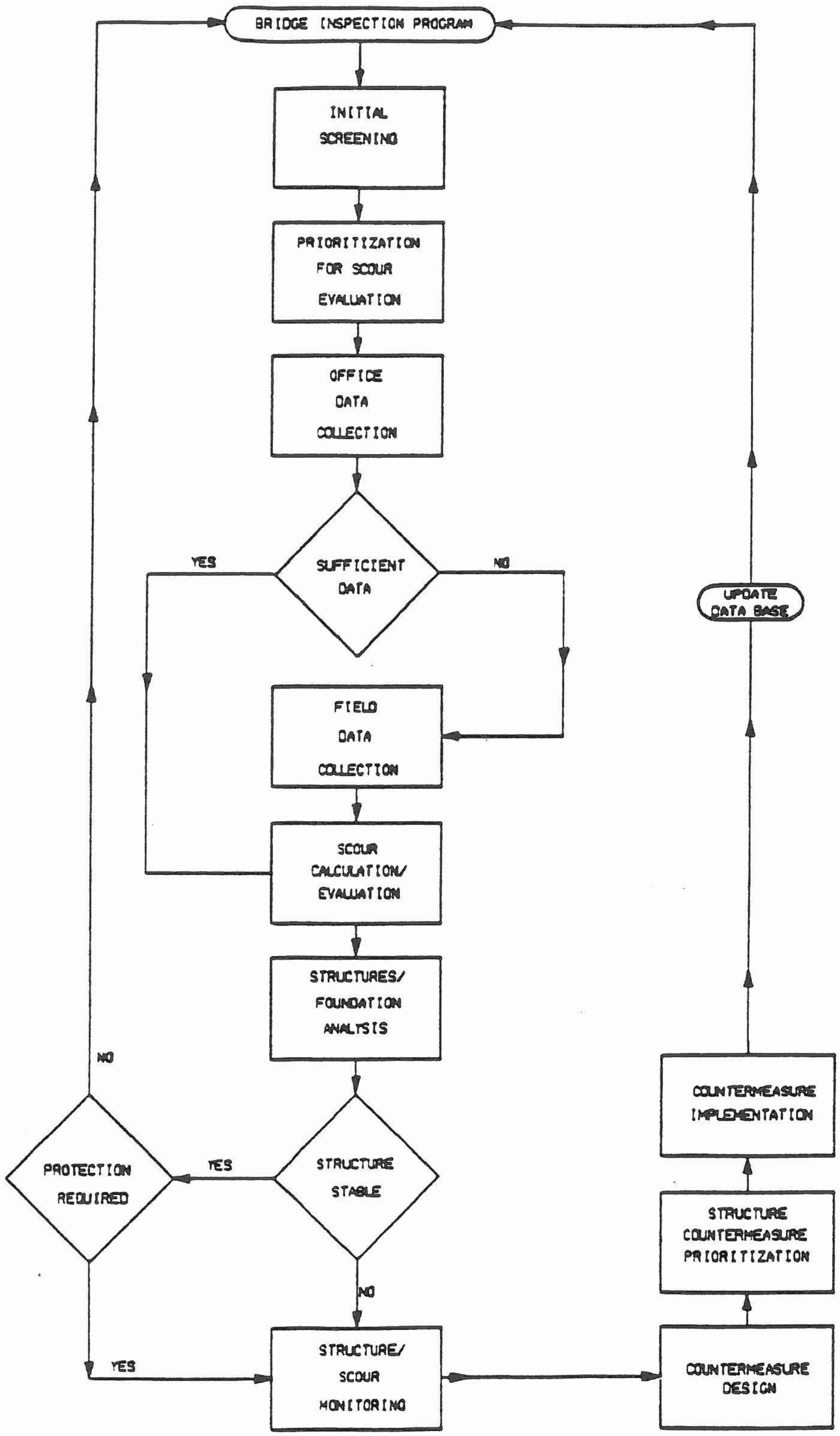


FIGURE 1

Section 3: INITIAL SCREENING

In April, 1990, North Carolina has approximately 16,900 State owned inventory structures of which approximately 14,600 are over water. Due to the massive number of structures over water, a method of prioritization for scour evaluation must be developed.

Table 1 shows data on existing structures in North Carolina which was considered in developing a Screening and Prioritization Process.

3.1 FHWA Requirements

By memorandum dated February 5, 1990, FHWA has established a requirement for the submission of biannual status reports covering bridge scour. See Figure 2 for the reporting format for this item (bridge scour) of the National Bridge Inspection Standards (NBIS). The status reports are due in Washington Headquarters each year by April 15 and November 15. FHWA has established a requirement that all screening to identify bridges which require scour analysis should be completed by March 31, 1991.

The FHWA memorandum suggests the screened structures be categorized into three categories:

A. Low Risk
B. Scour Susceptible
C. Unknown Foundations

The Initial Screening will prioritize structures for scour evaluation in accordance with the FHWA memorandum.

TABLE 1: DATA ON EXISTING STRUCTURES

April 1990

ITEM	STRUCTURES	BRIDGES	CULVERTS & PIPES (Greater Than 20 Feet)
INVENTORY OF STATE OWNED	16892	14147	2745
OVER WATER	14548	11803	2745
INTERSTATE (OVER WATER)	370	194	176
PRIMARY (OVER WATER)	2983	1923	1060
SECONDARY (OVER WATER)	11195	9686	1509
KNOWN SCOUR PROBLEMS	776	753	23
BUILT WITH STATE CONTRACT PROJECT NUMBER (OVER WATER)	2232	1514	718
BUILT BY BRIDGE MAINTENANCE, COUNTY, OR UNKNOWN(OVER WATER)	12316	10289	2027
INVENTORY OF MUNICIPAL OWNED	546	349	197
MUNICIPAL (OVER WATER)	455	264	191

NOTE: Unless otherwise noted on the individual table, the number of structures shown is for the North Carolina structure inventory which includes bridges less than 20 feet in length.

ATTACHMENT B

BRIDGE SCOUR

NUMBER OF BRIDGES

STATE________

DATE________

	FEDERAL AID SYSTEM	OFF SYSTEM	TOTAL NUMBER
OVER WATERWAYS			
SCREENED TOTAL			
A) LOW RISK			
B) SCOUR SUSCEPTIBLE			
C) UNKNOWN FOUNDATIONS			
D) CULVERTS & PIPES			
ANALYZED FOR SCOUR			
SCOUR CRITICAL			
COUNTERMEASURES PLANNED			
MONITORING PLANNED			

NOTE: CULVERTS & PIPES ARE INCLUDED IN THE TOTAL NUMBER OF STRUCTURES OVER WATERWAYS. D) CULVERTS & PIPES WERE ADDED SO THAT THE SCREENED TOTAL WOULD MATCH THE TOTAL OVER WATERWAY.

COMMENTS

3.2 Low Risk (Category A)

FHWA Memorandum of February 5, 1990, states "Many bridges can be screened as having reasonably risk-free or low-risk foundations, negating the need for further scour analysis." It is North Carolina's position that all bridges must be analyzed. However, placing some structures in a "low risk" classification is acceptable since it will provide for a more appropriate prioritization of potentially scour critical structures. The design of bridges in North Carolina since about 1976 has included detailed geological information with scour considered. A 1980 acceptance date was chosen to insure that bridges designed after 1976 are completed before being classified as low risk.

Bridges accepted (date built in the computer file) in 1980 or later and built with a State Contract Project number will be categorized as low risk for the following reasons:

1. North Carolina began obtaining geological information at Contract bridge sites in 1976. Scour was considered in the design phase when bottom of footing elevations and pile lengths were established. This scour consideration was based on the historical scour obtained from the geological information.

2. The only way to classify a bridge to be in this category using computer data is by date built which is the acceptance date.

3. Only bridges built with a State Contract Project number are included in the criteria for low risk because Bridge Maintenance has built bridges where scour was not considered.

Classifying these bridges as low risk does not indicate that they should not be evaluated for scour but postpones the time when they will be evaluated. Postponement of the time when these low risk bridges are evaluated allows other bridges which have a greater risk for damage from scour to be evaluated first. All bridges should be evaluated by the applicable parts of the Technical Advisory to be classified as not requiring further analysis for scour.

Bridges classified as low risk will be reclassified as scour susceptible if scour problems are detected.

3.3 Scour Susceptible (Category B)

Scour susceptible is defined in the Initial Screening Process as structures most likely to be susceptible to scour damage. Scour susceptible structures will require scour analysis.

The criteria for classifying structures as Scour Susceptible is as follows:

1. Structures with known scour problems or scour evaluation requested by a DOH Unit.
2. Bridge built with a State Contract Project Number before 1980.
3. Bridges built by Bridge Maintenance after 1965.

These structures can be generated from the computer data file.

3.3.1 Known Scour Problems

Structures that are identified as experiencing scour problems from site inspection or that have a history of scour problems as noted from maintenance records, experience, or bridge inspection records fall within this category.

An updated list of structures with known scour problems will be maintained. Any structure added to this list will also be screened into the scour susceptible category for further evaluation.

See Tables 2 and 3 for the number of structures with known scour problems as of April 1990.

3.3.2 Bridges built with a State Contract Project Number Before 1980

Bridges built with a State contract Project Number will generally have plans available, many will have hydraulic surveys, and some will have geologic information. Having this information available facilitates the scour evaluation.

A small number of bridges in this group will not have information on pile length or bottom of footing elevation. When initially evaluated, these bridges will be reclassified into the unknown foundation category.

See Tables 2 and 3 for the number of bridges built with a State Project Number.

3.3.3 Bridges built by Bridge Maintenance after 1965.

Bridges built by Bridge Maintenance after 1965 generally have foundation information available thru pile driving data.

The exact year Bridge Maintenance started keeping pile driving records is not precisely known; however, 1965 is the best estimate of the starting time.

There are some bridges built by Bridge Maintenance after 1965 that will not have this pile driving record. When initially evaluated, these bridges will be reclassified into the unknown foundation category.

See Tables 2 and 3 for the numbers of bridges built by Bridge Maintenance after 1965.

3.4 Unknown Foundations (Category C)

Data is not available in the computer file on bottom of footing elevation or pile length; therefore, a direct method of identifying bridges with unknown foundations is not available. Bridges with unknown foundations may also be scour susceptible; however, based on data not being available, scour evaluation will be delayed, unless the structure has been identified as a known scour problem structure.

All bridges which are not classified in the scour susceptible and low risk categories will be classified into the unknown foundations category.

See Tables 2 and 3 for bridges classified as having unknown foundations.

Bridges with unknown foundations will be coded on the Structure Inventory And Appraisal Sheet with a "6" in Item 113, Scour Critical Bridges. These bridges will be differentiated in the computer data file as "screened" unknown foundation structures from those structures for which a scour calculation/evaluation has not been made.

3.5 Non-Scour Critical (Category D)

Unless scour problems are identified, all culverts and pipes will be classified as non-scour critical structures requiring no evaluation due to the improbability of a catastrophic failure of a culvert or pipe from scour.

Any culvert or pipe which is discovered to have a scour problem will be added to the known scour problem list and be evaluated accordingly.

Culverts and pipes will be coded on the Structure Inventory And Appraisal Sheet with an "8" in Item 113, Scour Critical Bridges. These culverts and pipes will be differentiated in the computer data file as "screened" non-scour critical structures from those structures determined to be stable for the calculated scour above the top of footing condition.

See Tables 2 and 3 for the number of culverts and pipes classified as non-scour critical.

3.6 Conclusions

This Initial Screening Process allows postponement of scour evaluation for bridges with unknown foundations (where information cannot be obtained to evaluate the structure for scour) or low-risk bridges. It also allows culverts and pipes to be classified as non-scour critical with no evaluation required unless scour problems are detected. Structures classified as scour susceptible will be evaluated first. Any structure which is discovered to have a scour problem by the Bridge Inspection Program (either underwater or above water teams) will be added to the known scour problem list and evaluated accordingly.

Due to the potential safety risk to the traveling public which could result from the failure of a structure due to scour, all existing bridges over water in the bridge inventory will be eventually evaluated for scour.

See Figure 3 for "Screening, Prioritization And Coding for Scour Evaluation" Flow Chart. See Tables 2 and 3 for number of structures from Initial Screening.

Due to FHWA reporting requirements, the computer data file will be expanded in order to track the various components of the screening process. A computer program will be written to automate gathering data for FHWA reporting requirements.

TABLE 2: INITIAL SCREENING - STRUCTURES OVER WATER

ITEM		NO. OF STRUCTURES			CLASSIFICATION	CATEGORY
		FA	NFA	TOTAL		
BRIDGES BUILT 1980 AND LATER/W STATE CONTRACT PROJ. NO.		216	163	379	LOW RISK	A
KNOWN SCOUR PROBLEMS	BRIDGES	213	540	753	SCOUR SUSCEPTIBLE	B
	CULV. & PIPES	17	6	23		
	SUBTOTAL	230	546	776		
BUILT WITH STATE CONTRACT PROJECT NUMBER (BRIDGES) BEFORE 1980		632	578	1,210		
BUILT BY BRIDGE MAINTENANCE AFTER 1965 (BRIDGES)		92	631	723		
SUBTOTAL		954	1,755	2,709		
UNKNOWN FOUNDATIONS (BRIDGES)		1,598	7,140	8,738	UNKNOWN FOUNDATIONS	C
CULVERTS AND PIPES		1,409	1,313	2,722	NON-SCOUR CRITICAL	D
TOTALS		4,177	10,371	14,548		

TABLE 3: INTIAL SCREENING
STRUCTURES OVER WATER GREATER THAN 20 FEET

ITEM		NO. OF STRUCTURES			CLASSIFICATION	CATEGORY
		FA	NFA	TOTAL		
BRIDGES BUILT 1980 AND LATER /W STATE CONTRACT PROJ. NO		216	163	379	LOW RISK	A
KNOWN SCOUR PROBLEMS	BRIDGES	209	454	663	SCOUR SUSCEPTIBLE	B
	CULV. & PIPES	17	6	23		
	SUBTOTAL	226	460	686		
BUILT WITH STATE CONTRACT PROJECT NUMBER(BRIDGES) BEFORE 1980		632	578	1,210		
BUILT BY BRIDGE MAINTENANCE AFTER 1965 (BRIDGES)		91	619	710		
SUBTOTAL		949	1,657	2,606		
UNKNOWN FOUNDATIONS (BRIDGES)		1,533	5,874	7,407	UNKNOWN FOUNDATIONS	C
CULVERTS AND PIPES		1,409	1,313	2,722	NON-SCOUR CRITICAL	D
TOTALS		4,107	9,007	13,114		

Section 4: PRIORITIZATION FOR SCOUR EVALUATION

The Initial Screening process has defined broad categories of structures for scour evaluation. Since there are several thousand structures in some of the three categories: low-risk, scour susceptible, and unknown foundation; a priority order must be developed for scour evaluation of these structures.

4.1 Factors Considered for Prioritization Process

Structures will first be prioritized in broad areas which consider the following factors:

1. Structures with known scour problem or scour evaluation requested by a DOH Unit.
2. Interstate
3. ADT
4. Area of the State in which the structure is located.
5. Type of foundation.
6. Simple spans.
7. Latest inspection date.

4.1.1 Known Scour Problem or Scour Evaluation Requested

The top priority for scour evaluations will be those structures that are experiencing scour or that have a history of scour problems as identified from maintenance records, experience, bridge inspections records, etc.

An updated list of structures with known scour problems will be maintained. Any structure added to this list will also have top priority for "Scour Evaluation."

An equal prioritization criteria will be a Scour Evaluation Request from a DOH unit for a bridge over water that is proposed to be widened or rehabilitated. A bridge that is classified as Scour Critical will have an impact on decisions for:

1. Widening and/or rehabilitation vs. replacement.
2. Funding

A list of major structures in the Tidal Zone will be included in the priority as a Scour Evaluation Request.

Structures with a known scour problem or scour evaluation requested will be further prioritized by the following factors:

1. Interstate
2. ADT
3. Type Foundation
4. Simple spans
5. Latest inspection date

See Figure 3 for Screening, Prioritization and Coding Flow Chart.

See Appendix A for a partial listing of structures with known scour problems prioritized for scour evaluation.

4.1.2 Interstate

An initial assumption of the Interdisciplinary Scour Work Group was that the System Classification would be a prioritization factor. Concerns were expressed that lower ADT Primary System bridges would be evaluated before some Secondary System bridges with high ADT. The liability factor and disruption in the flow of traffic resulting from evaluating lower ADT Primary System bridges before high ADT Secondary System bridges was not considered acceptable if a failure due to scour should occur. Therefore, System Classification has been eliminated as a prioritization factor except for Interstate structures which were retained for the following reasons:

1. Interstate routes are part of the defense highway system.
2. The Interstate System is the highest order where a lane closure must be reported to the Washington Office of FHWA.
3. There are 25 Interstate bridges on the known scour problems list among the 194 Interstate bridges over water.
4. Interstate bridges are generally in the higher ADT categories.
5. Closure of an Interstate bridge would seriously disrupt Interstate Commerce due to lack of adequate detour and linkage routes for Interstate Commerce type traffic.

4.1.3 ADT

Average Daily Traffic (ADT) will be a prioritization factor because of the effects that a structure collapse would have on the safety of the traveling public and on the operation of the overall transportation system for the area or region.

ADT ranges less than or equal to 4,000 were obtained from "A LEVEL OF SERVICE SYSTEM FOR BRIDGE EVALUATION" developed for NCDOT by Dr. David W. Johnston of North Carolina State University for North Carolina in August 1983.

Initially ADT greater than 4,000 were placed in one group. In order to insure that structures with high ADT are evaluated before lower ADT structures, ADT ranges greater than 4,000 have been expanded.

ADT ranges for prioritization are as follows:

1. ADT > 50,000
2. ADT 25,001 - 50,000
3. ADT 10,001 - 25,000
4. ADT 4,001 - 10,000
5. ADT 2,001 -4,000
6. ADT 801 - 2,000
7. ADT less than or equal 800
8. Any other

See Tables 4 and 5 for Number of Structures By System and ADT ranges.

North Carolina pedestrian bridges over water will be included under the ADT prioritization range 8 (Any other).

TABLE 4: STRUCTURES OVER WATER BY ADT RANGES

ADT	STATE SYSTEM			ALL SYSTEMS	
	INTERSTATE	PRIMARY	SECONDARY	TOTAL	%
> 50,000	16	10	1	27	0.19
25,001 - 50,000	59	42	17	118	0.81
10,001 - 25,000	184	357	92	633	4.35
4,001 - 10,000	103	984	287	1374	9.45
2,001 - 4,000	2	692	427	1121	7.71
801 - 2,000	4	593	1331	1928	13.25
< 800	2	301	9,043	9346	64.24
TOTAL	370	2,979	11,198	14547	100
%	2.54	20.48	76.98	100	

TABLE 5: KNOWN SCOUR PROBLEMS BY ADT RANGES

ADT	STATE SYSTEM			ALL SYSTEMS	
	INTERSTATE	PRIMARY	SECONDARY	TOTAL	%
> 50,000	0	0	0	0	0
25,001 - 50,000	0	0	0	0	0
10,001 - 25,000	13	10	2	25	3.22
4,001 - 10,000	12	61	8	81	10.44
2,001 - 4,000	0	41	20	61	7.86
801 - 2,000	0	39	65	104	13.4
< 800	0	27	478	505	65.08
TOTAL	25	178	573	776	100
%	3.22	22.94	73.84	100	

4.1.4 Foundation Type

Structures will be prioritized by foundation type as follows:

1. Sill
2. Spread Footing
3. Pile Bent
4. Pile Footing
5. Other foundation types plus culverts and pipes.

A sill foundation is not a commonly recognized foundation type and consists of poured concrete or a timber member placed on the ground surface with posts placed on the sill to support the cap.

4.1.5 Location in State

North Carolina has three (3) geographical areas which are:

1. Mountains
2. Piedmont
3. Coastal Plain

An initial assumption of the Interdisciplinary Scour Work Group was that the Piedmont area would be the most susceptible to scour because naturally high stream velocities and occurrences of deep alluvial soils provide conditions conducive to foundation problems. The mountains were considered next in priority because of high stream velocities.

Analysis of the data for structures with known scour problems indicates there is not a "good fit" between the Piedmont area assumption and historical data for structures with known scour problems. Since data for structures with known scour problems is the only data available at this point in time, it was decided that location priority be established to parallel the data for the 776 structures with known scour problems.

Structures with known scour problems were tabulated by Major Rivers and Tributaries and by Highway Divisions. Analysis of the data indicated that neither of these factors could be correlatec in any pattern.

Table 6 shows structures with known scour problems tabulated by county in descending order by number of structures.

TABLE 6: NUMBER OF STRUCTURES WITH KNOWN SCOUR PROBLEMS BY COUNTY

LOCATION	COUNTY	NO. OF STRUCTURES
1	Iredell	70
	Surry	68
	Wilkes	64
	Alleghany	47
	Robeson	46
SUBTOTAL	5	295
2	Ashe	39
	Cumberland	32
	Catawba	31
	Yadkin	29
	Caldwell	28
	Buncombe	25
	Bladen	23
	Watauga	22
	Columbus	22
SUBTOTAL	9	251
3	Yancey	16
	Graham	13
	Scotland	13
	Alexander	12
	Mitchell	12
	Jackson	11
SUBTOTAL	6	77

LOCATION	COUNTY	NO. OF STRUCTURES
4	Cleveland	9
	Henderson	9
	Madison	8
	Cherokee	8
	Macon	8
	Haywood	7
	Rockingham	6
	Transylvania	6
	McDowell	6
	Clay	6
	Hyde	6
	Avery	5
	Burke	5
	Swain	5
	Caswell	4
	Forsyth	3
	Chatham	3
	Rutherford	3
	Northampton	3
	Lenior	3
	Halifax	3
	Union	3
	Rowan	3
	Polk	3
	Nash	3
	Lincoln	2
	Cabarrus	2
	Mecklenburg	2
	Davidson	2
	Duplin	2
	Pender	2
	Edgecombe	2
	Wilson	2
	Gaston	1
	Alamance	1
	Randolph	1
	Stokes	1
	Greene	1
	Brunswick	1
	Durham	1
	Anson	1
	Dare	1
SUBTOTAL	42	153
5	Remainder of Counties	0
SUBTOTAL	38	0
TOTAL	100	776

Analysis of the data in Table 6 indicates four (4) levels of structures with known scour problems. An additional level is one in which there are no structures with known scour problems. Location Prioritization Categories are as follows:

Location	Range of Structures In A County With Known Scour Problems
1	greater than 45
2	21 - 45
3	10 - 20
4	1 - 9
5	0

There will be five (5) categories of location priority which is shown in Table 7 under STRUCTURES WITH KNOWN SCOUR PROBLEMS.

It is recommended that Location Priority be reviewed and evaluated periodically as experience is gained in Scour Evaluation. Adjustment of the number of Counties in the five (5) categories may be required as experience is gained in Scour Evaluation.

Location in the state will not be a prioritization factor for structures with known scour problems since a structure with an identified scour problem is critical at any location in the state.

TABLE 7: PRIORITIZATION BY LOCATION

PRIORITY				
LOCATION 1	LOCATION 2	LOCATION 3	LOCATION 4	LOCATION 5
COUNTIES	COUNTIES	COUNTIES	COUNTIES	COUNTIES
Iredell	Ashe	Yancey	Cleveland	Bertie
Surry	Cumberland	Alexander	Henderson	Camden
Wilkes	Catawba	Mitchell	Madison	Chowan
Alleghany	Yadkin	Graham	Cherokee	Currituck
Robeson	Caldwell	Scotland	Haywood	Hertford
	Buncombe	Jackson	Rockingham	Martin
	Watauga		Transylvania	Pasquotank
	Bladen		McDowell	Perquimans
	Columbus		Caswell	Tyrrell
			Forsyth	Washington
			Chatham	Beaufort
			Rutherford	Carteret
			Northampton	Craven
			Lenior	Pamlico
			Halifax	New Hanover
			Lincoln	Onslow
			Cabarrus	Sampson
			Mecklenburg	Johnston
			Davidson	Franklin
			Duplin	Granville
			Pender	Person
			Edgcombe	Warren
			Wilson	Harnett
			Gaston	Guilford
			Alamance	Orange
			Randolph	Montgomery
			Stokes	Richmond
			Greene	Stanly
			Brunswick	Gates
			Durham	Jones
			Macon	Pitt
			Clay	Wayne
			Hyde	Vance
			Avery	Wake
			Burke	Hoke
			Swain	Lee
			Union	Moore
			Rowan	Davie
			Polk	
			Nash	
			Anson	
			Dare	

4.1.6 Simple Spans

Structures with simple spans are more susceptible to collapse due to scour than are continuous spans. Therefore simple spans will be evaluated before continuous spans.

4.1.7 Latest Inspection Date

After structures have been prioritized by the factors discussed, there could be several hundred structures in some of the combinations of groups. The latest inspection date criteria will prioritize these group combinations into manageable numbers of structures for scour evaluation.

Structures with the most current data will be evaluated first. The latest inspection date either underwater or above water will be utilized.

4.2 Prioritization For Scour Evaluation Flow Chart

Figure 3 is a flow chart for "Screening, Prioritization, And Coding For Scour Evaluation" of existing structures.

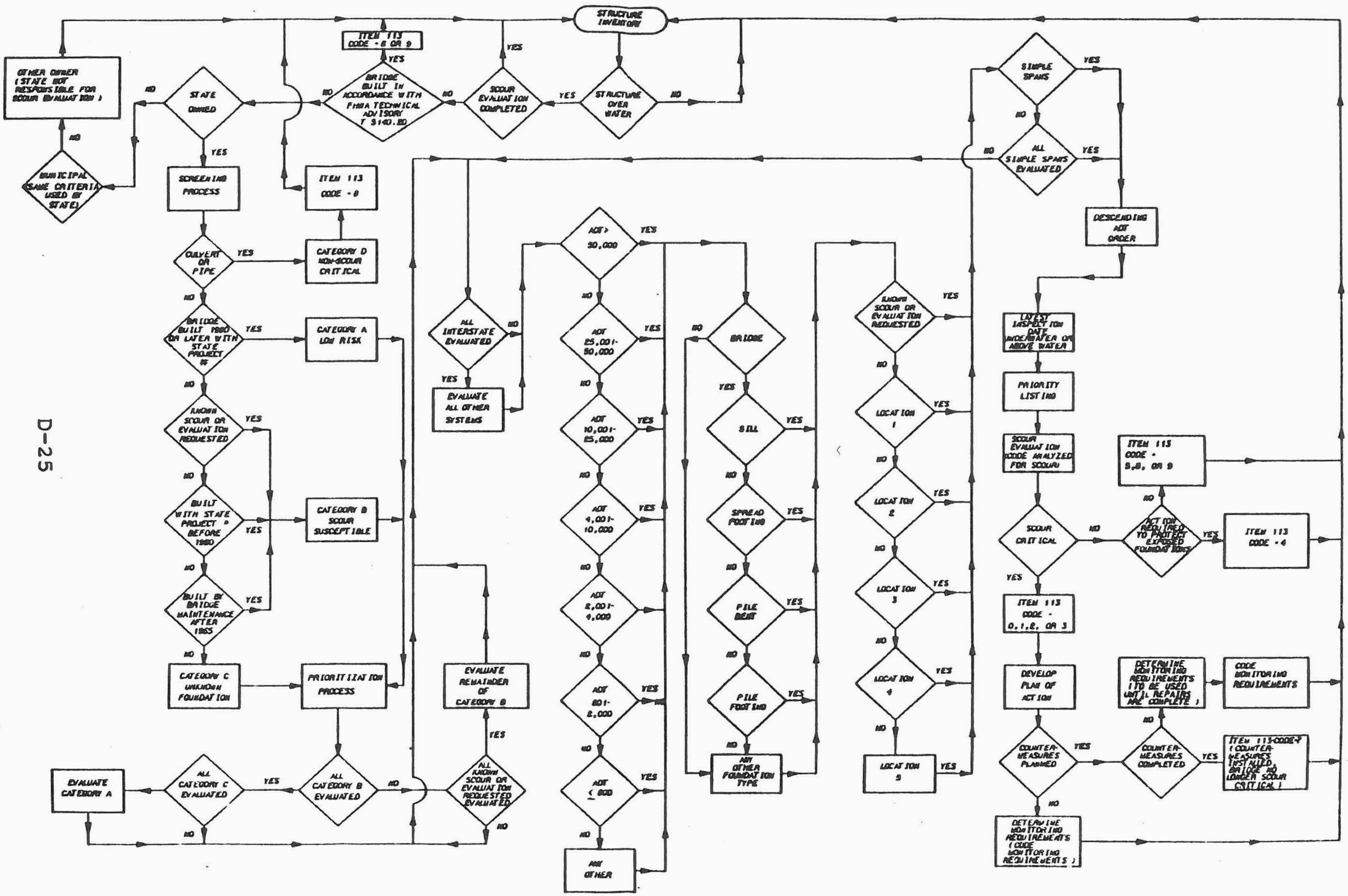
4.3 Prioritization For Scour Evaluation Data

See APPENDIX B for Prioritization For Scour Evaluation Data.

4.4 Conclusions

This process for Prioritization For Scour Evaluation of existing structures accounts for the effect that a structure collapse would have on the safety of the traveling public and on the operation of the overall transportation system.

A computer program will be written to automate Prioritization For Scour Evaluation. See discussion in APPENDIX B for justification.



SCREENING, PRIORITIZATION AND CODING FOR SCOUR EVALUATION

FIGURE 3

APPENDIX A

Structures With Known Scour Problems

Prioritized For Scour Evaluation

Section 1. Introduction

Structures with known scour problems are the top priority for Scour Evaluation. Table A1 shows the number of structures with known scour problems by ADT and Foundation Type. Table A2 shows the number of structures with known scour problems by County and Foundation Type.

Section 2. Prioritization For Scour Evaluation

Table A3 is a partial listing of structures with known scour problems. Table A3 lists structures in priority order in accordance with the Screening, Prioritization, And Coding For Scour Evaluation flow chart.

Table A3 was prepared manually. A computer program will be developed to automate this process.

Bridge Maintenance will be responsible for maintaining a priority list for structures with known scour problems.

TABLE A1: KNOWN SCOUR PROBLEMS - NUMBER OF STRUCTURES BY ADT

ADT	FOUNDATION TYPE					TOTAL
	SPREAD	SILL	PILE BENT	PILE FOOTING	OTHER	
10,001-25,000	10	0	10	3	2	25
4,001-10,000	47	0	20	12	2	81
2,001-4,000	30	0	23	8	2	63
801-2,000	36	5	2	51	9	103
> or = 800	193	214	83	6	8	504
TOTALS	316	219	138	80	23	776

TABLE A2: KNOWN SCOUR PROBLEMS
NUMBER OF STRUCTURES BY COUNTY - FOUNDATION TYPE

COUNTY	FOUNDATION TYPE					TOTAL
	SPR.	SILL	PILE BT.	PILE FTG.	OTHER	
Iredell	26	23	16	5	0	70
Surry	34	25	5	4	0	68
Wilkes	16	44	3	1	0	64
Alleghany	5	38	1	0	3	47
Robeson	3	0	30	12	1	46
Ashe	27	11	1	0	0	39
Cumberland	14	0	12	6	0	32
Catawba	13	3	6	8	1	31
Yadkin	11	12	4	1	1	29
Caldwell	17	8	1	2	0	28
Buncombe	18	4	3	0	0	25
Bladen	4	0	12	7	0	23
Watauga	14	6	0	1	1	22
Columbus	2	0	11	9	0	22
Yancey	6	9	0	0	1	16
Graham	3	10	0	0	0	13
Scotland	0	0	11	2	0	13
Alexander	6	2	3	1	0	12
Mitchell	7	5	0	0	0	12
Jackson	4	5	1	0	1	11
Cleveland	5	1	1	2	0	9
Henderson	6	2	0	1	0	9
Madison	5	1	0	0	2	8
Cherokee	3	2	0	1	2	8
Macon	6	1	0	0	1	8
Haywood	6	0	0	1	0	7
Rockingham	5	0	0	1	0	6
Transylvania	4	2	0	0	0	6
McDowell	1	1	3	1	0	6
Clay	6	0	0	0	0	6
Hyde	0	0	0	1	5	6
Avery	5	0	0	0	0	5

COUNTY	FOUNDATION TYPE					TOTAL
	SPR.	SILL	PILE BT.	PILE FTG.	OTHER	
Burke	3	0	2	0	0	5
Swain	5	0	0	0	0	5
Caswell	1	1	0	2	0	4
Forsyth	1	1	0	1	0	3
Chatham	2	0	1	0	0	3
Rutherford	1	1	1	0	0	3
Northampton	2	0	0	1	0	3
Lenior	0	0	2	0	1	3
Halifax	1	0	0	0	2	3
Union	3	0	0	0	0	3
Rowan	0	0	2	1	0	3
Polk	2	0	0	1	0	3
Nash	2	0	1	0	0	3
Lincoln	1	0	1	0	0	2
Cabarrus	0	1	1	0	0	2
Mecklenburg	0	0	2	0	0	2
Davidson	1	0	0	1	0	2
Duplin	0	0	0	2	0	2
Pender	1	0	0	1	0	2
Edgecombe	2	0	0	0	0	2
Wilson	2	0	0	0	0	2
Gaston	0	0	0	0	1	1
Alamance	0	0	0	1	0	1
Randolph	1	0	0	0	0	1
Stokes	1	0	0	0	0	1
Greene	1	0	0	0	0	1
Brunswick	0	0	0	1	0	1
Durham	0	0	1	0	0	1
Anson	1	0	0	0	0	1
Dare	0	0	0	1	0	1
TOTALS	316	219	138	80	23	776

TABLE A3: PRIORITY LISTING FOR STRUCTURES WITH KNOWN SCOUR PROBLEMS

COUNTY	BRIDGE NUMBER	ROUTE	FEATURE INTERSECTED	ADT	FOUNDATION TYPE	PRIORITY
HAYWOOD	142	I-40	PIGEON RIVER	11,600	SPREAD	1
CATAWBA	177	I-40	LYLE CREEK	10,750	SPREAD	2
CATAWBA	178	I-40	LYLE CREEK	10,750	SPREAD	3
NORTHAMPTON	9	I-95	ROANOAKE RIVER	10,200	SPREAD	4
NORTHAMPTON	11	I-95	ROANOAKE RIVER	10,200	SPREAD	5
MECKLENBURG	296	I-85	MALLARD CREEK	17,000	PILE BENT	6
MECKLENBURG	298	I-85	MALLARD CREEK	17,000	PILE BENT	7
ROBESON	147	I-95	LUMBER RIVER	15,000	PILE BENT	8
ROBESON	146	I-95	LUMBER RIVER	14,800	PILE BENT	9
IREDELL	52	I-77	REEDS CREEK	11,000	PILE BENT	10
IREDELL	53	I-77	REEDS CREEK	11,000	PILE BENT	11
ROBESON	C89	I-95	ASHPOLE SWAMP	21,000	OTHER	12
CATAWBA	C71	I-40	CREEK	20,000	OTHER	13
IREDELL	6	I-40	CATAWBA RIVER	9,350	SPREAD	14
IREDELL	7	I-40	CATAWBA RIVER	9,350	SPREAD	15
CUMBERLAND	35	I-95	ROCKFISH CREEK	9,300	SPREAD	16
CUMBERLAND	36	I-95	ROCKFISH CREEK	9,300	SPREAD	17
CUMBERLAND	109	I-95	CAPE FEAR RIVER	8,850	SPREAD	18
CUMBERLAND	111	I-95	CAPE FEAR RIVER	8,850	SPREAD	19
IREDELL	186	I-40	S.YADKIN R. & SR 2145	8,250	SPREAD	20
CUMBERLAND	85	I-95	CAPE FEAR R. SR 1739 & SR 1737	8,200	SPREAD	21
SURRY	121	I-77	MITCHELL RIVER	7,750	SPREAD	22
SURRY	123	I-77	MITCHELL RIVER	7,750	SPREAD	23
CUMBERLAND	77	I-95	ROCKFISH CREEK	7,550	SPREAD	24
CUMBERLAND	83	I-95	ROCKFISH CREEK	7,550	SPREAD	25
CUMBERLAND	23	NC 24	LOWER LITTLE RIVER	21,200	SPREAD	26
BUNCOMBE	76	US 25	SWANNANOA RIVER SOUTH R\R & SR 3556	15,400	SPREAD	27
CUMBERLAND	49	NC 210	LOWER LITTLE RIVER	11,900	SPREAD	28
ROCKINGHAM	75	NC 700	SMITH CREEK	11,700	SPREAD	29
WILKES	48	US 421	YADKIN RIVER	11,000	SPREAD	30
CUMBERLAND	71	SR 1400	BEAVER CREEK	15,100	PILE BENT	31
ROBESON	125	NC 41	LUMBER RIVER	14,700	PILE BENT	32
ROBESON	43	NC 72	LUMBER RIVER	13,000	PILE BENT	33
CUMBERLAND	70	SR 1404	BEAVER CREEK	11,600	PILE BENT	34
CALDWELL	16	US 64	ZACKS FORK CREEK	25,000	PILE FOOTING	35
CATAWBA	91	NC 127	CATAWBA RIVER	12,700	PILE FOOTING	36
FORSYTH	33	US 158	MUDDY CREEK	10,500	PILE FOOTING	37
SURRY	81	US 601	STEWARTS CREEK	9,900	SPREAD	38
SURRY	26	US 52	ARARAT RIVER	9,800	SPREAD	39
SURRY	184	US 52	ARARAT RIVER	9,700	SPREAD	40
CHEROKEE	48	US 19	VALLEY RIVER	9,400	SPREAD	41
CUMBERLAND	14	US 401	LAKE RIM RUNOFF	9,000	SPREAD	42
YADKIN	177	SR 1314	SOUTH DEEP CREEK	8,700	SPREAD	43
ROCKINGHAM	63	US 220	DAN RIVER	8,300	SPREAD	44
SURRY	332	SR 1190	YADKIN R.	8,100	SPREAD	45
CHEROKEE	14	US 19	HIWASSEE RIVER	8,000	SPREAD	46

TABLE A3: PRIORITY LISTING FOR STRUCTURES WITH KNOWN SCOUR PROBLEMS (continued)

COUNTY	BRIDGE NUMBER	ROUTE	FEATURE INTERSECTED	ADT	FOUNDATION TYPE	PRIORITY
HALIFAX	51	NC 48	ROANOAKE RIVER	7,500	SPREAD	47
BUNCOMBE	39	NC 51	SWANNANOA RIVER	7,200	SPREAD	48
HENDERSON	115	US 64	FRENCH BREAD R.	7,200	SPREAD	49
SURRY	21	US 21	YADKIN R.	7,000	SPREAD	50
LINCOLN	50	NC 73	CATAWBA R.	7,000	SPREAD	51
CLEVELAND	101	US 74	BUFFALO CREEK	6,950	SPREAD	52
AVERY	27	US 221	LINVILLE R.	6,900	SPREAD	53
SURRY	111	NC 89	LOVILLES CREEK	6,600	SPREAD	54
SURRY	126	US 52	TOMS CREEK	6,550	SPREAD	55
CATAWBA	139	NC 16	CATAWBA R.	6,500	SPREAD	56
CUMBERLAND	144	NC 24	SOUTH R.	6,400	SPREAD	57
ALEXANDER	4	NC 16	LOWER LITTLE R.	6,100	SPREAD	58
SURRY	122	US 52	TOMS CREEK	6,000	SPREAD	59
BUNCOMBE	265	NC 151	HOMINY CREEK	5,900	SPREAD	60
WATAUGA	53	NC 194	BAIRDS CREEK	5,900	SPREAD	61
ALEXANDER	6	US 64	LOWER LITTLE R.	5,500	SPREAD	62
IREDELL	56	SR 1109	LAKE NORMAN	5,000	SPREAD	63
YADKIN	35	NC 67	YADKIN R.	5,000	SPREAD	64
COLUMBUS	55	US 74	WHITE MARSH SWAMP	4,800	SPREAD	65
BUNCOMBE	649	SR 1002	FRENCH BROAD R. & SOUTH.R/R	4,600	SPREAD	66
HAYWOOD	176	NC 215	PIGEON R.	4,600	SPREAD	67
TRANSYLVANIA	69	US 64	N.FORK FRENCH BROAD R.	4,500	SPREAD	68
AVERY	4	US 19	NORTH TOE R.	4,300	SPREAD	69
CLAY	6	US 64	HIWASSEE R.	4,300	SPREAD	70
CATAWBA	50	NC 127	HENRY FORK R.	4,100	SPREAD	71
ANSON	81	US 74	PEE DEE R.	4,050	SPREAD	72
SURRY	185	US 52	AVARAT R.	9,700	PILE BENT	73
CUMBERLAND	68	NC 59	ROCKFISH CREEK	8,100	PILE BENT	74
LENOIR	43	US 70	NEUSE R.	7,850	PILE BENT	75
LENOIR	42	US 70	NEUSE R.	7,600	PILE BENT	76
DURHAM	217	SR 1116	CREEK	7,300	PILE BENT	77
LINCOLN	35	NC 150	S.FORK CATAWBA R.	7,000	PILE BENT	78
ROWAN	85	US 70	NORTH SOUND CREEK	7,000	PILE BENT	79
COLUMBUS	53	US 74	WHITE MARSH SWAMP	6,900	PILE BENT	80
ROBESON	33	US 74	BACK SWAMP CREEK	6,300	PILE BENT	81
BLADEN	6	NC 131	BRYANT SWAMP	6,300	PILE BENT	82
ROBESON	118	US 74	LUMBER R.	6,100	PILE BENT	83
SCOTLAND	22	US 74	GUM SWAMP CREEK	6,050	PILE BENT	84
IREDELL	45	SR 1100	CREEK	6,000	PILE BENT	85
COLUMBUS	54	US 74	WHITE MARSH SWAMP	5,720	PILE BENT	86
CALDWELL	15	US 64	SPAINHOUR CREEK	5,700	PILE BENT	87

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TABLE A3: PRIORITY LISTING FOR STRUCTURES WITH KNOWN SCOUR PROBLEMS (continued)

COUNTY	BRIDGE NUMBER	ROUTE	FEATURE INTERSECTED	ADT	FOUNDATION TYPE	PRIORITY
MCDOWELL	267	SR 1103	CATAWBA R.	5,200	PILE BENT	88
SCOTLAND	17	NC 15	GUM SWAMP	5,050	PILE BENT	89
ROBESON	16	NC 71	LUMBER R.	5,000	PILE BENT	90
ROBESON	420	SR 2289	LUMBER R.	5,000	PILE BENT	91
SCOTLAND	47	US 401	LUMBER R.	4,600	PILE BENT	92
BLADEN	17	NC 701	CAPE FEAR R.	9,200	PILE FOOTING	93
BLADEN	22	NC 211	BRYANT SWAMP	7,300	PILE FOOTING	94
CUMBERLAND	126	NC 24	CAPE FEAR R.	7,100	PILE FOOTING	95
MCDOWELL	104	US 221	ARMSTRONG CREEK	7,100	PILE FOOTING	96
CATAWBA	138	NC 150	LAKE NORMAN	6,700	PILE FOOTING	97
SCOTLAND	23	US 74	GUM SWAMP CREEK	6,050	PILE FOOTING	98
CLEVELAND	17	NC 18	HICKORY CREEK	5,800	PILE FOOTING	99
WATAUGA	72	US 221	GAP CREEK	5,400	PILE FOOTING	100
CATAWBA	97	NC 16	LYLE CREEK	5,000	PILE FOOTING	101
CUMBERLAND	219	SR 1006	CAPE FEAR R.	5,000	PILE FOOTING	102
CATAWBA	111	NC 16	BAKERS CREEK	4,900	PILE FOOTING	103
IREDELL	43	US 70	THIRD CREEK	4,350	PILE FOOTING	104
GASTON	C20	NC 27	DUTCHMAN'S CREEK	6,500	OTHER	105
MADISON	C35	US 25-70	WALNUT CREEK	5,200	OTHER	106
BUNCOMBE	292	NC 151	HOMINY CR. & SOUTH R\R	4,000	SPREAD	107
CUMBERLAND	21	NC 87	ROCKFISH CREEK	4,000	SPREAD	108
CUMBERLAND	60	US 40	LOWER LITTLE R.	4,000	SPREAD	109
CUMBERLAND	182	SR 1451	LITTLE R.	4,000	SPREAD	110
COLUMBUS	83	US 74	LIVINGSTON CREEK	3,750	SPREAD	111
HENDERSON	3	SR 1345	FRENCH BROAD R.	3,750	SPREAD	112
ROCKINGHAM	134	NC 700	DAN R.	3,400	SPREAD	113
AVERY	23	NC 194	ELK R.	3,300	SPREAD	114
SURRY	330	SR 2258	FISHER R.	3,300	SPREAD	115
YADKIN	54	US 601	YADKIN R. & SOUTH R\R	3,200	SPREAD	116
YADKIN	115	SR 1605	FORBUSH CREEK	3,200	SPREAD	117
WILSON	88	SR 1326	TOLSHOT RES.	3,100	SPREAD	118
JACKSON	52	NC 107	CONEY FORK CREEK	3,100	SPREAD	119
ROBESON	439	NC 72	LUMBER R.	3,100	SPREAD	120
TRANSYLVANIA	193	SR 1533	DAVISON R.	2,900	SPREAD	121
PENDER	28	NC 210	LONG CREEK	2,800	SPREAD	122
BLADEN	37	NC 211	BROWN MARSH SWAMP	2,800	SPREAD	123
BLADEN	48	NC 211	ELKTON SWAMP CK.	2,800	SPREAD	124
BUNCOMBE	511	SR 3413	HOMINY CREEK	2,800	SPREAD	125
IREDELL	91	US 21	DUTCHMAN CREEK	2,700	SPREAD	126
CATAWBA	141	NC 10	LYLE CREEK	2,600	SPREAD	127

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APPENDIX B

Prioritization For Scour Evaluation

(For all structures except those with known scour problems.)

Section 1. Introduction

All structures must be prioritized for scour evaluation. Table B1 shows the number of structures over water by System, ADT, and Foundation Type.

Section 2. Limitation on Computer Generated Data

Whether spans are simple or continuous can not be computer generated at this time. This data will be entered in the computer file beginning in early 1990.

Approximately 7 to 8 hours of computer time was required to generate the data contained in Table B1. Approximately 40 individual computer runs were required to generate this data. It took a technician 2 to 3 days to write the programs and check the output.

In order to run location in the State, it would require that each county be run individually. To run each county individually would increase computer time, number of individual runs, and technician time by a factor of approximately 100. The effort and expense in running the data in this manner is not justified by the benefits that would be gained.

A similar type of manual effort will be required to generate lists of individual structures for scour evaluations. A computer program will be developed to automate this process.

Section 3. Conclusions

Although the data presented does not accurately reflect the Screening, Prioritization, And Coding For Scour Evaluation Flow Chart, it does give a "feel" for the numbers of structures in some of the areas of the flow chart.

Lists of individual structures prioritized for scour evaluation will be developed as needed.

TABLE B1: STRUCTURES OVER WATER
BY
SYSTEM-ADT-FOUNDATION TYPE

FOUNDATION TYPE	ADT	INTERSTATE			PRIMARY			SECONDARY			ALL SYSTEMS		
		MULTI-SPAN	SINGLE SPAN	TOTAL	MULTI-SPAN	SINGLE SPAN	TOTAL	MULTI-SPAN	SINGLE SPAN	TOTAL	MULTI-SPAN	SINGLE SPAN	TOTAL
SPREAD FOOTING	> 4.000	110	0	110	380	48	428	70	32	102	560	80	640
	2.001-4.000	2	0	2	154	37	191	76	46	122	232	83	315
	801-2.000	1	0	1	123	33	156	174	143	317	298	176	474
	< or = 800	0	0	0	81	37	118	716	1.057	1.773	797	1.094	1.891
	Pedestrian	0	0	0	0	0	0	0	0	0	0	0	0
	Subtotal	113	0	113	738	155	893	1.036	1.278	2.314	1.387	1.433	3.320
SILL	> 4.000	0	0	0	5	0	5	10	8	18	15	8	23
	2.001-4.000	0	0	0	2	2	4	25	20	45	27	22	49
	801-2.000	0	0	0	8	3	11	91	135	226	99	138	237
	< or = 800	0	0	0	3	8	11	750	2.442	3.192	753	2.450	3.203
	Pedestrian	0	0	0	0	0	0	0	0	0	0	0	0
	Subtotal	0	0	0	18	13	31	876	2.605	3.481	894	2.618	3.512
PILE BENT	> 4.000	35	0	35	270	7	277	117	5	122	422	12	434
	2.001-4.000	3	0	3	174	5	179	153	19	172	330	24	354
	801-2.000	0	0	0	214	14	228	469	84	553	683	98	781
	< or = 800	0	0	0	81	8	89	2.370	623	2.993	2.451	631	3.082
	Pedestrian	0	0	0	0	0	0	0	0	0	0	0	0
	Subtotal	38	0	38	739	34	773	3.109	731	3.840	3.886	765	4.651
PILE FOOTING	> 4.000	40	0	40	129	10	139	17	1	18	186	11	197
	2.001-4.000	2	0	2	53	4	57	7	0	7	62	4	66
	801-2.000	0	0	0	18	0	18	18	1	19	36	1	37
	< or = 800	0	0	0	10	3	13	47	3	50	57	6	63
	Pedestrian	1	0	1	0	0	0	0	0	0	1	0	1
	Subtotal	43	0	43	210	17	227	89	5	94	342	22	364
CULVERT PIPE AND OTHER FOUNDATION TYPES	> 4.000	176	0	176	548	0	548	137	0	137	858	0	858
	2.001-4.000	0	0	0	261	0	261	81	0	81	342	0	342
	801-2.000	0	0	0	180	0	180	216	0	216	396	0	396
	< or = 800	0	0	0	70	0	70	1.032	0	1.032	1.105	0	1.105
	Pedestrian	0	0	0	0	0	0	0	0	0	0	0	0
	Subtotal	176	0	176	1.059	0	1.059	1.466	0	1.466	2.701	0	2.701
TOTALS		370	0	370	2.764	219	2.983	6.576	4.619	11.195	9.710	4.838	14.548

APPENDIX E

RECORDING AND CODING GUIDE
for the
STRUCTURAL INVENTORY and APPRAISAL
of the
NATION'S BRIDGES

This appendix contains relevant material for recording and coding the results of the evaluation of scour at bridges. The material is excerpted from the Federal Highway Administration document "Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges," dated December 1988.

Items 58 through 62 - Indicate the Condition Ratings

In order to promote uniformity between bridge inspectors, these guidelines will be used to rate and code Items 58, 59, 60, 61, and 62.

Condition ratings are used to describe the existing, in-place bridge as compared to the as-built condition. Evaluation is for the materials related, physical condition of the deck, superstructure, and substructure components of a bridge. The condition evaluation of channels and channel protection and culverts is also included. Condition codes are properly used when they provide an overall characterization of the general condition of the entire component being rated. Conversely, they are improperly used if they attempt to describe localized or nominally occurring instances of deterioration or disrepair. Correct assignment of a condition code must, therefore, consider both the severity of the deterioration or disrepair and the extent to which it is widespread throughout the component being rated.

The load-carrying capacity will not be used in evaluating condition items. The fact that a bridge was designed for less than current legal loads and may be posted shall have no influence upon condition ratings.

Portions of bridges that are being supported or strengthened by temporary members will be rated based on their actual condition; that is, the temporary members are not considered in the rating of the item. (See Item 103 - Temporary Structure Designation for the definition of a temporary bridge.)

Completed bridges not yet opened to traffic, if rated, shall be coded as if open to traffic.

Item 60 - Substructure 1 digit

This item describes the physical condition of piers, abutments, piles, fenders, footings, or other components. Rate and code the condition in accordance with the previously described general condition ratings. Code N for all culverts.

All substructure elements should be inspected for visible signs of distress including evidence of cracking, section loss, settlement, misalignment, scour, collision damage, and corrosion. The rating given by Item 113 - Scour Critical Bridges, may have a significant effect on Item 60 if scour has substantially affected the overall condition of the substructure.

The substructure condition rating shall be made independent of the deck and superstructure.

Integral-abutment wingwalls to the first construction or expansion joint shall be included in the evaluation. For non-integral superstructure and substructure units, the substructure shall be considered as the portion below the bearings. For structures where the substructure and superstructure are integral, the substructure shall be considered as the portion below the superstructure.

1. Item 60 - Substructure:

CONDITION RATING FOR ITEM 60

Code	Description
N	NOT APPLICABLE
9	EXCELLENT CONDITION
8	VERY GOOD CONDITION - no problems noted.
7	GOOD CONDITION - some minor problems.
6	SATISFACTORY CONDITION - structural elements show some minor deterioration.
5	FAIR CONDITION - all primary structural elements are sound but may have minor section loss, cracking, spalling, or scour.
4	POOR CONDITION - advanced section loss, deterioration. spalling, or scour.
3	SERIOUS CONDITION - loss of section, deterioration, spalling, or scour have seriously affected primary structural components. Local failures are possible. Fatigue cracks in steel or shear cracks in concrete may be present.
2	CRITICAL CONDITION - advanced deterioration of primary structural elements. Fatigue cracks in steel or shear cracks in concrete may be present or scour may have removed substructure support. Unless closely monitored it may be necessary to close the bridge until corrective action is taken.
1	"IMMINENT" FAILURE CONDITION - major deterioration or section loss present in critical structural components or obvious vertical or horizontal movement affecting structure stability. Bridge is closed to traffic but corrective action may put back in light service.
0	FAILED CONDITION - out of service - beyond corrective action.

Item 61 - Channel and Channel Protection

This item describes the physical conditions associated with the flow of water through the bridge such as stream stability and the condition of the channel, riprap, slope protection, or stream control devices including spur dikes. The inspector should be particularly concerned with visible signs of excessive water velocity which may affect undermining of slope protection or footings, erosion of banks, and realignment of the stream which may result in immediate or potential problems. Accumulation of drift and debris on the superstructure and substructure should be noted on the inspection form but not included in the condition rating.

Rate and code the condition in accordance with the previously described general condition ratings and the following descriptive codes:

Code	Description
N	Not applicable. Use when bridge is not over a waterway.
9	There are no noticeable or noteworthy deficiencies which affect the condition of the channel.
8	Banks are protected or well vegetated. River control devices such as spur dikes and embankment protection are not required or are in a stable condition.
7	Bank protection is in need of minor repairs. River control devices and embankment protection have a little minor damage. Banks and/or channel have minor amounts of drift.
6	Bank is beginning to slump. River control devices and embankment protection have widespread minor damage. There is minor stream bed movement evident. Debris is restricting the waterway slightly.
5	Bank protection is being eroded. River control devices and/or embankment have major damage. Trees and brush restrict the channel.
4	Bank and embankment protection is severely undermined. River control devices have severe damage. Large deposits of debris are in the waterway.
3	Bank protection has failed. River control devices have been destroyed. Stream bed aggradation, degradation or lateral movement has changed the waterway to now threaten the bridge and/or approach roadway.
2	The waterway has changed to the extent the bridge is near a state of collapse.
1	Bridge closed because of channel failure. Corrective action may put back in light service.
0	Bridge closed because of channel failure. Replacement necessary.

Item 71 - Waterway Adequacy

This item appraises the waterway opening with respect to passage of flow through the bridge. The following codes shall be used in evaluating waterway adequacy. Site conditions may warrant somewhat higher or lower ratings than indicated by the table (e.g., flooding of an urban area due to a restricted bridge opening).

Where overtopping frequency information is available, the descriptions given in the table for chance of overtopping mean the following:

Remote	-	greater than 100 years
Slight	-	11 to 100 years
Occasional	-	3 to 10 years
Frequent	-	less than 3 years

Adjectives describing traffic delays mean the following:

Insignificant	-	Minor inconvenience. Highway passable in a matter of hours.
Significant	-	Traffic delays of up to several days.
Severe	-	Long term delays to traffic with resulting hardship.

Functional Classification			
Principal Arterials - Interstates, Freeways, or Expressways	Other Principal and Minor Arterials and Major Collectors	Minor Collectors, Locals	Description
Code			
N	N	N	Bridge not over a waterway.
9	9	9	Bridge deck and roadway approaches above flood water elevations (high water). Chance of overtopping is remote.
8	8	8	Bridge deck above roadway approaches. Slight chance of overtopping roadway approaches.
6	6	7	Slight chance of overtopping bridge deck and roadway approaches.
4	5	6	Bridge deck above roadway approaches. Occasional overtopping of roadway approaches with insignificant traffic delays.

(codes continued on the next page)

Item 71 - Waterway Adequacy (cont'd)

Functional Classification			
Principal Arterials - Interstates, Freeways, or Expressways	Other Principal and Minor Arterials and Major Collectors	Minor Collectors, Locals	Description
Code			
3	4	5	Bridge deck above roadway approaches. Occasional overtopping of roadway approaches with significant traffic delays.
2	3	4	Occasional overtopping of bridge deck and roadway approaches with significant traffic delays.
2	2	3	Frequent overtopping of bridge deck and roadway approaches with significant traffic delays.
2	2	2	Occasional or frequent overtopping of bridge deck and roadway approaches with severe traffic delays.
0	0	0	Bridge closed.

Item 92 - Critical Feature Inspection

Using a series of 3-digit code segments, denote critical features that need special inspections or special emphasis during inspections and the designated inspection interval in months as determined by the individual in charge of the inspection program. The designated inspection interval could vary from inspection to inspection depending on the condition of the bridge at the time of inspection.

Segment	Description	Length
92A	Fracture Critical Details	3 digits
92B	Underwater Inspection	3 digits
92C	Other Special Inspection	3 digits

For each of 92A, B, and C, code the first digit Y for special inspection or emphasis needed and code N for not needed. The first digit of 92A, B, and C must be coded for all structures to designate either a yes or no answer. In the second and third digits of each segment, code a 2-digit number to indicate the number of months between inspections only if the first digit is coded Y. If the first digit is coded N, the second and third digits are left blank.

EXAMPLES:	Item	Code
A 2-girder system structure which is being inspected yearly and no other special inspections are required.	92A 92B 92C	Y12 N__ N__
A structure where both fracture critical and underwater inspection are being performed on a 1-year interval. Other special inspections are not required.	92A 92B 92C	Y12 Y12 N__
A structure has been temporarily shored and is being inspected on a 6-month interval. Other special inspections are not required.	92A 92B 92C	N__ N__ Y06

Item 93 - Critical Feature Inspection Date

Code only if the first digit of Item 92A, B, or C is coded Y for yes. Record as a series of 4-digit code segments, the month and year that the last inspection of the denoted critical feature was performed.

Segment	Description	Length
93A	Fracture Critical Details	4 digits
93B	Underwater Inspection	4 digits
93C	Other Special Inspection	4 digits

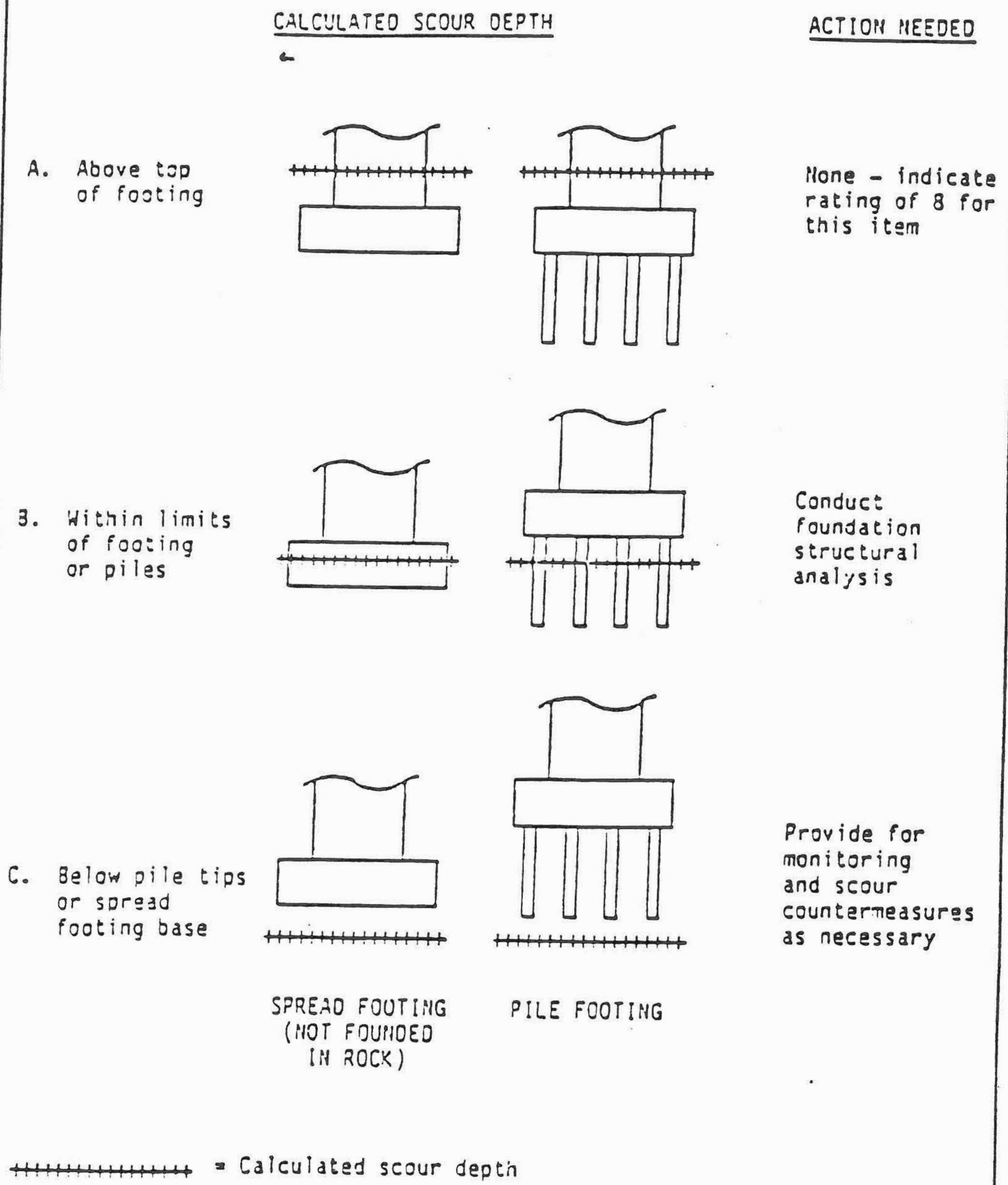
For each segment of this item, when applicable, code a 4-digit number to represent the month and year. The number of the month should be coded in the first 2 digits with leading zeros as required and the last 2 digits of the year coded as the third and fourth digits of the field. If the first digit of any part of Item 92 is coded N, then the corresponding part of this item shall be blank.

EXAMPLES:	Item	Code
A structure has fracture critical members which were last inspected in March 1986. It does not require underwater or other special feature inspections.	93A 93B 93C	0386 (blank) (blank)
A structure has no fracture critical details, but requires underwater inspection and has other special features (for example, a temporary support) for which the State requires special inspection. The last underwater inspection was done in April 1986 and the last special feature inspection was done in November 1985.	93A 93B 93C	(blank) 0486 1185

Item 113 - Scour Critical Bridges

Use a single-digit code as indicated below to identify the current status of the bridge regarding its vulnerability to scour. The scour calculations/analyses and field inspections for this determination shall be made by hydraulic/foundation engineers. Details on conducting a scour analysis are included in the FHWA Technical Advisory entitled, "Scour at Bridges." Whenever a rating factor of 4 or below is determined for this item, the rating factor for Item 60 - Substructure may need to be revised to reflect the severity of actual scour and resultant damage to the bridge. For foundations on rock where scour cannot be calculated, use the coding most descriptive of site conditions. A scour critical bridge is one with abutment or pier foundations which are rated as unstable due to (1) observed scour at the bridge site or (2) a scour potential as determined from a scour evaluation study.

Code	Description
N	Bridge not over waterway.
9	Bridge foundations (including piles) well above flood water elevations.
8	Bridge foundations determined to be stable for calculated scour conditions; calculated scour is above top of footing. (Example A).
7	Countermeasures have been installed to correct a previously existing problem with scour. Bridge is no longer scour critical.
6	Scour calculation/evaluation has not been made. (Use only to describe case where bridge has not yet been evaluated for scour potential.)
5	Bridge foundations determined to be stable for calculated scour conditions; scour within limits of footing or piles. (Example B).
4	Bridge foundations determined to be stable for calculated scour conditions; field review indicates action is required to protect exposed piles from effects of additional erosion and corrosion.
3	Bridge is scour critical; bridge foundations determined to be unstable for calculated scour conditions: - scour within limits of footing or piles (Example B) - scour below spread footing base or pile tips (Example C)
2	Bridge is scour critical; field review indicates that extensive scour has occurred at a bridge foundation. Immediate action is required to provide scour countermeasures.
1	Bridge is scour critical; field review indicates that failure of piers/abutments is imminent. Bridge is closed to traffic.
0	Bridge is scour critical. Bridge has failed and is closed to traffic.



EXAMPLES FOR CODING GUIDE ITEM 113 - SCOUR CRITICAL BRIDGES

APPENDIX F

SCOUR ANALYSIS

FOR

GREAT PEE DEE RIVER
U. S. Route 76/301

FLORENCE AND MARION COUNTIES
SOUTH CAROLINA

SOUTH CAROLINA DEPARTMENT
OF
HIGHWAYS AND PUBLIC TRANSPORTATION

July 7, 1988

By: WILLIAM H. HULBERET
Hydraulic Engineer

SCOUR ANALYSIS FOR
GREAT PEE DEE RIVER
AT U.S. ROUTE 76-301
FLORENCE AND MARION COUNTIES
SOUTH CAROLINA

A scour analysis was performed for the replacement of the bridges on the West Bound Lane over the Great Pee Dee River for U. S. Route 76/301. The purpose of the study was to determine the scour potential around the piers in the main channel so that the Bridge Design Section could set the footing elevations. The potential scour impacts on the bridge abutments and in the overflow bridge were also investigated.

The drainage area of the Great Pee Dee River at this location is 8,830 square miles. The drainage area extends along a north northwest line from just inside Virginia's southern border across North Carolina through South Carolina to the Coast at Georgetown. Above Albemarle, North Carolina, the river is called the Yadkin River.

Most of the South Carolina reach of the river is characterized by a wide flood plain and a meandering channel. A study of aerial photographs of the basin shows numerous oxbow lakes indicating that the channel has shifted location many times during the past.

At the Route 76/301 crossing, the flood plain is approximately 11,000 feet wide with the river located on the east edge. The CSX Railroad crosses the river approximately 900 feet upstream at the channel and 2800 feet upstream at the West edge of the flood plain.

The Route 76 crossing was originally completed by 1947 as a two lane road. In the 1960's, a parallel crossing was added making the roadway into a four lane divided section. An older crossing was constructed in the 1920's approximately 1.6 miles down stream. This project is to replace the bridge structures built in the 1940's.

The crossing has twin main bridges 4698 feet long over the river channel and twin overflow bridges 600 feet long. All abutments are spill through type. At the channel end of the existing main bridges, there are six spans supported by piers. Both bridges have three piers in the channel. The replacement bridge will have four piers with only two in the channel. All piers are skewed approximately 33 degrees to line up with the channel. The remaining substructure of the bridges consists of prestressed concrete pile bents oriented normal to the roadway.

A field inspection was made to determine the existing scour patterns on the crossing. The most apparent sign of scour is along the bank on the east side of the river. For approximately 50 feet back of the low water line, the high bank has been scoured to within a few feet of the normal water level. Concrete rubble has been placed along this area in an effort to stabilize the bank.

The only other sign of scour was at the first interior bent from the west end of the main bridge. At this bent there is a small scour hole approximately 6 feet wide and one foot deep. This is just at the toe of the abutment. Under the second span from the west end of the east bound lane overflow bridge there is a hole approximately one foot deep almost filling the area below the span. It is difficult to determine how much of this is caused by scour and how much by vehicle traffic from fishermen and hunters who obviously use the area.

At the pier located in the center of the main channel, there is a considerable accumulation of drift. This has caused a sand bar to develop around the pier. At the current low water levels this sand bar is almost exposed.

It should be noted that all end fills were protected by riprap. The area along side of the fills, between the bridges, and under the bridges is covered by a dense growth of underbrush.

HYDRAULIC ANALYSIS

In order to do the scour analysis, a detailed hydraulic analysis of the river was made. This study was made simpler by the presence of a U. S. Geological Survey Gage on the site. The gage was located at the older downstream bridge site from 1938 to 1947. It was relocated to the current site in 1947. The Weather Bureau had a stage gage located at the old site from 1924 to 1938.

The U. S. Army Corps of Engineers' computer program HEC-2 was used to analyze the crossing. The Federal Highway Administration Water Surface Profile (WSPRO) was considered, but the difficulty in modeling the CSX Railroad bridge with the specific bridge approach section distances required by the program ruled this out. A version of FHWA's Hydraulics of Bridge Backwater program HY-4 modified locally to analyze multiple bridge crossings was used to determine bridge losses. This program balances the flow distribution between bridges based on equalizing the backwater.

A gage rating, flood frequency relationship, and copies of four discharge measurements were furnished by the local office of the U. S. Geological Survey. The flood frequency relationship was computed by the Log Pearson Type III method using regionalized skew coefficients. The resulting frequencies, probabilities, discharges, and water surface elevation are summarized in the following table:

Frequency	Probability	Discharge cfs	Elevation ft.
2	.500	41,300	48.36
5	.200	63,100	50.93
10	.100	80,500	52.57
25	.040	106,000	54.47
50	.020	128,000	55.80
100	.010	153,000	57.00
500	.002	223,000	59.50

The four measured discharges were:
1. 61,100 cfs at elevation 50.69
2. 60,100 cfs at elevation 50.64
3. 72,200 cfs at elevation 51.48
4. 52,800 cfs at elevation 49.44

The maximum flood of record is the 1945 flood. The discharge for this flood was 223,000 cfs (cubic feet per second) or the same as the 500 year flood estimate.

A summary of other available high water data for the river from Department records and gage records follows:

Location	Distance from Rt. 76	Year	Elevation
I-95	9.47 mi.	1945	64.0
76-301	---	1908	57.0
76-301	---	1945	58.75
Old 76	1.6 mi	1928	53.06
Old 76	" "	1936	50.56
Old 76	" "	1945	56.76
US 378	29.2 mi.	1908	41.0 (est.)
US 378	" "	1928	38.67
US 378	" "	1936	37.61
US 378	" "	1945	37.61

The data used to develop the HEC-2 model, came from U. S. G. S. topographic maps and from the Road Plans for the old and existing crossings. The distance that the model needed to be extended downstream was computed using the method from the Corps of Engineers' <u>Accuracy of Computed Water Surface Profiles</u>. The computation was based on the 100 year flood.

$$Ldn = 8000\,(HD)^{.8}/S = 8000X(15.82)^{.8}/.459 = 158{,}700 \text{ ft.} = 30 \text{ mi.}$$

where Ldn = required length
HD = hydraulic depth
S = slope in ft./mile (Based on slope from Mannings Equation computed at rated 100 year high water elevation)

The computed value is too long for practical purposes. Based on the topography a distance of approximately 14 miles was used. Since the gage rating is available, the starting elevation for each profile was adjusted until the computed elevation matched the gage elevations.

It was not possible to match the discharge distribution for the overflow bridge with the HEC-2 model or with the bridge back-water model. The discharges computed by the bridge backwater program were 22% too low while the HEC-2 values were 87% too low. To estimate the flow for various floods in the overflow bridge the discharges computed by the bridge backwater model were increased by adjusting the computed values by the 22%. This problem indicates that this crossing should have been modeled using the U. S. G. S.'s two dimensional flow model. However, there is not enough data available to support this model. Adjusting the computed data on the basis of the gage records and measurements should give sufficient accuracy to compute the scour produced by the bridge crossing.

SCOUR ANALYSIS

The scour analysis was computed using the methods listed in "FHWA Technical Advisory Scour at Bridges" by E. V. Richardson. Scour is computed in three parts: 1. contraction scour, due to the contraction forcing more water into the channel, 2. local scour at the piers, due to the turbulence caused by the piers, and 3. local scour at the abutments due to the turbulence at the abutment. Scour computations from the three sources are added together to compute total scour depths. Since this is the first time that the Department has used this particular method, several different ways of computing each type of scour were used where they were available.

Soils data supporting the scour analysis was from a report prepared by Foundation And Materials Engineering, Inc. for the site. Their study included test boring data and seive analysis of samples collected. The test data revealed the presence of a hard silt sand layer called the Black Creek Formation at an average elevation of 11.6 throughout the flood plain. Under the channel this layer ran as low as elevation 5.0 to 6.0. Above the Black Creek in the flood plain, the soils are a loose silt sand clay mixture, which took low blow counts, generally less than 10 per foot. In the river bottom, this upper layer contained wood fragments, which were evidence of previous scour events.

Several borings were made close to existing piers to detect signs of previous scour. The results were inconclusive.

Due to the length of the main bridge, the hydraulic analysis did not reveal any increase in discharge in the channel through the bridge area. Therefore, there will be no contraction scour in the channel.

There are two methods for computing local scour at piers in Richardson's advisory. Richardson has an equation for live bed scour which predicts equilibrium scour. Maximum scour will be 30% higher. This equation is:

$$Ys/Y1 = 2.0\ K1\ K2\ (a/Y1)^{0.65}\ Fr1^{0.43}$$

Ys = scour depth
a = pier width
K1 = correction for pier shape
k2 = Correction for flow angle of attack to pier
Y1 = flow depth just upstream from pier
Fr1 = Froude No. $V1/(g\ Y1\ S)^{0.5}$
g = gravitational acceleration
S = slope

The second equation was developed by F. M. Chang for live bed and clear water scour as an envelope curve for maximum scour. It is as Follows:

$$Ys/a = 1.6\ K1\ K2\ (Y1/a)^{0.33}$$

with terms defined as above.

Both methods were used to compute scour for piers for the main channel. Computations were made for the 2, 5, 10, 25, 50, 100, and 500 year floods. The slopes used, came from the HEC-2 energy grade for the sections at the bridge. The velocity form of Mannings equation was used to compute the velocity.

$$V = (1.486/n)R^{2/3}\ S^{1/2}$$

n = Mannings roughness coefficient
R = Hydraulic radius, in this case equal to the depth

Computation Summary

Freq. (Yr.)	Elev. (Ft.)	Slope (ft/ft)	Depth (ft)	Vel. (ft/sec)	Fr #	Ys(R) (ft.)	Ymax(R) (ft.)	Ymax(C) (ft.)
2	48.41	.000066	26.41	2.92	.10	5.88	7.64	12.2
5	52.05	.000083	29.05	3.66	.12	6.56	8.53	12.6
10	52.68	.000096	30.68	4.08	.13	6.93	9.01	12.8
25	54.39	.000121	32.39	4.75	.15	7.45	9.69	13.1
50	55.85	.000137	33.85	5.20	.16	7.80	10.1	13.2
100	57.18	.000157	35.18	5.71	.17	8.16	10.6	13.4
500	59.45	.000237	37.45	7.32	.21	9.16	11.9	13.7

The Richardson equation seems to give a more realistic variation in scour with discharge. However, at the 500 year flood level, the difference is only 1.8 feet. The value of 13.7 feet from the Chang equation would be a more conservative estimate for design purposes. Using the Richardson equation, the elevation of the top of the footings could be set at elevation 10.0. The top of the footings should be at elevation 8.0 if the Chang equation is considered more appropriate. Borings taken near the proposed footing locations indicate that the firmer Black Creek formation begins at elevation 9.0 to 9.5. The footings should be set no higher than the top of this material. Based on the borings and the scour computations, the recommended footing elevation is 8.0.

Abutment scour was computed for the abutments at each end of both bridges using three different computation methods. The first two use a relationship developed by E.M. Laurson for clear water scour at abutments. Since the ends of the bridges are well away from the channel in a densely vegetated flood plain, the clear water equations should apply. The basic equation is:

$$\frac{a}{Y1} = 2.75\,\frac{Ys}{Y1}\left[\frac{\left(\frac{1}{11.5}\,\frac{Ys}{Y1} + 1\right)^{(7/6)}}{(\tau_1/\tau_c)^{(1/2)}} - 1\right]$$

Where: a = distance for abutment to the edge of the flood plain or to the flow divide between bridges.

τ_1 = γY1 S =Shear stress on the overbank area upstream of the abutment.

γ = Specific weight of water

τ_c = Critical shear stress on material in overbank area.

$$(\tau_1/\tau_c) = \frac{V1^{(2)}}{120\ Y1^{(1/3)}\ d50^{(2/30)}}$$

(This relationship is from Laursen's 1958 report on "Scour at Bridge Crossings".)

V1 = Velocity upstream from the abutment

All other variables are as defined previously.

A second computation method used the same equation but (τ_1/τ_c) was taken from a graph in Richardson's report. Both of these solutions of Laursen's equation require a trial and error solution. This was readily accomplished using a programmable calculator.

The third method uses an equation which Richardson recommended as a limiting value for a/Y1 > 25. This equation is:

$$Ys/Y1 = 4(Fr1)^{0.33}$$

The results of these computations are summarized below.
Note: Ys(1) are the results using the computed (τ_1/τ_c).
Ys(2) uses the graph value of (τ_1/τ_c).
Ys(3) uses the limiting equation.

Computation Summary
West End Main Bridge

Freq. (yr.)	Ys(1) (ft.)	Ys(2) (ft.)	Ys(3) (ft.)
2	1.5	1.1	8.4
5	3.2	1.9	11.7
10	4.6	2.4	13.8
25	7.1	3.2	16.5
50	9.6	3.9	18.4
100	13.3	4.9	20.4
500	23.9	7.8	24.2

Nearest sample d50 = .33mm, 18.4% passing #200 seive, graph value of (τ_1/τ_c) =.014.

East End Main Bridge

Freq. (yr.)	Ys(1) (ft.)	Ys(2) (ft.)	Ys(3) (ft.)
2	dry	--	--
5	0.7	1.2	1.9
10	1.9	2.5	3.7
25	4.6	4.5	5.9
50	7.5	6.4	7.6
100	12.4	9.3	9.2
500	28.0	12.2	13.0

Nearest boring sample 80% passed #200 seive for computation assume d50 = 0.074 mm, graph value of (τ_1/τ_c)= 0.075.

West End of Overflow Bridge

Freq. (yr.)	Ys(1) (ft.)	Ys(2) (ft.)	Ys(3) (ft.)
2	0.4	0.4	6.4
5	1.0	0.7	9.4
10	2.5	1.0	11.5
25	2.6	1.4	14.1
50	3.8	1.7	16.0
100	5.4	2.2	17.9
500	11.7	3.8	*

Nearest Boring sample d50 = .11 mm, 37.3% passing #200 seive, graph value of (τ_1/τ_c) = .035.
* a/Y1 = 23.7 > 25

East End of Overflow Bridge

Freq. (yr.)	Ys(1) (ft.)	Ys(2) (ft.)	Ys(3) (ft.)
2	3.0	2.7	6.4
5	5.7	4.1	9.4
10	8.3	5.2	11.5
25	12.9	7.2	14.1
50	17.6	8.9	16.0
100	23.9	11.0	17.9
500	42.4	17.5	17.9

Nearest sample d50 = .088 mm, 45.5% passing #200 seive, graph value of (τ_1/τ_c) = 0.05.

In view of the physical evidence at the site these computed values appear to be far too high. The maximum flood that has occurred since the construction of this crossing in 1947, was the 1979 flood with 103,000 cfs. This is approximately equivalent to the 25 year flood. There have been seven floods which equaled or exceeded the five year flood of 63,100 cfs in this period. As noted above, there is no evidence of scour at any of the abutments and there was no sign of any repairs to the abutments or to the flood plain at the toe of the abutments.

The discrepancy between the predicted scour and the apparent lack of any actual occurrence, indicates that the models used to make the prediction are extremely conservative or do not apply to a crossing of this nature. There are two possible reasons for the discrepancy. The first was suggested by Stanley R. Davis, FHWA Hydraulic Branch Chief, in a telephone conversation with the author of this report. In his work to develop the models, Laurson considered the bridge opening as a long constriction. Davis suggested that the long constriction model may not be applicable to bridge crossings of this nature. This may certainly be true in the case of this crossing of the Great Pee Dee River where the 100 foot long constriction is approximately 1% of the 11,000 foot wide flood plain.

The second reason for the discrepancy is due to the effects of the dense under growth. The laboratory models used in the research relied on sand beds in flumes to simulate the flood plain. This completely ignored the ability of the plant material to armor the soil and resist scour.

General scour in the over flow bridge was computed using Laurson's equation for clear water scour.

$$\frac{Y2}{Y1} = \left(\frac{W1}{W2}\right)^{6/7}\left[\frac{V1^{2}}{120\ Y1^{1/3}\ d50^{2/3}}\right]^{3/7}$$

Where $W1$ = the flood plain width to to flow divide between the two bridges.
$W2$ = the width of the bridge opening.
$Y2$ = the depth in the bridge including scour so that $Ys = Y2 - Y1$
The other variables are as defined previously.

Computation Summary for
General Scour in the Overflow Bridge
$D50 = 0.11$ mm

Freq. (Yr.)	W1 (ft.)	W2 (ft.)	Y1 (ft.)	Ys (ft.)
2	2492	544	10.0	1.7
5	2435	555	15.6	4.6
10	2442	562	17.4	4.8
25	2469	569	23.7	9.3
50	2483	575	29.8	14.0
100	2485	580	31.5	14.5
500	2507	589	45.1	25.6

Here again the computed values do not reflect field conditions. The same reasoning for the discrepancy applies.

Pier scour for the overflow bridge was computed using Chang's equation, since the overflow bridge will have clear water with no sediment supply. The value of "A" will be 1.5 feet, reflecting the 18 inch square prestressed piles. K1 will be 1.1 and K2 will be 1.0.

Computation Summary for
Pier Scour for Overflow Bridge

Freq. (yr.)	Ys (ft.)
2	4.7
5	5.1
10	5.3
25	5.6
50	5.7
100	5.9
500	6.2

These values may be acceptable since Chang's equation predicts the maximum scour that could occur. If there were no vegetation present, the predicted maximums may be reasonable.

Conclusions and Recommendations

The total scour that is predicted to occur within the bridges is the sum of all the different types of scour. But the results of the computations when compared with field conditions indicate that the only reasonably accurate predictions are for pier scour in both the channel and overflow areas. As long as the pile bearing is achieved in the Black Creek formation in the overflow areas and the footings in the channel are set at elevation 8.0, no significant scour should occur around the piers or bents. Riprap protection should be sufficient to protect the abutments.

Much research, including considerable field work, must be done before reliable scour predictions can be made. The effects of vegetation and debris accumulation should be investigated.

The author has observed abutment failure due to scour at a number of bridges during floods that have occurred in the 22 year period that he has been with the Department. Other observed scour failures were due to the effects caused by extremely high accumulations of debris. The ability of the current methods to predict abutment scour as shown by this study is not reliable.

APPENDIX G

SCOUR DETECTION EQUIPMENT

In the past scour measurements have been made by 3 methods: pole, leadline, and fathometer. In shallow water a pole with graduated markings is used while the lead line is used in the areas with deeper water. However these are difficult to use in channels with faster currents since the current tends to carry them downstream. The electronic depth finders (fathometers) are useful in the deeper, faster moving streams. Also many of these units are equipped with an internal recording device that will provide agraphic representation of the channel bottom. This feature can bea real time saver for plotting river bottom profiles and cross sections. Any one of these methods can be used to determine the configuration of the stream and measure existing scour.

Scour is most prevalent during a flood, which is the time when monitoring is most difficult. Although a number of different types of permanently installed scour meters are presently being evaluated, no economical and reliable meters of this type are currently available for general use. Obtaining scour measurements from the bridge or by boat during peak flood flows has not been widely attempted because of the hazardous conditions, complex flow patterns, presence of drift and debris and problems getting personnel to the bridge site during peak flow conditions.

Geophysical Tools

After a flood, the stream velocity decreases which may result in the sediment being redeposited in the scour hole, also referred to as infilling. Since this material often has a different density than the adjacent unscoured material, we can measure the true extent of scour by determining the interface where the density change occurs. Methods for determining this include standard penetration testing, cone penetrometer exploration and geophysical techniques. While standard penetration testing is accurate it is expensive, time consuming and does not provide a continuous profile. Less expensive geophysical methods are available however which will provide continuous subsurface profiles by providing information on the physical properties.

The three geophysical tools which can be used to measure scour after infilling occurs are: ground penetrating radar, tuned transducer and color fathometer. Each of these methods has it's advantages and limitations. However if applied properly they can yield meaningful data in a very short period of time.

The U.S. Geological Survey in cooperation with the Federal Highway Administration has used each of these tools to study the extent of scour and the findings are documented a report entitled "The Use of Surface Geophysical Methods in Studying River Bed Scour". The following descriptions are taken from that report by S.R. Gorin and F.P. Haeni of the U.S. Geological Survey.

Ground Penetrating Radar

Ground penetrating radar (GPR) can be used to obtain high resolution, continuous, subsurface profiles on land or in relatively shallow water (less than 25 feet). This device transmitters short, 80 to 800 MHz electromagnetic pulses into the subsurface and measures the two way travel time for the signal to return to the receiver. When the electromagnetic energy reaches an interface between two materials with differing physical properties, a portion of the energy is reflected back to the surface, whilesome of it is attenuated and a portion is transmitted to deeper layers. The penetration depth of GPR is dependent upon the electrical properties of the material through which the signal is transmitted and the frequency of the signal transmitted. Highly conductive (low resistivity) materials such as clay materials severely attenuate radar signals. Similarly, sediments saturated with or overlain by salt water will yield poor radar results. Fresh water also attenuates the radar signal and limits the use of radar to sites with less than 25 feet of water. The lower frequency signals yield better penetration and reduced resolution, where as higher freqency signals yield higher resolution and less penetration. Ground penetrating radar systems which include a transmitter, receiver, high density tape recorder and player for storage of records and antenna cost approximately $50,000.

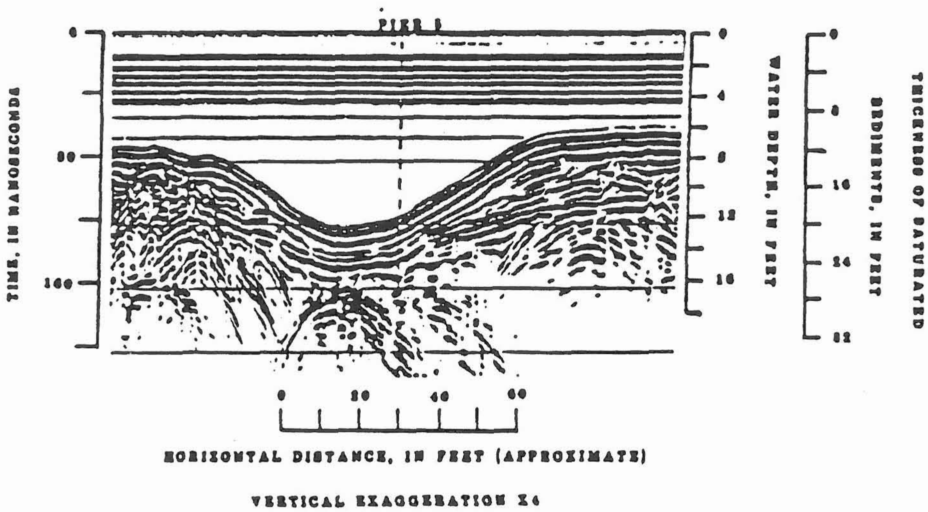


Figure 1 above shows a cross section generated by a ground penetrating radar signal upstream of a bridge pier. The scour hole is approximately 7 feet deeper than the river bottom base level and 60 to 70 feet wide. Two different infilled layers can be observed at this location. The apparent thickness of the infilled material at the center of the hole is 3 feet to the first interface and 6 feet to the second interface.

Tuned Transducer

The tuned transducer and the color fathometer are both seismic systems which operate through the transmission and reception of acoustic waves. A portion of the seismic signal is reflected back to the surface when there is a change in acoustical impedance between two layers. The major variable which separates these two devices from the fathometer is the frequency. The tuned transducer and color fathometer have lower frequency signals (3 20 KHz) which yield better penetration at the expense of resolution. High frequency fathometers (200 KHz) have good resolution with little or no penetration. In fine grained materials up to 100 feet of penetration can be obtained with a 3 to 7 KHz transducer, while in coarser material subsurface penetration may be limited to a few feet. The tuned transducer system cost approximately $25,000.

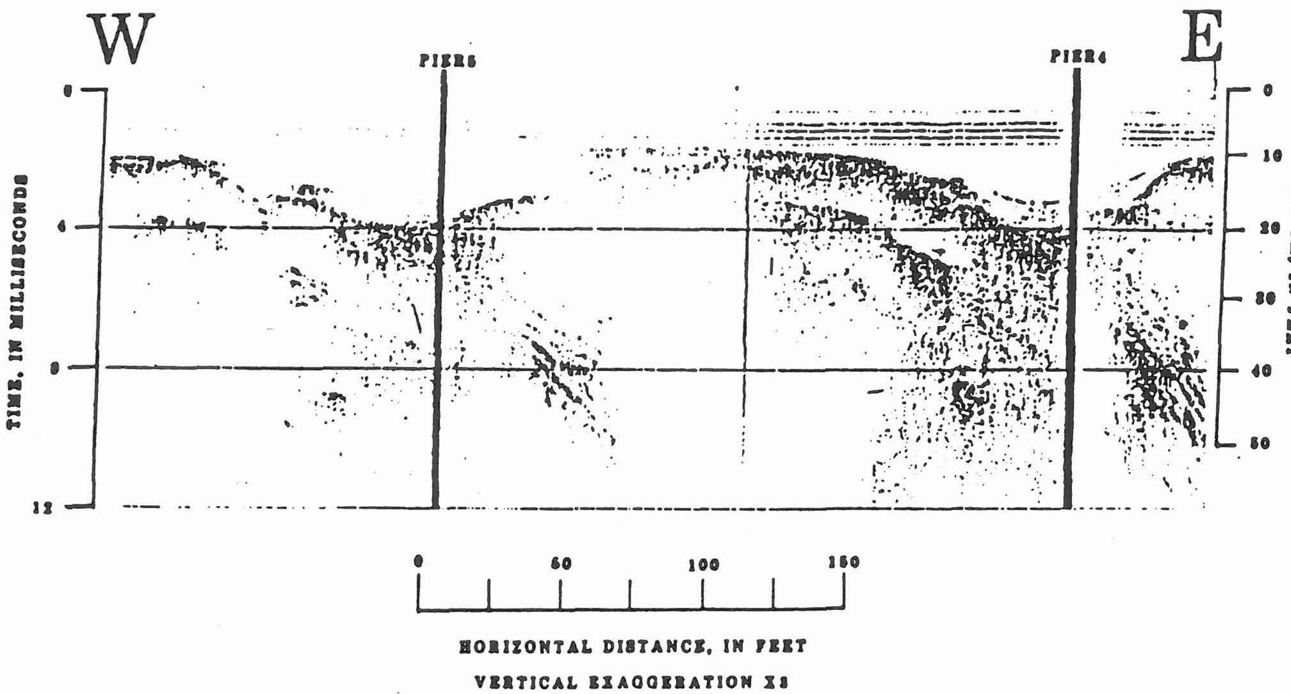


Figure 2 above shows a cross section record provided by a 14 KHz tuned transducer. This is the same location as the GPR record in figure 1. The record shows 6 feet of infilled material. The 2 layers which could be seen on the radar record are not evident on the tuned transducer record.

Color Fathometer

The color fathometer is a variable frequency seismic system that digitizes the reflected signal and displays a color image on a monitor. This system measures the reflected signal in decibels and it distinguishes between different interfaces by assigning color changes to a given degree of decibel change. Since decibel changes in the reflected signal are related to density, porosity and median grain size it is able to identify and define shallow interfaces in the subsurface. Where infilling has occurred the soft material iseasily penetrated and shown to have low reflectivity as opposed to denser materials which have high reflectivity. Typically the materials which have a low reflectivity are assigned the "cool"colors such as blue and green while the denser material is represented by the "hot" colors such as red and orange. Since the data is displayed on a color monitor a hard copy is not readily available, however it can be stored on a cassette tape for playback and processing. The U.S. Geological Survey is presently working on developing a computer program to process the color fathometer record in order to remove some the extraneous and undesirable signals which make interpretation more difficult.

Black and White Fathometer

Even though the black and white fathometer is unable to penetrate the channel except in very soft mud, it is still considered an excellent tool for defining the channel bottom. The graphic recorder is easy to use, reasonably inexpensive and will provide an accurate bottom profile very quickly. Also when used in conjunction with the other tools it adds a degree of certainty to the other geophysical data. A 200 KHz fathometer with graphics capabilities can be purchased for approximately $1000.

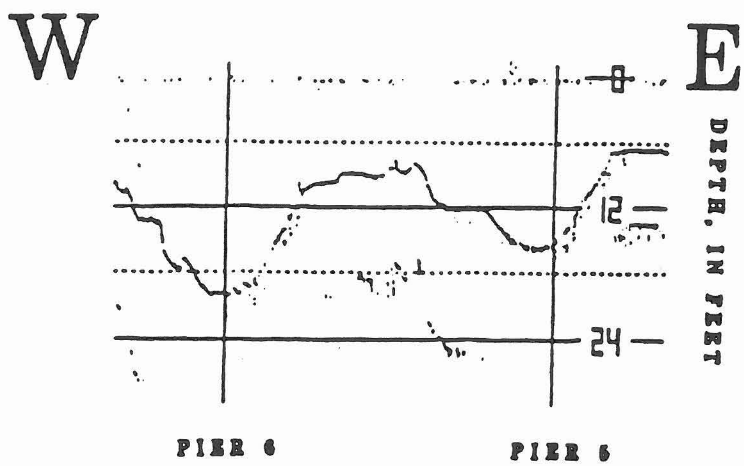


Figure 3 above shows a cross section using a 200 KHz fathometer. This record correlates with the radar and tuned transducer record shown in Figures 1 and 2 with the exception that the radar record was run 6 feet further upstream.

The FHWA Demonstration Projects Division is developing a project to demonstrate each of the devices discussed. This project entitled "Demonstration Project No. 80 Bridge Inspection Techniques and Equipment" will give participants an opportunity to view andparticipate in the operation of these and other underwater inspection equipment. Questions concerning this project can be directed to Dennis Decker at 202-366-1131.

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Stream Stability at Highway Structures

Research, Development, and Technology
Turner-Fairbank Highway Research Center
6300 Georgetown Pike
McLean, Virginia 22101-2296

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16. Abstract

This document provides guidelines for identifying stream instability problems at highway stream crossings and for the selection and design of appropriate countermeasures to mitigate potential damages to bridges and other highway components at stream crossings. The HEC-20 manual covers geomorphic and hydraulic factors that affect stream stability and provides a step-by-step analysis procedure for evaluating stream stability problems. Guidelines and criteria for selecting countermeasures for stream stability problems are summarized, and the design of three countermeasures (spurs, guide banks, and check dams) is presented in detail. Conceptual design considerations for many other countermeasures are summarized.

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GLOSSARY

abrasion:	Removal of streambank material due to entrained sediment, ice, or debris rubbing against the bank.
afflux:	Backwater; the increase in water surface elevation upstream of a bridge relative to the elevation occurring under natural conditions.
aggradation:	General and progressive buildup of the longitudinal profile of a channel bed due to sediment deposition.
alluvial channel:	Channel wholly in alluvium; no bedrock is exposed in channel at low flow or likely to be exposed by erosion.
alluvial fan:	A fan-shaped deposit of material at the place where a stream issues from a narrow valley of high slope onto a plain or broad valley of low slope. An alluvial cone is made up of the finer materials suspended in flow while a debris cone is a mixture of all sizes and kinds of materials.
alluvial stream:	A stream which has formed its channel in cohesive or non-cohesive materials that have been and can be transported by the stream.
alluvium:	Unconsolidated material deposited in water by a stream.
alternating bars:	Elongated deposits found alternately near the right and left banks of a channel.
anabranch:	Individual channel of an anabranched stream.
anabranched stream:	A stream whose flow is divided at normal and lower stages by large islands or, more rarely, by large bars; individual islands or bars are wider than about three times water width; channels are more widely and distinctly separated than in a braided stream.
apron:	Protective material laid on a streambed to resist scour.
apron, launching:	An apron designed to settle and protect the side slopes of a scour hole after settlement.
armor:	Surfacing of channel bed, banks, or embankment slope to resist erosion and scour.
armoring:	(a) Natural process whereby an erosion-resistant layer of relatively large particles is formed on a streambed due to the removal of finer particles by stream flow; (b) placement of a covering to resist erosion.

articulated concrete mass:	Rigid concrete slabs which can move without separating as scour occurs; usually hinged together with corrosion-resistant wire fasteners; primarily placed for lower bank protection.
average velocity:	Velocity at a given cross section determined by dividing discharge by cross sectional area.
avulsion:	A sudden change in the channel course that usually occurs when a stream breaks through its banks; usually associated with a flood or a catastrophic event.
backwater:	The increase in water surface elevation relative to the elevation occurring under natural channel and flood plain conditions, induced by a bridge or other structure that obstructs or constricts a channel. Backwater also can occur downstream of a constriction where flow expands, as in wide, wooded flood plain.
backwater area:	The low-lying lands adjacent to a stream that may become flooded due to backwater.
bank:	The side slopes of a channel between which the flow is normally confined.
bank, left (right):	The side of a channel as viewed in a downstream direction.
bank full discharge:	Discharge that, on the average, fills a channel to the point of overflowing.
bank protection:	Engineering works for the purpose of protecting streambanks from erosion.
bank revetment:	Erosion-resistant materials placed directly on a streambank to protect the bank from erosion.
bar:	An elongated deposit of alluvium within a channel, not permanently vegetated.
bed:	The bottom of a channel bounded by banks.
bed form:	A recognizable relief feature on the bed of a channel, such as a ripple, dune or bar.
bed layer:	A flow layer, several grain diameters thick (usually two) immediately above the bed.
bed load:	Sediment that is transported in a stream by rolling, sliding, or skipping along the bed or very close to it; considered to be within the bed layer.

bed load discharge (or bed load): The quantity of bed load passing a cross section of a stream in a unit of time.

bed material: Material found on the bed of a stream (May be transported as bed load or in suspension).

bedrock: The solid rock underlying soils and overlying the mantle rock, ranging from surface exposure to depths of several hundred feet.

bed shear (tractive force): The force per unit area exerted by a fluid flowing past a stationary boundary.

braid: A subordinate channel of a braided stream.

braided stream: A stream whose flow is divided at normal stage by small mid-channel bars or small islands; the individual width of bars and islands is less than about three times water width; a braided stream has the aspect of a single large channel within which are subordinate channels.

bridge opening: The cross-sectional area beneath a bridge that is available for conveyance of water.

bridge waterway: The area of a bridge opening available for flow, as measured below a specified stage and normal to the principal direction of flow.

bulkhead: A vertical, or near vertical, wall that supports a bank or an embankment; also may serve to protect against erosion.

caving: The collapse of a bank caused by undermining due to the action of flowing water. Also, the falling in of the concave side of a bend of which the curvature is changing.

channel: The bed and banks that confine the surface flow of a stream.

channelization: Straightening or deepening of a natural channel by artificial cutoffs, grading, flow-control measures, or diversion of flow into a man-made channel.

cellular-block mattress: Regularly cavitated interconnected concrete blocks placed directly on a streambank or filter to resist erosion. The cavities can permit bank drainage and the growth if vegetation where synthetic filter fabric is not used between the mattress and bank.

channel diversion: The removal of flows by natural or artificial means from a natural length of channel.

channel pattern: The aspect of a stream channel in plan view, with particular reference to the degree of sinuosity, braiding, or anabranching.

channel process:	Behavior of a channel with respect to shifting, erosion and sedimentation.
check dam:	A low dam or weir across a channel used to control stage or degradation.
choking (of flow):	Severe backwater effect resulting from excessive constriction.
cobble:	A fragment of rock whose diameter is in the range of 64 to 250 mm.
concrete revetment:	Plain or reinforced concrete slabs placed on the channel bed to protect it from erosion.
confluence:	The junction of two or more streams.
constriction:	A natural or artificial control section, such as a bridge crossing, channel reach or dam, with limited flow capacity in which the upstream water surface elevation is related to discharge.
contact load:	Sediment particles that roll or slide along in almost continuous contact with the stream bed.
contraction:	The effect of channel constriction on flow streamlines.
countermeasure:	A measure intended to prevent, delay or reduce the severity of hydraulic problems.
contraction scour:	See General Scour.
Coriolis force:	The inertial force caused by the earth's rotation that deflects a moving body to the right in the Northern Hemisphere.
crib:	A frame structure filled with earth or stone ballast, designed to reduce energy and to deflect stream flow away from a bank or embankment.
critical shear stress:	The minimum amount of shear stress required to initiate soil particle motion.
crossing:	The relatively short and shallow reach of a stream between bends; also crossover.
cross section:	A section normal to the trend of a channel.
current:	Water flowing through a channel.
cut bank:	The concave wall of a meandering stream.

cutoff: (a) A direct channel, either natural or artificial, connecting two points on a stream, thereby shortening the original length of the channel and increasing its slope; (b) A natural or artificial channel which develops across the neck of a meander loop.

cutoff wall: A wall, usually of sheet piling or concrete, that extends down to scour-resistant material or below the expected scour depth.

daily discharge: Discharge averaged over one day.

debris: Floating or submerged material, such as logs or trash, transported by a stream.

deflector: Alternative term of "spur."

degradation (bed): A general and progressive lowering of the channel bed due to scour.

density of water-sediment mixture: Bulk density (mass per unit volume) including both water and sediment.

depth of scour: The vertical distance a streambed is lowered by scour below a reference elevation.

dike: An impermeable linear structure for the control or containment of overbank flow. A dike trending parallel with a streambank differs from a levee in that it extends for a much shorter distance along the bank, and it may be surrounded by water during floods.

dike (groin, spur, jetty): A structure extending from a bank into a channel that is designed to: (a) reduce the stream velocity as the current passes through the dike, thus encouraging sediment deposition along the bank (permeable dike); or (b) deflect erosive currents away from the stream bank (impermeable dike).

dominant discharge: (a) The discharge which is of sufficient magnitude and frequency to have a dominating effect in determining the characteristics and size of the stream course, channel and bed. (b) That discharge which determines the principal dimensions and characteristics of a natural channel. The dominant formative discharge depends on the maximum and mean discharge, duration of flow, and flood frequency. For hydraulic geometry relationships, it is taken to be the bank full discharge which has a return period of approximately 1.5 years in many natural channels.

drift: Alternative term for "debris."

eddy current: A vortex-type motion of a fluid flowing contrary to the main current, such as the circular water movement that occurs when the main flow becomes separated from the bank.

entrenched stream:	Stream cut into bedrock or consolidated deposits.
emphemeral stream:	A stream or reach of stream that does not flow for parts of the year. As used here, the term includes intermittent streams with flow less than perennial.
erosion:	Displacement of soil particles on the land surface due to water or wind action.
erosion control matting:	Fibrous matting (e.g., jute, paper, etc.) placed or sprayed on a streambank for the purpose of resisting erosion or providing temporary stabilization until vegetation is established.
estuary:	Tidal reach at the mouth of a stream.
Fabriform:	Grout-filled fabric mattress used for stream bank protection.
fetch:	The area in which waves are generated by wind having a rather constant direction and speed; sometimes used synonymously with fetch length.
fetch length:	The horizontal distance (in the direction of the wind) over which wind generates waves and wind setup.
fill slope:	Side or end slope of an earth fill embankment.
filter:	Layer of fabric, sand, gravel, or graded rock placed between bank revetment and soil for one or more of three purposes: (1) to prevent the soil from moving through the revetment by piping, extrusion, or erosion; (2) to prevent the revetment from sinking into the soil; (3) and to permit natural seepage from the streambank, thus preventing the buildup of excessive hydrostatic pressure.
filter blanket:	A layer of graded sand and gravel laid between fine-grained material and riprap to prevent wash-out of the finer material.
filter cloth:	Fabric of synthetic plastic strands that serves the same purpose as a granular filter blanket.
fine sediment load (or wash load):	That part of the total sediment load that is composed of particle sizes finer than those represented in the bed. Normally, the fine-sediment load is finer than 0.062 mm for a sand-bed channel. Silts, clays and sand could be considered wash load in coarse gravel and cobble bed channels.
flanking:	Erosion resulting from stream flow between the bank and the landward end of a countermeasure for stream stabilization.

flashy stream:	Stream characterized by rapidly rising and falling stages, as indicated by a sharply peaked hydrograph. Most flashy streams are ephemeral, but some are perennial.
flood plain:	A nearly flat, alluvial lowland bordering a stream, that is subject to inundation by floods.
flow-control structure:	A structure either within or outside a channel that acts as a countermeasure by controlling the direction, depth, or velocity of flowing water.
flow hazard:	Flow characteristics (discharge, stage, velocity, or duration) that are associated with a hydraulic problem or that can reasonably be considered of sufficient magnitude to cause a hydraulic problem or to test the effectiveness of a countermeasure.
flow slide:	Saturated soil materials which behave more like a liquid than a solid. A flow slide on a channel bank can result in a bank failure.
Froude number:	A dimensionless number that represents the ratio of inertial to gravitational forces. High Froude numbers can be indicative of high flow velocity and the potential for scour.
gabion:	A basket or compartmented rectangular contained made of steel wire mesh. When filled with cobbles or other rock of suitable size, the gabion becomes a flexible and permeable block with which flow-control structures can be built.
general (contraction) scour:	Scour in a channel or on a flood plain that is not localized at a pier, abutment, or other obstruction to flow. In a channel, general/contraction scour usually affects all or most of the channel width.
geomorphology/ morphology:	That branch of both physiography and geology that deals with the form of the earth, the general configuration of its surface, and the changes that take place due to erosion of the primary elements and in the buildup of erosional debris.
grade-control structure (sill, check dam):	Structure placed bank to bank across a stream channel (usually with its central axis perpendicular to flow) for the purpose of controlling bed slope and preventing scour or head-cutting.
graded stream:	A geomorphological term used for streams that have apparently achieved a state of equilibrium between the rate of sediment transport and the rate of sediment supply throughout long reaches.

groin:	A structure built from the bank of a stream in a direction transverse to the current. Many names are given to this structure, the most common being "spur," "spur dike," "transverse dike," "jetty," etc. Groins may be permeable, semi-permeable or impermeable.
guide bank:	Preferred term for spur dike.
hardpoint:	A streambank protection structure whereby "soft" or erodible materials are removed from a bank and replaced by stone or compacted clay. Some hard points protrude a short distance into the channel to direct erosive currents away from the bank. Hard points also occur naturally along streambanks as passing currents remove erodible materials leaving nonerodible materials exposed.
headcutting:	Channel degradation associated with abrupt changes in the bed elevation (headcut) that migrates in an upstream direction.
helical flow:	Three-dimensional movement of water particles along a spiral path in the general direction of flow. These secondary-type currents are of most significance as flow passes through a bend; their net effect is to remove soil particles from the cut bank and deposit this material on the point bar.
hydraulic radius:	The cross-sectional area of a stream divided by its wetted perimeter.
hydraulic problem:	An effect of stream flow, tidal flow, or wave action such that the integrity of the highway facility is destroyed, damaged, or endangered.
incised reach:	A stretch of stream with an incised channel that only rarely overflows its banks.
incised stream:	A stream which has cut its channel through the bed of the valley floor, as opposed to one flowing on a flood plain.
island:	A permanently vegetated area, emergent at normal stage, that divides the flow of a stream. Islands originate by establishment of vegetation on a bar, by channel avulsion, or at the junction of minor tributary with a larger stream.
jack:	A device for flow control and protection of banks against lateral erosion consisting of three mutually perpendicular arms rigidly fixed at the center. Kellner jacks are made of steel struts strung with wire, and concrete jacks are made of reinforced concrete beams.
jack field:	Rows of jacks tied together with cables, some rows generally parallel with the banks and some perpendicular thereto or at an angle. Jack fields may be placed outside or within a channel.

jetty:	(a) An obstruction built of piles, rock, or other material extending from a bank into a stream, so placed as to induce scouring or bank building, or to protect against erosion. (b) A similar obstruction to influence stream, lake, or tidal currents, or to protect a harbor.
lateral erosion:	Erosion in which the removal of material is extended in a lateral direction, as contrasted with degradation and scour in a vertical direction.
launching:	Release of undercut material (stone riprap, rubble, slag, etc.) downslope or into a scoured area.
levee:	An embankment, generally landward of top bank, that confines flow during high water periods, thus preventing overflow into lowlands.
littoral drift:	The transport of material along a shoreline.
local scour:	Scour in a channel or on a flood plain that is localized at a pier, abutment, or other obstruction to flow.
lower bank:	That portion of a streambank having an elevation less than the mean water level of the stream.
mattress:	A blanket or revetment materials interwoven or otherwise lashed together and placed to cover an area subject to scour.
meander or full meander:	A meander in a river consists of two consecutive loops, one flowing clockwise and the other anti-clockwise.
meander belt:	The distance between lines drawn tangent to the extreme limits of successive fully developed meanders.
meander length:	The distance along a stream between corresponding points at the extreme limits of successive fully developed meanders.
meander loop:	An individual loop of a meandering or sinuous stream lying between inflection points with adjoining loops.
meander ratio:	The ratio of meander width to meander length.
meander width:	The amplitude of swing of a fully developed meander measured from midstream to midstream.
meandering:	A stream which follows a sinuous path due to natural physical causes not imposed by external restraint, and is characterized by curved flow and alternating shoals and bank erosion.

meandering channel:	A channel exhibiting a characteristic process of bank erosion and point bar deposition associated with systematically shifting meanders.
meander scrolls:	Low, concentric ridges and swales on a flood plain, marking the successive positions of former meander loops.
meandering stream:	A stream having a sinuosity greater than some arbitrary value. The term also implies a moderate degree of pattern symmetry, imparted by regularity of size and repetition of meander loops.
median diameter:	The particle diameter of the 50 percentile point on a size distribution curve such that half of the particles (by weight for samples of sand, silt, or clay and by number for samples of gravel) are larger and half are smaller.
mid-channel bar:	A bar lacking permanent vegetal cover that divides the flow in a channel at normal stage.
middle bank:	The portion of a streambank having an elevation approximately the same as that of the mean water level of the stream.
migration:	Change in position of a channel by lateral erosion of one bank and simultaneous accretion of the opposite bank.
natural levee:	A low ridge formed along streambanks during floods by deposition that slopes gently away from the channel banks.
nominal sediment diameter:	Equivalent speherical diameter of a hypothetical sphere of the same volume as a given stone.
nonalluvial channel:	A channel whose boundary is completely in bedrock.
normal stage:	The water stage prevailing during the greater part of the year.
overbank flow:	Water movement over top bank either due to stream stage or to inland surface water runoff.
oxbow:	The abandoned bow-shaped or horseshoe-shaped reach of a former meander loop that remains after a stream cuts a new, shorter channel across the narrow neck between closely approaching bends of a meander.
perennial stream:	A stream or reach of a stream that flows continuously for all or most of the year.
phreatic line:	The upper boundary of the seepage water surface landward of a streambank.

pile dike:	A type of permeable structure for the protection of banks against caving; consists of a cluster of piles driven into the stream, braced and lashed together.
piping:	Removal of soil material through subsurface flow of seepage water that develops channels or "pipes" within the soil bank.
point bar:	An alluvial deposit of sand or gravel lacking permanent vegetal cover occurring in a channel at the inside of a meander loop, usually somewhat downstream from the apex of the loop.
poised stream (stable stream):	A stream which, as a whole, maintains its slopes, depths, and channel dimensions without any noticeable raising or lowering of its bed. Such condition may be temporary from a geological point of view, but for practical engineering purposes, the stream may be considered stable.
quarry-run stone:	Stone as received from a quarry without regard to gradation requirements.
railbank protection:	A type of countermeasure composed of rock-filled wire fabric supported by steel rails or posts driven into the streambed.
rapid drawdown:	Lowering the water against a bank more quickly than the bank can drain without becoming unstable.
reach:	A segment of stream length that is arbitrarily bounded for purposes of study.
regime:	The condition of a stream or its channel as regards stability. A stream is in regimen (regime) if its channel has reached a stable form as a result of its flow characteristics. According to Lacey, a regime channel is a stable channel in incoherent alluvium and transporting a regime silt charge. A "regime silt charge" is the maximum transported load consistent with a fully active bed. Full activity is such that any reduction would lead to partial rigidity, and at the limit to complete rigidity and immobility of the bed. Silt is understood as sediment or detritus.
regime channel:	Alluvial channel that has attained, more or less, a state of equilibrium with respect to erosion and deposition.
regime change:	A change in channel characteristics resulting from such things as changes in imposed flows, sediment loads or slope.
regime formula:	A formula relating stable alluvial channel dimensions or slope to discharge and sediment characteristics.

reinforced-earth bulkhead:	A retaining structure consisting of vertical panels and attached to reinforcing elements embedded in compacted backfill for supporting a streambank.
reinforced revetment:	A streambank protection method consisting of a continuous stone toe-fill along the base of a bank slope with intermittent fillets of stone placed perpendicular to the toe and extending back into the natural bank.
retard (retarder structure):	A permeable or impermeable linear structure in a channel, parallel with the bank and usually at the toe of the bank, intended to reduce flow velocity, induce deposition, or deflect flow from the bank.
revetment:	Rigid or flexible armor placed to inhibit scour and lateral erosion. (See bank revetment).
riffle:	A natural, shallow flow area extending across a streambed in which the surface of flowing water is broken by waves or ripples. Typically, riffles alternate with pools along the length of a stream channel.
riparian:	Pertaining to anything connected with or adjacent to the banks of a stream.
riprap:	In the restricted sense, layer or facing of broken rock or concrete dumped or placed to protect a structure or embankment from erosion; also the broken rock or concrete suitable for such use. Riprap has also been applied to almost all kinds of armor, including wire-enclosed riprap, grouted riprap, sacked concrete, and concrete slabs.
river training:	Engineering works with or without the construction of embankment, built along a stream or reach of stream to direct or to lead the flow into a prescribed channel.
river training structure:	Any structure configuration constructed in a stream or placed on, adjacent to, or in the vicinity of a streambank that is intended to deflect currents, induce sediment deposition, induce scour, or in some other way alter the flow and sediment regimes of the stream.
rock-and-wire mattress:	A flat or cylindrical wire cage or basket filled with stone or other suitable material and placed as protection against erosion.
roughness coefficient:	Numerical measure of the frictional resistance to flow in a channel, as in the Manning and Strikler formulas.
rubble:	Rough, irregular fragments of materials of random size used to retard erosion. The fragments may consist of broken concrete slabs, masonry, or other suitable refuse.

sack revetment:	Sacks (e.g., burlap, paper, or nylon) filled with mortar, concrete, sand, stone or other available material used as protection against erosion.
saltation load:	Sediment bounced along the streambed by energy, turbulence of flow, and by other moving particles.
scour:	Erosion due to flowing water; usually considered as being localized as opposed to general bed degradation.
scoured depth:	Total depth of the water from water surface to a scoured bed level (compare with "depth of scour").
sediment or fluvial sediment:	Fragmental material transported, suspended, or deposited by water.
sediment concentration:	Weight or volume of sediment relative to quantity of transporting or suspending fluid or fluid-sediment mixture.
sediment discharge:	The quantity of sediment that is carried past any cross section of a stream in a unit of time. Discharge may be limited to certain sizes of sediment or to a specific part of the cross section.
sediment load:	Amount of sediment being moved by a stream.
sediment yield:	The total sediment outflow from a watershed or a drainage area at a point of reference and in a specified time period. This outflow is equal to the sediment discharge from the drainage area.
seepage:	The slow movement of water through small cracks and pores of the bank material.
seiche:	Long-period oscillation of a lake or similar body of water.
set-up:	Raising of water level due to wind action.
shallow water (for waves):	Water of such a depth that waves are noticeably affected by bottom conditions; customarily, water shallower than half the wavelength.
shoal:	A submerged sand bank. A shoal results from natural deposition on a streambed which has resisted all erosion; thus, the water is of necessity compelled to pass over it.
sill:	(a) A structure built under water, across the deep pools of a stream with the aim of changing the depth of the stream. (b) A low structure built across an effluent stream, diversion channel or outlet to reduce flow or prevent flow until the main stream stage reaches the crest of the structure.

sinuosity:	The ratio between the thalweg length and the valley length of a sinuous stream.
slope (of channel or stream):	Fall per unit length along the channel centerline.
slope protection:	Any measure such as riprap, paving, vegetation, revetment, brush or other material intended to protect a slope from erosion, slipping or caving, or to withstand external hydraulic pressure.
sloughing:	Sliding of overlying material; same ultimate effect as caving, but usually occurs when a bank or an underlying stratum is saturated.
slope-area method:	A method of estimating unmeasured flood discharges in a uniform channel reach using observed high-water levels.
slump:	A sudden slip or collapse of a bank, generally in the vertical direction and confined to a short distance, probably due to the substratum being washed out or having become unable to bear the weight above it.
soil-cement:	A designed mixture of soil and Portland cement compacted at a proper water content to form a veneer or structure that can resist erosion.
sorting:	Progressive reduction of size (or weight) of particles of the load carried down a stream.
spatial concentration:	The dry weight of sediment per unit volume of water-sediment mixture in place or the ratio of dry weight of sediment or total weight of water-sediment mixture in a sample or unit volume of the mixture.
spillthrough abutment:	A bridge abutment having a fill slope on the streamward side.
spur:	A permeable or impermeable linear structure that projects into a channel from the bank to alter flow direction, induce deposition, or reduce flow velocity along the bank.
spur dike/guide bank:	A dike extending upstream from the approach embankment at either or both sides of the bridge opening. Guide banks may also extend downstream from the bridge.
stability:	A condition of a channel when, though it may change slightly at different times of the year as the result of varying conditions of flow and sediment charge, there is no appreciable change from year to year; that is, accretion balances erosion over the years.

stable channel:	A condition that exists when a stream has a bed slope and cross section which allows its channel to transport the water and sediment delivered from the upstream watershed without aggradation, degradation, or bank erosion.
stage:	Water-surface elevation of a stream with respect to a reference elevation.
stone riprap:	Natural cobbles, boulders, or rock dumped or placed as protection against erosion.
stream:	A body of water that may range in size from a large river to a small rill flowing in a channel. By extension, the term is sometimes applied to a natural channel or drainage course formed by flowing water whether it is occupied by water or not.
streambank erosion:	Removal of soil particles or a mass of particles from a bank surface due primarily to water action. Other factors such as weathering, ice and debris abrasion, chemical reactions, and land use changes may also directly or indirectly lead to bank erosion.
streambank failure:	Sudden collapse of a bank due to an unstable condition such as due to removal of material at the toe of the bank by scour.
streambank protection:	Any technique used to prevent erosion or failure of a streambank.
suspended sediment discharge:	The quantity of suspended sediment passing through a stream cross section above the bed layer in a unit of time.
sub-bed material:	Material underlying that portion of the streambed which is subject to direct action of the flow.
submeander:	A small meander contained within the banks of a perennial stream channel. These are caused by relatively low discharges after the flood has subsided.
subcritical, supercritical flow:	Open channel flow conditins with Froude number less than and greater than unity, respectively.
tetrahedron:	Component of river-training works made of six steel or concrete struts fabricated in the shape of a pyramid.
tetrapod:	Bank protection component of precast concrete consisting of four legs joined at a central joint, with each leg making an angle of 109.5 degrees with the other three.
thalweg:	The line extending down a channel that follows the lowest elevation of the bed.

tieback:	Structure placed between revetment and bank to prevent flanking.
timber or brush mattress:	A revetment made of brush, poles, logs, or lumber interwoven or otherwise lashed together. The completed mattress is then placed on the bank of a stream and weighted with ballast.
toe of bank:	That portion of a stream cross section where the lower bank terminates and the channel bottom or the opposite lower bank begins.
toe protection:	Loose stones laid or dumped at the toe of an embankment, groin, etc., or masonry or concrete wall built at the junction of the bank and the bed in channels or at extremities of hydraulic structures to counteract erosion.
total sediment load (or total load):	The sum of suspended load and bedload or the sum of bed material load and washload of a stream.
trench-fill revetment:	Stone, concrete, or masonry material placed in a trench dug behind and parallel to an eroding streambank. When the erosive action of the stream reaches the trench, the material placed in the trench armors the bank and thus retards further erosion.
turbulence:	Motion of fluids in which local velocities and pressures fluctuate irregularly in a random manner as opposed to laminar flow where all particles of the fluid move in distinct and separate lines.
uniform flow:	Flow of constant cross section and velocity through a reach of channel at a given instant. Both the energy slope and the water slope are equal to the bed slope under conditions of uniform flow.
unit discharge:	Discharge per unit width (may be average over a cross section, or local at a point).
unit shear force (shear stress):	The force or drag developed at the channel bed by flowing water. For uniform flow, this force is equal to a component of the gravity force acting in a direction parallel to the channel bed on a unit wetted area. Usually expressed in units of stress, lb/ft^2.
unsteady flow:	Flow of variable cross section and velocity with respect to time.
upper bank:	The portion of a streambank having an elevation greater than the average water level of the stream.
velocity:	The rate of motion in a fluid on a stream or of the objects or particles transported therein, usually expressed in ft/s.
velocity-weighted sediment concentration:	The dry weight of sediment discharged through a cross section during unit time.

wandering channel:	A channel exhibiting a more or less non-systematic process of channel shifting, erosion and deposition, with no definite meanders or braided pattern.
wandering thalweg:	A thalweg whose position in the channel shifts during floods and typically serves as an inset channel that conveys all or most of the stream flow at normal or lower stages.
wash load:	Suspended material of very small size (generally clays and colloids) originating primarily from erosion on the land slopes of the drainage area and present to a negligible degree in the bed itself.
waterway opening width (area):	Width (area) of bridge opening at (below) a specified stage, measured normal to the principal direction of flow.
weephole:	A hole in an impermeable wall or revetment to relieve the neutral stress or pore pressure in the soil.
windrow revetment:	A row of stone placed landward of the top of an eroding streambank. As the windrow is undercut, the stone is launched downslope, thus armoring the bank.
wire mesh:	Wire woven to form a mesh; where used as an integral part of a countermeasure, openings are of suitable size and shape to enclose rock or broken concrete or to function on fence-like spurs and retards.

LIST OF SYMBOLS

A_{n2} = Cross-sectional flow area at bridge opening at normal stage (ft^2)

b = Length of the bridge opening (ft)

C_d = coefficient of drag

D_c = diameter of sediment particle at incipient motion conditions (ft)

D_i = the ith percentile size of bed material

ΔZ = the difference in water surface elevation between the concave and convex banks (ft)

D_{50} = the median sediment size

d_m = Tail water depth (ft)

d_s = Local scour depth for free overfall (ft)

F = the impact imparted by the debris (lbs/ft)

F_d = drag force per unit of length (lbs/ft)

g = acceleration of gravity (ft/s^2)

H = depth of submergence (ft)

H_t = Total drop in head, measured from the upstream to the downstream energy grade line (ft)

K = constant

L = the effective length of spur, or the distance between arcs describing the toe of spurs and the desired bank line (ft)

L_s = Projected length of guide bank

M = the mass of the debris (lb/sec^2/ft)

m = roughness correction factor for sinuosity of the channel

$n =$ = Manning's roughness coefficient

n_b = the base value for 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

P_c = decimal fraction of material coarser than the armoring size

Q = the discharge, total discharge (ft^3/sec)

Q_f = Lateral or floodplain discharge (cfs)

$\frac{Q_f}{Q_{100}}$ = Guide bank discharge ratio

Q_s = the sediment discharge (ft^3/sec)

Q_{100} = Discharge in 100 feet of stream adjacent to the abutment (cfs)

q = Discharge per unit width (cfs/foot)

R = hydraulic radius (ft)

r_c = radius of the center of the stream (ft)

r_i = radius of the inside bank (ft)

r_o = radius of the outside bank at the bend (ft)

S = stopping distance (ft)

S = the energy slope or channel slope (ft/ft)

S = the spacing between spurs at the toe (ft)

V = velocity or average velocity of flow (ft/sec)

V_{n2} = Average velocity through the bridge opening (cfs)

Y = Depth (ft)

y_a = thickness of the armoring layer

Z = Bed elevation referenced to a common datum (ft)

γ = specific weight of water (lb/ft^3)

γ_s = specific weight of sediment (lb/ft^3)

ρ = density of water ($slugs/ft^3$)

τ = boundary shear stress (lb/ft^2)

θ = the expansion angle downstream of spur tips (degrees)

1.0 INTRODUCTION

1.1 Purpose

The purpose of this document is to provide guidelines for identifying stream instability problems at highway-stream crossings and for the selection and design of appropriate countermeasures to mitigate potential flood damages to bridges and other highway components at stream crossings.

1.2 Background

Approximately 86 percent of the 577,000 bridges in the National Bridge Inventory (NBI) are built over streams. A large proportion of these bridges span alluvial streams that are continually adjusting their beds and banks. Many, especially those on more active streams, will experience problems with scour and bank erosion during their useful life. The magnitude of these problems is demonstrated by the average annual flood damage repair costs of approximately $50 million for highways on the Federal-aid system. The Federal-aid system contains less than half of all bridges in the NBI.

1.3 Factors that Affect Stream Stability

Factors which affect stream stability and, potentially, bridge stability at highway stream crossings can be classified as geomorphic factors and hydraulic factors. Rapid and unexpected changes can occur in streams in response to man's activities in the watershed and/or natural disturbances of the fluvial system, making it important to anticipate changes in channel geomorphology, location and behavior. Geomorphic characteristics of particular interest to the highway engineer are the alignment, geometry, and form of the stream channel. The behavior of a stream at a highway crossing depends not only on the apparent stability of the stream at the bridge, but also on the behavior of the stream system of which it is a part. Upstream and downstream changes may affect future stability at the site. Natural disturbances such as floods, drought, earthquakes, landslides, forest fires, etc., may result in large changes in sediment load in a stream and major changes in the stream channel. These changes can be reflected in aggradation, degradation, or lateral migration of the stream channel.

The bed material of a stream can be a cohesive material, sand, gravel, cobbles, boulders, or bedrock. Bank material is also composed of these materials and may be dissimilar from the bed material. Obviously, the stability and the rate of change in a stream is dependent on the material in the bed and banks.

Man-made changes in the drainage basin and the stream channel, such as alteration of vegetative cover and changes in pervious (or impervious) area can alter the hydrology of a stream, sediment yield and channel geometry. Channelization, stream channel straightening, streamside levees and dikes, bridges and culverts, reservoirs, gravel mining, and changes in land use can have major effects on stream flow, sediment transport, and channel geometry and location. Geomorphic factors are discussed in Chapter 2.0.

Hydraulic factors which affect stream channel and bridge stability are numerous and include bed forms and their effects on sediment transport, resistance to flow, flow velocities and flow depths; the magnitude and frequency of floods; characteristics of floods, (i.e., duration, time to peak, and time of recession); flow classification (e.g., unsteady, nonuniform, turbulent, supercritical or subcritical); ice and other floating debris in the flow; flow constrictions; bridge length, location, orientation, span lengths, pier location and design; superstructure elevation and design; the location and design of countermeasures; and the effects of natural and man-made changes which affect the hydrology and hydraulic flow conditions of the stream. In the bridge reach, bridge design and orientation can induce contraction scour and local scour at piers and abutments. Hydraulic factors are discussed in Chapter 3.0

1.4 Countermeasures

Numerous measures are available to counteract the actions of man and nature which contribute to the instability of alluvial streams. These include measures installed in or near the stream to protect highways and bridges by stabilizing a local reach of the stream, and measures which can be incorporated into the highway design to ensure the structural integrity of the highway in an unstable stream environment. The selection, location, and design of countermeasures are dependent on hydraulic and geomorphic factors that contribute to instability, as well as costs and construction and maintenance considerations.

1.5 Manual Organization

This manual is organized to: (1) familiarize the user with the important geomorphic and hydraulic factors which are indicators of and contributors to potential and existing stream and bridge stability problems (Chapters 2.0 and 3.0), (2) provide a procedure for the analysis of potential and existing stability problems (Chapter 4.0), and (3) provide guidance for selecting and designing appropriate countermeasures to mitigate instability problems (Chapters 5.0 and 6.0, respectively). Finally, Chapter 7.0 contains selected references, and Appendix A is an illustrative example of Stream Stability Analysis.

2.0 GEOMORPHIC FACTORS AND PRINCIPLES

2.1 Introduction

Most streams that highways cross or encroach upon are alluvial; that is, the streams are formed in materials that have been and can be transported by the stream. In alluvial stream systems, it is the rule rather than the exception that banks will erode; sediments will be deposited; and floodplains, islands and side channels will undergo modification with time. Alluvial channels continually change position and shape as a consequence of hydraulic forces exerted on the bed and banks. These changes may be gradual or rapid and may be the result of natural causes or man's activities.

Many streams are not alluvial. The bed and bank material is very coarse, and except at extreme flood events, does not erode. These streams are classified as sediment supply deficient, i.e., the transport capacity of the stream flow is greater than the availability of bed material for transport. The bed and bank material of these streams may consist of cobbles, boulders or even bed rock. In general, these streams are stable but should be carefully analyzed for stability at large flows.

A study of the plan and profile of a stream is very useful in understanding stream morphology. Plan view appearances of streams are varied and result from many interacting variables. Small changes in a variable can change the plan view and profile of a stream, adversely affecting a highway crossing or encroachment. This is particularly true for alluvial streams. Conversely, a highway crossing or encroachment can inadvertently change a variable, adversely affecting the stream.

Each of the geomorphic properties listed in the left column of Figure 1 could be used as the basis of a valid stream classification. The stream classification presented here is based on stream properties observed on aerial photographs and in the field. Its major purpose is to facilitate the assessment of streams for engineering purposes, particularly regarding lateral stability of a stream. Each property has limited usefulness when considered alone, however, classification based on combinations of more than a few properties and categories of each become unwieldy. Since the most common stream types represent a characteristic association of properties, these common types will be described and their engineering significance discussed. Data and observations are derived from a study of case histories of 224 bridge sites in the U. S. and Canada[1,2]. The following section is organized according to Figure 1. No particular significance is assigned to the order of the figure, and association of characteristics should not be inferred with descriptions above or below in the figure.

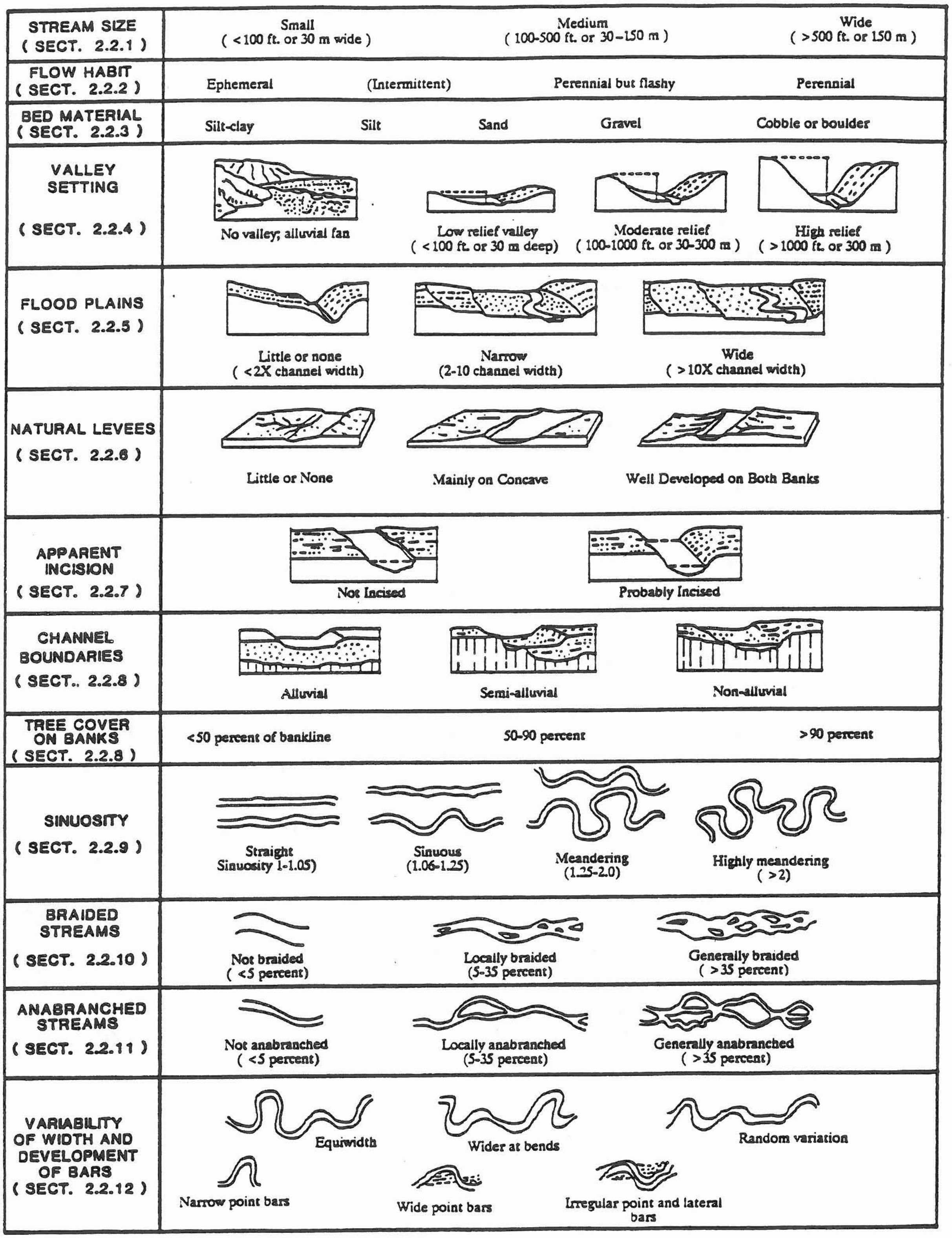


Figure 1. Geomorphic factors that affect stream stability (Adapted from [1]).

2.2 Geomorphic Factors Affecting Stream Stability

2.2.1 Stream Size

Stream depth tends to increase with size, and potential for scour increases with depth. Thus, potential depth of scour increases with increasing stream size.

The potential for lateral erosion also increases with stream size. This fact may be less fully appreciated than the increased potential for deep scour. Brice et al., cite as examples the lower Mississippi River, with a width of about 5,000 feet, which may shift laterally 100 feet or more in a single major flood; the Sacramento River, where the width is about 1,000 feet, is unlikely to shift more than 25 feet in a single flood; and streams whose width is about 100 feet are unlikely to shift more than 10 feet in a single flood.[1] Except for the fact that the potential for lateral migration increases with stream size, no generalization is possible regarding migration rates.

The size of a stream can be indicated by discharge, drainage area, or some measure of channel dimensions, such as width or cross sectional area. No single measure of size is satisfactory because of the diversity of stream types. For purposes of stream classification (Figure 1), bank-to-bank channel width is chosen as the most generally useful measure of size, and streams are arbitrarily divided into three size categories on the basis of width. The width of the stream does not include the width of the floodplain, but floodplain width is an important factor in bridge design if significant overbank flow occurs.

Bank-to-bank width is sometimes difficult to define for purposes of measurement when one of the banks is indefinite. This is particularly true at bends, where the outside bank is likely to be vertical and sharply defined but the inside bank slopes gradually up to floodplain level. The position of the line of permanent vegetation on the inside bank is the best available indicator of the bank line, and it tends to be rather sharply defined along many rivers in humid regions. The width of a stream is measured along a perpendicular drawn between its opposing banks, which are defined either by their form or as the riverward edge of a line of permanent vegetation. For sinuous or meandering streams, width is measured at straight reaches or at the inflections between bends, where it tends to be most consistent. For multiple channel streams, width is the sum of the widths of individual, unvegetated channels.

The Topographic Division of the U. S. Geological Survey uses, insofar as possible, the so-called "normal" stage or the stage prevailing during the greater part of the year for representing streams on topographic maps. It finds that the "normal" stage for a perennial river usually corresponds to the water level filling the channel to the line of permanent vegetation along its banks. Normal stage is also adopted here to define channel width.

2.2.2 Flow Habit

The flow habit of a stream may be ephemeral, perennial but flashy, or perennial. An ephemeral stream flows briefly in direct response to precipitation, and as used here, includes intermittent streams. A perennial stream flows all or most of the year, and a perennial but flashy stream responds to precipitation by rapid changes in stage and discharge. Perennial streams may be relatively stable or unstable, depending on other factors such as channel boundaries and bed material.

In arid regions, ephemeral streams may be relatively large and unstable. They may pose problems in determining the stage-discharge relationship and in estimating the depth of scour. A thalweg that shifts with stage and channel degradation by headcutting may also cause problems. In humid regions, ephemeral streams are likely to be small and pose few problems of instability.

2.2.3 Bed Material

Streams are classified, according to the dominant size of the sediment on their beds, as silt-clay bed, sand bed, gravel bed, and cobble or boulder bed. Accurate determination of the particle size distribution of bed material requires careful sampling and analysis, particularly for coarse bed material, but for most of the bed material designations, rough approximations can be derived from visual observation.

No relation has been found between bed material size and the incidence of scour problems at piers, abutments, or embankments.[1] It has been shown that particle size has only a small effect on the depth of scour produced by vortex and wake action around piers. The greatest depths of scour are usually found on streams having sand or sand-silt beds. The general conclusion is that scour problems are as common on streams having coarse bed material as on streams having fine bed material. However, very deep scour is more probable in fine bed material.

2.2.4 Valley Setting

Valley relief is used as a means of indicating whether the surrounding terrain is generally flat, hilly, or mountainous. For a particular site, relief is measured (usually on a topographic map) from the valley bottom to the top of the highest adjacent divide. Relief greater than 1,000 feet is regarded as mountainous, and relief in the range of 100 to 1,000 feet as hilly. Streams in mountainous regions are likely to have steep slopes, coarse bed materials, narrow floodplains and be non-alluvial, i.e., supply-limited sediment transport rates. In many regions, channel slope increases as the steepness of valley side slopes increases. Brice et al., reported no specific hydraulic problems at bridges at 23 study sites in mountainous terrain, at which all have beds of gravel or cobble-boulder.[2] Streams in regions of lower relief are usually alluvial and exhibit more problems because of lateral erosion in the channels.

Streams on alluvial fans or on piedmont slopes in arid regions pose special problems. A piedmont slope is a broad slope along a mountain front, and streams issuing from the mountain front may have shifting courses and poorly defined channels, as on an alluvial fan. Alluvial fans are among the few naturally occurring cases of aggradation problems at transverse highway crossing. They occur wherever there is a change from a steep to a flat gradient. As the bed material and water reaches the flatter section of the stream, the coarser bed materials are deposited because of the sudden reduction in both slope and velocity. Consequently, a cone or fan builds out as the material is dropped with the steep side of the fan facing the floodplain. Although typically viewed as a depositional zone, alluvial fans are also characterized by unstable channel geometries and rapid lateral movement. Deposition tends to be episodic, being interrupted by periods of fan trenching and sediment reworking.

The occurrence of deposition verses fan trenching on an alluvial fan surface are important factors in the assessment of stream stability at bridge crossings (Figure 2). On an untrenched fan, the sediment depositional zone will be nearer the mountain front, possibly creating more channel instability on the upper fan surface than on the lower fan surface. In contrast, a fan that is trenched will promote sediment movement across the fan and move the depositional zone closer to the toe of the fan, suggesting that the upper fan surface will be more stable that the lower fan surface. However, the general instability of fan channels and their tendency for rapid changes during large floods, and the possible channel avulsion created by deposition near the fan head, suggest that any location of an alluvial fan surface is, or could easily become, an area where channel stability is a serious concern to bridge safety.

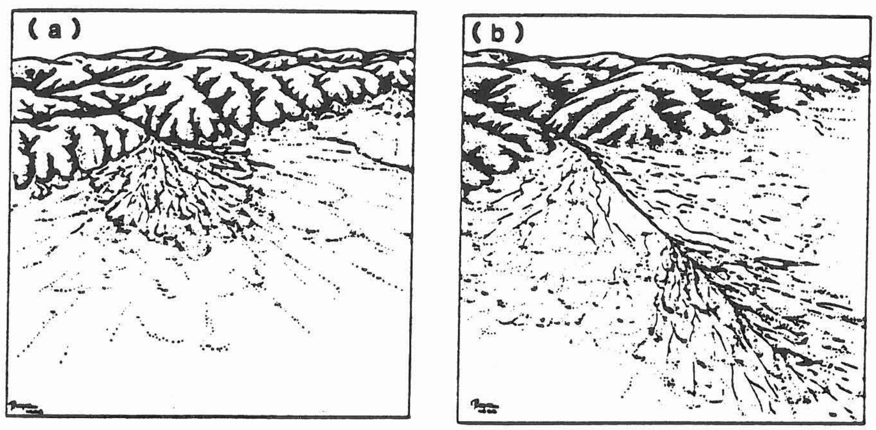


Figure 2. Diverse morphology of alluvial fans: (a) area of deposition at fan head, (b) fan-head trench with deposition at fan toe (After [3]).

There is considerable similarity between deltas and alluvial fans. Both result from reductions in slope and velocity, have steep slopes at their outer edges and tend to reduce upstream slopes. Deposits very similar to a delta develop where a steep tributary enters a larger stream. The steep channel tends to drop part of its sediment load in the main channel building out into the main stream. In some instances, drastic changes can occur in the main stream channel as a result of deposition from the tributary stream.

2.2.5 Floodplains

Floodplains are described as the nearly flat alluvial lowlands bordering a stream that are subject to inundation by floods. Many geomorphologists prefer to define a floodplain as the surface presently under construction by the stream, which is flooded with a frequency of about 1.5 years. According to this definition, surfaces flooded less frequently are terraces, abandoned floodplains, or flood-prone areas. However, flood prone areas are considered herein as part of the floodplain. Vegetative cover, land use, and flow depth on the floodplain are also significant factors in stream channel stability. In Figure 1, floodplains are categorized according to width relative to channel width.

Over time, the highlands of an area are worn down, streams erode their banks, and the material that is eroded is utilized farther downstream to build banks and bars. Streams move laterally, pushing the highlands back. Low, flat valley land and floodplains are formed. As streams transport sediment to areas of flatter slopes and, in particular, to bodies of water where the velocity and turbulence are too small to sustain transport of the material, the material is deposited forming deltas. As deltas build outward, the upstream portion of the channel is elevated through deposition and becomes part of the floodplain. Also, the stream channel is lengthened and the slope is further reduced. The upstream streambed is filled in and average flood elevations are increased. As the stream works across the stream valley, deposition causes the total floodplain to raise in elevation. Hence, even old streams are far from static. Old rivers meander, and they are affected by changes in sea level, influenced by movements of the earth's crust, changed by delta formations or glaciation, and subject to modifications due to climatological changes and as a consequence of man's development.

2.2.6 Natural Levees

Natural levees form during floods as the stream stage exceeds bankfull conditions. Sediment is then deposited on the floodplain due to the reduced velocity and transporting capacity of the flood in these overbank areas. The natural levees formed near the stream are rather steep because coarse material drops out quickly as the overbank velocity is smaller than the stream velocity. Farther from the stream, the gradients are flatter and finer materials drop out. Swamp areas are found beyond the levees.

Classification based on natural levees is illustrated in Figure 1. Streams with well-developed natural levees tend to be of constant width and have low rates of lateral migration. Well-developed levees usually occur along the lower courses of streams or where the floodplain is submerged for several weeks or months a year. If the levee is breached, the stream course may change through the breach. Areas between natural levees and the valley sides may drain, but slowly. Streams tributary to streams with well-developed natural levees may flow approximately parallel with the larger stream for long distances before entering the larger stream.

2.2.7 Apparent Incision

The apparent incision of a stream channel is judged from the height of its banks at normal stage relative to its width. For a stream whose width is about 100 feet, bank heights in the range of 6 to 10 feet are about average, and higher banks indicate probable incision. For a stream whose width is about 1,000 feet, bank heights in the range of 10 to 15 feet are about average, and higher banks indicate probable incision. Incised streams tend to be fixed in position and are not likely to bypass a bridge or to shift in alignment at a bridge. Lateral erosion rates are likely to be slow, except for western arroyos with high, vertical, and clearly unstable banks.

2.2.8 Channel Boundaries and Vegetation

Although no precise definitions can be given for alluvial, semi-alluvial, or non-alluvial streams, some distinction with regard to the erosional resistance of the earth material in channel boundaries is needed. In geology, bedrock is distinguished from alluvium and other surficial materials mainly on the basis of age, rather than on resistance to erosion. A compact alluvial clay is likely to be more resistant than a weakly cemented sandstone that is much older. Nevertheless, the term "bedrock:" does carry a connotation of greater resistance to erosion, and it is used here in that sense. An alluvial channel is in alluvium, a non-alluvial channel is in bedrock or in very large material (cobbles and boulders) that do not move except at very large flows, and a semi-alluvial channel has both bedrock and alluvium in its boundaries. The bedrock of non-alluvial channels may be wholly or partly covered with sediment at low stages, but is likely to be exposed by scour during floods.

Most highway stream crossings are over alluvial streams which are susceptible to more hydraulic problems than non-alluvial streams. However, the security of foundations in bedrock depends on the quality of the bedrock and the care with which foundations are set. Serious problems and failures have developed at bridges with foundations on shale, sandstone, limestone, glacial till, and other erodible rock. The New York State Thruway Schoharie Creek bridge failure is a recent catastrophic example of such a failure. Bed material at the bridge site was highly cemented glacial till.

Changes in channel geometry with time are particularly significant during periods when alluvial channels are subjected to high flows, and few changes occur during relatively dry periods. Erosive forces during high flow periods may have a capacity as much as 100 times greater than those forces acting during periods of intermediate and low flow rates. When considering the stability of alluvial streams, in most instances it can be shown that approximately 90 percent of all changes occur during that small percentage of the time when the discharge exceeds dominant discharge. A discussion of dominant discharge may be found in reference [4], but the bank full flow condition is recommended for use where a detailed analysis of dominant discharge is not feasible.

The most significant property of materials of which channel boundaries are comprised is particle size. It is the most readily measured property, and, in general, represents a sufficiently complete description of the sediment particle for many practical purposes. Other properties such as shape and fall velocity tend to vary with size in a roughly predictable manner.

In general, sediments have been classified into boulders, cobbles, gravel, sands, silts, and clays on the basis of their nominal or sieve diameters. The size range in each general class is given in Table 1. Noncohesive material generally consists of silt (0.004 - 0.062 mm), sand (0.062 - 2.0 mm), gravel (2.0 - 64 mm), or cobbles (64 - 250 mm).

The appearance of the stream bank is a good indication of relative stability. A field inspection of a channel will help to identify characteristics which are associated with erosion rates:

* Unstable banks with moderate to high erosion rates usually have slopes which exceed 30 percent, and a cover of woody vegetation is rarely present. At a bend, the point bar opposite an unstable cut bank is likely to be bare at normal stage, but it may be covered with annual vegetation and low woody vegetation, especially willows. Where very rapid erosion is occurring, the bank may have irregular indentations. Fissures, which represent the boundaries of actual or potential slump blocks along the bank line indicate the potential for very rapid bank erosion.

* Unstable banks with slow to moderate erosion rates may be partly reshaped to a stable slope. The degree of instability is difficult to assess, and reliance is placed mainly on vegetation. The reshaping of a bank typically begins with the accumulation of slumped material at the base such that a slope is formed, and progresses by smoothing of the slope and the establishment of vegetation.

Table 1. Sediment grade scale.

SIZE				Approximate Sieve Mesh Openings per Inch		CLASS
Millimeters		Microns	Inches	Tyler	U.S. Standard	
4000-2000	—	—	160-80	—	—	Very large boulders
2000-1000	—	—	80-40	—	—	Large boulders
1000-500	—	—	40-20	—	—	Medium boulders
500-250	—	—	20-10	—	—	Small boulders
250-130	—	—	10-5	—	—	Large cobbles
130-64	—	—	5-2.5	—	—	Small cobbles
64-32	—	—	2.5-1.3	—	—	Very coarse gravel
32-16	—	—	1.3-0.6	—	—	Coarse gravel
16-8	—	—	0.6-0.3	2-1/2	—	Medium gravel
8-4	—	—	0.3-0.16	5	5	Fine gravel
4-2	—	—	0.16-0.08	9	10	Very fine gravel
2-1	2.00-1.00	2000-1000	—	16	18	Very coarse sand
1-1/2	1.00-0.50	1000-500	—	32	35	Coarse sand
1/2-1/4	0.50-0.25	500-250	—	60	60	Medium sand
1/4-1/8	0.25-0.125	250-125	—	115	120	Fine sand
1/8-1/16	0.125-0.062	125-62	—	250	230	Very fine sand
1/16-1/32	0.062-0.031	62-31	—			Coarse silt
1/32-1/64	0.031-0.016	31-16	—			Medium silt
1/64-1/128	0.016-0.008	16-8	—			Fine silt
1/128-1/256	0.008-0.004	8-4	—			Very fine silt
1/256-1/512	0.004-0.0020	4-2	—			Coarse clay
1/512-1/1024	0.0020-0.0010	2-1	—			Medium clay
1/1024-1/2048	0.0010-0.0005	1-0.5	—			Fine clay
1/2048-1/4096	0.0005-0.0002	0.5-0.24	—			Very fine clay

* Stable banks with very slow erosion rates tend to be graded to a smooth slope of less than about 30 percent. Mature trees on a graded bank slope are convincing evidence of bank stability. In most regions of the United States, the upper parts of stable banks are vegetated, but the lower part may be bare at normal stage, depending on bank height and flow regime of the stream. Where banks are low, dense vegetation may extend to the water's edge at normal stage. Where banks are high, occasional slumps may occur on even the most stable graded banks. Shallow mountain streams that transport coarse bed sediment tend to have stable banks.

Active bank erosion can be recognized by falling or fallen vegetation along the bank line, cracks along the bank surface, slump blocks, deflected flow patterns adjacent to the bank line, live vegetation in the flow, increased turbidity, fresh vertical faces, newly formed bars immediately downstream of the eroding area, and, in some locations, a deep scour pool adjacent to the toe of the bank. These indications of active bank erosion can be noted in the field and on stereoscopic pairs of aerial photographs. Color infrared photography is particularly useful in detecting most of the indicators listed above, especially differences in turbidity. Figure 3 illustrates some of the features which indicate that a bank line is actively eroding.

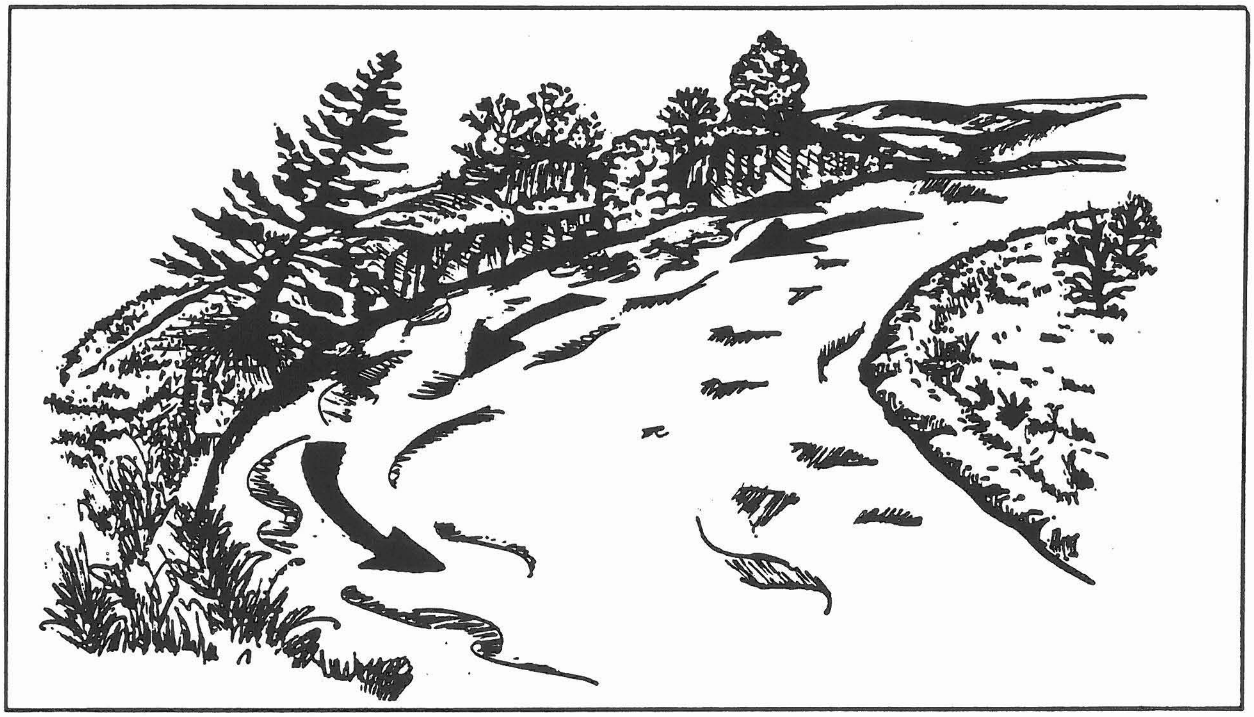


Figure 3. Active bank erosion illustrated by vertical cut banks, slump blocks, and falling vegetation (After [5]).

Bank Materials. Resistance of a streambank to erosion is closely related to several characteristics of the bank material. Bank material deposited in the stream can be broadly classified as cohesive, noncohesive, and composite. Typical bank failure surfaces of various materials are shown in Figure 4 and described as follows:[6]

* Noncohesive bank material tends to be removed grain by grain from the bank. The rate of particle removal, and particle movement, 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 of and fluctuations in the shear stress exerted on the banks, seepage force, piping, and wave forces. Figure 4(a) illustrates failure of banks of noncohesive material from flow slides resulting from a loss of shear strength because of saturation, and failure from sloughing resulting from the removal of materials in the lower portion of the bank.

* 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. Failure mechanisms for cohesive banks are illustrated in Figure 4(b).

* Composite or stratified banks consist of layers of materials of various sizes, 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 also vulnerable to erosion and sliding as a consequence of subsurface flows and piping. Typical failure modes are illustrated in Figure 4(c).

Piping. Piping is a phenomenon common to alluvial streambanks. With stratified banks, flow is induced in more permeable layers by changes in stream stage and by waves. If flow through the permeable lenses is capable of dislodging and transporting particles, the material is slowly removed, undermining portions of the bank. Without this foundation material to support the overlying layers, a block of bank material drops down and results in the development of tension cracks as sketched in Figure 4(c). These cracks allow surface flows to enter, further reducing the stability of the affected block of bank material. Bank erosion may continue on a grain-by-grain basis or the block of bank material may ultimately slide downward and outward into the channel, with bank failure resulting from a combination of seepage forces, piping, and mass wasting.

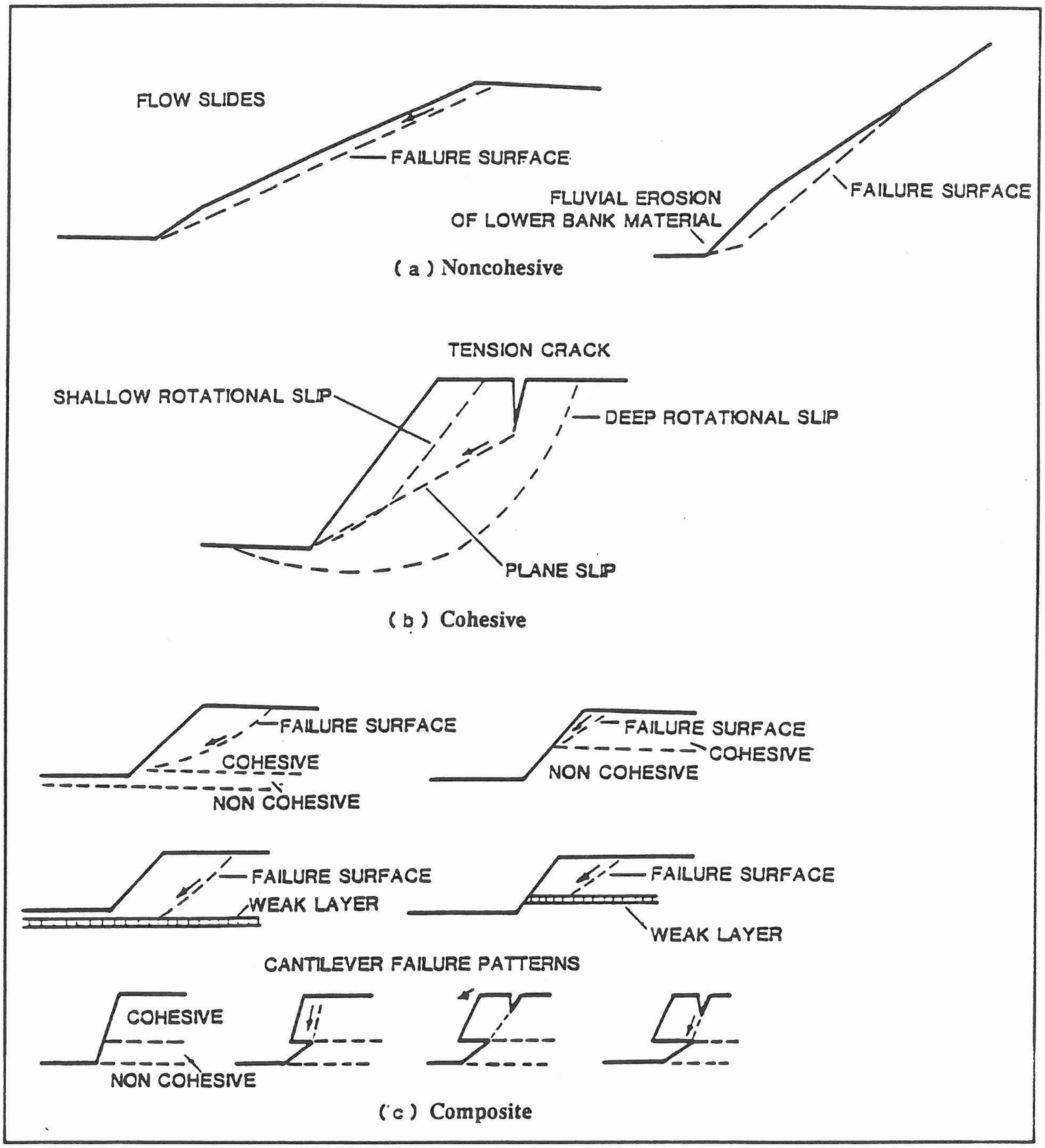


Figure 4. Typical bank failure surfaces: (a) noncohesive, (b) cohesive, and (c) composite (After [6]).

Mass Wasting. Local mass wasting is another form of bank failure. If a bank becomes saturated and possibly undercut by flowing water, blocks of the bank may slump or slide into the channel. Mass wasting may be caused or aggravated by the construction of homes on river banks, operation of equipment adjacent to the banks, added gravitational force resulting from tree growth, location of roads that cause unfavorable drainage conditions, agricultural uses on adjacent floodplain, saturation of banks by leach fields from septic tanks, and increased infiltration of water into the floodplain as a result of changing land-use practices.

Various forces are involved in mass wasting. Landslides, the downslope movement of earth and organic materials, result from an imbalance of forces. These forces are associated with the downslope gravity component of the slope mass. Resisting these downslope forces are the shear strength of the materials and any contribution from vegetation via root strength or man's slope reinforcement activities. When the toe of a slope is removed, as by a stream, the slope materials may move downward into the void in order to establish a new equilibrium. Oftentimes, this equilibrium is a slope configuration with less than original surface gradient. The toe of the failed mass then provides a new buttress against further movements. Erosion of the toe of the slope then begins the process over again.

2.2.9 Sinuosity

Sinuosity is the ratio of the length of a stream reach measured along its centerline, to the length measured along the valley centerline or along a straight line connecting the ends of the reach. The valley centerline is preferable when the valley itself is curved. Sometimes, sinuosity is defined as the ratio of stream slope to valley slope. Straight stream reaches have a sinuosity of one, and the maximum value of sinuosity for natural streams is about four. Inasmuch as the sinuosity of a stream is rarely constant from one reach to the next, no very refined measurement of sinuosity is warranted. The four classes of sinuosity in Figure 1 are arbitrary.

A straight stream, or one that directly follows the valley centerline, sometimes has the same slope as the valley. As the sinuosity of the stream increases, its slope decreases in direct proportion. Similarly, if a sinuous channel is straightened, the slope increases in direct proportion to the change in length.

The size, form, and regularity of meander loops are aspects of sinuosity. Symmetrical meander loops are not very common, and a sequence of two or three identical symmetrical loops is even less common. In addition, meander loops are rarely of uniform size. The largest is commonly about twice the diameter of the smallest. Statistically the size-frequency distribution of loop radii tends to have a normal distribution.

There is little relation between degree of sinuosity and lateral stream stability. A highly meandering stream may have a lower rate of lateral migration than a sinuous stream of similar size (Figure 1). Stability is largely dependent on other properties, especially bar development and the variability of channel width.

Streams are broadly classified as straight, meandering or braided. Any change imposed on a stream system may change its planform geometry.

Straight Streams. A straight stream has small sinuosity at bank full stage. At low stage, the channel develops alternate sandbars, and the thalweg meanders around the sandbars in a sinuous fashion. Straight streams are considered a transitional stage to meandering, since straight channels are relatively stable only where sediment size and load are small, gradient, velocities, and flow variability are low, and the channel width-depth ratio is relatively low. Straight channel reaches of more than 10 channel widths are not common in nature.

Meandering Streams. Alluvial channels of all types deviate from a straight alignment. The thalweg oscillates transversely and initiates the formation of bends. In a straight stream, alternate bars and the thalweg are continually changing; thus, the current is not uniformly distributed through the cross section but is deflected toward one bank and then the other. Sloughing of the banks, nonuniform deposition of bed load, debris such as trees, and the Coriolis force due to the earth's rotation have been cited as causes for the meandering of streams. When the current is directed toward a bank, the bank is eroded in the area of impingement, and the current is deflected and impinges on the opposite bank farther downstream. The angle of deflection of the current is affected by the curvature formed in the eroding bank and the lateral depth of erosion. Figure 5 shows bars, pools, and crossings typical of a meandering channel and the effects on water surface profiles.

Sinuous, meandering, and highly meandering streams have more or less regular inflections that are sinuous in plan, consisting of a series of bends connected by crossings. In the bends, deep pools are carved adjacent to the concave bank by the relatively high velocities. Because velocities are lower on the inside of bends, sediments are deposited in this region, forming point bars. Also, the centrifugal force in the bend causes a transverse water surface slope and helicoidal flow with a bottom velocity away from the outer bank toward the point bar. These transverse velocities enhance point bar building by sweeping the heavier concentrations of bed load toward the convex bank where they are deposited to form the point bar. Some transverse currents have a magnitude of about 15 percent of the average channel velocity. The bends are connected by crossings (short straight reaches) which are quite shallow compared to the pools in the bendways. At low flow, large sandbars form in the crossings if the channel is not well confined. Scour in the bend causes the bend to migrate downstream and sometimes laterally. Lateral movements as large as 2500 feet per year have been observed in alluvial rivers. Much of the sediment eroded from the outside bank is deposited in the crossing and on the point bar in the next bend downstream. The variability of bank materials and the fact that the stream encounters/produces such features as clay plugs causes a wide variety of river forms. The meander belt formed is often fifteen to twenty times the channel width.

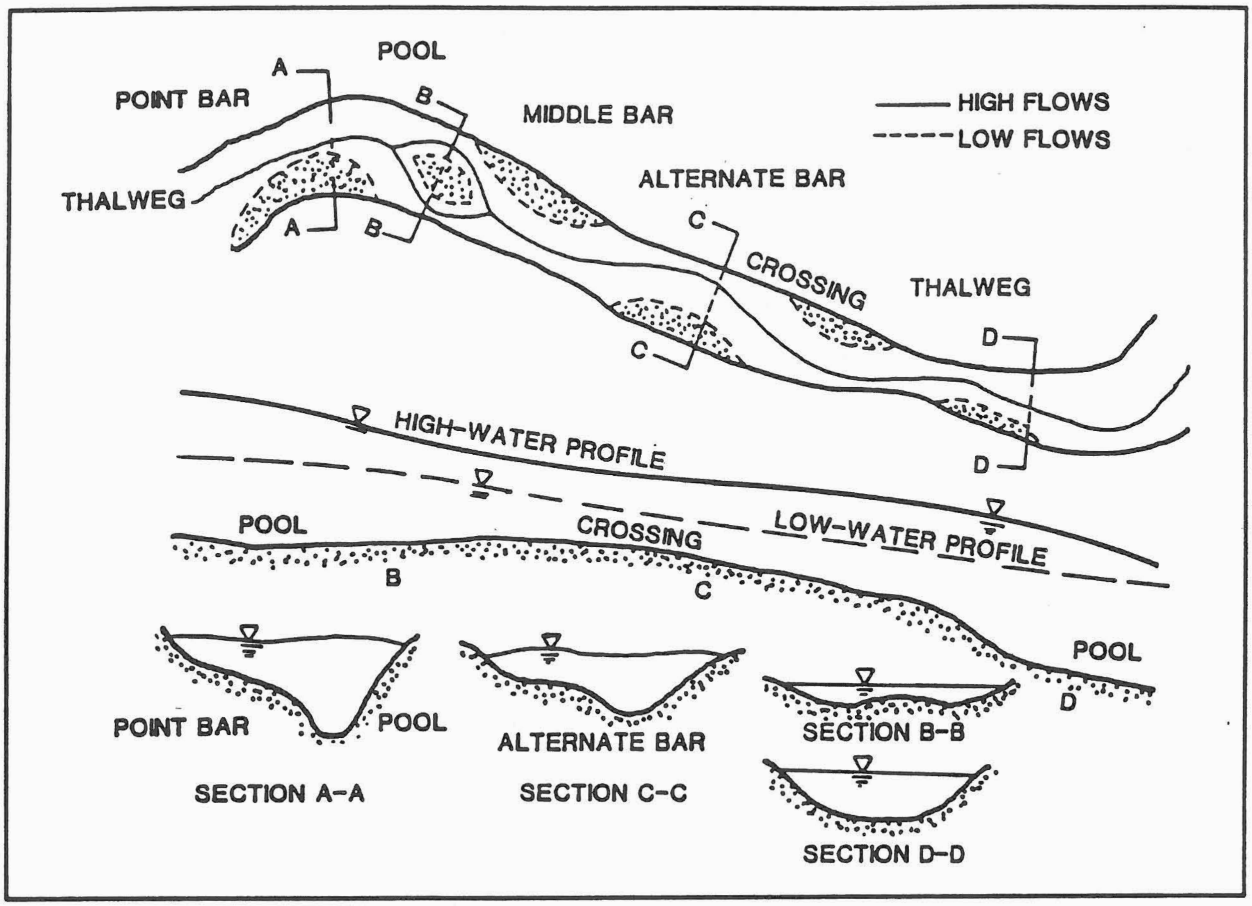


Figure 5. Plan view and cross section of a meandering stream (After [4]).

On a laterally unstable channel, or at actively migrating bends on an otherwise stable channel, point bars are usually wide and unvegetated and the opposite bank is cut and often scalloped by erosion. The crescent-shaped scars of slumping may be visible from place to place along the bank line. The presence of a cut bank opposite a point bar is evidence of instability. Sand or gravel on the bar appears as a light tone on aerial photographs. The unvegetated condition of the point bar is attributed to a rate of outbuilding that is too rapid for vegetation to become established. However, the establishment of vegetation on a point bar is dependent on factors other than the rate of growth of the point bar, such as climate and the timing of floods. Therefore, the presence of vegetation on a point bar is not conclusive evidence of stability. If the width of an unvegetated point bar is considered as part of the channel width, the channel tends to be wider at bends.

As a meandering stream system moves laterally and longitudinally, meander loops move at unequal rates because of unequal erodibility of the banks. This causes the channel to appear as a slowly developing bulb-form. Channel geometry depends upon the local slope, bank material, and the geometry of adjacent bends. Years may be required before a configuration characteristic of average conditions in the stream is attained.

If the proposed highway or highway stream crossing is located near a meander loop, it is useful to have some insight into the probable way in which the loop will migrate or develop, as well as its rate of growth. No two meanders will behave in exactly the same way, but the meanders on a particular stream reach tend to conform to one of the several modes of behavior illustrated in Figure 6, which is based on a study of about 200 sinuous or meandering stream reaches[1].

Mode a (Figure 6) represents the typical development of a loop of low amplitude, which decreases in radius as it extends slightly in a downstream direction. Mode b rarely occurs unless meanders are confined by artificial levees or by valley sides on a narrow floodplain. Well developed meanders on streams that have moderately unstable banks are likely to follow Mode c. Mode d applies mainly to larger loops on meandering or highly meandering streams. The meander has become too large in relation to stream size and flow, and secondary meanders develop, converting it to a compound loop. Mode e also applies to meandering or highly meandering streams, usually of the equiwidth, point-bar type. The banks have been sufficiently stable for an elongated loop to form without being cut off, but the neck of the loop is gradually being closed and cutoff will eventually occur at the neck. Modes f and g apply mainly to locally braided, sinuous, or meandering streams having unstable banks. Loops are cut off by chutes that break diagonally or directly across the neck.

Oxbow lakes are formed by the cutoff of meander loops, which occurs either by gradual closure of the neck (neck cutoffs) or by a chute that cuts across the neck (chute cutoffs). Neck cutoffs are associated with relatively stable channels, and chute cutoffs with relatively unstable channels. Recently formed oxbow lakes along a channel are evidence of recent lateral migration. Commonly, a new meander loop soon forms at the point of cutoff and grows in the same direction as the previous meander. Cutoffs tend to induce rapid bank erosion at adjacent meander loops. The presence of abundant oxbow lakes on a floodplain does not necessarily indicate a rapid channel migration rate because an oxbow lake may persist for hundreds of years.

Usually the upstream end of the oxbow lake fills quickly to bank height. Overflow during floods and overland flow entering the oxbow lake carry fine materials into the oxbow lake area. The lower end of the oxbow remains open and drainage entering the system can flow out from the lower end. The oxbow gradually fills with fine silts and clays which are plastic and cohesive. As the stream channel meanders, old bendways filled with cohesive materials (referred to as clay plugs) are sufficiently resistant to erosion to serve as semipermanent geologic controls which can drastically affect planform geometry.

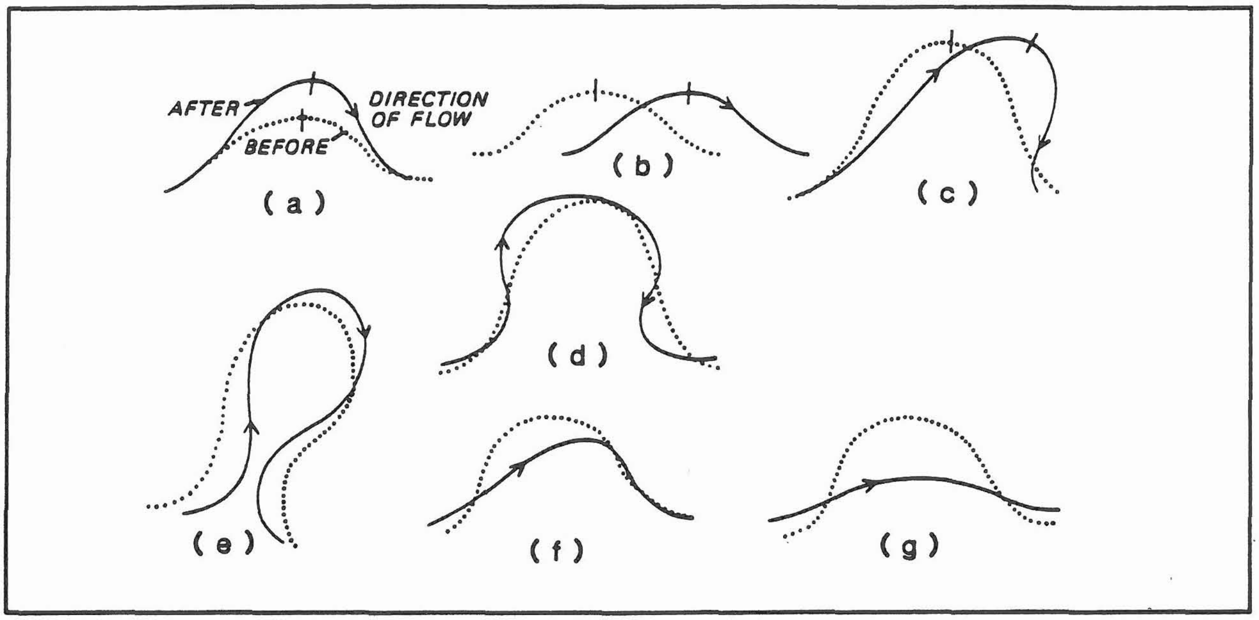


Figure 6. Modes of meander loop development: (a) extension; (b) translation; (c) rotation; (d) conversion to a compound loop; (e) neck cutoff by closure; (f) diagonal cutoff by chute; and (g) neck cutoff by chute (After [1]).

The local increase in channel slope due to cutoff usually results in an increase in the growth rate of adjoining meanders, and an increase in channel width at the point of cutoff. On a typical wide-bend point-bar stream, the effects of cutoff do not extend very far upstream or downstream. The consequences of cutoffs are an abruptly steeper stream gradient at the point of the cutoff, scour at the cutoff, and a propagation of the scour in an upstream direction. Downstream of a cutoff, the gradient of the channel is not changed and, therefore, the increased sediment load caused by upstream scour will usually be deposited at the site of the cutoff or below it, forming a large bar.

In summary, there is little relation between degree of sinuosity, as considered apart from other properties, and lateral stream stability.[1] A highly meandering stream may have a lower rate of lateral migration than a sinuous stream of similar size. Assessment of stability is based mainly on additional properties, especially on bar development and the variability of channel width. However, many hydraulic problems are associated with the location of crossings at a meander or bend. These include the shift of flow direction at flood stage, shift of thalweg toward piers or abutments, and lateral channel erosion at piers, abutments, or approaches.

Since random factors seem to be involved in the migration of meanders, the exact rate or place of erosion is probably not predictable. However, the most rapid bank erosion is generally at the outside of meanders, downstream from the apex of the loop.

The cutoff of a meander, whether done artificially or naturally, causes a local increase in channel slope and a more rapid growth rate of adjoining meanders. Adjustment of the channel to increase in slope seems to be largely accomplished by increase in channel width (wetted perimeter) at and near the point of cutoff.

Some generalizations can be made, from general knowledge of stream behavior, about the probable consequences of controlling or halting the development of a meander loop by the use of countermeasures. The most probable consequences relate to change in flow alignment (or lack of change, if the position of a naturally eroding bank is held constant). The development of a meander is affected by the alignment of the flow that enters it. Any artificial influence on flow alignment is likely to affect meander form. Downstream bank erosion rates are not likely to be increased, but the points at which bank erosion occurs are likely to be changed. In the case where flow is deflected directly at a bank, an increase in erosion rates would be expected. The recent failure of a major bridge on the Hatchie River near Covington, Tennessee has been attributed, in part, to lateral migration of the channel in the bridge reach.

2.2.10 Braided Streams

A braided stream is one that consists of multiple and interlacing channels (Figure 1). In general, a braided channel has a large slope, a large bed-material load in comparison with its suspended load, and relatively small amounts of silts and clays in the bed and banks. The magnitude of the bed load is more important than its size. If the flow is overloaded with sediment, deposition occurs, the bed aggrades, and the slope of the channel increases in an effort to obtain a graded state. As the channel steepens, velocity increases, and multiple channels develop. Multiple channels are generally formed as bars of sediment and deposited within the main channel, causing the overall channel system to widen. However, braided streams may occur with a graded state that is neither aggrading nor degrading.

The formation of multiple, mid channel islands and bars is characteristic of streams that transport large bed loads. The presence of bars obstructs flow and scour occurs, either lateral erosion of banks on both sides of the bar, scour of the channels surrounding the bar, or both. This erosion will enlarge the channel and, with reduced water levels, an island may form at the site of a gravel or sand bar. The worst case will be where major bar or island forms at a bridge site. This can produce erosion of both banks of the stream and bed scour along both sides of the island. Reduction in the flow capacity beneath the bridge can result as a vegetated island forms under the bridge. An island or bar that forms upstream or downstream of a bridge can change flow alignment and create bank erosion or scour problems at the bridge site.

Island shift is easily identified because active erosion at one location and active deposition at another on the edge of an island can be recognized in the field. Also, the development or abandonment of flood channels and the joining together of islands can be detected by observing vegetational differences and patterns of erosion and deposition.

The degree of channel braiding is indicated by the percent of reach length that is divided by bars and islands, as shown in Figure 1. Braided streams tend to be common in arid and semiarid parts of the western United States and regions having active glaciers.

Braided streams may present difficulties for highway construction because they are unstable, change alignment rapidly, carry large quantities of sediment, are very wide and shallow even at flood flow and are, in general, unpredictable. Deep scour holes can develop downstream of a gravel bar or island where the flow from two channels comes together.

Braided streams generally require long bridges if the full channel width is crossed or effective flow-control measures if the channel is constricted. The banks are likely to be easily erodible, and unusual care must be taken to prevent lateral erosion at or near abutments. The position of braids is likely to shift during floods, resulting in unexpected velocities, angle of attack, and depths of flow at individual piers. Lateral migration of braided streams takes place by lateral shift of a braid against the bank, but available information indicates that lateral migration rates are generally less than for meandering streams. Along braided streams, however, migration is not confined to the outside of bends but can take place at any point by the lateral shift of individual braids.

2.2.11 Anabranched Streams

An anabranched stream differs from a braided stream in that the flow is divided by islands rather than bars, and the islands are large relative to channel width. The anabranches, or individual channels, are more widely and distinctly separated and more fixed in position than the braids of a braided stream. An anabranch does not necessarily transmit flow at normal stage, but it is an active and well-defined channel, not blocked by vegetation. The degree of anabranching is arbitrarily categorized in Figure 1 in the same was as the degree of braiding was described.

Although the distinction between braiding and anabranching may seem academic, it has real significance for engineering purposes. Inasmuch as anabranches are relatively permanent channels that may convey substantial flow, diversion and confinement of an anabranched stream is likely to be more difficult than for a braided stream. Problems associated with crossings on anabranched streams can be avoided if a site where the channel is not anabranched can be chosen. If not, the designer may be faced with a choice of either building more than one bridge, building a long bridge, or diverting anabranches into a single channel. Problems with flow alignment may occur if a bridge is built at or near the junction of anabranches.

Where anabranches are crossed by separate bridges, the design discharge for the bridges may be difficult to estimate. If one anabranch should become partly blocked, as by floating debris or ice, an unexpected amount of flow may be diverted to the other.

2.2.12 Variability of Width and Development of Bars

The variability of unvegetated channel width is a useful indication of the lateral stability of a channel. The visual impression of unvegetated channel width on aerial photographs depends on the relatively dark tones of vegetation as contrasted with the lighter tones of sediment or water. A channel is considered to be of uniform width (equiwidth) if the unvegetated width at bends is not more than 1.5 times the average width at the narrowest places.

The relationship between width variability and lateral stability is based on the rate of development of point bars and alternate bars. If the concave bank at a bend is eroding slowly, the point bar will grow slowly and vegetation will become established on it. The unvegetated part of the bar will appear as a narrow crescent. If the bank is eroding rapidly, the unvegetated part of the rapidly growing point bar will be wide and conspicuous. A point bar with an unvegetated width greater than the width of flowing water at the bend is considered to be wider than average. Lateral erosion rates are probably high in stream reaches where bare point bars tend to exceed average width. In areas where vegetation is quickly established, as in rainy southern climates, cut banks at bends may be a more reliable indication of instability than the unvegetated width of point bars.

Three categories of width variability are distinguished in Figure 1, but the relative lateral stability of these must be assessed in connection with bar development and other properties. In general, equiwidth streams having narrow point bars are the most stable laterally, and random-width streams having wide, irregular point bars are the least stable. Vertical stability, or the tendency to scour, cannot be assessed from these properties. Scour may occur in any alluvial channel. In fact, the greatest potential for deep scour might be expected in laterally stable equiwidth channels, which tend to have relatively deep and narrow cross sections and bed material in the size range of silt and sand.

2.3 Lane Relation and Other Geomorphic Concepts

The major complicating factors in river mechanics are: (1) the large number of interrelated variables that can simultaneously respond to natural or imposed changes in a stream system, and (2) the continual evolution of stream channel patterns, channel geometry, bars and forms of bed roughness with changing water and sediment discharge. In order to understand the responses of a stream to the actions of man and nature, a few simple geomorphic concepts are presented here.

The dependence of stream form on slope, which may be imposed independent of other stream characteristics, is illustrated schematically in Figure 7. Any natural or artificial change which alters channel slope can result in modifications to the existing stream pattern. For example, a cutoff of a meander loop increases channel slope. Referring to Figure 7, this shift in the plotting position to the right could result in a shift from a relatively tranquil, meandering pattern toward a braided pattern that varies rapidly with time, has high velocities, is subdivided by sandbars, and carries relatively large quantities of sediment. Conversely, it is possible that a slight decrease in slope could change an unstable braided stream into a meandering one.

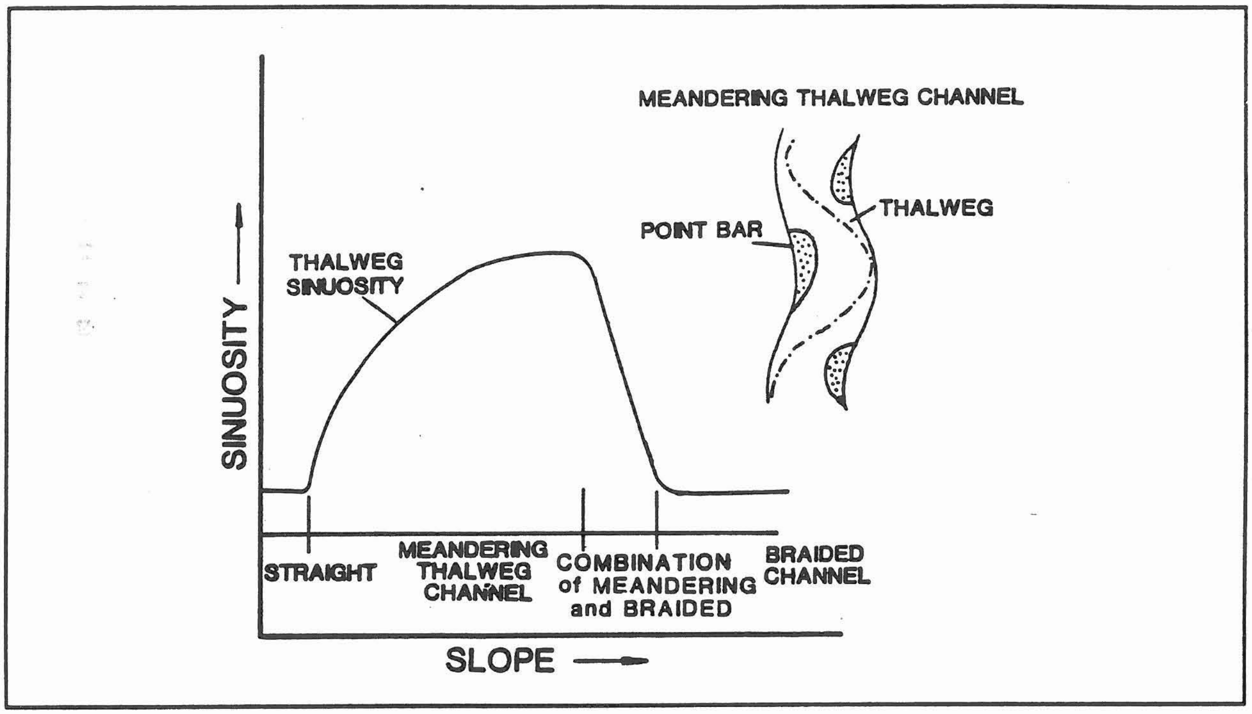


Figure 7. Sinuosity vs. slope with constant discharge (After [4]).

The significantly different channel dimensions, shapes, and patterns associated with different quantities of discharge and amounts of sediment load indicate that as these independent variables change, major adjustments of channel morphology can be anticipated. Further, a change in hydrology may cause changes in stream sinuosity, meander wave length, and channel width and depth. A long period of channel instability with considerable bank erosion and lateral shifting of the channel may be required for the stream to compensate for the hydrologic change. The reaction of a channel to changes in discharge and sediment load may result in channel dimension changes contrary to those indicated by many regime equations. For example, it is conceivable that a decrease in discharge together with an increase in sediment load could cause a decrease in depth and in increase in width.

Figure 8 illustrates the dependence of sand bed stream form on channel slope and discharge. According to Lane, a sand bed channel meanders where:[7]

$$SQ^{\frac{1}{4}} \leq .0017 \tag{1}$$

Similarly, a sand bed channel is braided where:

$$SQ^{\frac{1}{4}} \geq .010 \tag{2}$$

Where:

S = channel bed slope, ft/ft

Q = mean discharge, ft^3/sec

The zone between the lines defining braided streams and meandering streams in Figure 8 is the transitional range, i.e., the range in which a stream can change readily from one stream form to the other.

Many U. S. rivers, classified as intermediate sand bed streams, plot in this zone between the limiting curves defining meandering and braided stream. If a stream is meandering but its discharge and slope borders on the transitional zone, a relatively small increase in channel slope may cause it to change, with time, to a transitional or braided stream.

Leopold and Wolman plotted slope and discharge for a variety of natural streams.[8] They observed that a line could separate meandering from braided streams. The equation of this line is:

$$SQ^{.44} = 0.06 \tag{3}$$

Streams classified as meandering by Leopold and Wolman are those whose sinuosity is greater than 1.5. Braided streams are those which have relatively stable alluvial islands and, therefore, two or more channels. They note that sediment size is related to slope and channel pattern but do not try to account for the effect of sediment size on the morphology of streams. They further note that braided and meandering streams can be differentiated based on combinations of slope, discharge, and width/depth ratio, but regard width as a variable dependent mainly on discharge.

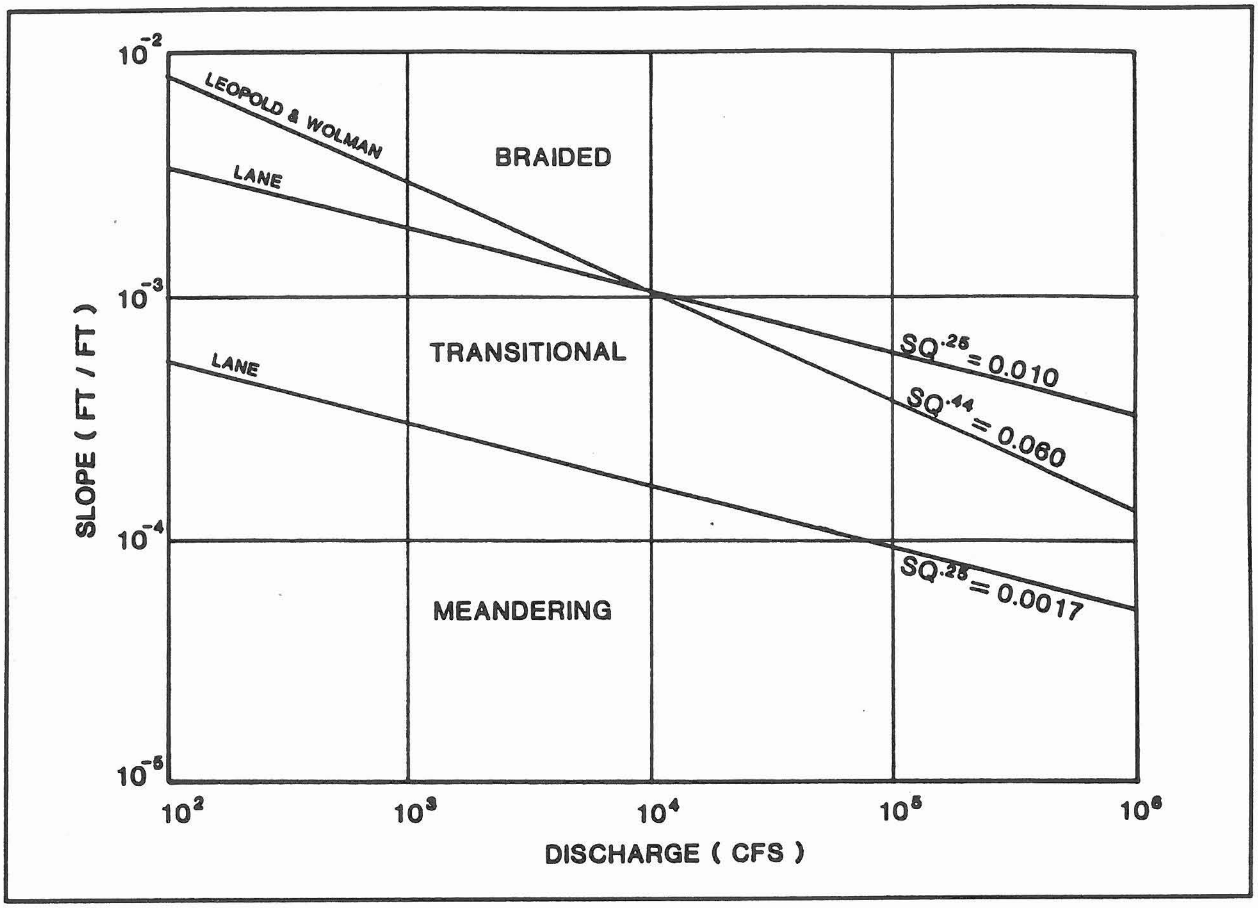


Figure 8. Slope-discharge relationship for braiding or meandering in sand bed streams (After [7]).

Long reaches of many streams have achieved a state of equilibrium, for practical engineering purposes. These stable reaches are called "graded" streams by geologists and "poised" streams by engineers. However, this condition does not preclude significant changes over a short period of time or over a period of years. Conversely, many streams contain long reaches that are actively aggrading or degrading. These aggrading and degrading channels pose definite hazards to highway crossings and encroachments, as compared with poised streams.

Regardless of the degree of channel stability, man's activities may produce major changes in stream characteristics locally and throughout an entire reach. All too frequently, the net result of a stream "improvement" is a greater departure from equilibrium than existed prior to "improvement." Designers of stream channel modifications should invariably seek to enhance the natural tendency of the stream toward equilibrium and a stable condition. This requires an understanding of the direction and magnitude of change in channel characteristics which will result from the actions of man and nature. This understanding can be obtained by: (1) studying the stream in a natural condition; (2) having knowledge of the sediment and water discharge; (3) being able to predict the effects and magnitude of man's future activities; and (4) applying to these a knowledge of geology, soils, hydrology, and hydraulics of alluvial rivers.

Predicting the response to channel modifications is a very complex task. There are large numbers of variables involved in the analysis that are interrelated and can respond to changes in a stream system in the continual evolution of stream form. The channel geometry, bars, and forms of bed roughness all change with changing water and sediment discharges. Because such a prediction is necessary, useful methods have been developed to qualitatively and quantitatively predict the response of channel systems to changes.

Quantitative prediction of response can be made if all of the required data are known with sufficient accuracy. However, available data are usually not sufficient for quantitative estimates, and only qualitative estimates are possible. Examples of studies that have been undertaken by various investigators for qualitative estimates follow. Lane studied the changes in stream morphology caused by modifications of water and sediment discharges.[9] Similar but more comprehensive treatments of channel response to changing conditions in streams have been presented by Leopold and Maddock, [10] Schumm, [11] and Santos-Cayado.[12] All research results support the relationship originally proposed by Lane:

$$QS \sim Q_s D_{50} \tag{4}$$

where:

Q is the discharge

S is the energy slope

Q_s is the sediment discharge

D_{50} is the median sediment size

Equation (4) is very useful to predict qualitatively channel response to climatological changes, stream modifications, or both. The geomorphic relation expressed is only an initial step in analyzing long-term channel response problems. However, this initial step is useful because it warns of possible future difficulties related to channel modifications. Examples of its use are given in reference [4].

2.4 Aggradation/Degradation and the Sediment Continuity Concept

2.4.1 Aggradation/Degradation

Aggradation and degradation are the vertical raising and lowering, respectively, of the stream bed over relatively long distances and time frames. Such changes, which are sometimes referred to as gradation changes, can be the result of both natural and man-induced changes in the watershed. The sediment continuity concept is the primary principle applied in both qualitative and quantitative analysis of gradation changes. After an introduction to the concept of sediment continuity, some of factors causing gradation change are reviewed.

2.4.2 Overview of the Sediment Continuity Concept

The amount of material transported, eroded, or deposited in an alluvial channel is a function of sediment supply and channel transport capacity. Sediment supply is provided from the tributary watershed and from any erosion occurring in the upstream channel. Sediment transport capacity is a function of the size of sediment, the discharge of the stream, and the geometric and hydraulic properties of the channel. When the transport capacity equals sediment supply, as state of equilibrium exists.

Application of the sediment continuity concept to a single channel reach illustrates the relationship between sediment supply and transport capacity. Technically, the sediment continuity concept states that the sediment inflow minus the sediment outflow equals the time rate of change of sediment volume in a given reach. More simply stated, during a given time period the amount of sediment coming into the reach minus the amount leaving the downstream end of the reach equals the change in the amount of sediment stored in that reach (see Figure 9). The sediment inflow to a given reach is defined by the sediment supply from the watershed (upstream of the study reach plus any significant lateral input directly to the study reach). The transport capacity of the channel within the given reach defines the sediment outflow. Changes in the sediment volume within the reach occur when the total input to the reach (sediment supply) is not equal to the downstream output (sediment transport capacity). When the sediment supply is less than the transport capacity, erosion will occur in the reach so that the transport capacity at the outlet is satisfied, unless controls exist that limit erosion. Conversely, when the sediment supply is greater than the transport capacity deposition will occur in the reach.

Controls that limit erosion may either be man made or natural. Man made controls included bank protection works, grade control structures, and stabilized bridge crossings. Natural controls can be geologic, such as outcroppings, or the presence of significant coarse sediment material in the channel. The presence of coarse material can result in the formation of a surface armor layer of larger sediments that are not transported by average flow conditions.

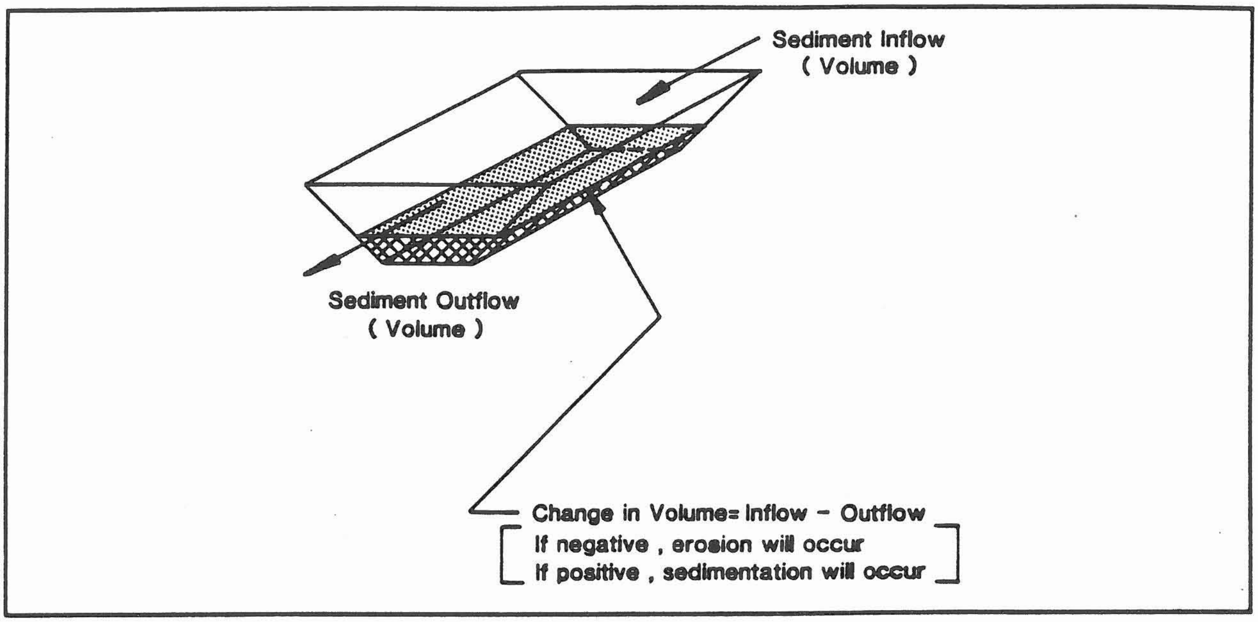


Figure 9. Definition sketch of sediment continuity concept applied to a given channel reach over a given time period.

2.4.3 Factors Initiating Gradation Changes

Man-induced Changes. Man's activities are the major cause of streambed gradation problems. Very few gradation changes are due to natural causes, although some may be the result of both natural and man-induced causes. The most common activities which result in gradation problems caused by man are channel alterations, streambed mining, dams and reservoirs, and land-use changes. Highway construction, including the construction of bridges and channel alterations of limited extent, usually affect stream vertical stability only locally.

Channel Alterations. Dredging, channelization, straightening, the construction of cutoffs to shorten the flow path of a stream, and clearing and snagging to increase channel capacity are the major causes of streambed elevation changes. An increase in slope resulting from a shorter flow path, or an increase in flow capacity results in increased velocities and a corresponding increase in sediment transport capacity. If the stream was previously in equilibrium (supply equal to transport capacity) the channel may adjust, either by increasing its length or by reducing its slope by degradation, in order to reestablish equilibrium. The most frequent response is a degrading streambed followed by bank erosion and a new meander pattern.

Constrictions in a stream channel, as in river control projects to maintain a navigation channel or highway crossings, also increase velocities and the sediment transport capacity in the constricted reach. The resulting degradation can be considered local, but it may extend through a considerable reach of stream, depending on the extent of the river control project. Constrictions may also cause local aggradation problems downstream.

The response to an increased sediment load in a stream that was near equilibrium conditions (i.e., supply now greater than transport capacity) is normally deposition in the channel downstream of the alteration. The result is an increase in flood stages and overbank flooding in downstream reaches. In time, the aggradation will progress both upstream and downstream of the end of the altered channel, and the stream reach may become locally braided as it seeks a new balance between sediment supply and sediment transport capacity.

Streambed Mining. Streambed mining for sand or gravel can be beneficial or detrimental, depending on the balance between sediment supply and transport capacity. Where the sediment supply exceeds the stream's transport capacity because of man's activities in the watershed or from natural causes, controlled removal of gravel bars and limited mining may enhance both lateral and vertical stability of the stream.

The usual result of streambed mining is an imbalance between sediment supply and transport capacity. Upstream of the operation, the water surface slope is increased and bank erosion and headcutting or a nick point may result. The extent of the damage that can result is a function of the volume and depth of the sand and gravel pit relative to the size of the stream, bed material size, flood hydrographs, upstream sediment transport, and the location of the pit. If the size of the borrow pit is sufficiently large, a substantial quantity of the sediment inflow will be trapped in the pit and degradation will occur downstream. If bank erosion and headcutting upstream of the pit produce a sediment supply greater than the trap capacity of the pit and the transport capacity downstream, aggradation could occur. However, this circumstance is unlikely and streambed mining generally causes degradation upstream and downstream of the pit.

Dams and Reservoirs. Storage and flood control reservoirs produce a stream response both upstream and downstream of the reservoir. A stream flowing into a reservoir forms a delta as the sediment load is deposited in the ponded water. This deposition reduces the stream gradient upstream of the reservoir and causes aggradation in the channel. Aggradation can extend many miles upstream.

Downstream of reservoirs, stream channel stability is affected because of the changed flow characteristics and because flow releases are relatively sediment-free. Clear water releases pick up a new sediment load and degradation can result. The stream channel and stream gradient that existed prior to the construction of the dam was the cumulative result of past floods of various sizes and subject to change with each flood. Post-construction flows are usually of lesser magnitude and longer duration and the stream will establish a new balance in time consistent with the new flow characteristics.

It is possible for aggradation to occur downstream of a reservoir if flow releases are insufficient to transport the size or volume of sediment brought in by tributary streams. Stream flow regulation, which is an objective in dam construction and reservoir operation, is sometimes overlooked in assessing stream system response to this activity by man. The reduction in flood magnitude and stage downstream of dams as a result of reservoir operation can result in greatly increased hydraulic gradients and degradation in tributaries downstream of the dam. A notorious bridge failure on the Big Sioux River was, in part, attributable to such a condition.

Land Use Changes. Agricultural activities, urbanization, commercial development, and construction activities also contribute to gradation problems in streams. Clear cutting of forests, and the destruction of grasslands by overgrazing, burning and cultivation can accelerate erosion, causing streams draining these areas to become overloaded with sediment (i.e., excess sediment supply). As the overload persists, the stream system aggrades and increases its slope to increase its sediment transport capacity.

Construction and developing urban and commercial areas can affect stream gradient stability. Fully developed urban areas are low sediment producers because of impervious areas and lawns, but tend to increase the magnitude of runoff events and reduce their duration. The response of a small stream system to these changes is degradation, changes in planform (eg., increased sinuosity), and channel widening downstream of the urbanized area. However, if the urbanized area is small relative to the basin of the stream in which it is located, the net effect will probably be small.

Natural Changes. Natural causes of stream gradient instability are primarily natural channel alterations, earthquake, tectonic and volcanic activities, climatic change, fire, and channel bed and bank material erodibility.

Cutoffs and chute development associated with channel straightening are the most common natural channel alterations. This results in a shorter flow path, a steeper channel gradient, and an increase in sediment transport capacity. Significant bank erosion and degradation progressing to an upstream control can result. Downstream of the cutoff, aggradation will occur.

Severe landslides, mud flows, uplifts and lateral shifts in terrain, and liquefaction of otherwise semi-stable materials are associated with earthquakes and tectonic activities. The response to these activities include channel changes, scour or deposition locally or system-wide, headcutting and bank instability.

Alluvial fans, discussed under Valley Setting, are among the few naturally occurring cases of channel aggradation.

2.4.4 Stream System Response

Streambed aggradation or degradation affects not only the stream in which the gradation change is initiated, but also tributaries to the stream and the stream to which it is tributary. Thus, the stream system is in an imbalanced sediment supply-sediment transport capacity condition, and it will seek a new state of equilibrium. A few examples are cited to illustrate the system-wide response to gradation changes. These examples also illustrate the use of several geomorphic concepts introduced in Section 2.3 and the discussion of Section 2.4.3.

Example 1. A degrading principal stream channel will cause tributaries to the stream to degrade, thus contributing additional sediment load to the degrading stream. This larger sediment load will slow the rate of degradation in the principal stream channel and may halt or reverse it for a period of time if the contribution is large enough or if a tributary transports material which armors the bed of the degrading stream.

Using equation 4, the basic response of the principal stream can be expressed as:

$$Q S^{+} \sim Q_s^{+} D_{50}$$

Here, it is assumed that water discharge (Q) and sediment size (D_{50}) remain unchanged. (Note: When neither + or - appears as a superscript in the Lane relationship, conditions remain unchanged). Thus, the increase in sediment discharge (Q_s^{+}) derived from the tributary stream must result in an increase in slope (S^{+}) on the master stream if the geomorphic balance expressed by the Lane relationship is to hold. This increase in slope on the principal stream then slows or reverses the original degradation of the principal stream which initiated the stream system response.

Example 2. The sediment supply available for transport by a reach of stream may be reduced by changes in the watershed which reduce erosion, mining of sand and gravel from the streambed upstream of the reach, or the construction of a dam to impound water upstream of the reach. In general, for the two latter cases, sediment transported by the stream is trapped in the mined areas or reservoir and mostly clear water is released downstream. Figure 10 illustrates the principle by use of the example of a dam. Referring to equation (4), a decrease in sediment discharge Q_s will cause a decrease in slope S, if the discharge Q and median sediment size D_{50} remain constant, or:

$$Q S^{-} \sim Q_s^{-} D_{50}$$

The original equilibrium channel gradient (Figure 10) is represented by the line CA. A new equilibrium grade represented by $C'A$ will result from a decrease in sediment supply. The dam is a control in the channel which prevents the effects from extending upstream. Except for the channel control formed by the dam, similar effects are experienced at any location which undergoes a reduction in sediment supply.

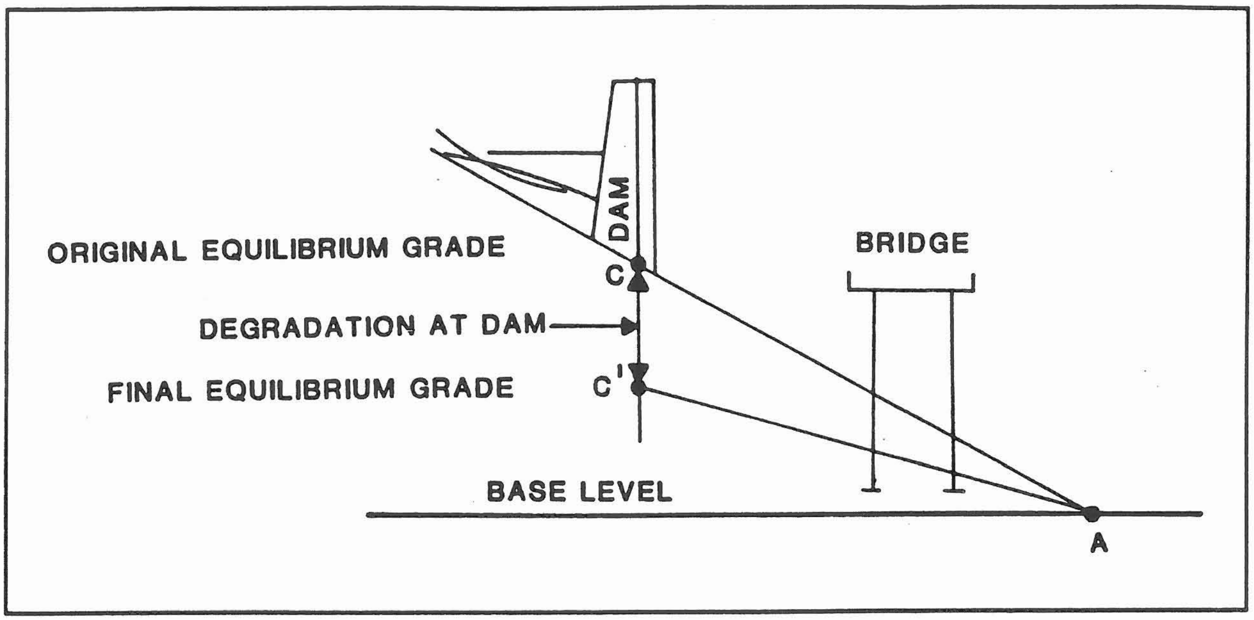
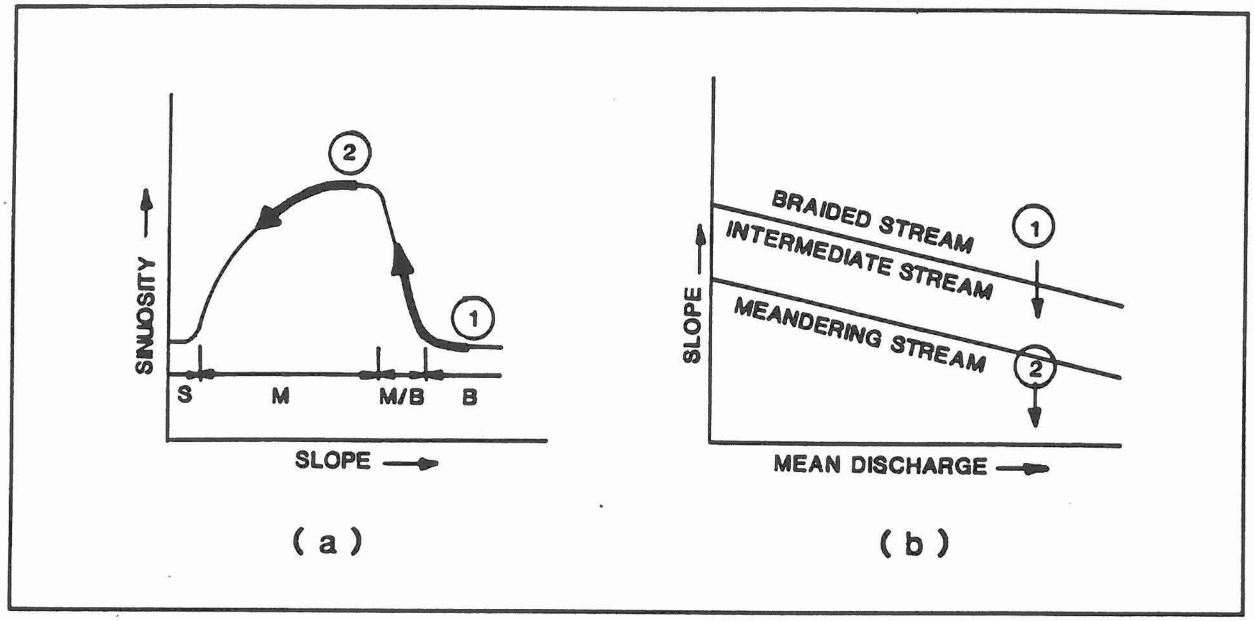


Figure 10. Changes in channel slope in response to a decrease in sediment supply at point C.

Referring to Figure 7, for a low sinuosity braided stream, this decrease in slope below the dam could result in an increase in sinuosity and a change in planform toward a combination meandering/braided stream. If the stream below the dam were initially a meandering stream at near maximum sinuosity for the original slope, the decrease in slope below the dam could shift the planform of the stream toward a reduced sinuosity, meandering thalweg channel. These changes in plotting position are illustrated as (1) and (2), respectively, on Figure 11a.

A similar result can be derived from Figure 8. For an initially braided channel pattern below the dam ((1) on Figure 11b), a decrease in slope below the dam could indicate a tendency to shift the stream's plotting position downward, possibly into the intermediate stream range (i.e., a combination of meandering and braided as on Figure 11a). For an initially meandering stream ((2) on Figure 11b), the decrease in slope below the dam could indicate a tendency toward less meandering channel (as on Figure 11a). It should be noted that both of these cases have assumed a constant discharge (Q).

As discussed in Section 2.4.3, the effects downstream of a dam are more complex than a simple reduction in sediment supply. If the reservoir is relatively small and water flow rates downstream are little affected, degradation may occur downstream initially and aggradation may then occur after the reservoir fills with sediments. Except for local scour downstream of the dam, the new equilibrium grade may approach line CA (Figure 10) over the long term. This could apply to a diversion dam or other small dam in a stream.



Figures 11a and b. Use of geomorphic relationships of Figures 7 and 8 in a qualitative analysis.

Dams constructed to impound water for flood control or water supply usually have provisions for sediment storage. Over the economic life of the project, essentially clear water is released downstream. For practical purposes, the sediment supply to downstream reaches is permanently reduced. Reservoirs developed for these purposes, however, also reduce the water flow rates downstream. Referring to equation (4), a reduction in discharge Q^- may have a moderating effect on the reduction in slope S and, consequently, on degradation at the dam CC' in Figure 10. If sediment discharge or sediment size remain constant below the dam (e.g., a tributary downstream continues to bring in a large sediment discharge), this would be expressed as:

$$Q^- S^+ \sim Q_s D_{50}$$

Considering the more likely scenario of stream response to a dam, both water discharge (Q) and sediment discharge (Q_s) would decrease. It is also possible that sediment size (D_{50}) in the reach below the dam would increase due to armoring or tributary sediment inflow. Using equation 4, this complex result could be expressed as:

$$Q^- S^{\pm} \sim Q_s^- D_{50}^+$$

Here, the resulting response in slope ($S^{\pm}$) would depend on the relative magnitude of changes in the other variables in the relationship.

3.0 HYDRAULIC FACTORS AND PRINCIPLES

3.1 Introduction

The design of highway stream crossings and countermeasures to prevent damage from stream flow requires assessment of factors that characterize stream flow and channel conditions at the bridge site. The importance of hydraulic or flow factors in the crossing design process is influenced by the importance of the bridge and by land use on the floodplain, among other things. Each of the hydraulic factors listed in the left column of Figure 12 has an effect on stream stability at a bridge crossing. Since the geometry and location of the bridge crossing can also affect stream stability, the most significant factors related to bends, confluences, alignment, and highway profile are summarized in Figure 12. Hydraulic factors are discussed in the following section. This is followed by a discussion of the geometry and location of the highway stream crossings, and some general concepts related to the hydraulic design of bridges.

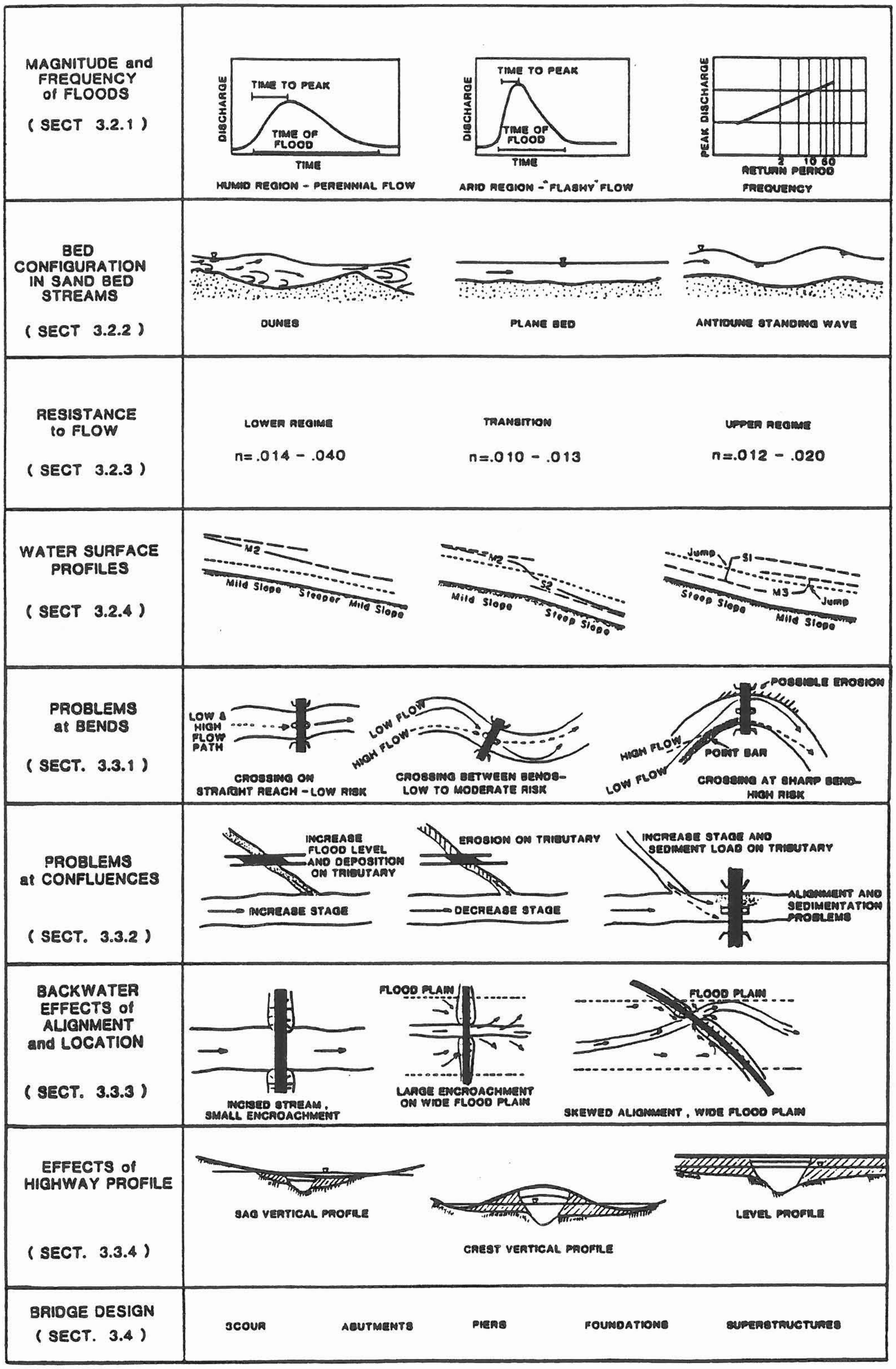
3.2 Hydraulic Factors Affecting Stream Stability

3.2.1 Magnitude and Frequency of Floods

The hydrologic analysis for a stream crossing consists of establishing peak flow-frequency relationships and such flow-duration hydrographs as may be necessary. Flood-frequency relationships are generally defined on the basis of a regional analysis of flood records, a gaging station analysis, or both. Regional analyses have been completed for all states by the U. S. Geological Survey, and the results are generally applicable to watersheds which are unchanged by man. Flood-frequency relationships at gaged sites can be established from station records which are of sufficient length to be representative of the total population of flood events on that particular stream. The Pearson Type III distribution with log transformation of flood data is recommended by the Water Resources Council (1981) for station flood data analysis.[13] Where flood estimates by regional analysis vary from estimates by station analysis, factors such as gaging station record length and the applicability of the regional analysis to that specific site should be considered, as well as high water information, flood data, and information of floods at existing bridges on the stream.

The term "design flood" is purposely avoided in the above discussion because of the implication that a stream crossing can be designed for a unique flood event. In reality, a range of events should be examined to determine which design condition is most advantageous, insofar as costs and risks are concerned. If a design flood is designated for purposes of stream stability analysis, it probably should be that event which causes the greatest stress to the highway stream crossing system, that is, the flood magnitude and stage which is at incipient overtopping of the highway.

Figure 12. Hydraulic and location factors that affect stream stability.



Hydrologic analysis establishes the probability of occurrence of a flood of given magnitude in any one year period. It also is the first step in establishing the probability of occurrence of the flood event which will pass through bridge waterways in the highway-stream crossing system without overtopping the highway. The FHWA HEC-19 document (Hydrology)[14] should be referred to for more detailed information and guidelines on hydrologic analysis. The second step is the determination of the stage-discharge relationship, flow and velocity distributions, backwater, scour, etc., (i.e., the hydraulics of the crossing system, as discussed in the remainder of this section).

3.2.2 Bed Configurations in Sand Bed Streams

In sand bed streams, sand material is easily eroded and is continually being moved and shaped by the flow. The interaction between the flow of the water-sediment mixture and the sand bed creates different bed configurations which change the resistance to flow, velocity, water surface elevation and sediment transport. Consequently, an understanding of the different types of bed forms that may occur and a knowledge of the resistance to flow and sediment transport associated with each bed form can help in analyzing flow in an alluvial channel. More specific to this discussion, it is necessary to understand what bed forms will be present so that the resistance to flow can be estimated and flood stages and water surface profiles can be computed.

Flow Regime. Flow in alluvial channels is divided into two regimes separated by a transition zone.[4] Forms of bed roughness in sand channels are shown in Figure 13a, while Figure 13b shows the relationships between water surface and bed configuration. The flow regimes are:

* The lower flow regime, where resistance to flow is large and sediment transport is small. The bed form 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 bed form; 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 bed form.

* 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 bed form changes.

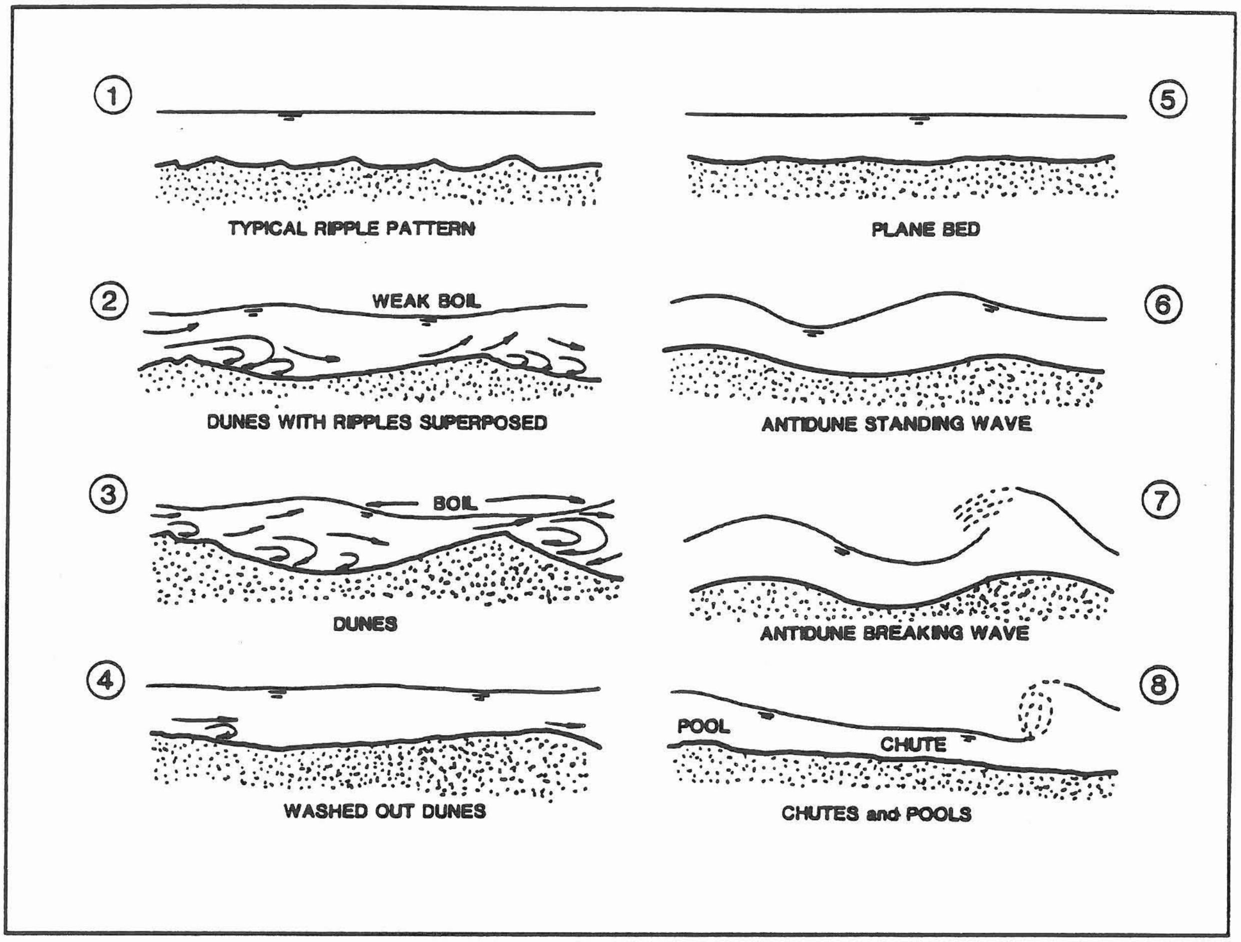


Figure 13(a). Forms of bed roughness in sand channels (After [4]).

* The upper flow regime, in which resistance to flow is small and sediment transport is large. The usual bed forms 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.

<u>Effects of Bedforms at Stream Crossings</u>. At high flows, most sand bed stream channels shift from a dune bed to a transition or a plane bed configuration. The resistance to flow is then decreased to one-half to one-third of that preceding the shift in bed form. The increase in velocity and corresponding decrease in depth may increase scour around bridge

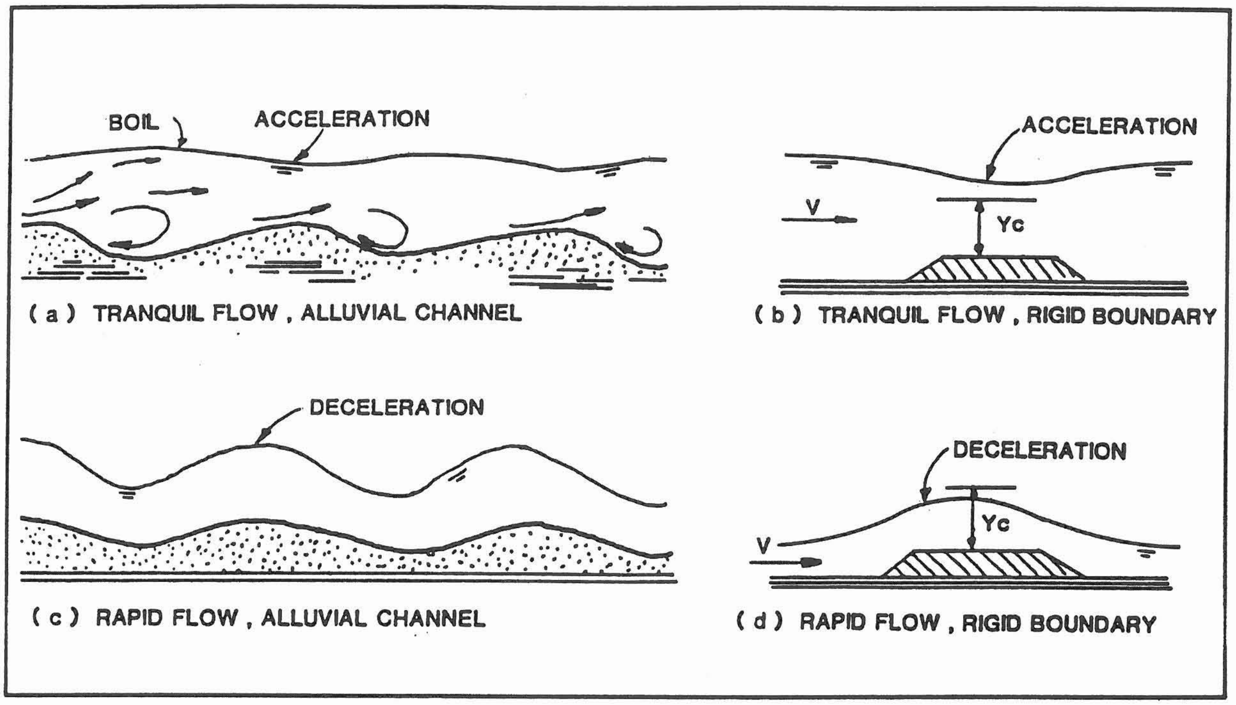


Figure 13(b). Relation between water surface and bed configuration (After [4]).

piers, abutments, spur dikes or banks and may increase the required size of riprap. However, maximum scour depth with a plane bed can be less than with dunes because of the absence of dune troughs. On the other hand, the decrease in stage resulting from planing out of the bed will decrease the required elevation of the bridge, the height of embankments across the floodplain, the height of any dikes, and the height of any channel control works that may be needed. The converse is also true.

Another effect of bed form on a highway crossing is that with dunes on the bed, there is a fluctuating pattern of scour on the bed and around piers. The average height of dunes is approximately 1/3 to 1/2 of the average depth of flow, and the maximum height of a dune may approach the average depth of flow. If the depth of flow is 10 feet, maximum dune height may be on the order of 10 feet, half of which would be below the mean elevation of the bed. With the passage of a dune through a bridge opening, an increase in local scour would be anticipated when the trough of the dune arrives at the bridge. It has been determined experimentally that local scour increases by 30 percent or more over equilibrium scour depth with the passage of a dune trough.

A very important effect of bed forms and bars is the change of flow direction in channels. At low flow, the bars can be residual and cause high velocity flow along or at a pier or other structures in the streambed, causing deeper than anticipated scour.

Care must be used in analyzing crossings of sand bed streams in order to anticipate changes that may occur in bed forms and the impact of these changes on the resistance to flow, sediment transport, and the stability of the reach and highway structures. As described in Section 3.2.3, with a dune bed, the Manning n could be as large as 0.040. Whereas, with a plane bed, the n value could be as low as 0.012. A change from a dune bed to a plane bed, or the reverse, can have an appreciable effect on depth and velocity. In the design of a bridge or a stream stability or scour countermeasure, it is good engineering practice to assume a dune bed (large n value) when establishing the water surface elevations, and a plane bed (low n value) for calculations involving velocity.

3.2.3 Resistance to Flow

Use of the Manning's equation to compute flow in open channels and floodplains assumes one-dimensional flow. Procedures for summing the results of computations for subsections to obtain results for the total cross section involve use of the following assumptions: (1) mean velocity in each subsection is the same, (2) the total force resisting flow is equal to the sum of forces in the subsections, and (3) total flow in the cross section is equal to the sum of the flows in the subsections. This implies that the slope of the energy grade line is the same for each subsection. Assumption (3) is the basis for computing total conveyance for a cross section by adding conveyances of subsections.

Resistance to Flow in Channels. The general approach for estimating the 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.[4,15] Equation (5) is used to compute the equivalent material roughness coefficient "n" for a channel:

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

where: n_b = the base value for 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, and

m = correction factor for sinuosity of the channel

Table 2 provides base n values for stable channels and sand channels, while Table 3 provides adjustment factors for use in equation (5). Reference [4,16] provides more detailed descriptions of conditions that affect the selection of appropriate values.

Table 2. Base values of Manning's n (n_b).

Channel or flood-plain type	Median size, bed material: Millimeters	Median size, bed material: Inches	Base n value: Benson and Dalrymple	Base n value: Chow
Sand channels (Only for upper regime flow where grain roughness is predominant.)	0.2	—	0.012	—
	.3	—	.017	—
	.4	—	.020	—
	.5	—	.022	—
	.6	—	.023	—
	.8	—	.025	—
	1.0	—	.026	—
Stable channels and flood plains				
Concrete	—	—	0.012 - 0.018	0.011
Rock cut	—	—	—	.025
Firm soil	—	—	.025 - .032	.020
Coarse sand	1 - 2	—	.026 - .035	—
Fine gravel	—	—	—	.024
Gravel	2 - 64	0.08 - 2.5	.028 - .035	—
Coarse gravel	—	—	—	.026
Cobble	64 -256	2.5 - 10.1	.030 - .050	—
Boulder	< 256	< 10.1	.040 - .070	—

Resistance to Flow in Sand Bed Channels. The value of n varies greatly in sand bed channels because of the varying bed forms that occur with lower and upper flow regimes. Figure 14 shows the relative resistance to flow in channels in lower regime, transition, and upper regime flow and the bed forms which exit in each for regime.

Table 3. Adjustment factors for the determination of n values for channels.

	Conditions	n Value	Remarks
n_1	Smooth	0	Smoothest Channel
	Minor	0.001-0.005	Slightly Eroded Side Slopes
	Moderate	0.006.0.010	Moderately Rough Bed and Banks
	Severe	0.011-0.020	Badly Sloughed & Scalloped Banks
n_2	Gradual	0	Gradual Changes
	Alternating Occasionally	0.001-0.005	Occasional Shifts From Large to Small Sections
	Alternating Frequently	0.010-0.015	Frequent Changes in Cross-Sectional Shape
n_3	Negligible	0-0.004	Obstructions < 5% of Cross-Section Area
	Minor	0.005-0.015	Obstruction < 15% of Cross-Section Area
	Applicable	0.020-0.030	Obstruction 15-50% of Cross-Section Area
	Severe	0.040-0.060	Obstruction > 50% of Cross-Section Area
n_4	Small	0.002-0.010	Flow Depth > 2 x 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	Minor	1.00	Sinuosity < 1.2
	Applicable	1.15	1.2 Sinuosity < 1.5
	Severe	1.30	Sinuosity > 1.5

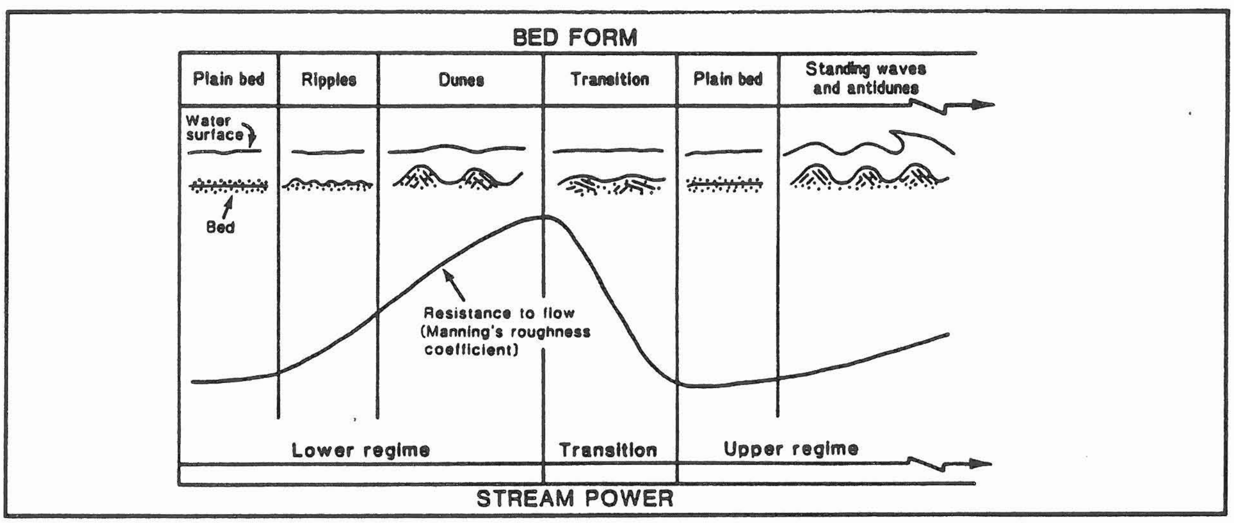


Figure 14. Relative resistance to flow in sand bed channels (After [16]).

Sand bed channels with bed materials having a median diameter from 0.14 mm to 0.4 mm usually plane out during high flows. Manning's n-values change from as large as 0.040 at low flows to as small as 0.012 at high flow. Table 4 provides typical ranges of n-values for sand bed channels.

Table 4. Manning's roughness coefficients for alluvial sand bed channels (no vegetation[1]).

Subcritical flow, FR < 1	
plane bed	0.014 - .020
ripples	.018 - .030
dunes	.020 - .040
washed out dunes or transition	.014 - .025
plane bed	.010 - .013
Supercritical flow, FR > 1	0.010 - 0.015
standing waves	.012 - .020
antidunes	.012 - .020

[1] Data is limited to sand channels with $D_{50} < 1.0$ mm.

Resistance to Flow in Coarse Material Channels. A coarse-material channel may range from a gravel bed channel up to the cobble/boulder channels typical of mountainous regions. The latter type channels may have bed material that is only partly submerged making it difficult to determine the channel roughness. However, for gravel and small cobble/boulder bed channels analysis of data from many rivers, canals and flumes shows that channel roughness can be predicted by the equation:[17]

$$n = 0.04 D_{50}^{\frac{1}{6}} \qquad (6)$$

where: D_{50} is measured in inches.

Alternately, Limerinos developed equation (7) from samples on streams having bed materials ranging in size from small gravel to medium size boulders.[18]

$$n = \frac{(0.0926) R^{\frac{1}{6}}}{1.16 + 2.0 \log\left(\frac{R}{D_{84}}\right)} \qquad (7)$$

where: R = hydraulic radius

D_{84} = the 84 percentile size of bed material

All dimensions are in feet. Flow depth, Y_o, may be substituted for the hydraulic radius, R, in wide channels ($W/Y_o > 10$). Note that equation (7) also applies to sand bed channels in upper regime flow.[16]

The alternative to use of equations (6) or (7) for gravel bed streams is to select a value of n from Table 2. Because of the range of values in the table, it would be advisable to verify the selected value by use of one of the above equations if flow depth or velocities will significantly affect a design. Reference [4] also gives equations for this case.

Resistance to Flow on Floodplains. Arcement and Schneider modified equation (5) for channels to make it applicable to the estimation of n-values for floodplains.[16] The correction factor for sinuosity, m, becomes 1.0 for floodplains, and the value for variations in size and shape, n_2, is assumed equal to zero. Equation (5), adapted for use on floodplains, becomes:

$$n = n_b + n_1 + n_3 + n_4 \tag{8}$$

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

n_1 = value to correct for surface irregularities

n_3 = value for obstructions

n_4 = value for vegetation

Selection of the base value for floodplains is the same as for channels. Reference [16] is recommended for a detailed discussion of factors which affect flow resistance in floodplains.

3.2.4 Water Surface Profiles

The water surface profile in a stream or river is a combination of gradually varied flow over long distances, and rapidly varied flow over short distances. Due to various obstructions in the flow, such as bridges, the actual flow depth over longer reaches is either larger or smaller than the normal depth defined by Mannings 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 (eg. see [4]), which can be implemented manually or more commonly by computer programs such the FHWA WSPRO program, or the Corps of Engineers HEC-2 program. A qualitative analysis of the general characteristics of the backwater curve is often useful prior to quantitative evaluation. Such an analysis requires locating control points, determining the type of profile upstream and downstream of the control points, and then sketching the backwater curves. For example, Figure 12 illustrates several typical profiles that would result from a control represented by a change in bed slope. Reference [4] provides a detailed discussion of water surface profiles for gradually varied flow.

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 (see Figure 15) are a combination of analytical and empirical relationships. The FHWA document Hydraulics of Bridge Waterways ([19]), 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 WSPRO and HEC-2 include analysis of bridge backwater, but do not calculate undular jump conditions or the flow through the bridge when flow accelerations are large, that is, large change in velocity either in magnitude or direction.

Superelevation of Water Surface at Bends. Because of the change in flow direction which results in centrifugal forces, there is a superelevation of the water surface in bends. The water surface is higher at the concave bank than at the convex bank (Figure 16). The resulting transverse slope can be evaluated quantitatively. By assuming velocity equal to average velocity, the following equation was derived for superelevation for subcritical flow:[20]

$$\Delta Z = \frac{V^2}{g r_c}(r_o - r_i) \tag{9}$$

where: g = acceleration of gravity, ft/s^2;

r_o = radius of the outside bank at the bend, ft;

r_i = radius of the inside bank, ft;

r_c = radius of the center of the stream, ft;

ΔZ = the difference in water surface elevation between the concave and convex banks, ft; and

V = average velocity, ft/s

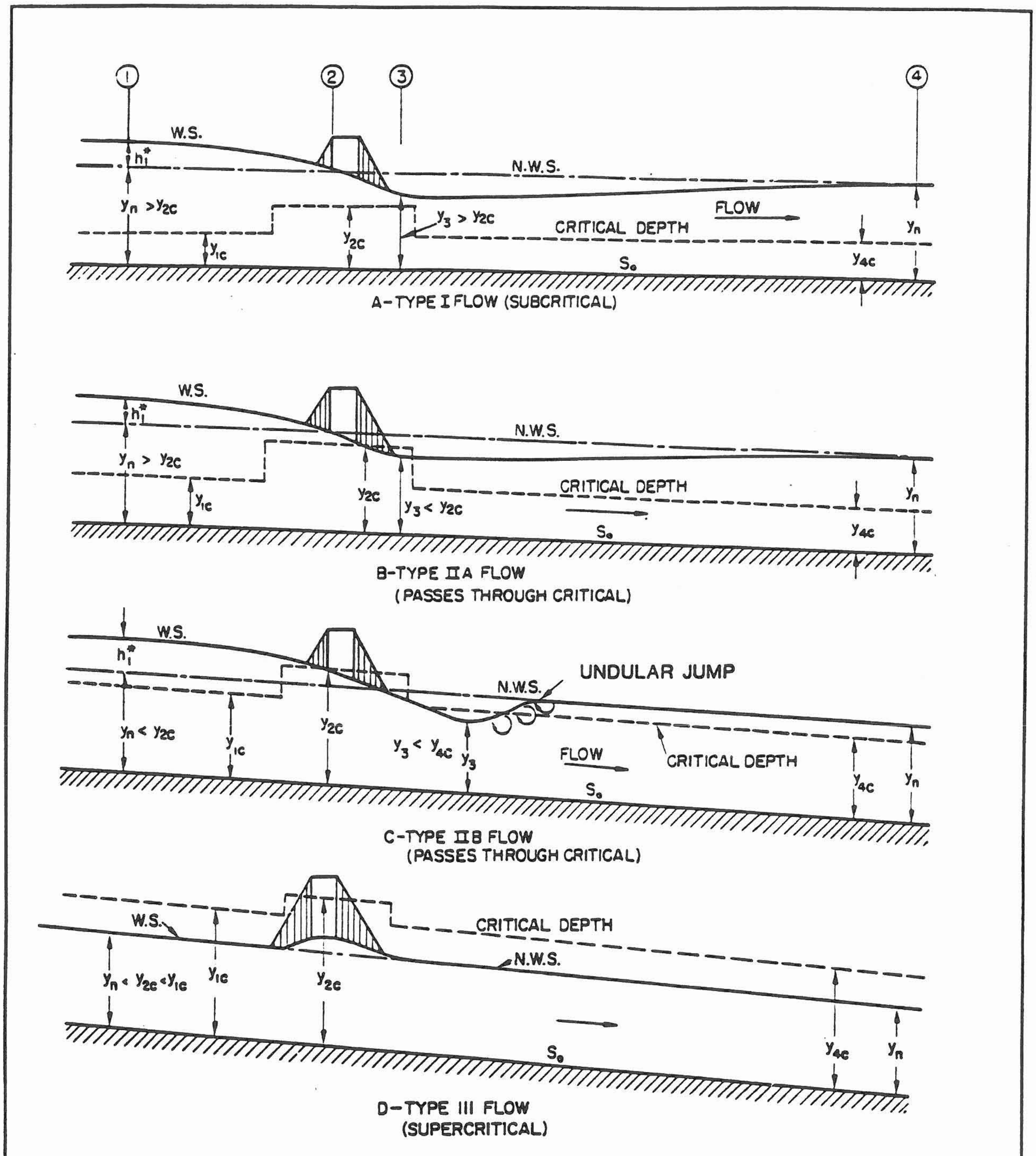


Figure 15. Types of water surface profiles through bridge openings (After [19]).

Other equations for superelevation are given in [4].

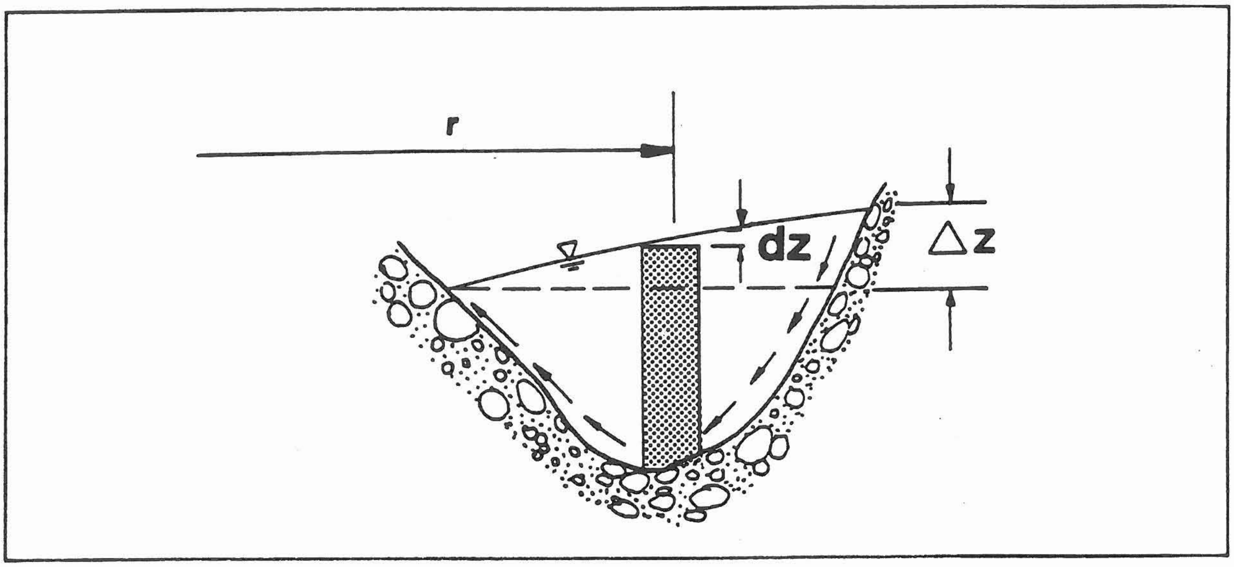


Figure 16. Superelevation of water surface in a bend.

3.3 Geometry and Location of Highway Stream Crossings

3.3.1 Problems at Bends

The highway stream crossing location is important because of the inherent instability of streams at some locations (See for example Chapter 2.0) and because the crossing system can contribute to instability. In general, a crossing on a straight reach is preferred as stability problems are usually minor. Low flow and high flow paths (thalwegs) are generally similar for a straight reach, reducing the risk of problems related to alignment and orientation of bridge piers and superstructures (see Figure 12).

For a relatively stable meandering stream, a bridge crossing at the inflection point between bends generally reduces the risk of instability problems. Here the low flow and high flow paths are comparable (as shown in Figure 12) and the crossing is in a zone where deposition and erosion are usually moderate. However, countermeasures against meander migration may still be required.

More hydraulic problems occur at alluvial stream crossings at or near bends than at all other locations because bends are naturally unstable. In addition, ice and floating debris tend to be greater problems in bends than in straight reaches. Other problems at bends include the shifting thalweg which can result in unanticipated scour at piers because of changes in flow direction and velocities, and non-uniform velocity distribution which causes scour of the bed and bank at the outside of the bend and deposition in the inside of the bend (see Figure 12). The high velocities at the outside of the bend or downstream of the bend can substantially contribute to local scour on abutments and piers.

3.3.2 Problems at Confluences

Hydraulic problems may also be experienced at crossings near stream confluences. Crossings of tributary streams are affected by the stage of the main stream. (See Chapter 2.0). Aggradation of the channel of the tributary may occur if the stage of the main stream is high during a flood on the tributary, and scour in the tributary may occur if the stage in the main stream is low. Similarly, problems at a crossing of the larger stream can result from varying flow distribution and flow direction at various stages in the stream and its tributary, and from sediment deposited in the stream by the tributary. (See Figure 12). Tributaries entering the main channel downstream of a main channel bridge can also cause varying flow distribution and direction at various stages (flows) in the main channel and the tributary.

3.3.3 Backwater Effects of Alignment and Location

As flow passes through a channel constriction, most of the energy losses occur as expansion losses downstream of the contraction. This loss of energy is reflected by a rise in the water surface and the energy line upstream from the constriction. Upstream of bridges, the rise in water level above the normal water surface (that which would exist without the bridge) is referred to as the bridge backwater (see Figure 15). However, many bridges do not cause backwater even at high flows even though they constrict the flows.[4] Hydraulic engineers are concerned with backwater with respect to flooding upstream of the bridge; backwater elevation with respect to the highway profile; and the effects on sediment deposition upstream, scour around embankments, contraction scour due to the constriction, and local scour at piers.

The effects of highway-stream crossing alignments on backwater conditions shown in Figure 17 are based on:

* Backwater resulting from a long skewed or curved roadway embankment (Figure 17a) may be quite large for wide floodplains. In effect, the bridge opening is located up-valley from one end of the embankment and the water level at the downstream extreme of the approach roadway, as at point A in Figure 17a, can be significantly higher than at the bridge.

* Backwater in an incised stream channel without substantial overbank flow (Figure 17b) is seldom large, but contraction and local scour may be severe. Backwater results from encroachment in the channel by approach embankments and from piers located in the channel.

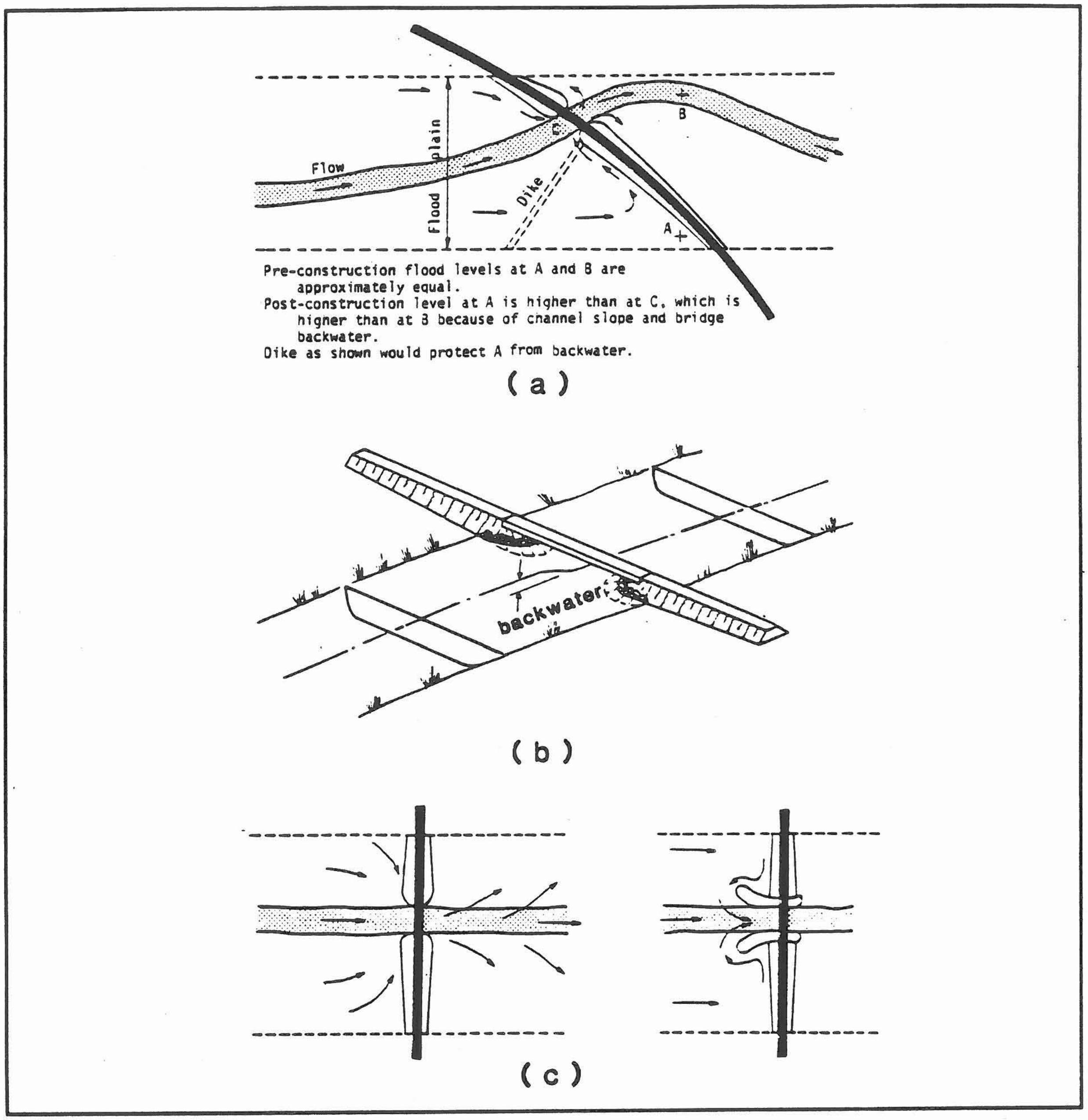


Figure 17. Backwater effect associated with three types of stream crossings: (a) a skewed alignment across a floodplain, (b) constriction of channel flow, and (c) constriction of over-bank flow (After [21]).

* Backwater resulting from a normal crossing of the valley where road approach fills block overbank flow (Figure 17c) may be significantly greater than in an incised channel. General and local scour may be severe if a significant quantity of flow is diverted from the floodplain to the bridge waterway.

3.3.4 Effects of Highway Profile

A highway stream crossing is a system consisting of the stream and its floodplain, the bridge(s) provided to pass floods, and the approach roadways on the floodplain. All floods which occur during the life of the crossing system will pass either through the bridge waterways provided or through the waterways and over the highway. The highway profile and alignment control the quantity of flow which must pass through waterway openings. Flood frequency should be considered in the design of bridge components and may influence highway profile and alignment. Consideration of the flood magnitude and frequency which must pass through bridge openings does not preclude acceptance or acknowledgement of possible damage during an extreme event.

The stage-discharge relationship for the stream and backwater associated with a crossing design are the hydraulic considerations for establishing the highway profile. Profile alternatives available for consideration are dependent on site topography and other site constraints, such as land use, traffic requirements, and flood damage potential. Figure 18a, b, and c illustrate profile alternatives, namely, a sag vertical curve, a crest vertical curve on the bridge or a rolling profile, and a level profile. A distinctive aspect of the sag vertical curve, as depicted in Figure 18a, and the level profile, Figure 18c, is the certainty that the bridge structure will be submerged before overflow of the roadway will occur. Therefore, the magnitude and probability of occurrence of such a flood event should be considered in the design of the waterway opening and bridge components. A variation of the sag vertical curve where the low point of the curve is located on a floodplain rather than on the bridge affords relief to the bridge waterway. Bridges on level profiles and sag vertical curves are susceptible to debris accumulation on the superstructure, impact forces, buoyant forces, and accentuated contraction and local scour.

The rolling profile illustrated in Figure 18b provides protection to the bridge in that flood events exceeding the stage of the low point in the sag vertical curve will, in part, flow over the roadway. This relieves the bridge and the bridge waterway of stresses to which bridges on sag vertical curves and level profiles are subjected.

When this superstructure is submerged (pressure flow through the bridge), pier scour is increased. In some cases the local scour with pressure flow will be two to three times deeper than for free flow.

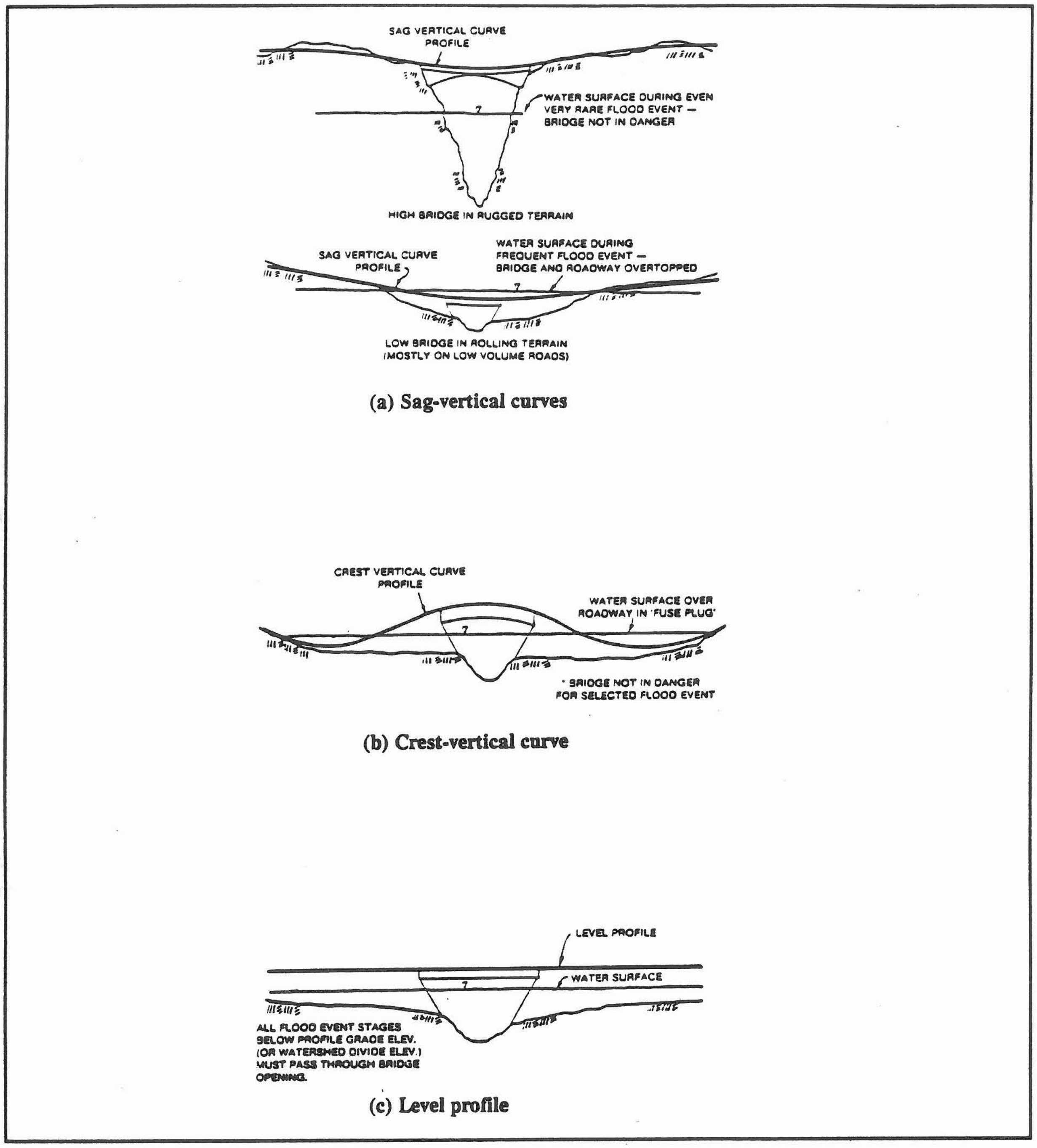


Figure 18. Various highway profiles: (a) sag-vertical curves, (b) crest-vertical curve, and (c) level profile.[22]

3.4 Bridge Design

The design of bridge components is of significant importance to the local stability of a stream because of scour that is attributable to the encroachment on the stream. Since the stability of bridges is dependent on countermeasures against stream instability, it is prudent to utilize designs which minimize undesirable stream response, to the extent practicable. This applies to component design as well as to the design of the total crossing system. The term countermeasure, as used here, is not necessarily an appurtenance to the highway stream crossing, but may be an integral part of the highway or bridge.

For example, the location and size of waterway openings influence stream stability locally. Encroachment in the stream channel at abutments and by piers reduces the channel section and may cause significant contraction scour. Severe constriction of floodplain flow may cause bank failures and exaggerated contraction scour in the bridge waterway. Auxiliary (relief) openings should be carefully designed to avoid excessive diversion of floodplain flow to main channel bridge openings on wide floodplains and at skewed crossings of floodplains.

3.4.1 Scour at Bridges

Scour at bridges consists of three components: (1) long-term aggradation or degradation of the stream channel (natural or man-induced), (2) contraction scour due to constriction or the location of the bridge, and (3) local scour. In general, the three components are additive.

Scour can be related to the following factors: (1) channel slope and alignment; (2) channel shifting; (3) bed sediment size distribution; (4) antecedent floods and surging phenomena; (5) accumulation of debris, logs, or ice; (6) flow contraction, flow alignment, and flow depth; (7) pier geometry and location; (8) type of foundation; (9) natural or man-induced modification of the stream; and (10) failure of a nearby structure.

The rate of scour depends on the erosive forces exerted on the channel boundary and the resistance of the material to erosion. Resistance to erosion in fine cohesive material results from chemical bonding. Cohesionless materials do not exhibit such properties and resistance to erosion depends primarily on bed sediment size distribution and density.

Under steady flow conditions, scour situations gradually reach equilibrium condition. However, most streams are active during a very short period of time and equilibrium scour conditions are not necessarily attained during a single event. Bridge crossings are generally subjected to unsteady flow conditions, and a series of events are required to reach equilibrium or maximum scour depth. During a typical flood hydrograph, experiments indicated that scour tends to lag behind discharge and maximum scour depth occurs after the flood peak. Deposition often occurs during the recession of the hydrograph, and the maximum scour depth measured after the flood is generally less than the maximum depth of scour reached during the flood event. Where erodible bed material is stratified, more resistant layers can cover more easily eroded material and special protective measures may be taken to prevent scour of the resistant layer. While armoring of the bed by the coarser material size fraction can temporarily reduce the rate of degradation and stabilize the stream system, armoring cannot be counted on as a long-term solution. Flows exceeding a given design event could disrupt the armor layer, resulting in degradation (see Section 4.6.6).

Gravel mining in the streambed can cause severe stream instability. Therefore, it is essential to monitor sand and gravel mining so that countermeasures can be installed to stabilize the stream in the vicinity of a highway facility, and, where possible, mining should be managed so that instabilities in the stream system will be minimized. In most cases, removal of sand and gravel has caused deepening and widening of the channel. These wider, deeper reaches act as sinks for the sediment loads and may trap the finer clays, altering the environment. (See additional discussion in Section 2.4.3).

Methods and equations for determining scour at piers and abutments are given in HEC No. 18 - Evaluating Scour at Bridges [23], [4], and [24].

3.4.2 Abutments

Bridge abutments are classified as spillthrough or vertical. Both types of abutment are susceptible to damage by scour dependent of flow distribution, foundation materials, velocities and other factors. However, scour at spillthrough abutments is about 50 percent smaller than at vertical abutments subjected to the same scouring actions.

In addition to the effects of abutment shape, scour at abutments is affected by the skew of approach flow at the abutment, soils materials subjected to the scouring action, encroachment on the floodplain and in the channel, and the amount of overbank flow diverted to the bridge waterway by approach fills to the bridge. Equations and methods for computing abutment scour and countermeasures for scour are provided in [23] and [4].

3.4.3 Piers

The number of piers in any stream channel should be limited to a practical minimum, and piers should not be located in the channel of small streams, if it is possible to avoid such locations. Piers properly oriented with the flow do not contribute significantly to bridge backwater, but they can contribute to contraction scour. In some locations, severe scour has developed immediately downstream of bridges because of eddy currents and because piers occupy a significant area in the channel. Lateral instability as well as vertical scour may occur.

Piers should be aligned with flow direction at flood stage in order to minimize the opportunity for drift to be caught, to reduce the contraction effect of piers in the waterway, to minimize ice forces and the possibility of ice dams forming at the bridge, and to minimize backwater and local scour. Pier orientation is difficult where flow direction changes with stage or time. Cylindrical piers or some variation thereof, are probably the best alternative if orientation at other than flood stage is critical. Raudkivi reported that a row of cylindrical columns will produce shallower scour than a solid pier where the angle of attack is greater than 5 to 10 degrees.[25] He also found that cylindrical piers with five-diameter spacing produced about 1.2 times the local scour depth at a single column of equal diameter. Pier shape is also a factor in local scour. A solid pier will not collect as much debris as a pile bent or a multiple-column bent. Rounding or streamlining the leading edges of piers helps to decrease the accumulation of debris and reduces local scour at the pier and contraction scour occasioned by the increased constriction. Recent studies [26] have provided additional data on the effects of footings and the behavior or pile groups.

Piers located on a bank or in the stream channel near the bank are likely to cause lateral erosion of the bank. Piers located near the streambank in the floodplain are vulnerable because they can cause bank scour. They are also vulnerable to failure from undermining by meander migration and bank caving. Piers which must be placed in locations where they will be especially vulnerable to scour damage should be founded at elevations safe from undermining or otherwise protected.

3.4.4 Bridge Foundations

The foundation is the bridge component which is most vulnerable to attack by floods. Examination of boring logs and plots of the profiles of various subsurface materials is important to the prediction of potential scour depths as well as to estimation of the bearing capacity of the materials. Refer to [23,24] for a complete discussion of scour mechanics.

The types of foundations used for bridges include spread footings, footings on piles or drilled shafts, and caissons. Spread footings are used where sound rock is relatively shallow. Failures have occurred where spread footings were set in erodible rock and where armoring in the streambed was inadequate to prevent scour.

Piling usually are dependent on the surrounding material for skin friction and lateral stability. In some locations, they can be carried to bedrock or other dense materials for bearing capacity. Tip elevation for piling should be based on estimates of potential scour depths as well as bearing in order to avoid losing lateral support and load carrying capacity after scour. Pile bearing capacity derived from driving records has little validity if the material through which the piles were driven is scoured away during a flood.

Caissons are used in large rivers and are usually sunk to dense material by excavation inside the caisson. Founding depths are such that scour is not usually a problem after construction is completed. Severe contraction scour has developed at some bridges, however, because of contraction of flow from the large piers.

Attention should be given to potential scour and the possibility of channel shifts in designing foundations on floodplains. Also, the thalweg in channels should not be considered to be in a fixed location. Consideration should be given, therefore, to duplicating the foundation elevations of the main channel piers on adjacent floodplain piers. The history of stream channel activity can be very useful in establishing foundation elevations. (See Chapter 2.0).

3.4.5 Superstructures

Hydraulic forces that should be considered in the design of a bridge superstructure include buoyancy, drag, and impact from ice and floating debris. The configuration of the superstructure should be influenced by the highway profile, the probability of submergence, expected problems with ice and debris, and flow velocities, as well as the usual economic, structural and geometric considerations. Superstructures should be made a structurally integral part of the piers and abutments to provide structural redundancy, that is, alternate load paths in case of failure of one bridge element.

Buoyancy. The weight of a submerged or partially submerged bridge superstructure is the weight of the superstructure less the weight of the volume of water displaced. The volume of water displaced may be much greater than the volume of the superstructure components if air is trapped between girders. Also, solid parapet rails and curbs on the bridge deck can increase the volume of water displaced and increase bouyant forces. The volume of air trapped under the superstructure can be reduced by providing holes (vents) through the deck between structural members. Superstructures should be anchored to piers to counter buoyant forces and to resist drag forces. Continuous span designs are also less susceptible to failure from buoyancy than simple span designs.

Drag Forces. Drag forces on a submerged or partially submerged superstructure can be calculated by equation (10):

$$F_d = C_d \rho H \frac{V^2}{2} \tag{10}$$

where: F_d = drag force per unit of length, lbs/ft

C_d = coefficient of drag

ρ = density of water, slugs/ft^3

H = depth of submergence, ft

V = velocity of flow, ft/sec

The coefficient of drag can be taken as 2.0 to 2.2 based on usual Reynolds numbers in natural streams and the usual shape of bridge superstructures. The density of fresh water is usually taken as 1.94 slugs/ft^3.

Floating Debris and Ice. Where bridges are destroyed by debris and ice, it usually is due to accumulations against bridge components. Waterways may be partially or totally blocked, creating hydraulic conditions that cause or increase scour at pier foundations and bridge abutments, structural damage from impact and uplift, and overtopping of roadways and bridges. Floating debris is a hydraulic problem at highway stream crossings nation-wide, but the greatest problems are in the Pacific Northwest and in the upper and lower Mississippi River Valley. Many debris problems exist in forested areas with active logging operations. Debris hazards occur more frequently in unstable streams where bank erosion is active and in streams with mild to moderate slopes, as contrasted with headwater streams. Debris hazards are often associated with large floods, and most debris is derived locally along the streambanks upstream from the bridge. After being mobilized, debris typically moves as individual logs which tend to concentrate in the thalweg of the stream. It is usually possible to evaluate the abundance of debris upstream of a bridge crossing and then to implement mitigation measures, such as removal and or containment, to minimize potential problems during a major flood.

Ice Forces. Superstructures may be subjected to impact forces from floating ice, static pressure from thermal movements or ice jams, or uplift from adhering ice in water of fluctuating levels. The latter is usually associated with relatively large bodies of water and superstructures in these locations should normally be high enough to be unaffected. Research is needed to define the static and dynamic loads that can be expected from ice under various conditions of ice strength and stream flow.

In addition to forces imposed on bridge superstructures by ice loads, ice jams at bridges can cause exaggerated backwater and a sluicing action under the ice. There are numerous examples of foundation failures from this orifice flow under ice as well as superstructure damage and failure from ice forces. Accumulations of ice or drift may substantially increase local pier and abutment scour especially if they are allowed to extend down to near the channel bed.

Ice also has serious effects on bank stability. For example, ice may form in bank stabilization materials, and large quantities of rock and other material embedded in the ice may be floated downstream and dumped randomly when the ice breaks up. Ice jams also threaten the stability of bridges because of the gradual increase in stage followed by the sudden release of a surge of water and ice blocks after the breakup. Banks are subjected to piping forces during the drawdown of water surface elevation after the breakup.

Debris Forces. Information regarding methods for computing forces imposed on bridge superstructures by floating debris is also lacking despite the fact that debris causes or contributes to many failures. Floating debris may consist of logs, trees, house trailers, automobiles, storage tanks, lumber, houses, and many other items representative of floodplain usage. This complicates the task of computing impact forces since the mass and the resistance to crushing of the debris contribute to the impact force.

The equation for computing impact forces is:

$$F = M dv/dt = \frac{MV^2}{2S} \tag{11}$$

where: F = the impact imparted by the debris, lbs/ft

M = the mass of the debris, lb-sec^2/ft

S = stopping distance, ft

V = the velocity of the floating debris prior to impact, ft/sec.

In addition to impact forces, a buildup of debris increases the effective depth of the superstructure and the drag coefficient may also be increased. Perhaps the most hazardous result of debris buildup is partial or total clogging of the waterway. This can result in a sluicing action of flow under the debris which can result in scour and foundation failure or a shift in the channel location from under the bridge.

4.0 ANALYSIS PROCEDURE

4.1 Problem Statement

A stable channel does not change in size, form, or position with time; however, all alluvial channels change to some extent and are somewhat unstable. For highway engineering purposes, a stream channel can be considered unstable if the rate or magnitude of change is great enough that the planning, location, design, or maintenance considerations for a highway encroachment are significantly affected. The kinds of changes that are of concern are: (1) lateral bank erosion, including the erosion that occurs from meander migration; (2) aggradation or degradation of the streambed that progresses with time; and (3) short-term fluctuations in streambed elevation that are usually associated with the passage of a flood (scour and fill). These changes are associated with instability in a stream system or in an extensive reach of stream.

Local instability caused by the construction of a highway crossing or encroachment on a stream is also of concern. This includes general scour caused by contraction of the flow, and local scour due to the disturbance of streamlines at an object in the flow, such as at a pier or an abutment. The purpose of this Section is to outline the analysis procedures that may be utilized to evaluate stream instability.

4.2 General Solution Procedure

The analysis of any complex problem should begin with an overview or general evaluation, including a qualitative assessment of the problem and its solution. This fundamental initial step should be directed towards providing insight and understanding of significant physical processes, without being too concerned with the specifics of any given component of the problem. The understanding generated from such analyses assures that subsequent detailed analyses are properly designed.

The progression to more detailed analyses should begin with application of basic principles, followed as required, with more complex solution techniques. This solution approach, beginning with qualitative analysis, proceeding through basic quantitative principles and then utilizing, as required, more complex or state-of-the-art solution procedures assures that accurate and reasonable results are obtained while minimizing the expenditure of time and effort.

The inherent complexities of stream stability, further complicated by highway stream crossings, requires such a solution procedure. The evaluation and design of a highway stream crossing or encroachment should begin with a qualitative assessment of stream stability. This involves application of geomorphic concepts to identify potential problems and alternative solutions. This analysis should be followed with quantitative analysis using basic hydrologic, hydraulic and sediment transport engineering concepts. Such analyses could include evaluation of flood history, channel hydraulic conditions (up to and including, for example, water surface profile analysis) and basic sediment transport analyses such as evaluation of watershed sediment yield, incipient motion analysis and scour calculations. This analysis can be considered adequate for many locations if the problems are resolved and the relationships between different factors affecting stability are adequately explained. If not, a more complex quantitative analysis based on detailed mathematical modeling and/or physical hydraulic models should be considered.

In summary, the general solution procedure for analyzing stream stability could involve the following three levels of analysis:

Level 1: Application of Simple Geomorphic Concepts and other Qualitative Analyses.

Level 2: Application of Basic Hydrologic, Hydraulic and Sediment Transport Engineering Concepts

Level 3: Application of Mathematical or Physical Modeling Studies.

4.3 Data Needs

The types and detail of data required to analyze a highway crossing or encroachment on a stream 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.

4.3.1 Level 1: Geomorphic and Other Qualitative Analyses

The data required for preliminary stability analyses include maps, aerial photographs, notes and photographs from field inspections, historic channel profile data, information on man's activities, and changes in stream hydrology and hydraulics over time.

The National Bridge Inspection Standards (NBIS) Program involves inspections on a 2-year cycle of the 577,000 bridges on the National Bridge Inventory. The FHWA December 1988 publication "The Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges" specifies the bridge and channel hydraulics and scour data that are evaluated and reported within the NBIS. Item 61, Channel and Channel Protection, Item 71, Waterway Adequacy, and Item 113, Scour Critical Bridges, are included. Typically, a cross section of the bridge waterway at the time of each inspection will provide a chronological picture of the bridge waterway.

Area maps, vicinity maps, site maps, geologic maps, soils maps, and land use maps each provide essential information. Unstable stream systems upstream or downstream of the encroachment site can cause instability at the site. Area maps are needed to locate unstable reaches of streams relative to the site. Vicinity maps help to identify more localized problems.

They should include a sufficient reach of stream to permit identification of stream classification, 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 deposits and rock formations and outcrops that control stream stability. Soils and land use maps provide information on soil types, vegetative cover, and land use which affect the character and availability of sediment supply.

Aerial photographs record much more ground detail than maps and are frequently available at 5-year intervals. This permits measurement of the rate of progress of bend migration and other stream changes that cannot be measured from maps made less frequently. A highway authority should periodically obtain aerial photographs of actively unstable streams that threaten highway facilities, including immediately after major floods. However, aerial photographs taken after the passage of an ice jam or immediately after a major flood must be interpreted with care and may provide misleading information regarding the rate of change.

Notes and photographs from field inspections are important to gaining an understanding of stream stability problems, particularly local stability. Field inspections should be made during high flow 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.

If historic stream profile data is available, it will provide information on channel stability. Stage trends at stream gaging stations and comparisons of streambed elevations with elevations before construction at structures will provide information on changes in stream profile. As-built bridge data and cross sections are frequently useful. Structure-induced scour should be taken into consideration where such comparisons are made.

Man's activities in a watershed are frequently the cause of stream instability. Information on urbanization, land clearing, snagging in stream channels, channelization, bend cutoffs, streambed mining, dam construction, reservoir operations, navigation projects, and other activities, either existing or planned, are necessary to evaluate the impact on stream stability.

Data on changes in morphology are important because change in a stream is rarely at a constant rate. Stream instability can often be associated with an event, such as an extreme flood or a particular activity in the watershed or stream channel. If association is possible, the rate of change can be more accurately assessed.

Similarly, information on changes in hydrology or hydraulics can sometimes be associated with activities that caused the change. Where changes in stream hydraulics are associated with an activity, changes in stream morphology are also likely to have occurred.

4.3.2 Level 2: Basic Engineering Analyses

Data requirements for basic hydrologic, hydraulic and sediment transport engineering analysis are dependent the types of analysis that must be completed. Hydrologic data needs include dominant discharge (or bank full flow), flow duration curves, and flow frequency curves. Discussion of hydrologic methods is beyond the scope of this manual; however, information can be obtained from [14] and State Highway Agency manuals. Hydraulic data needs include cross sections, channel and bank roughness estimates, channel alignment, and other data for computing channel hydraulics, up to and including water surface profiles calculations. Analysis of basic sediment transport conditions requires information on land use, soils, and geologic conditions, sediment sizes in the watershed and channel, and available measured sediment transport rates (eg., from U. S. Geological Survey gaging stations).

More detailed quantitative analyses require data on the properties of bed and bank materials and, at times, field data on bed load and suspended load transport rates. Properties of bed and bank materials that are important to a study of sediment transport include size, shape, fall velocity, cohesion, density, and angle of repose.

4.3.3 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, water and sediment routing by mathematical models (e.g., BRISTARS or HEC-6), and construction of a physical model, would both require detailed channel cross section 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, results in a relatively large scope of work.

4.4 Data Sources

Preliminary stability data may be available from government agencies such as the U. S. Army Corps of Engineers, Soil Conservation Service, local river basin commissions, and local watershed districts. These agencies may have information on historic streambed profiles, stage-discharge relationships, and sediment load characteristics. They may also have information on past and planned activities that affect stream stability. Table 5 provides a list of sources for the various data needed to assess stream stability at a site.

4.5 Level 1: Qualitative and Other Geomorphic Analyses

A flow chart of the typical steps in qualitative and other geomorphic analyses is provided in Figure 19. The six identified steps are generally applicable to most stream stability problems. These steps are discussed in more detail in the following paragraphs. As shown on Figure 19, the qualitative evaluation leads to a conclusion regarding the need for more detailed (Level 2) analysis or a decision to proceed directly to selection and design of countermeasures based only on the qualitative and other geomorphic analyses. Selection and design of countermeasures are discussed in Chapters 5.0 and 6.0, respectively.

Table 5. List of data sources (After [24]).

Topographic Maps:

(1) Quadrangle maps - U. S. Department of the Interior, Geological Survey, Topographic Division; and U. S. Department of the Army, Army Map Service.

(2) River plans and profiles - U. S. Department of the Interior, Geological Survey, Conservation Division.

(3) National parks and monuments - U. S. Department of the Interior, National Park Service.

(4) Federal reclamation project maps - U. S. Department of the Interior, Bureau of Reclamation.

(5) Local areas - commercial aerial mapping firms.

(6) American Society of Photogrammetry.

Planimetric Maps:

(1) Plats of public land surveys - U. S. Department of the Interior, Bureau of Land Management

(2) National forest maps - U. S. Department of Agriculture, Forest Service.

(3) County maps - State Highway Agency.

(4) City plats - city or county recorder.

(5) Federal reclamation project maps - U. S. Department of the Interior, Bureau of Reclamation.

(6) American Society of Photogrammetry.

(7) ASCE Journal - Surveying and Mapping Division.

Aerial Photographs:

(1) The following agencies have aerial photographs of portions of the United States: U. S. Department of the Interior, Geological Survey, Topographic Division; U. S. Department of Agriculture, Commodity Stabilization Service, Soil Conservation Service and Forest Service; U. S. Air Force; various state agencies; commercial aerial survey; National Oceanic and Atmospheric Administration; and mapping firms.

(2) American Society of Photogrammetry.

(3) Photogrammetric Engineering.

(4) Earth Resources Observation System (EROS) - Photographs from Gemini, Apollo, Earth Resources Technology Satellite (ERTS) and Skylab.

Transportation Maps:

(1) State Highway Agency.

Triangulation and Benchmarks:

(1) State Engineer.

(2) State Highway Agency.

Geologic Maps:

(1) U. S. Department of the Interior, Geological Survey, Geologic Division; and state geological surveys or departments. (Note - some regular quadrangle maps show geological data also).

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, Bureau of Reclamation.

(4) Hydraulic laboratory reports - U. S. Department of the Interior, 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 U. S. Department of the Army, Corps of Engineers.

(5) Cooperative study reports - U. S. Department of Commerce, National Oceanic and Atmospheric Administration; and U. S. Department of the Interior, Bureau of Reclamation.

Stream Flow Data:

(1) Water supply papers - U. S. Department of the Interior; Geological Survey, Water Resources Division.

(2) Reports of state engineers.

(3) Annual reports - International Boundary and Water Commission, United States and Mexico.

(4) Annual reports - various interstate compact commissions.

(5) Hydraulic laboratory reports - U. S. Department of the Interior, Bureau of Reclamation.

(6) Bureau of Reclamation.

(7) Corps of Engineers, U. S. Army, Flood control studies.

Sedimentation Data:

(1) Water supply papers - U. S. Department of the Interior, Geological Survey, Quality of Water Branch.

(2) Reports - U. S. Department of the Interior, Bureau of Reclamation; and U. S. Department of Agriculture, Soil Conservation Service.

(3) Geological Survey Circulars - U. S. Department of the Interior, Geological Survey.

Quality of Water Reports:

(1) Water supply papers - U. S. Department of the Interior, 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, Bureau of Reclamation.

(5) Environmental Protection Agency, regional offices.

(6) State water quality agency.

Irrigation and Drainage Data:

(1) Agriculture census reports - U. S. Department of Commerce, Bureau of the Census.

(2) Agricultural statistics - U. S. Department of Agriculture, Agricultural Marketing Service.

(3) Federal reclamation projects - U. S. Department of the Interior, Bureau of Reclamation.

(4) Reports and progress reports - U. S. Department of the Interior, Bureau of Reclamation.

Power Data:

(1) Directory of Electric Utilities - McGraw Hill Publishing Co.

(2) Directory of Electric and Gas Utilities in the United States - Federal Power Commission.

(3) Reports - various power companies, public utilities, state power commissions, etc.

Basin and Project Reports and Special Reports:

(1) U. S. Department of the Army, Corps of Engineers.

(2) U. S. Department of the Interior, Bureau of Land Management, Bureau of Mines, Bureau of Reclamation, Fish and Wildlife Service, and National Park Service.

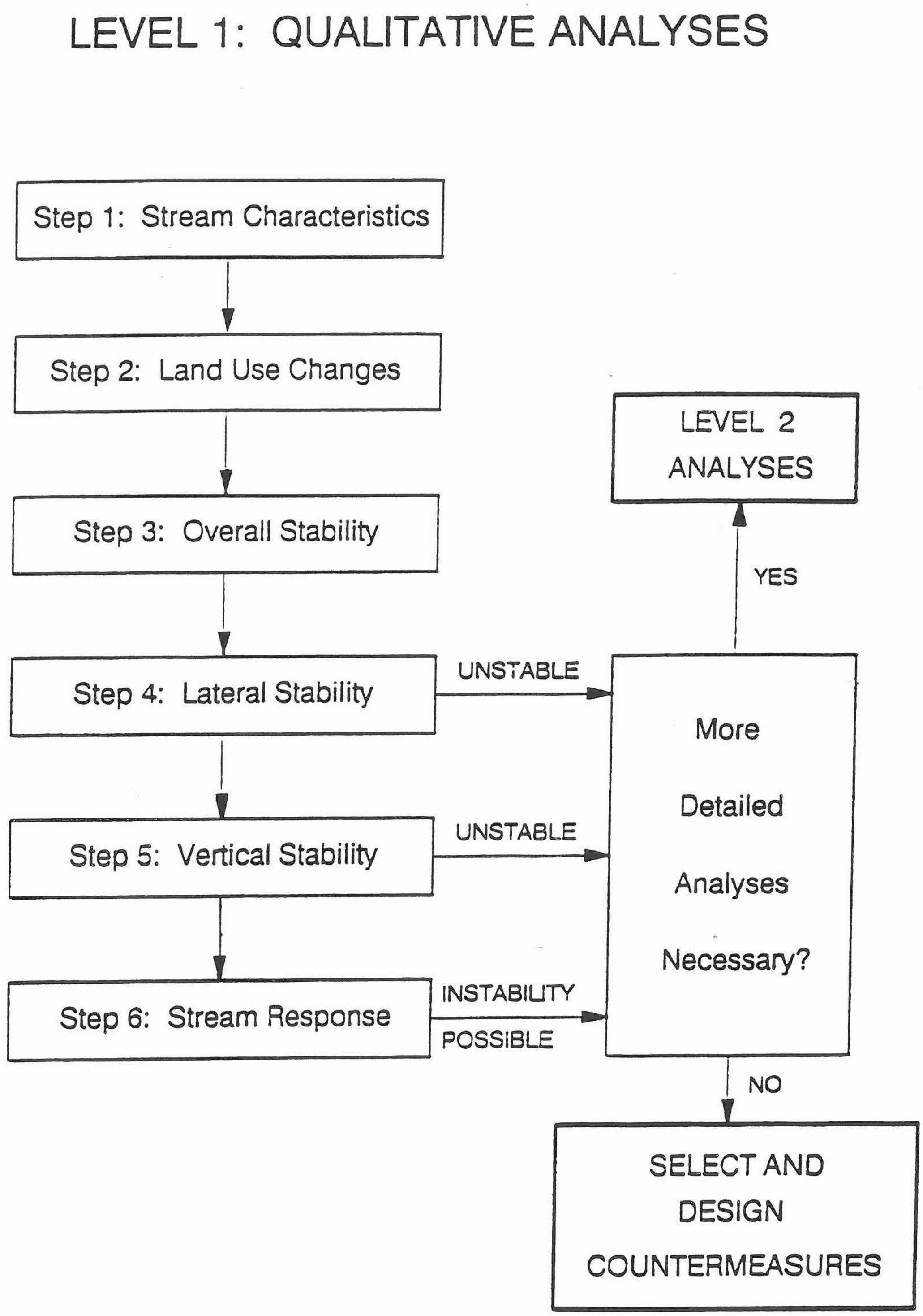


Figure 19. Flow chart for Level 1: Qualitative Analyses.

4.5.1 Step 1. Define Stream Characteristics

The first step in stability analysis is to identify stream characteristics according to the factors discussed in Chapter 2.0, Geomorphic Factors and Principles. Defining the various characteristics of the stream according to this scheme provides insight into stream behavior and response, and information on impacting activities in the watershed.

4.5.2 Step 2. Evaluate Land Use Changes

Water and sediment yield from a watershed is a function of land-use practices. Thus, knowledge of the land use and historical changes in land use is essential to understanding conditions of stream stability and potential stream 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, logging, land conversion and urbanization can either increase or decrease the total water and sediment yield from a watershed. For example, fire and logging tend to increase water and sediment yield, while urbanization promotes increased water yield and peak flows, but decreased sediment yield from the watershed. Urbanization may increase sediment yield from the channel.

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.). Additionally, analysis of historical aerial photographs can provide significant insight on land use changes. Land use change due to urbanization can be classified based on estimated changes in pervious and impervious cover. Changes in vegetative cover can be classified as simply as no change, vegetation increasing, vegetation damaged and vegetation destroyed. The relationship or correlation between changes in channel stability and land use changes can contribute to a qualitative understanding of system response mechanisms.

4.5.3 Step 3. Assess Overall Stream Stability

Table 6 summarizes possible channel stability interpretations according to stream characteristics discussed in Chapter 2.0 (Figure 1), as well as additional factors that commonly influence stream stability. Figure 20 is also useful in making a qualitative assessment of stream stability based on stream characteristics. It shows that straight channels are relatively stable only where flow velocities and sediment load are low. As these variables increase, flow meanders in the channel causing the formation of alternate bars and the initiation of a meandering channel pattern. Similarly, meandering channels are progressively less stable with increasing velocity and bed load. At high values of these variables, the channel becomes braided. The presence and size of point bars and middle bars are indications of the relative lateral stability of a stream channel.

Table 6. Interpretation of observed data.
(After [27]).

OBSERVED CONDITION	CHANNEL RESPONSE			
	STABLE	UNSTABLE	DEGRADING	AGGRADING
Alluvial Fan \1				
Upstream		X		X
Downstream		X	X	
Dam and Reservoir				
Upstream		X		X
Downstream		X	X	
River Form				
Meandering	X	X	Unknown	Unknown
Straight		X	Unknown	Unknown
Braided		X	Unknown	Unknown
Bank Erosion		X	Unknown	Unknown
Vegetated Banks	X		Unknown	Unknown
Head Cuts		X	X	
Diversion				
Clear water diversion		X		X
Overloaded w/sediment		X	X	
Channel Straightened		X	X	
Deforest Watershed		X		X
Drought Period	X			X
Wet Period		X	X	
Bed Material Size				
Increase		X		X
Decrease		X	Unknown	X

\1The observed condition refers to location of the bridge on the alluvial fan, i.e., on the upstream or downstream portion of the fan.

Bed material transport is directly related to stream power, and relative stability decreases as stream power increases as shown by Figure 20. Stream power is the product of shear stress at the bed and the average velocity in the channel section. Shear stress can be determined from the gross shear stress equation (γ RS) where γ is the specific weight of water, R is the hydraulic radius, and S is the slope of the energy grade line.

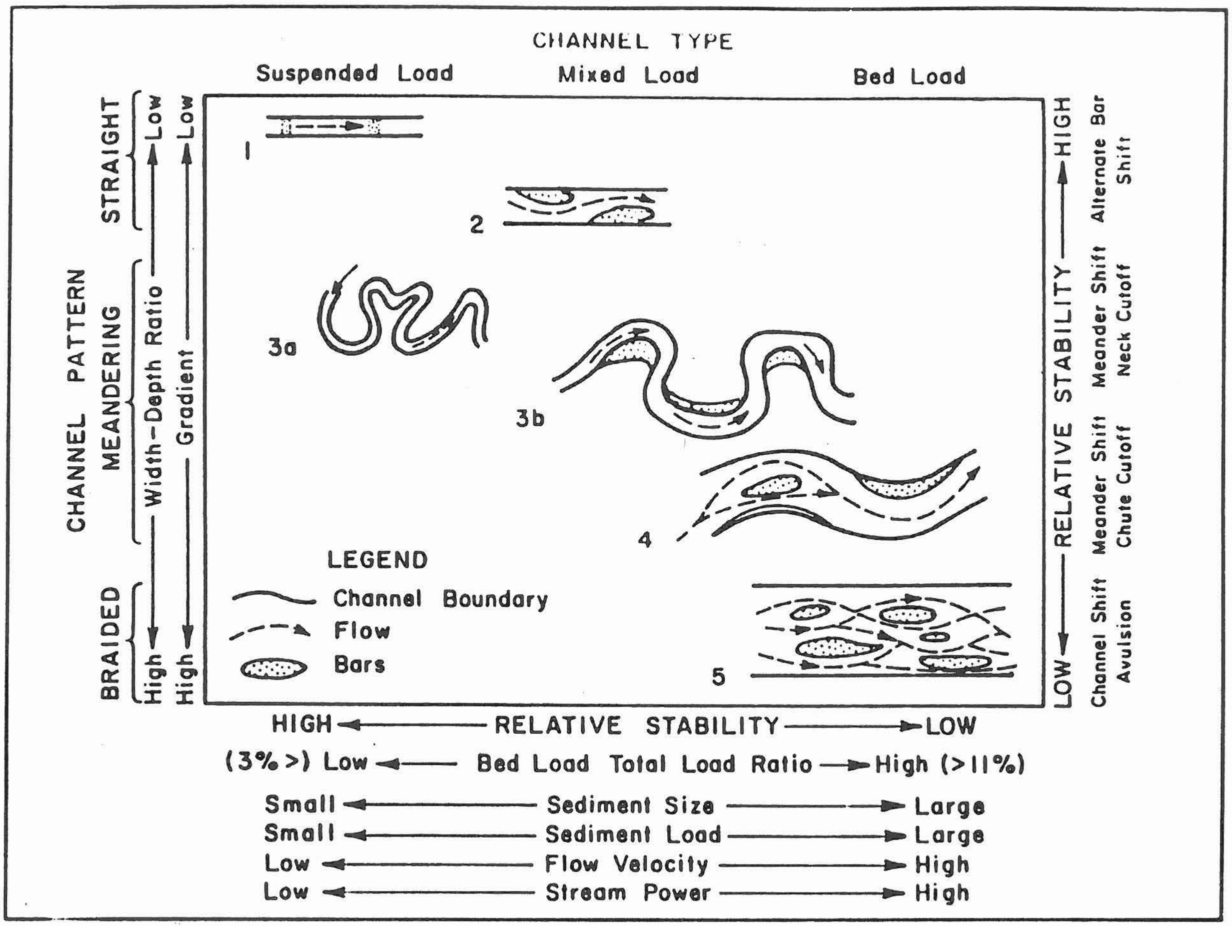


Figure 20. Channel classification and relative stability as hydraulic factors are varied (After [5]).

4.5.4 Step 4. Evaluate Lateral Stability

The effects of lateral instability of a stream at a bridge are dependent on the extent of the bank erosion and the design of the bridge. Bank erosion can undermine piers and abutments located outside the channel and erode abutment spill slopes or breach approach fills. Where bank failure is by a rotational slip, lateral pressures on piers located within the slip zone may cause cracks in piers or piling or displacement of pier foundations. Migration of a bend through a bridge opening changes the direction of flow through the opening so that a pier designed and constructed with a round-nose acts as a blunt-nosed, enlarged obstruction in the flow, thus accentuating local and general scour. Also, the development of a point bar on the inside of the migrating bend can increase contraction at the bridge if the outside bank is constrained from eroding. Figure 21 illustrates some of the problems of lateral erosion at bridges.

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 2.2.8 helps qualify bank stability. Similarly, field observations of bank material, composition and existing failure modes can provide insight on bank stability, based on the descriptions of cohesive, non-cohesive and composite banks given in Section 2.2.8. An evaluation of lateral stability in conjunction with the design of a bridge should take the performance of existing nearby bridges into account. The experience of such structures which have been subjected to the impacts of the stream can provide insight into response at a nearby structure.

Lateral stability assessment can also be completed from records of the position of a bend at two or more different times; aerial photographs or maps are usually the only records available. Surveyed cross sections are extremely useful although rarely available. Some progress is being made on the numerical prediction of loop deformation and bend migration (Level 3 type analyses). At present, however, the best available estimates are based on past rates of lateral migration at a particular reach. In using the estimates, it should be recognized that erosion rates may fluctuate substantially from one period of years to the next.

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

A site of potential avulsion (channel shifting to new flow path) in the vicinity of a highway stream crossing should be identified so that steps can be taken to mitigate the effects of avulsion when it occurs. A careful study of aerial photographs will show where overbank flooding has been taking place consistently and where a channel exists that can capture the flow in the existing channel. In addition, topographic maps and special surveys may show that the channel is indeed perched above the surrounding alluvial surface, with the inevitability of avulsion. Generally avulsion, as the term is used here, will only be a hazard on alluvial fans, alluvial plains, deltas, and wide alluvial valleys. In a progressively aggrading situation, as on an alluvial fan, the stream will build itself out of its channel and be very susceptible to avulsion. In other words, in a cross profile on an alluvial fan or plain, it may be found that the river is flowing between natural levees at a level somewhat higher than the surrounding area. In this case, avulsion is inevitable.

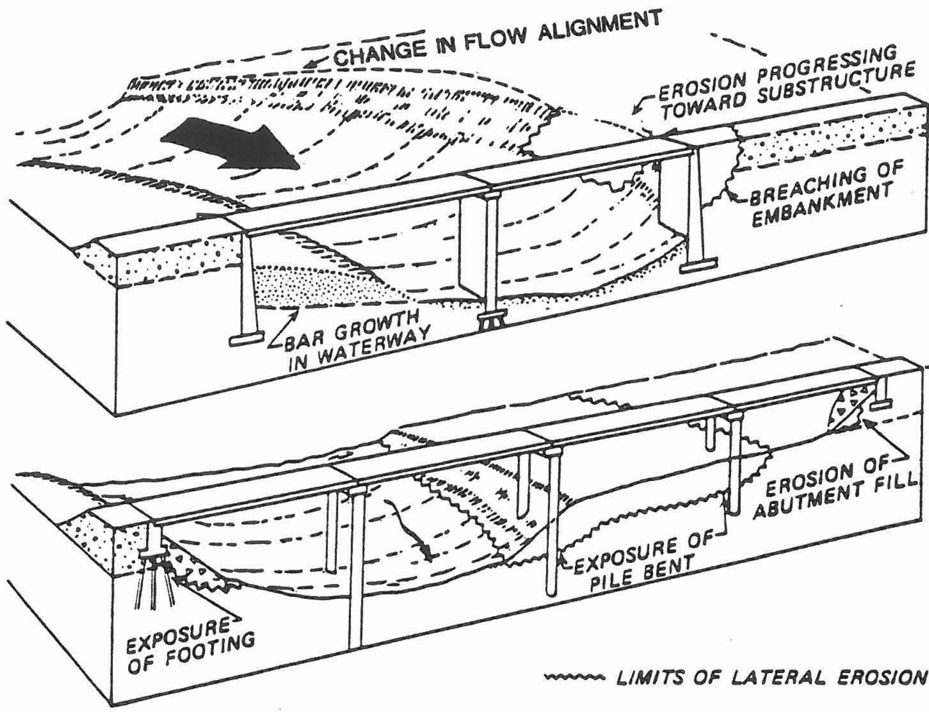


Figure 21. Hydraulic problems at bridges attributed to erosion at a bend or to lateral migration of the channel (After [1,2]).

4.5.5 Step 5. Evaluate Vertical Stability

The typical effects associated with gradation (bed elevation) changes at highway bridges are erosion at abutments and the exposure and undermining of foundations with degradation, and a reduction in flow area under bridges resulting in more frequent flow over the highway with aggradation. Bank caving associated with degradation poses the same problems at bridges as lateral erosion from bend migration, but the problems may be more severe because of the lower elevation of the streambed. Aggrading stream channels also tend to become wider as aggradation progresses, eroding floodplain areas and highway embankments on the floodplain. The location of the bridge crossing upstream, downstream, or on tributaries may cause gradation problems.

Brown et al., reported that their study indicated that there are serious problems at about three degradation sites for every aggradation site.[28] This is a reflection of the fact that degradation is more common than aggradation, and also the fact that aggradation does not endanger the bridge foundation. It is not an indication that aggradation is not a serious problem in some areas of the United States.

Problems other than those most commonly associated with degrading channels include the undermining of cutoff walls, other flow-control structures, and bank protection. Bank sloughing because of degradation often greatly increases the amount of debris carried by the stream and increases the hazard of clogged waterway openings and increased scour at bridges. 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 damage. Bridge decks and approach roadways become inundated more frequently, disrupting traffic, subjecting the superstructure of the bridge to hydraulic forces that can cause failure, and subjecting approach roadways to overflow that can erode and cause failure of the embankment. Where lateral erosion or increased flood stages accompanying aggradation increases the debris load in a stream, the hazards of clogged bridge waterways and hydraulic forces on bridge superstructures are increased.

Data records for at least several years are usually needed to detect gradation problems. This is due to the fact that the channel bottom often is not visible and changes in flow depth may indicate changes in the rate of flow rather than gradation changes. 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. On many large streams, the long-term trends have been analyzed and documented by agencies such as the U. S. Geological Survey and the U. S. Army Corps of Engineers.

4.5.6 Step 6. Evaluate Channel Response to Change

The knowledge and insight developed from evaluation of present and historical channel and watershed conditions, as developed above through Steps 1-5, provides an understanding of potential channel response to previous impacts and/or proposed changes, such as construction of a bridge. Additionally, the application of simple, predictive geomorphic relationships, such as the Lane Relationship (see Section 2.3) can assist in evaluating channel response mechanisms. Section 2.4.4 illustrated the evaluation of stream response based on geomorphic and other qualitative considerations. Additional applications of Level 1 analyses techniques to bridge related stream stability problems can be found in Chapter VIII of reference [4].

4.6 Level 2: Basic Engineering Analyses

A flow chart of the typical steps in basic engineering analyses is provided in Figure 22. The flow chart illustrates the typical steps to be followed if a Level 1 qualitative analysis resulted in a decision that Level 2 analyses were required (Figure 19). The eight basic engineering steps are generally applicable to most stream stability problems and are discussed in more detail in the paragraphs which follow. The basic engineering analysis steps lead to a conclusion regarding the need for more detailed (Level 3) analysis or a decision to proceed to selection and design of countermeasures without more complex studies. Selection and design of countermeasures are discussed in Chapters 5.0 and 6.0, respectively.

4.6.1 Step 1. Evaluate Flood History and Rainfall-Runoff Relations

Detailed discussion of hydrologic analysis techniques, in particular the analysis of flood magnitude and frequency, is presented in HEC-19 [14] and will not be repeated here. However, several hydrologic concepts of particular significance to evaluation of stream stability are summarized.

Consideration of flood history is an integral step in attempting to characterize watershed response and morphologic evolution. Analysis of flood history is of particular importance to understanding arid region stream characteristics. Many dryland streams flow only during the spring and immediately after major storms. For example, Leopold, et al. [29] found that arroyos near Santa Fe, New Mexico, flow only about three times a year. As a consequence, dryland stream response can be considered to be 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 a dryland system due to the infrequency of hydrologically significant events. Thus, the absence of significant morphological changes in a dryland stream or river, even over a period of years, should not necessarily be construed as indicative of system stability.

Although the occurrence of single large storms can often be directly related to system change in any region of the country, this is not always the case. In particular, the succession of morphologic change may be linked to the concept of geomorphic thresholds as proposed by Schumm [3]. Under this concept, although a single major storm may trigger an erosional event in a system, the occurrence of such an event may be the result of a cumulative process leading to an unstable geomorphic condition.

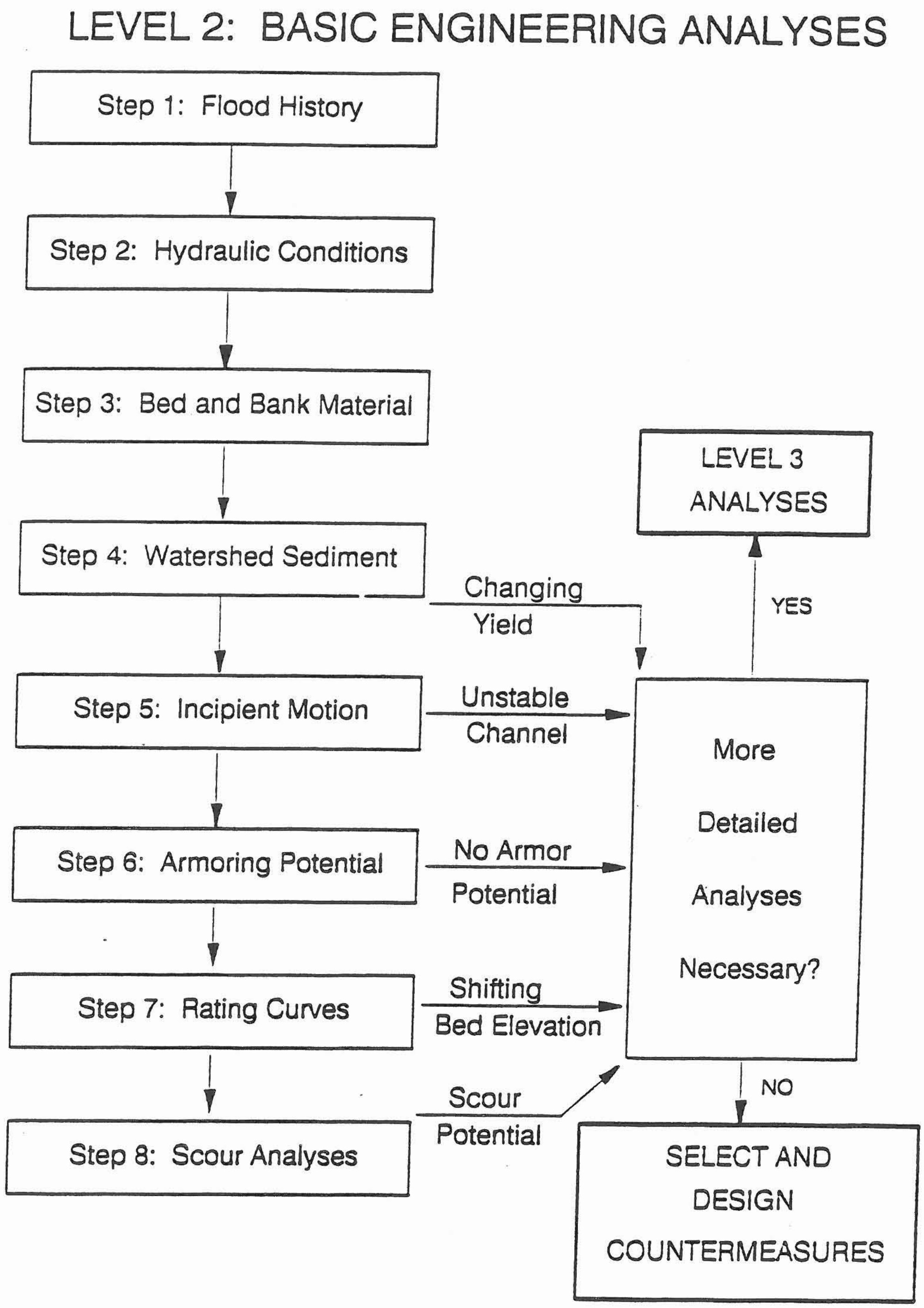


Figure 22. Flow chart for Level 2: Basic Engineering Analyses.

Where available, the study of flood records and corresponding system responses, as indicated by time-sequenced aerial photography or other physical information, may help determine the relationship between morphological change and flood magnitude and frequency. Evaluation of wet-dry cycles can also be beneficial to an understanding of historical system response. Observable historical change may be found to be better correlated with the occurrence of a sequence of events during a period of above average rainfall and runoff than with the single large event. The study of historical wet-dry trends may explain certain aspects of system response. For example, a large storm preceded by a period of above-average precipitation may result in less erosion, due to better vegetative cover, than a comparable storm occurring under dry antecedent conditions; however, runoff volumes might be greater due to saturated soil conditions.

A good method to evaluate wet-dry cycles is to plot annual rainfall amounts, runoff volumes and maximum annual mean daily discharge for the period of record. A comparison of these graphs will provide insight to wet-dry cycles and flood occurrences. Additionally, a plot of the ratio of rainfall to runoff is a good indicator of watershed characteristics and historical changes in watershed condition.

4.6.2 Step 2. Evaluate Hydraulic Conditions

Knowledge of basic hydraulic conditions, such as velocity, flow depth and top width, etc., for given flood events is essential for completion of Level 2 stream stability analysis. Incipient motion analysis, scour analysis, assessment of sediment transport capacity, etc. all require basic hydraulic information. Hydraulic information is sometimes required for both the main channel and overbank areas, such as in the analysis of contraction scour.

Evaluation of hydraulic conditions is based on the factors and principles reviewed in Chapter 3.0. For many river systems, particularly near urban 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 based on appropriate analytical techniques will be required prior to completing other quantitative analyses in a Level 2 stream stability assessment. The most common computer models for analysis of water surface profiles and hydraulic conditions are the Corps of Engineers HEC-2 and the Federal Highway Administration WSPRO. For the analysis and design of bridge crossings, WSPRO is generally considered a better model. The computational procedure in WSPRO for evaluating bridge loss is superior to that utilized in other models, and the input structure of the model has been specifically developed to facilitate bridge design.

4.6.3 Step 3. Bed and Bank Material Analysis

Bed material is the sediment mixture of which the streambed is composed. Bed material ranges in size from huge boulders to fine clay particles. The erodibility or stability of a channel largely depends on the size of the particles in the bed. Additionally, knowledge of bed sediment is necessary for most sediment transport analyses, including evaluation of incipient motion, armoring potential, sediment transport capacity and scour calculations. Many of these analyses require knowledge of particle size gradation, and not just the median (D_{50}) sediment size.

Bank material usually consists of particles the same size as, or smaller than, bed particles. Thus, banks are often more easily eroded than the bed, unless protected by vegetation, cohesion, or some type of man-made protection.

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 reference [4]. The following information briefly discusses sediment sampling considerations.

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, and 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 dig down some distance to establish bed-material characteristics. For example, in sand/gravel bed systems the potential existence of a thin surface layer of coarser sediments (armor layer) on top of relatively undisturbed subsurface material must be considered in any sediment sampling. 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, bed-material transport during a large flood (i.e., large enough to disturb the surface layer) is important, then the underlying layer may be more significant. Methods of analysis are given in reference [4].

4.6.4 Step 4. Evaluate Watershed Sediment Yield

Evaluation of watershed sediment yield, and in particular, the relative increase in yield as a result of some disturbance, can be an important factor in stream stability assessment. Sediment eroded from the land surface can cause silting problems in stream channels 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, thus promoting channel degradation and headcutting. A radical change in sediment yield as a result of some disturbance, such as a recent fire or long term land use changes, would suggest that stream instability conditions either already exist, or might readily develop.

Assessment of watershed sediment yield first requires understanding the sediment sources in the watershed and the types of erosion that are most prevalent. The physical processes causing erosion can be classified as sheet erosion, rilling, gullying and stream channel erosion. Other types of erosional processes are classified under the category of mass movement, eg., soil creep, mudflows, landslides, etc. Data from publications and maps produced by the Soil Conservation Service and the Geological Survey can be used along with field observations to evaluate the area of interest.

Actual quantification of sediment yield is at best an imprecise science. The most useful information is typically obtained not from analysis of absolute magnitude of sediment yield, but rather the relative changes in yield as a result of a given disturbance. One useful approach to evaluating sediment yield from a watershed was developed by the Pacific Southwest Interagency Committee [30]. This method, which was designed as an aid for broad planning purposes only, consists of a numerical rating of nine factors affecting sediment production in a watershed, which then defines ranges of annual sediment yield in acre feet per square mile. The nine factors are surficial geology, soil climate, runoff, topography, ground cover, land use, upland erosion, and channel erosion and transport.

Other approaches to quantifying sediment yield are based on regression equations, as typified by the Universal Soil Loss Equation (USLE). The USLE is an empirical formula for predicting annual soil loss due to sheet and rill erosion, and is perhaps the most widely recognized method for predicting soil erosion. Wischmeier and Smith [31] provide detailed descriptions of this equation and its terms.

4.6.5 Step 5. Incipient Motion Analysis

An evaluation of relative channel stability can be made by evaluating incipient motion parameters. 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.

The Shields diagram may be used to evaluate the particle size at incipient motion for a given discharge (see [4]). For most river flow conditions the following equation, derived from the Shields diagram, is appropriate for evaluation of incipient motion:

$$D_c = \frac{\tau}{0.047(\gamma_s - \gamma)} \tag{12}$$

where D_c is the diameter of the sediment particle at incipient motion conditions, τ is the boundary shear stress (see [4] for equations defining the boundary shear stress), γ_s and γ are the specific weights of sediment and water, respectively, and 0.047 is a dimensionless coefficient often referred to as the Shields parameter.

As originally proposed the Shields parameter was 0.06 for flow conditions in the turbulent range. The value of 0.047 was suggested by Meyer-Peter and Muller [32], and further supported by Gessler [33]. Recent research has indicated that this coefficient is not constant (values range from 0.02 to 0.10), and equations have been derived as a function of surface and subsurface particle size. However, as a first estimate the use of 0.047 should provide reasonable results in most situations.

Evaluation of the incipient motion size for various discharge conditions provides insight on channel stability and what flood might potentially disrupt channel stability. The results of such an analysis are generally more useful for analysis of gravel or cobble-bed systems. 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.

4.6.6 Step 6. Evaluate Armoring Potential

The armoring process begins as the non-moving coarser particles segregate from the finer material in transport. The coarser particles are gradually worked down into the bed, where they accumulate in a sublayer. Fine bed material is leached up through this coarse sublayer to augment the material in transport. As sediment movement continues and degradation progresses, an increasing number of non-moving particles accumulate in the sublayer. Eventually, enough coarse particles can accumulate to shield, or "armor" the entire bed surface.

An armor layer sufficient to protect the bed against moderate discharges can be disrupted during high flow, but may be restored as flows diminish. Therefore, as in any hydraulic design, the analysis must be based on a certain design event. If the armor layer is stable for that design event, it is reasonable to conclude that 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 representative bed-material composition. In this case the representative bed-material composition is that which is typical of the depth of anticipated degradation. For given hydraulic conditions the incipient motion particle size can be computed as given above in Step 5. If no sediment of the computed size or larger is present in significant quantities in the bed, armoring will not occur.

The D_{90} or D_{95} size of the representative bed material is frequently found to be the size "paving the channel" when degradation is arrested. Within practical limits of planning and design, the D_{95} size is considered to be about the maximum size for pavement formation [33]. Therefore, armoring is probable when the computed incipient motion size is equal to or smaller than the D_{95} size in the bed material.

By observing the percentage of the bed material equal to or larger than the armor particle size (D_c) the depth of scour necessary to establish an armor layer can be calculated [35]:

$$Y_s = y_a\left(\frac{1}{P_c} - 1\right) \tag{13}$$

where y_a is the thickness of the armoring layer and P_c is the decimal fraction of material coarser than the armoring size. The thickness of the armoring layer (y_a) ranges from one to three times the armor particle size (D_c), depending on the value of D_c. Field observations suggest that a relatively stable armoring condition requires a minimum of two layers of armoring particles.

4.6.7

4.6.8 Step 7. Evaluation of Rating Curve Shifts

When stream gage data is available, such as that collected by the U.S. Geological Survey, an analysis of the stage-discharge rating curve over time can provide insight on stream stability. For example, a rating curve that was very stable for many years, but suddenly shifts might indicate a change in watershed conditions causing increased channel erosion or sedimentation, or a some other change related to channel stability. Similarly, a rating curve that shifts continually would be a good indicator that channel instability exists. However, it is important to note that not all rating curve shifts are the result of channel instability. Other factors promoting a shift in a rating curve include changes in channel vegetation, ice conditions, beaver activity, etc.

The most common cause of rating curve shifts in natural channel control sections is generally scour and fill [36]. A positive shift in the rating curve results from scour, and the depth, and hence, the discharge are increased for a given stage. Conversely, a negative shift results from fill, and the depth and discharge will be less for a give stage.

Shifts may also be the result of changes in channel width. Channel width may increase due to bank-cutting, or decrease due to undercutting of steep streambanks. In meandering streams, changes in channel width can occur as point bars are created or destroyed.

Analysis of rating curve shifts is typically available from the agency responsible for the stream gage. If such information is not available, field inspection combined with the methods described by the [36] can be utilized to analyze observed rating curve shifts. If the shifts can be traced to scour, fill or channel width changes, such information will be a reliable indicator of potential channel instability.

Gaging stations at which continuous sediment data are collected may also provide clues to the existence of gradation problems. Any changes in the long-term sediment load may indicate lateral movement of the channel, gradation changes, or a change in sediment supply from the watershed.

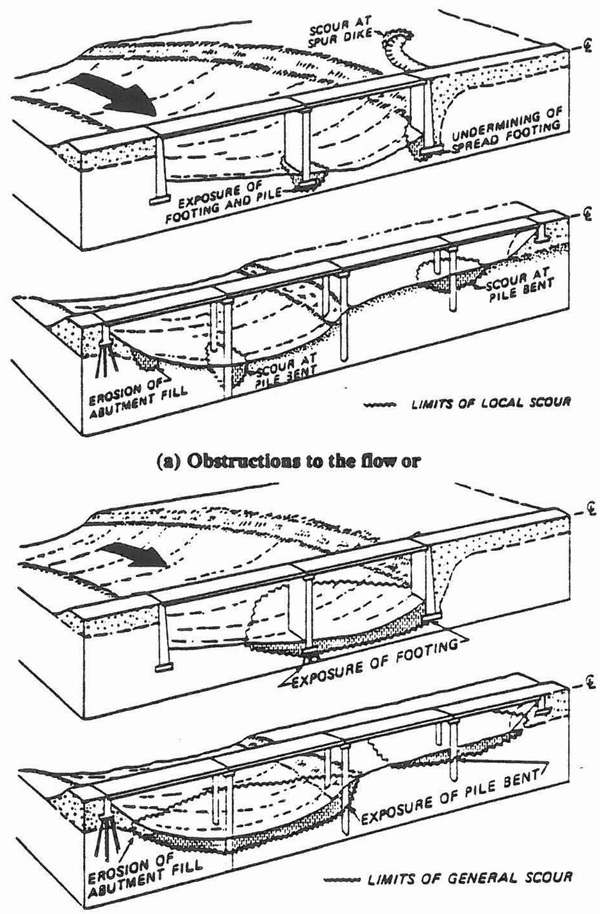
4.6.8 Step 8. Evaluate Scour Conditions

Section 3.4.1 provided an overview of scour at bridge crossings and reference [23] provides detailed computational procedures. Figure 23 illustrates common scour related problems at bridges. These problems are attributable to the effects of obstructions to the flow (local scour) and contraction of the flow or channel deepening at the outside of a bend. Calculation of the three components of scour, local scour, contraction scour and aggradation/degradation, quantifies the potential instability at a bridge crossing. Scour suspectible bridges are those that show potentially large amounts of any one of the scour components, and/or their cumulative amount is large. Such bridges should be carefully monitored and/or countermeasures installed.

4.7 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. 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.

Similarly, physical model studies completed in a hydraulics laboratory can provide detailed information on flow conditions and to some extent, sediment transport conditions, at a bridge crossing. The hydraulic laws and principles involved 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 what is readily available from mathematical models, due to the complexity of the process and the limitations of 2- and 3-dimensional mathematical models. Often the use of both physical and mathematical models can provide complementary information.



(a) Obstructions to the flow or

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

Figure 23. 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 [1,2].

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, considering the importance of stream stability to the safety and integrity of all bridges suggests that Level 3 type analyses should be completed routinely. The widespread use of personal computers and the continued development of more sophisticated software have greatly facilitated completion of Level 3 type investigations and have reduced the level of effort and cost required.

4.8 Illustrative Examples

The FHWA manual, "Highways in the River Environment" [4], provides a discussion of "Design Considerations for Highway Encroachment and River Crossings" in Chapter VII. This discussion includes principal factors for design, procedures for evaluation and design, and conceptual examples. The procedures for evaluation and design of river crossings and encroachments parallel the three-level approach of this chapter. A series of short conceptual discussions in Chapter VII of HIRE (pp. VII-13 to VII-32) illustrate the application of qualitative (level 1) techniques, and a series of short case studies (pp. VII-33 to VII-60) provide various applications. Finally, Chapter VII of HIRE presents five "Overview Examples" which illustrate various steps in the three-level approach.

To illustrate the application of Level 1 and Level 2 analysis techniques as presented in this chapter, one of the overview examples from HIRE [4] has been edited to correspond to the specific steps as outlined in Figures 19 and 22. This illustrative example is given in Appendix A.

5.0 SELECTION OF COUNTERMEASURES FOR STREAM INSTABILITY

5.1 Introduction

A countermeasure is defined as a measure incorporated into a highway-stream crossing system to monitor, control, inhibit, change, delay, or minimize stream and bridge stability problems or action plan for monitoring structures during and/or after flood events. This would include river stabilizing works over a reach of the river up and downstream of the crossing. Countermeasures may be installed at the time of highway construction or retrofitted to resolve stability problems at existing crossings. Retrofitting is good economics and good engineering practice in many locations because the magnitude, location, and nature of potential stability problems are not always discernible at the design stage, and indeed, may take a period of several years to develop. Also, a countermeasure does not need to be a separate structure, but may be an integral part of the highway. For example, relief bridges on floodplains are countermeasures which alleviate general scour from flow contraction at the bridge over the stream channel. Some features that are integral to the highway design serve as countermeasures to minimize stream stability problems. Abutments and piers oriented with flow direction serve to achieve the most efficient utilization of available waterway to convey flow and also serve to reduce local scour and scour due to contraction.

Countermeasures which are not integral to the highway may serve one function at one location and a different function at another. For examples, bank revetment may be installed to control bank erosion from meander migration, or it may be used to stabilize stream banks in the contracted area at a bridge. Other countermeasures are useful for one function only. This category of countermeasures includes spurs constructed in the stream channel to control meander migration.

In selecting a countermeasure it is necessary to evaluate how the stream might respond, and also how the stream may respond as the result of the activities of other parties.

A countermeasure for scour critical bridges and unknown foundations could also be monitoring a bridge during and/or after a flood event. If monitoring is selected and if the risk of scour failure is high, interim protection such as riprap or instrumentation should be provided. At this time the sizing of riprap to resist scour is not fail-safe. Therefore, even if riprap is placed around piers or abutments, the high risk bridge should be monitored during floods and inspected after floods. If monitoring is selected and the risk of scour failure is low, an action plan should be implemented which includes a notification process, flood watch procedures, a highway closure process, documentation of available detours, inspection procedures, assessment procedures, and a repair notification process.

This chapter provides some general criteria for the selection of countermeasures for stream instability. Then, the selection of countermeasures for specific stream instability problems is discussed. Finally, case histories of hydraulic problems at bridge sites are summarized to provide information on the relative success of various countermeasures for stream stabilization.

5.2 Criteria for the Selection of Countermeasures

The selection of an appropriate countermeasure for a specific bank 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 bulkheads. Spurs, longitudinal dikes, and area retardance structures can be used to establish a new flow path and channel alignment, or to constrict flow in a channel. Bulkheads may be used for any of the 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 gradation stability of the stream. These mechanisms are described in [4].

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; ice and debris; and floodplains.

Channel Width. Channel width influences only the use of spur-type countermeasures. On smaller streams (<250 feet wide), 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.

Bank Height. Low banks (< 10 feet) may be protected by any of the countermeasures, including bulkheads. 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.

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, bulkheads, revetments, and riprap have been used to control bank erosion resulting from stream migration. On anabranching streams, revetments, riprap, and spurs have been used to control bank erosion and channel shifting. Also, channels that do not carry large flows can and have been closed off. In one case, [4] reports that a large channel was closed off and revetment and riprap used to control erosion in the other channel.

Channel Material. Spurs, revetments, riprap, jack fields, or check dams can be used in any type of channel material if they are designed correctly. However, jack fields should only be placed on streams that carry appreciable debris and sediment in order for the jacks to cause deposition and eventually be covered up.

Bank Vegetation. Vegetation such as willows 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.

Sediment Transport. The 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, and an armor layer of coarse-grained material can develop 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.

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.

Channel Velocities and Flow Depth. Channel hydraulics affect countermeasure selection because structural stability and induced 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.

Ice and Debris. Ice and debris 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.

Floodplains. In selecting countermeasures for stream stability and scour, the amount of flow on the floodplain is an important factor. For example, if there is appreciable overbank flow, then guide banks to protect abutments should be considered. Also, spurs perpendicular to the approach embankment may be needed to control erosion.

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 for countermeasures which are located in stream channels include considerations of constructing and maintaining a structure which may be partially under water at all times, 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 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.

5.2.5 Costs

Cost comparisons should be used to study alternative countermeasures with an understanding that the measures were installed under widely varying stream conditions, that the conservatism (or lack thereof) of the designer is 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.

Figure 24 provides some insight regarding the relative costs of major countermeasure types. Although the study was done in 1985 and costs have increased, the relative cost probably has not changed. The bars represent the cost range for each countermeasure included in the comparison and the darkened portion of each bar represents the dominant range of costs. Numbers following the countermeasure type are the number of sites included in the cost analysis. The figure shows that rock spurs, horizontal wood slat spurs, rock windrow revetments, vegetation, jack retardance structures, wood-fence retardance structures, and rock toe dikes are usually the least expensive. Henson-type (vertical wood slat) spurs, cellular block revetments, and concrete-filled mats are generally the most expensive. Rock riprap revetment costs per foot of bank protection vary widely, but the dominant range of costs are not out of line with costs for other countermeasures.

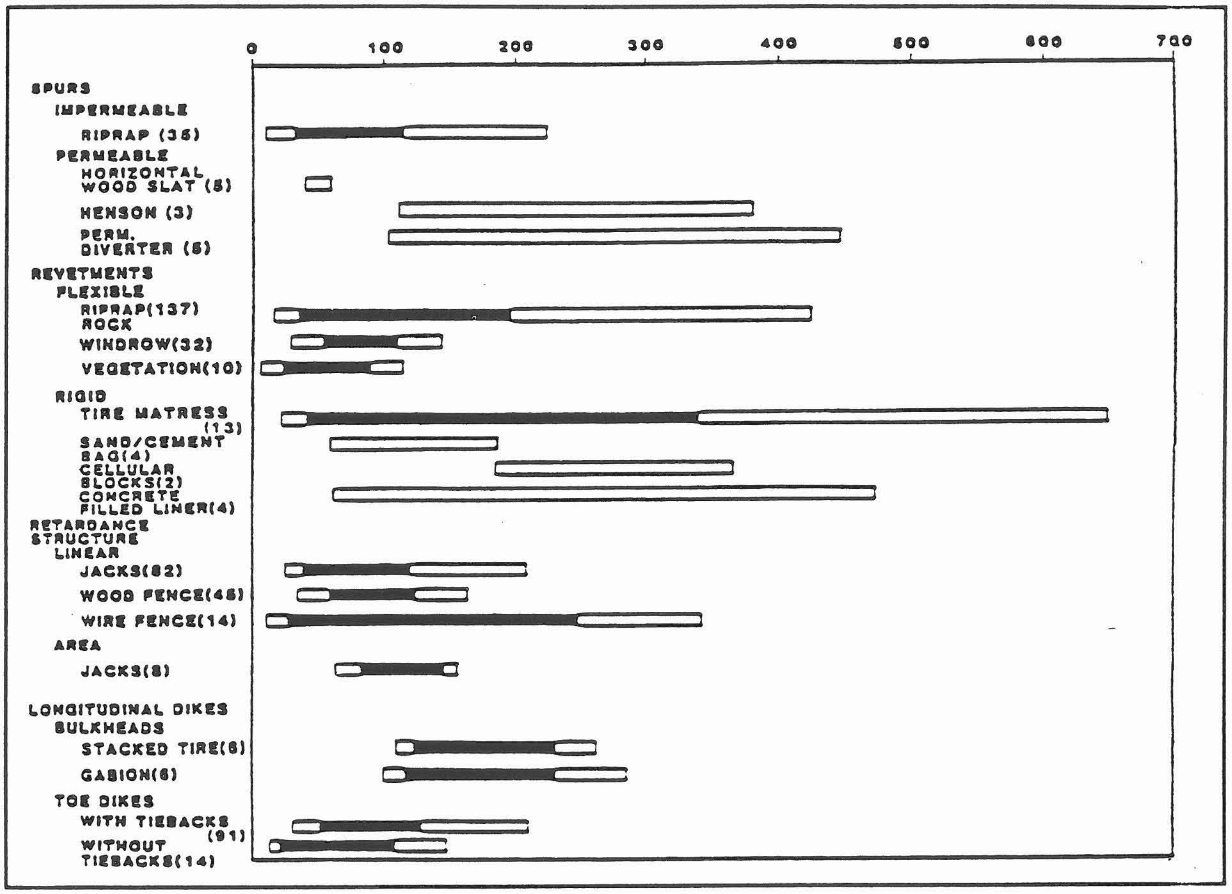


Figure 24. Countermeasure costs per foot of bank protected (After [37]).

5.3 Countermeasures for Meander Migration

The best countermeasure against meander migration is to locate the bridge crossing on a relatively straight reach of stream between bends. At many such locations, countermeasures may not be required for several years because of the time required for the bend to move to a location where it becomes threatening to the highway facility. However, bend migration rates on other streams may be at such a rate that countermeasures will be required after a few years or a few flood events and, therefore, should be installed during initial construction.

Stabilizing channel banks at a highway stream crossing can cause a change in the channel cross section and an increase in stream sinuosity upstream of the stabilized banks. Figure 25(a) 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 deposition in the remainder of the section. Figure 25(b) 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 the countermeasure and the bridge. It should also be recognized that the thalweg location and flow direction can change as sinuosity upstream increases.

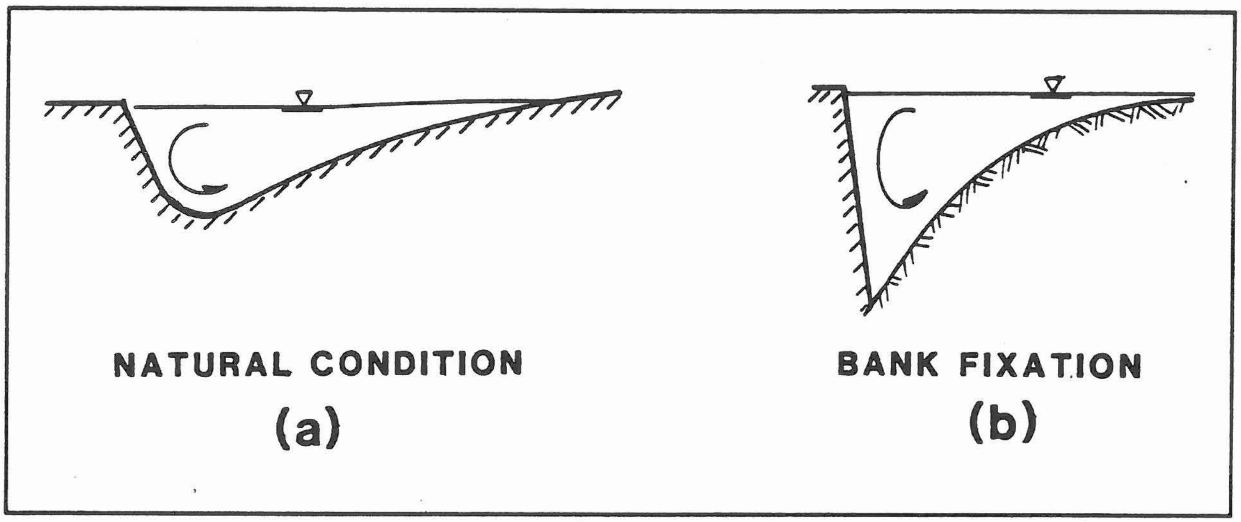


Figure 25. Comparison of channel bend cross sections (a) for natural conditions, and (b) for stabilized bend. (After [37]).

Figure 26(a) illustrates meander migration in a natural stream and Figure 26(b), the effects of bend stabilization on upstream sinuosity. As sinuosity increases, meander amplitude may increase, meander 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, as at the bridge location shown in Figure 26(b). Ultimately, cutoffs can occur. These changes can also result in changing hydraulic problems downstream of the stabilized bend.

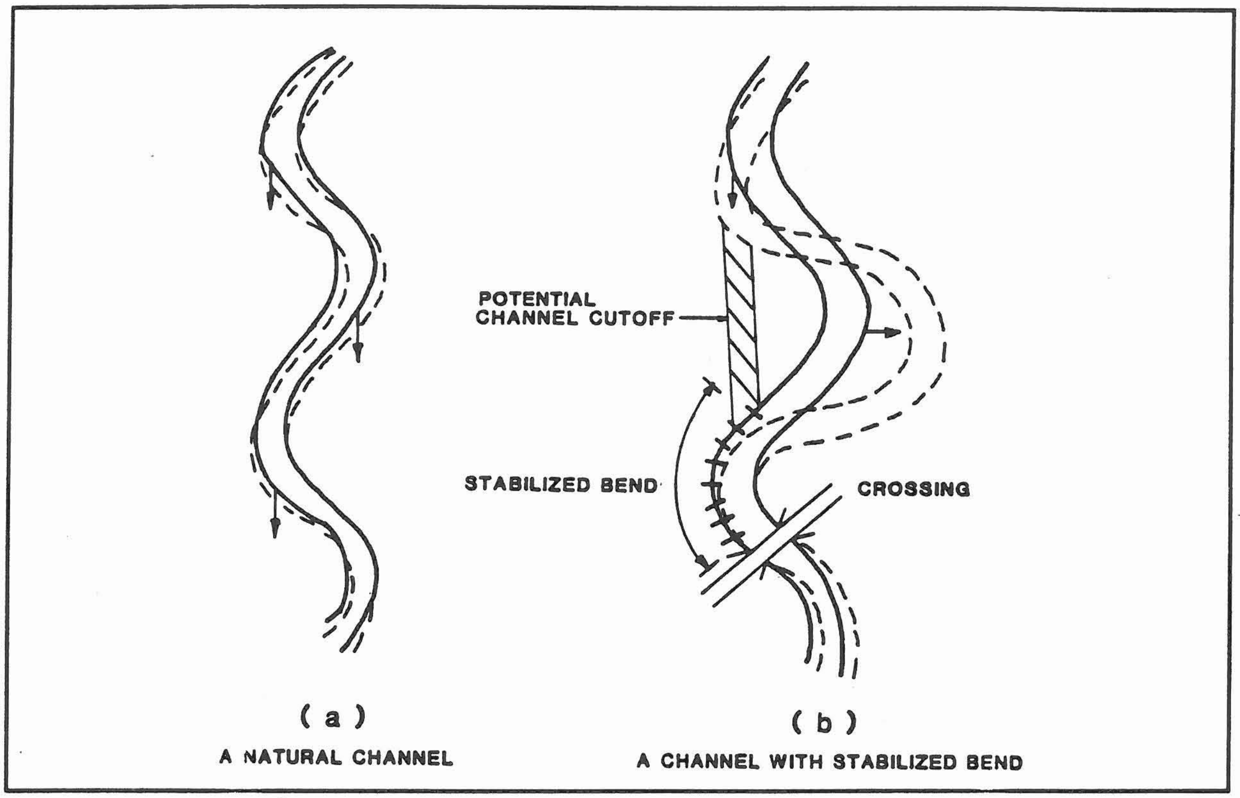


Figure 26. Meander migration in (a) a natural channel, and (b) a channel with stabilized bend (After [37]).

Countermeasures for meander migration include those that:

* protect an existing bank line,
* establish a new flow line or alignment, and
* control and constrict channel flow

The classes of countermeasures identified for bank stabilization and bend control are bank revetments, spurs, retardance structures, longitudinal dikes, vane dikes, bulkheads, and channel relocations. Also, a carefully planned cutoff may be an effective way to counter problems created by meander migration. These measures may be used individually or in combination to combat meander migration at a site. Some of these countermeasures are also applicable to bank erosion from causes other than bend migration. Descriptions and design recommendations are included in Chapter 6.0 - Countermeasure Design.

5.4 Countermeasures for Scour at Bridges

Scour is the result of the erosive action of running water, excavating and carrying away material from the bed and banks of streams. Different materials scour at different rates. Loose granular soils are rapidly eroded under water action while cohesive or cemented soils are more scour resistant. However, ultimate scour in cohesive or cemented soils can be as deep as scour in sandbed streams. Scour will reach its maximum depth in sand and gravel bed materials in hours; cohesive bed materials in days; glacial tills, poorly cemented sand stones and shales in months; hard, dense and cemented sandstone or shales and limestones in years; and dense granites in centuries. Massive rock formations with few discontinuities can be highly resistant to scour and erosion during the lifetime of a typical bridge.

Designers and inspectors need to carefully study site specific subsurface information in evaluating scour potential at bridges, giving particular attention to foundations on rock.

Total Scour. Total scour at a highway crossing is comprised of three components. These components are:

* Aggradation and Degradation. These are long-term stream bed elevation changes due to natural or man induced causes within the reach of the river on which the bridge is located.

* Contraction Scour. This type of scour involves the removal of material from the bed and banks across all or most of the width of a channel. This scour can result from a contraction of the flow by the approach embankments to the bridge encroaching onto the floodplain and/or into the main channel, a change in downstream control of the water surface elevation, or the location of the bridge in relation to a bend. In each case, the scour is caused by an increase in transport of the bed material in the bridge cross section.

* Local Scour. This scour occurs around piers, abutments, spurs, and embankments and is caused by the acceleration of the flow and the development of vortex systems induced by these obstructions to the flow.

In addition to the types of scour mentioned above, lateral movement of the stream may also erode the approach roadway to the bridge or change the total scour by changing the angle of the flow in the waterway at the bridge crossing. Factors that affect lateral movement and the stability of a bridge are the geomorphology of the stream, location of the crossing on the stream, flood characteristics, and the characteristics of the bed and bank materials.

Scour estimating procedures and countermeasure selection are presented in detail in [23,24].

There are two ways by which bridges can be made fail-safe against scour, but neither is a viable alternative at many alluvial stream crossing sites. Nevertheless, where the alternative is available and practicable, consideration can be given to making the crossing or bridge fail-safe. The first fail-safe design alternative is a one-span structure which spans stream flow at flood stage. This includes a superstructure that will not be submerged or partially submerged at flood stage. Obviously, may only be practicable for small, relatively stable, incised streams.

The second fail-safe bridge design alternative is a foundation in sound rock and a superstructure above the elevation of flood flow. Other countermeasures may be necessary to inhibit scour of stream banks, abutment spill slopes and approach fills, but the bridge will be fail-safe if adequately anchored in sound rock.

A third design alternative which is necessary for all bridges over alluvial streams is to locate the bridge foundation, pile tips or drilled shafts, at an elevation at which sufficient support will be retained after scour occurs.[23] Where pile bearing capacity is based on skin friction, driving the piles to refusal may be inadequate. Pile tip elevation should be applied as a second criterion, and tip elevation should be based on bearing capacity after scour.

The FHWA September 1988 Technical Advisory, subject "Scour at Bridges" [24] states the following with regard to new and existing bridges:

a. Interdisciplinary Team. Scour evaluations of new and existing bridges should be conducted by an interdisciplinary team comprised of structural, hydraulic, and geotechnical engineers. (...)

b. New Bridges. Bridges over tidal and non-tidal waterways with scourable beds should withstand the effects of scour from a superflood (a flood exceeding the 100-year flood) without failing, i.e., experiencing foundation movement of a magnitude that requires corrective action.

   (1) Hydraulic studies should be prepared for bridges over waterways in accordance with Article 1.3.2 of the standard Specifications for Highway Bridges of the American Association of State Highway and Transportation Officials (AASHTO) and the floodplain regulation of the FHWA as set forth in 23 CFR 650, Subpart A.

   (2) Hydraulic studies should include estimates of scour at bridge piers and abutments. Bridge foundations should be designed to withstand the effects of scour without failing for the worst conditions resulting from floods equal to or less than the 100-year flood in accordance with the interim procedures in Chapters 3 and 4 of the Attachment. Bridge

foundations should be checked to ensure that they will not fail due to scour resulting from the occurrence of a superflood on the order of magnitude of a 500-year flood. (...)

(3) The geotechnical analysis of bridge foundations should be performed on the basis that all stream bed material in the scour prism above the total scour line for the design flood (for scour) has been removed and is not available for bearing or lateral support. In addition, the ratio of ultimate to applied loads should be greater than 1.0 for conditions of scour for the superflood. (...)

(4) Data on scour at bridge piers and abutments should be collected and analyzed in order to improve existing procedures for estimating scour. (...)

c. Existing Bridges. All existing bridges over tidal and non-tidal waterways with scourable beds should be evaluated for the risk of failure from scour during the occurrence of a superflood on the order of magnitude of a 500-year flood. (...)

(1) An initial screening process should be developed to identify bridges most likely to be susceptible to scour damage and to establish a priority list for evaluation. (...)

(2) Bridge scour evaluations should be conducted for each bridge to determine whether it is scour critical. A scour critical bridge is one with abutment or pier foundations which are rated as unstable due to:

a observed scour at the bridge site or

b a scour potential as determined from scour evaluation study. (...)

d. Scour Critical Existing Bridges. A plan of action should be developed for each existing bridge determined to be scour critical. (...)

(1) The plan of action should include instructions regarding the type and frequency of inspections to be made at the bridge, particularly in regard to monitoring the performance and closing of the bridge, if necessary, during and after flood events.

(2) The plan of action should include a schedule for the timely design and construction of scour countermeasures determined to be needed for the protection of the bridge. (...)

As a practical matter, fail-safe bridge designs are usually not feasible, but strategies against scour are available. Design alternatives, in addition to the three discussed above, are integral to the highway facility. Countermeasures are appurtenances to the highway-stream crossing system. Design alternatives and countermeasures are both discussed in the following sections in regard to contraction scour and local scour problems.

5.4.1 Contraction Scour

Severe contraction of flow at highway stream crossings has resulted in numerous bridge failures at abutments, approach fills, and piers from contraction scour. Lessening contraction scour can be accomplished by reducing the amount of flow contraction and by reducing the effects of flow contraction. Design alternatives to lessen contraction scour include longer bridges, relief bridges on the floodplain, superstructures at elevations above flood stages of extreme events, and a rolling profile on approach roadways to provide for overtopping during floods exceeding the design flood event. These design alternatives are integral features of the highway facility which reduce the contraction at bridges and, therefore, reduce the magnitude of contraction scour. Further discussion of the elevation of superstructures and the roadway profile is warranted since the impact of these features on the magnitude of scour is less obvious than the effects of other features of the design.

The elevation of bridge superstructures is recognized as important to the integrity of the bridge because of hydraulic forces that may damage the superstructure. These include buoyancy and impact forces from ice and other floating debris. Contraction scour is another consideration in setting the superstructure elevation. When the superstructure of a bridge becomes submerged or when ice or debris lodged on the superstructure cause the flow to contract at the surface, flow is accelerated and more severe scour occurs. For this reason, where contraction scour is of concern, bridge superstructures should be located with clearance for debris, and, if practicable, above the stage of floods larger than the design flood.

Where stream flow at flood stages includes overbank flow, maximum flow through bridge waterways occurs at incipient overtopping of the roadway. As the flood stage increases, backwater and flow through bridge openings decrease, at least temporarily. Therefore, highway profiles are significant to the contraction of flow and contraction scour at stream crossings.

Another design feature which should be considered relative to contraction scour is the effective depth of the superstructure. Present day superstructures often include bridge railings which are solid parapets. These increase the effective depth of the superstructure and the importance of locating the bridge superstructure above high water with clearance for debris passage. It also increases the importance of alternate provisions for the passage of flood waters in the event of debris blockage of the waterway or superstructure submergence. Possible alternate provisions include relief bridges on the floodplain and a highway profile which provides for overtopping before the bridge superstructure begins to become submerged.

Similarly, pier design, span length, and pier location become more important contributors to contraction scour where debris can lodge on the piers and further contract flow in the waterway. In streams which carry heavy loads of debris, longer, higher spans and solid piers will help to reduce the collection of debris. Where practicable, piers should be located out of the main current in the stream, i.e., outside the thalweg at high flow. There are numerous locations where piers occupy a significant area in the stream channel and contribute to contraction scour, especially where devices to protect piers from ship traffic are provided.

The stream channel cross section under bridges is sometimes designed to increase the waterway area and thereby decrease backwater upstream of the bridge and contraction scour in the waterway. In streams which carry large sediment loads, deposition may occur in the enlarged section of channel during smaller floods and on the recession of larger floods, thus rendering the channel excavation ineffective. However, for streams which do not carry a significant sediment load and on floodplains, excavation within the bridge waterway area will compensate for some of the lateral contraction of flow and reduce contraction scour. The option of substituting excavation for scour is site-specific and may be undesirable on some floodplains because of high water tables and standing water.

Countermeasures used to reduce flow contraction include measures which retard flow along highway embankments on floodplains. Flow along highway fills usually intersects with flow within bridge openings at large angles. This causes additional contraction of the flow, vortices, and turbulence which produce local scour. The contraction of flow can be reduced by using spurs on the upstream side of the highway embankment to retard flow parallel with the highway.[19]

Guide banks (also referred to as spur dikes) at bridge ends serve a similar purpose in addition to the purpose of aligning flow in the bridge opening.[19] They reduce contraction scour because they increase the efficiency of the bridge opening and hence reduce flow contraction. The primary purpose of these guide banks, however, is to reduce local scour at abutments.

The principal countermeasure used for reducing the effects of contraction is revetment on channel banks and fill slopes at bridge abutments. However, guide banks may be used to reduce the effects of contraction by moving the site of local scour caused by the turbulence of intersecting flows and contraction away from the bridge end.

The potential for undesired effects from stabilizing all or any portion of the channel perimeter at a contraction should be considered. Stabilization of the banks may only result in exaggerated scour in the streambed near the banks or, in a relatively narrow channel, across the entire channel. Stabilization of the streambed may also result in exaggerated lateral scour in any size stream. Stabilization of the entire stream perimeter may result in downstream scour or failure of some portion of the countermeasures used on either the stream bed or banks.

5.4.2 Local Scour

Local scour occurs in bridge openings at piers and abutments. In general, design alternatives against structural failure from local scour consist of measures which reduce scour depth, such as pier shape and orientation, and measures which retain their structural integrity after scour reaches its maximum depth, such as placing foundations in sound rock and using deep piling. Countermeasures which prevent scour from occurring include riprap.

Abutments. Countermeasures for local scour at abutments consist of measures which improve flow orientation at the bridge end and move local scour away from the abutment, as well as revetments and riprap placed on spill slopes to resist erosion.

Guide banks (spur dikes) are earth or rock embankments placed at abutments. They are sometimes called spur dikes but this nomenclature can lead to confusion with spurs which have a different configuration and are used to retard flow velocities and/or divert flow away from stream banks. Flow disturbances, such as eddies and cross-flow, will be eliminated where a properly designed and constructed guide bank is placed at a bridge abutment. Guide banks also protect the highway embankment, reduce local scour at the abutment and adjacent piers, and move local scour to the end of the guide bank.

Local scour also occurs at abutments as a result of expanding flow downstream of the bridge. This is especially true of bridges on wide, wooded floodplains cleared for construction of the highway. Short guide banks extending to the tree line will move this scour away from the abutment, and the trees will retard velocities so that flow redistribution can occur with minimal scour.

The effectiveness of guide banks is a function of stream geometry, the quantity of flow on the floodplain, and the size of bridge opening. A typical guide bank at a bridge opening is shown in Figure 27.

Revetments may be pervious rock or rigid concrete. Rock riprap revetment provides an effective countermeasure against erosion on spill slopes.[38] Rigid revetments have been more successful where abutments are on the floodplain rather than in stream channels because hydrostatic pressure behind the revetments is not usually a problem. Precautions against undermining of the toe and upstream terminus of all revetments are always required.

Where guide banks are used, local scour is located away from the abutment spill slope and revetment on the spill slope may not be necessary.

Other countermeasures have been successfully used to inhibit scour at abutments where the abutment is located at the stream bank or within the stream channel. These measures include dikes to constrict the width of braided streams and retards to reduce velocities near the stream bank.

Piers. Three basic methods may be used to prevent damage from local scour at piers. The first method is to place the foundation of the structure at such a depth that the structural stability will not be at risk with maximum scour. This must be done on all new or replacement bridges.[23] The second is to provide protection at or below the stream bed to inhibit the development of a scour hole. The third measure is to prevent erosive vortices from forming or to reduce their strength and intensity. The first method is often expensive and there are uncertainties involved in estimating total scour at a pier, but it is the only measure that can be recommended without reservation in the absence of other measures. It should be noted that armored stream beds, cemented materials and shales will erode in time. Scour depths in streams with these bed materials can be greater than in streams which transport significant sediment loads. It should also be noted that the thalweg of alluvial channels shifts with time, sometimes moving from one side of the channel to the other during a flood.

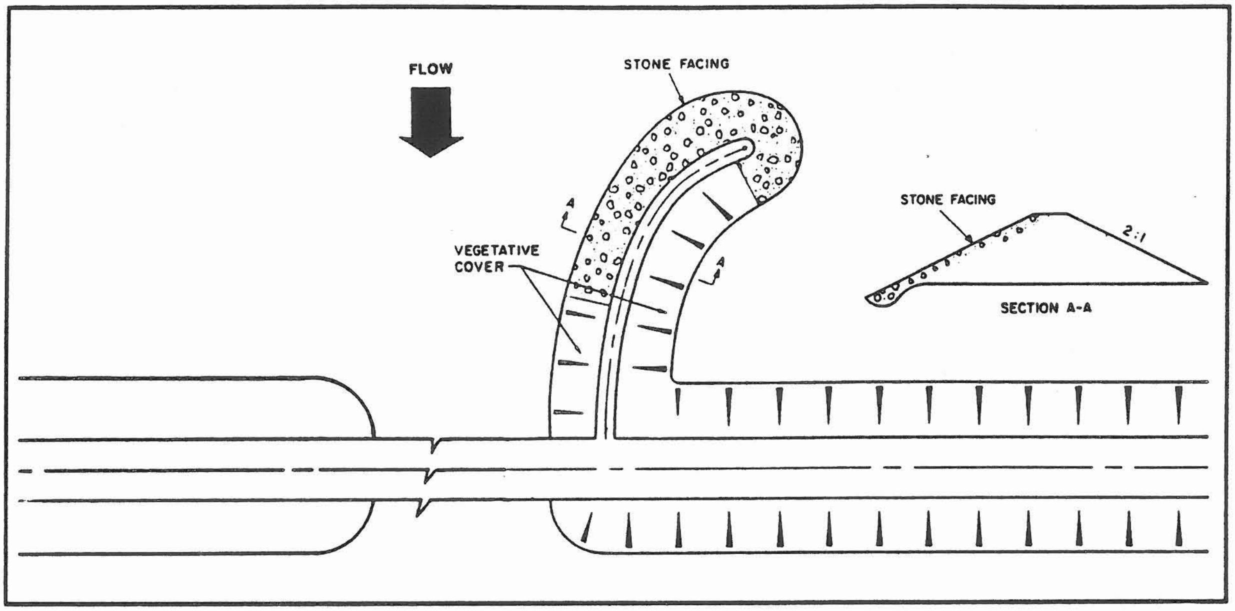


Figure 27. Typical guide bank layout and section (After [19]).

Streamlined pier noses decrease flow separation at the face of the pier, reducing the strength of the horseshoe vortices which form at piers. Practical application of this principle involves the use of rounded or circular shapes at the upstream and downstream faces of piers in order to reduce the flow separation. However, flow direction can and does change with time and with stage on some streams. Piers oriented with flow direction at one stage or at one point in time may be skewed with flow direction at another. Also, flow direction changes with the passage of bed forms. In general, piers should be aligned with the main flow direction and skew angles greater than 5 to 10 degrees should be avoided. Where this is not possible, a single cylindrical pier or a row of cylindrical columns will produce a lesser depth of local scour. For example, columns at a spacing of five diameters produce local scour about 1.2

times as deep as the local scour at a single column of equal diameter.[25] The tendency of a row of columns to collect debris should be considered in selecting this alternative. Debris can greatly increase scour depths. Webwalls have been used between columns to add to structural strength and to reduce the tendency to collect debris. These should be constructed at the elevation of stream flood stages which carry floating debris; otherwise, the row of columns will act as a solid pier if webwalls are extended to the elevation of the stream bed.

Riprap is commonly used to inhibit local scour at piers. This practice is not recommended as an adequate substitute for foundations or piling located below expected scour depths. It is recommended as a retrofit or interim measure where scour threatens the integrity of a pier. The practice of heaping stones around a pier is not recommended except as an interim measure because experience has shown that continual replacement is usually required. Success rates have been better with alluvial bed materials where the top of the riprap was placed at or below the elevation of the stream bed.

Piles (sheet, H beams or concrete) have been successfully used as a retrofit measure to lower the effective foundation elevation of structures where footings or pile caps have been exposed by scour. The piling is placed around the pile footings and anchored to the pile cap or seal to retain or restore the bearing capacity of the foundation. This will produce greater depth of scour, however.

Where sheet pile cofferdams are used during construction, the sheet piling should be removed or cut off below the level of expected contraction scour in order to avoid contributing to local scour. Cofferdams should not be much wider than the pier itself since the effect may be to greatly increase local scour depth. Leaving or removing cofferdams must be carefully evaluated because leaving a cofferdam that is higher than the contraction scour elevation may increase local scour depth. Recent work by Jones [26] gives a method to evaluate the expected scour depths for cofferdams.

5.4.3 Temporary Countermeasures

Monitoring or closing a bridge during high flows and inspection after the flood may be an effective temporary countermeasure. However, monitoring of bridges during high flow may not determine that they are about to collapse from scour. It also may not be practical to close the bridge during high flow because of traffic volume, no (or poor) alternate routes, the need for emergency vehicles to use the bridge, etc. Under these circumstances, temporary scour countermeasures such as riprap could be installed. A temporary countermeasure installed at a bridge along with monitoring during and inspection after high flows could provide for the security of the public without closing the bridge.

5.5 Countermeasures for Channel Braiding and Anabranching

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 low flow forms multiple interlaced channels. In an anabranched stream, flow is divided by islands rather than bars, and the anabranches are more permanent than braided channels and generally convey more flow.

Meandering streams may change to a braided stream if slope is increased by channel straightening or the dominant discharge is increased. Lane's [7] relation, Figure 8, may be used to determine if there can be a shift from a meandering channel to a braided one. If, after a change in discharge or slope the stream still plots in the meandering zone, then it will remain a meandering stream. However, if it moves closer to or into the braided zone, then the stream may become braided

Braided channels change alignment rapidly, and are very wide and shallow even at flood flow. They present problems at bridge sites because of the high cost of bridging the complete channel system, unpredictable channel locations and flow directions, and difficulties with eroding channel banks and in maintaining bridge openings unobstructed by bars and islands.

Countermeasures used on braided and anabranched 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 dikes constructed from the limits of the multiple channels to the channel over which the bridge is constructed. Guide banks at bridge ends 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.

Since anabranches are permanent channels that may convey substantial flow, diversion and confinement of an anabranched stream is likely to be more difficult than for a braided stream. The designer may be faced with a choice of either building more than one bridge, building a long bridge, or diverting anabranches into a single channel.

5.6 Countermeasures for Degradation and Aggradation

Gradation problems are common on alluvial streams. Degradation in streams can cause the loss of bridge piers in stream channels and can contribute to the loss of piers and abutments located on caving banks. Aggradation causes the loss of waterway opening in bridges and, where channels become wider because of aggrading stream beds, overbank piers and abutments can be undermined. At its worst, aggradation may cause streams to abandon their original channels and establish new flow paths which may sever highways from the existing bridge.

5.6.1 Countermeasures to Control Degradation

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, cutoff walls, and drop flumes. A check-dam is a low dam or weir constructed across a channel to prevent degradation.

Channel linings of concrete and riprap have proved unsuccessful at stopping degradation. To protect the lining, a check-dam may have to be placed at the downstream end to key it to the channel bed. Such a scheme would provide no more protection than would a check dam alone, in which case the channel lining would be redundant.

Bank erosion is a common hydraulic hazard in degrading streams. As the channel bed degrades, bank slopes become steeper and bank caving failures occur. The U. S. Army Corps of Engineers 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.[39]

The following is a condensed list of recommendations and guidelines for the application of countermeasures at bridge crossings experiencing degradation:

* Check-dams or drop structures are the most successful technique for halting degradation on small to medium streams.
* Channel lining alone may not be a successful countermeasure against degradation problems.
* Combinations of bulkheads and riprap revetment have been successfully used to protect abutments where steep stream banks threaten abutment fill slopes.
* Riprap on channel banks and spill slopes will fail if unanticipated channel degradation occurs.
* Successful pier protection involves providing deeper foundations at piers and pile bents.
* Jacketing piers with steel casings or sheet piles has also proved successful where expected degradation extends only to the top of the original foundation.
* The most economical solution to degradation problems at new crossing sites on small to medium size streams is to minimize the number of piers in the flow channel and provide adequate foundation depths. Adequate setback of abutments from slumping banks is also necessary.
* Rock-and-wire mattresses are recommended for use only on small (< 100 foot) channels experiencing lateral instability and little or no vertical instability.

* 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, may be necessary where installations are limited to the vicinity of the highway stream crossing.

5.6.2 Countermeasures to Control Aggradation

Currently, measures used in attempts to alleviate aggradation problems at highways include channelization, debris basins, bridge modification, and/or continued maintenance, or combinations of these. Channelization may include excavating and clearing channels, constructing small dams to form debris basins, constructing cutoffs to increase the local slope, 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. Cutoffs must be designed with considerable study as they can cause erosion upstream and deposition downstream. These studies would involve the use of sediment transport relations given in [4] or the use of sediment transport models such as BRISTARS. The most common bridge modifications are increasing the bridge length by adding spans and increasing the effective flow area beneath the structure by raising the bridge deck.

A program of continuing maintenance has been successfully used to control problems at bridges on aggrading streams. In such a program, a monitoring system is set up to survey the affected crossing at regular intervals. When some preestablished deposition depth is reached, the bridge opening 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 an adequate waterway under the bridge. 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 channelization, bridge alterations, or relocations. 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 bridge site. 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 degradation profile downstream, potentially impacting the bridge or other structures.

Following is a list of guidelines regarding aggradation countermeasures:

* 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.

* Alteration or replacement of a bridge is often required to accommodate maximum aggradation depths.

* 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.

* 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.

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

5.7 Case Histories

Case histories of hydraulic problems at bridge sites are used in this section to provide information on the relative success of the various countermeasures used to stabilize streams. All case histories are taken from Brice et al., [2,40] and Brown et al.[28] Site data are from Report No. FHWA-RD-78-163.[2] This compilation of case histories at 224 bridge sites is recommended reference material for those responsible for selecting countermeasures for stream instability. Additional case histories are given in reference [4].

5.7.1 Flexible Revetment

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 et al. documented the use of rock riprap at 110 sites (Volume 1, Table 2[1]). 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 concluded that riprap used in Oklahoma performed without significant failure and provides basic and efficient bank control on the meandering streams in Oklahoma.[41]

A review of the causes of failure at the sites studied by Brice et al. is instructive (Volume 1, Table 3[1]). They found the absence of a filter blanket clearly the cause of the failure at a site subject to tides and wave action. 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. At one site, failure was attributed to saturation of a high fill by impounded water in a reservoir. Wave action also probably contributed to the failure. The other site is difficult to include as a riprap failure because the rock was not placed as riprap revetment. Thirty-three freight car loads of rock were dumped as an emergency measure to stop erosion at a bridge abutment during high flow releases from a reservoir. The rock was displaced, and the high streambanks and highway fill are still susceptible to slumps. At both sites, riprap failed to prevent slumps in high fills.

Inadequate rock size and size gradation was given as the cause 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 Corps of Engineers. Riprap installations on the streambanks, at bridge abutments and in the stream bed 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.

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.

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 equidimensional. 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.

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.

5.7.2 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.

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[1,2]. 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.

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).

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 rigid revetments.

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.

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 stairstep construction can be used on steeper slopes (i.e., typically one to one), which reduces channel excavation costs. For many applications, soil cement is generally more aesthetically pleasing than other types of revetment.

5.7.3 Bulkheads

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 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 [1,2], but in each case, the probable cause was undermining.

5.7.4 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[42], spurs may protect a stream bank at less cost than riprap revetment, and 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.

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. Although no failures from ice damage were cited for impermeable spurs, North Dakota uses steel sheet pile enclosed earth fill spurs because of the potential for ice damage. At one site, such a spur sustained only minor damage from 2.5 feet of ice.

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.

5.7.5 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 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 [1,2] 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 U. S. Army Corps of Engineers reported that longitudinal rock toe dikes were the most effective bank stabilization measure studied for channels having very dynamic and/or actively degrading beds.[39]

5.7.6 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 flood water 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.

5.7.7 Guide Banks (Spur Dikes)

The major use of guide banks (or spur dikes) in the United States is to prevent erosion by eddy action at bridge abutments or piers where concentrated flood flow travelling along the upstream side of an approach embankment enters the main flow at the bridge. By establishing smooth parallel streamlines in the approaching flow, guide banks improve flow conditions in the bridge waterway. Scour, if it occurs, is near the upstream end of the dike away from the bridge. A guide bank differs from dikes described above in that a dike is intended to contain overbank flow while a guide bank only seeks to align overbank flow with flow through the bridge opening. An extension of the usual concept of the purpose for guide banks, but not in conflict with that concept, is the use of guide banks and highway fill to constrict braided channels to one channel. At three sites studied [1,2], guide banks only or guide banks plus revetment on the highway fill were used to constrict wide braided channels rather severely, and the installations have performed well.

Guide bank performance was found to be generally satisfactory at all study sites. Performance is theoretically affected by construction materials, shape, orientation, and length.

Most guide banks are constructed of earth with revetment to inhibit erosion of the dike. At two sites, guide banks of concrete rubble masonry performed well. Revetment of riprap is most common, but concrete revetment with rock riprap toe protection, rock-and-wire mattress, gabions, and grass sod have also performed satisfactorily. Since partial failure of a guide bank during a flood usually will not endanger the bridge, wider consideration should be given to the use of vegetative cover for protection. Partial failure of any countermeasure is usually of little significance so long as the purpose of protecting the highway stream crossing is accomplished.

Guide banks of elliptical shape, straight, and straight with curved ends performed satisfactorily at study sites, although there is evidence at one site that flow does not follow the nose of the straight guide bank. Clear evidence of the effect of guide bank orientation was not found at study sites although the conclusion by Colson and Wilson that guide banks (spur dikes) should be oriented with valley flow for skewed crossings of wooded floodplains was cited.[43] There was evidence at one site that a guide bank may be severely tested where a large flow is diverted along the roadway embankment, as at a skewed crossing or on a wide floodplain which is severely constricted by the bridge. At these locations, embankment spurs may be advisable to protect the embankment from erosion and to reduce the potential for failure of the spur dike.

Guide banks at study sites tended to be longer than recommended by Bradley [19] at most sites, except at five sites where they ranged from 15 feet to 75 feet. All guide banks appeared to perform satisfactorily. Not enough short guide banks were included in the study to reach conclusions regarding length.[1,2]

5.7.8 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 the channel in order to protect the substructure foundation of bridges. 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.[1,2]

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 just upstream of a bridge.

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.

5.7.9 Jack or Tetrahedron Fields

Jacks and tetrahedrons function as flow control measures by reducing the water velocity along a bank, which in turn results in an accumulation of sediment and the establishment of vegetation. Steel jacks, or Kellner jacks which consist of six mutually perpendicular arms rigidly fixed at the midpoints and strung with wire are the most commonly used. Tetrahedrons apparently are not currently used by highway agencies. Jacks are usually deployed in fields consisting of rows of jacks tied together with cables.

Four sites where steel jack fields were used are included in the case histories.[1,2] At two sites, the jack fields performed satisfactorily. Jacks were buried in the stream bed and rendered ineffective at one site, and jacks were damaged by ice at one site, but apparently continued to perform satisfactorily. From Keeley's observations[41] of the performance of jack fields used in Oklahoma and findings of the study of countermeasures by Brice et al.[1], the following conclusions were reached regarding performance:

* The probability of satisfactory performance of jack fields is greatly enhanced if the stream transports small floating debris and sediment load in sufficient quantity to form accumulations during the first few years after construction.

* Jack fields may serve to protect an existing bank line, or to alter the course of a stream if the stream course is realigned and the former channel backfilled before the jack field is installed.

* On wide shallow channels, which are commonly braided, jack fields may serve to shift the bank line channelward if jacks of large dimensions are used.

5.7.10 Special Devices for Protection of Piers

Countermeasures at piers have been used to combat abrasion of piers, to deflect debris, to reduce local scour, and to restore structural integrity threatened by scour. Retrofit countermeasures installed after problems develop are common. The usual countermeasure against abrasion consists of steel armor on the upstream face of a pier in the area affected by bed load. At one site, a pointed, sloping nose on a massive pier, called a special "cutwater" design, and a concrete fender debris deflector has functioned to prevent debris accumulation at the pier. At another site, a steel rail debris deflector worked until channel degradation caused all countermeasures to fail.

Countermeasures for local scour at piers are discussed above, except for a measure installed on a bridge over an estuary in Florida where about 37 feet of scour had occurred. This measure consists of flat plates installed around piers to deflect plunging currents. The plates are eight feet in diameter and are installed around 20 inch diameter piles. It was recommended that the plates be installed at or slightly below the elevation of the stream bed, but strong tidal currents prevented underwater installation at uniform locations. Two years after installation, some deposition had occurred but performance could not be judged.

Countermeasures used to restore structural integrity of bridge foundations included in the case histories include underpinning, sheet pile driven around the pier, and grout curtain around pier foundation.

5.7.11 Investment in Countermeasures

While it is often possible to predict that bank erosion will occur at or near a given location in an alluvial stream, one can frequently be in error about the location or magnitude of potential erosion. At some locations, unexpected lateral erosion occurs because of a large flood, a shifting thalweg, or from other actions of the stream or activities of man. Therefore, where the investment in a highway crossing is not in imminent danger of being lost, it is often prudent to delay the installation of countermeasures until the magnitude and location of the problem becomes obvious. In many, if not most, of the case histories collected by Brice et al., highway agencies invested in countermeasures after a problem developed rather than in anticipation of a problem.[1,2,40]

6.0 COUNTERMEASURE DESIGN

6.1 Introduction

Chapters 2.0 through 4.0 of this manual discuss factors influencing stream stability and response. For stream stability problems which adversely impact highway crossings, specific countermeasure recommendations were made in Chapter 5. In this chapter the design of these specific countermeasures are discussed with references to appropriate design documents.

This chapter discusses in detail the design of three basic countermeasures: 1) Spurs; 2) Guide Banks (often referred to as Spur Dikes); and 3) Check Dams. Respectively, these three countermeasures are used to: 1) protect banks and redirect flow in the vicinity of the highway crossing; 2) channelize and direct flows through the bridge opening; and 3) control degradation (either long-term or contraction scour) and maintain the bed elevation at the bridge opening. With these countermeasures, most adverse impacts on highway crossings can be controlled.

In some cases other countermeasures may be required. Therefore, a separate section for the conceptual design of other countermeasures is also presented. Details of the design of these other countermeasures are not given, but a schematic and references are provided, where available.

6.2 General Design Guidelines

The objective of highway agencies at crossings of streams is to protect highway users and the investment in the highway facility, and to avoid causing damage to other properties, to the extent practicable. Countermeasures should be designed and installed to stabilize only a limited reach of stream and to ensure the structural integrity of highway components in an unstable stream environment. Countermeasures are often damaged or destroyed by the stream, and stream banks and beds often erode at locations where no countermeasure was installed, but so long as the primary objectives are achieved in the short-term as a result of countermeasure installation, the countermeasure installation can be deemed a success. Therefore, the highway agency's interest in stream stability often entails long-term protection of costly structures by committing to maintenance, reconstruction, and construction of additional countermeasures as the response of streams and rivers to natural and man-induced changes are identified.

The design of any countermeasure for the protection of highway crossings requires the designer to be cognizant of the factors which effect stream stability and the morphology of the stream. In most all cases, the installation of any countermeasure will cause the bed and banks to respond to the change in hydraulic conditions imposed by the countermeasure. Thus, the analyses procedures outlined and illustrated in Chapter 4.0 are a necessary prerequisite to the detailed design of specific countermeasures provided in this chapter. The goal in any countermeasure design is to achieve a response which is beneficial to the protection of the highway crossing and to minimize adverse effects either upstream or downstream of the crossing.

In many cases, a combination of two or more countermeasures are required due to site-specific problems or as a result of changing conditions after the initial installation. The great number of possible countermeasure combinations makes it impractical to suggest design procedures for combined countermeasures. However, combined countermeasures should complement each other. That is to say that the design of one countermeasure must not adversely impact on another or the overall protection of the highway crossing. The principles of river mechanics, as discussed in [4] and in Chapters 2.0 through 4.0 of this text, coupled with sound engineering judgment should be used to design countermeasure strategies involving two or more countermeasures.

6.3 Spurs

A spur is a pervious or impervious structure projecting from the stream bank into the channel. Spurs are used to deflect flowing water away from, or to reduce flow velocities in critical zones near the stream bank, to prevent erosion of the bank, and to establish a more desirable channel alignment or width. The main function of spurs is to reduce flow velocities near the bank, which in turn, encourages sediment deposition due to these reduced velocities. Increased protection of banks can be achieved over time, as more sediment is deposited behind the spurs. Because of this, spurs may protect a stream bank more effectively and at less cost than revetments. Furthermore, by moving the location of any scour away from the bank, partial failure of the spur can often be repaired before damage is done to structures along and across the stream.

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. The use of spurs to establish and maintain a well-defined channel location, cross-section, and alignment in braided streams can decrease the required bridge lengths, thus decreasing the cost of bridge construction and maintenance.

Spur types are classified based upon their permeability as retarder spurs, retarder/deflector spurs, and deflector spurs. The permeability of spurs is defined simply as the percentage of the spur surface area facing the stream flow that is open. Deflector spurs are impermeable spurs which function by diverting the primary flow currents away from the bank. Retarder/deflector spurs are permeable and function by retarding flow velocities at the bank and diverting flow away from the bank. Retarder spurs are more permeable and function by retarding flow velocities near the bank.

Table 7 can be used as an aid in the selection of an appropriate spur type for a given situation [6]. The primary factors influencing the selection of a specific spur type are listed across the top, and primary spur types are evaluated in terms of those selection criteria. A scale from 1 to 5 is used to indicate the applicability of a specific spur for a given condition. A value of 1 indicates a disadvantage in using that spur type for given condition, and a value of 5 indicates a definite advantage. The table can be used by summing values horizontally for given site conditions to select the best spur type for the specific site. It should be recognized however, that adherence to the results of such a procedure assigns equal weight to each of the factors listed across the top of the table and places undue reliance on the accuracy and relative merit of values given in the rating table. It is possible that assigned values of 1 should have been negative and assigned values of 5 should not be assigned quantitative significance. It is recommended that values given in the table be used only for a qualitative evaluation of expected performance. Spur type selection should be based on the results of this evaluation as well as estimated costs, availability of materials, construction and maintenance requirements, and experience with the stream in which the spur installation is to be placed.

SPUR TYPE	FUNCTION			EROSION MECHANISM				SEDIMENT ENVIRONMENT			FLOW ENVIRONMENT						BEND RADIUS			ICE/DEBRIS ENVIRONMENT		
											VELOCITY			STAGE								
	Protect Ext, Bank	Re-est. Prev. Align.	Flow Construction	Transport	Shear Stress - Toe	Shear Stress - Upper Bank	Abrasion	Regime/Low Threshold	Medium Threshold	High Threshold/Rigid	Low	Medium	High	Low	Medium	High	Large	Medium	Small	Minimal	Light Debris	Large Debris/Ice
RETARDER																						
Fence Type	3	2	2	3	3*	1	1	4	3	2	3	3	2	3	2	1	3	2	1	3	3	2
Jack/Tetrahedron	3	3	1	3	3	1	1	4	3	1	3	2	1	3	2	1	3	2	1	2	4	1
RETARDER/DEFLECTOR																						
Light Fence	3	3	3	3	3	2	2	3	3	2	3	3	2	3	3	2	3	3	2	3	4	2
Heavy Diverter	3	4	4	3	3	4	3	2	3	3	3	3	2	3	4	4	3	3	2	3	4	3
DEFLECTOR																						
Hardpoint	3	4	4	3	3	3	4	2	3	4	3	3	4	3	3	2	3	4	4	3	3	5
Transverse Dike	3	4	4	3	3	3	4	2	3	4	3	3	4	3	3	2	3	4	3	3	3	5

*Henson spur jetties are rated a 4 for this condition

1. Definite disadvantage to the use of this type structure.
2. Some disadvantage to the use of this type structure.
3. Adequate for condition.
4. Some advantage to the use of this type structure.
5. Significant advantage to the use of this type structure.

Table 7. Spur type performance (After [6]).

6.3.1 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.

Longitudinal Extent of Spur Field

The longitudinal extent of channel bank requiring protection is discussed in [38]. Figure 28 was developed from Corps of Engineers studies of the extent of protection required at meander bends.[39] The minimum extent of bank protection determined from Figure 28 should be adjusted according to field inspections to determine the limits of active scour, channel surveys at low flow, and aerial photography and field investigations at high flow. Investigators of field installations of bank protection have found that protection commonly extends farther upstream than necessary and not far enough downstream. However, such protection may have been necessary at the time of installation. The lack of a sufficient length of protection downstream is generally more serious, and the downstream movement of meander bends should be considered in establishing the downstream extent of protection.

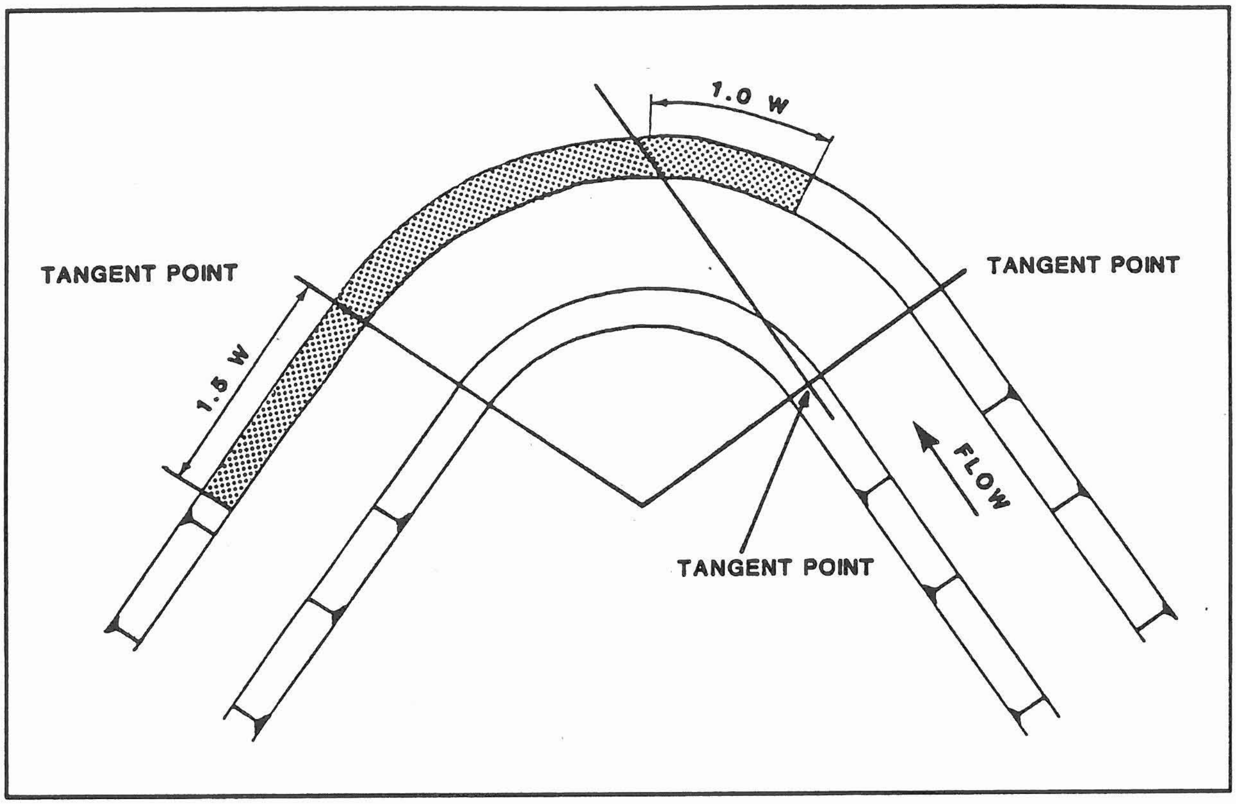


Figure 28. Extent of protection required at a channel bend (After [39]).

Spur Length

Spur length is taken here as the projected length of spur normal to the main flow direction or from the bank. Where the bank is irregular, spur lengths must be adjusted to provide for an even curvature of the thalweg. The length of both permeable and impermeable spurs relative to channel width affects local scour depth at the spur tip and the length of bank protected. Laboratory tests indicate that diminishing returns are realized from spur lengths greater than 20 percent of channel width. The length of bank protected measured in terms of projected spur length (LBP / PL) is essentially constant up to spur lengths of 20 percent of channel width for permeable and impermeable spurs. Field installations of spurs have been successful with lengths from 3 to 30 percent of channel width. Impermeable spurs are usually installed with lengths of less than 15 percent while permeable spurs have been successful with lengths up to 25 percent of channel width. However, only the most permeable spurs were effective at greater lengths.

The above discussion assumes that stabilization of the bend is the only objectives when spur lengths are selected. It also assumes that the opposite bank will not erode. Where flow constriction or changing the flow path is also an objective, spur lengths will depend on the degree of constriction required or the length of spur required to achieve the desired change in flow path. At some locations, channel excavation on the inside of the bend may be required where spurs would constrict the flow excessively. However, it may be acceptable to allow the stream to do its own excavation if it is located in uniformly graded sand.

Spur Orientation

Spur orientation refers to spur orientation with respect to the direction of the main flow current in a channel. Figure 29 defines the spur angle such that an acute spur angle means that the spur is angled in an upstream direction and an angle greater than 90 degrees indicates that the spur is oriented in a downstream direction.

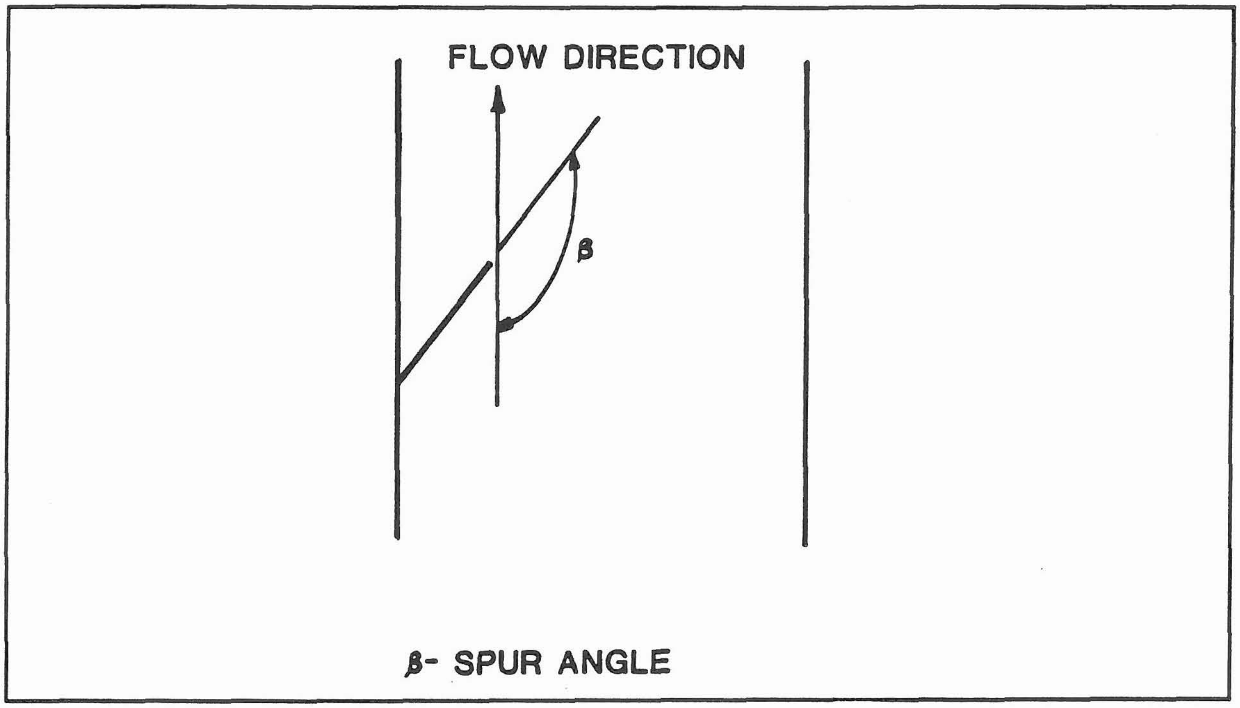


Figure 29. Definition sketch for spur angle (After [44]).

Permeable retarder spurs are usually designed to provide flow retardance near the stream bank, and they perform this function equally as well without respect to the spur angle. Since spurs oriented normal to the bank and projecting a given length into the channel are shorter than those at any other orientation, all retarder spurs should be constructed at 90 degrees with the bank for reasons of economy.

No consensus exists regarding the orientation of permeable retarder/deflector spurs and impermeable deflector spurs. There is some agreement that spurs oriented in an upstream direction do not protect as great a length of channel bank downstream of the spur tip, result in greater scour depth at the tip, and have a greater tendency to accumulate debris and ice. There is also some agreement that the first spur in an array should be at a spur angle of approximately 150 degrees in order to provide more gradual flow transition through the bend.[44]

Spur orientation affects spur spacing, the degree of flow control achieved by the spur, and scour depth at the tip of the spur. Scour depth at the tip of impermeable spurs and retarder/deflector spurs with permeability of 35 percent or less oriented normal to the channel bank can be estimated by use of Figure 30. This graph, which extends to a limiting value of $a / Y_1 < 25$ is based on laboratory studies and represents the equilibrium depth of scour for spurs oriented normal to the wall of the flume. Maximum scour depth can be as much as 30 percent greater than equilibrium scour depth. The curve representing values of $a/Y_1 > 25$ was taken from data collected at rock spurs in the Mississippi River and is believed to represent the limit in scale for scour depths. For the equations on this figure, Y_s is the equilibrium scour depth measured from the mean stream bed elevation, Y_1 is the flow depth upstream, F_{rl} is the upstream Froude number, and, a is the spur length measured normal to the wall of the flume. It should be noted that available information on the depth of scour at spurs is based on sand bed streams. In gravel bed streams, armoring of the scour hole by selective transport of material forming the stream bed will reduce the depth of scour.

Spur orientation at approximately 90 degrees has the effect of forcing the main flow current (thalweg) farther from the concave bank than spurs oriented in an upstream or downstream direction. Therefore, more positive flow control is achieved with spurs oriented approximately normal to the channel bank. Spurs oriented in an upstream direction cause greater scour than if oriented normal to the bank, and spurs oriented in a downstream direction cause less scour.

It is recommended that the spur furthest upstream be angled downstream to provide a smoother transition of the flow lines near the bank and to minimize scour at the nose of the leading spur. Subsequent spurs downstream should all be set normal to the bank line to minimize construction costs.

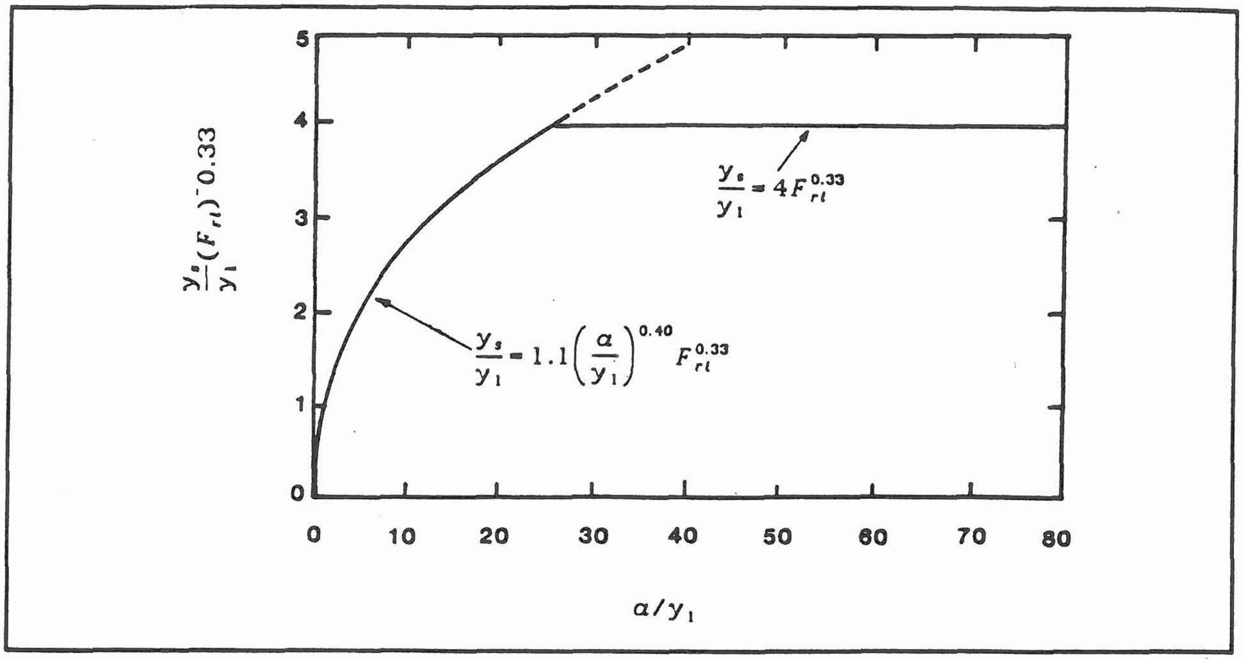


Figure 30. Recommended prediction curves for scour at the end of spurs with permeability up to about 35 percent (After [4]).

Figure 31 can be used to adjust scour depth for orientation. It should be noted that permeability also affects scour depth. A method to adjust scour depth for permeability is presented in the following section.

The lateral extent of scour is nearly always determinable from the depth of scour and the natural angle of repose of the bed material.

The expansion angle downstream of a spur, i.e., the angle of flow expansion downstream of the contraction at the spur is about 17 degrees for impermeable spurs for all spur angles. The implication is that spur orientation affects the length of bank protected only because of the projected length of the spur along the channel bank.

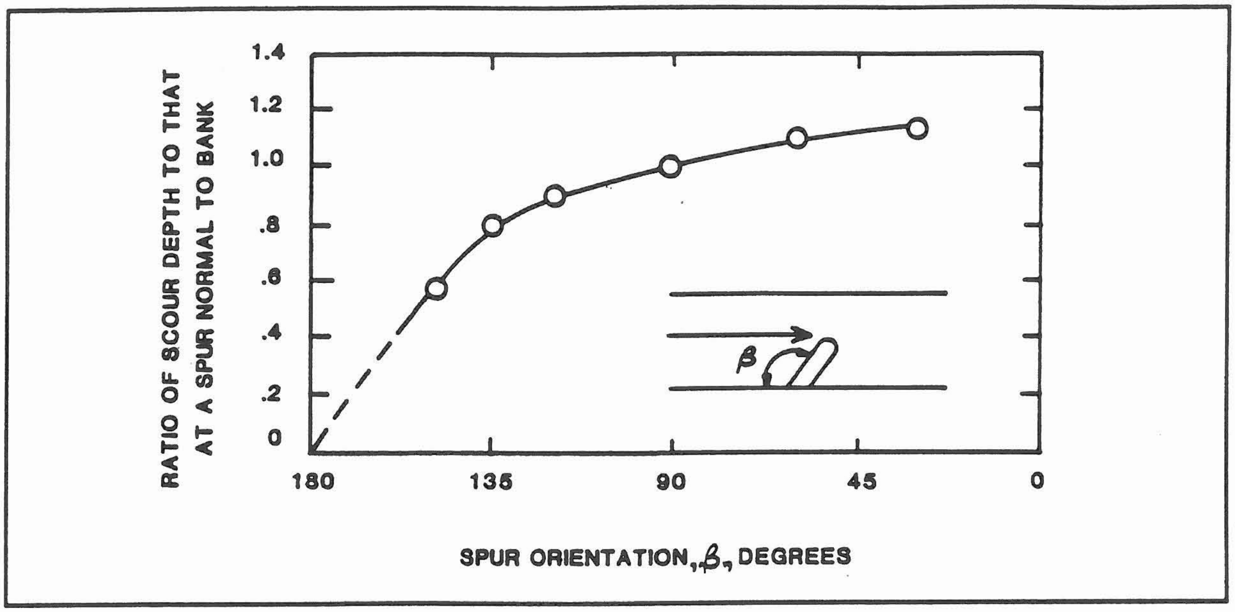


Figure 31. Scour adjustment for spur orientation (Modified from [4]).

Spur Permeability

The permeability of the spur depends on stream characteristics, the degree of flow retardance and velocity reduction required, and the severity of the channel bend. Impermeable spurs can be used on sharp bends to divert flow away from the outer bank. Where bends are mild and only small reductions in velocity are necessary, highly permeable retarder spurs can be used successfully. However, highly permeable spurs can also provide required bank protection under more severe conditions where vegetation and debris will reduce the permeability of the spur without destroying the spur. This is acceptable provided the bed load transport is high.

Scour along the stream bank and at the spur tip are also influenced by the permeability of the spur. Impermeable spurs, in particular, can create erosion of the stream bank at the spur root. This can occur if the crest of impermeable spurs are lower than the height of the bank. Under submerged conditions, flow passes over the crest of the spur generally perpendicular to the spur as illustrated in Figure 32. Laboratory studies of spurs with permeability greater than about 70 percent were observed to cause very little bank erosion, while spurs with permeability of 35 percent or less caused bank erosion similar to the effect of impermeable spurs.[44]

Figure 33 illustrates the effect of spur permeability on relative scour depth at various spur orientations with the channel bank. Spur angles are measured from the bank

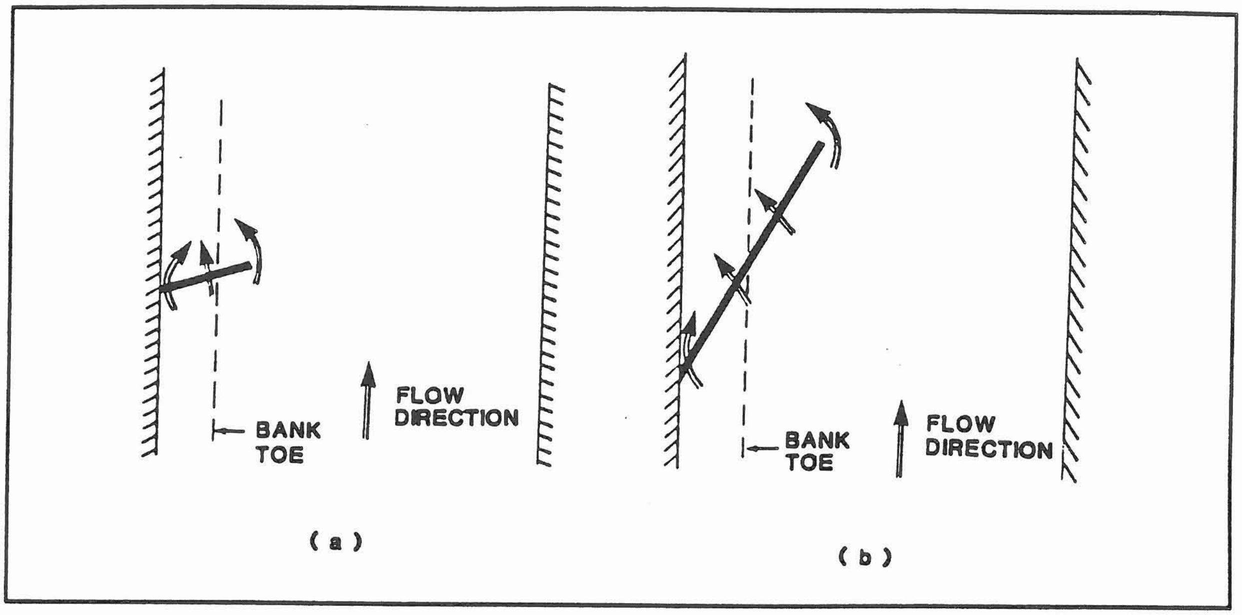


Figure 32. Flow components in the vicinity of spurs when the crest is submerged (After [44]).

upstream of the spur to the centerline of the spur. This graph was derived from laboratory studies which relate spur permeability to scour depth relative to depths for an impermeable spur set perpendicular to the bank.

Tests were conducted with projected spur lengths equal to 20 percent of the channel width. If the permeability of spurs is less than 35 percent, Figures 30 and 31, from the previous section should be used to evaluate the scour depth. If the permeability of the spurs being designed is greater than 35 percent, Figure 33 should be used to adjust scour depths obtained from Figure 30 presented in the previous section.

Permeability up to about 35 percent does not affect the length of channel bank protected by the spur. Above a permeability of 35 percent, the length of bank protected decreases with increasing permeability. Figure 34 shows the results of laboratory tests of the effects of permeability and orientation on the expansion angle of flow downstream of spurs. For this figure, spur lengths were 20 percent of the channel width projected normal to the bank.[44]

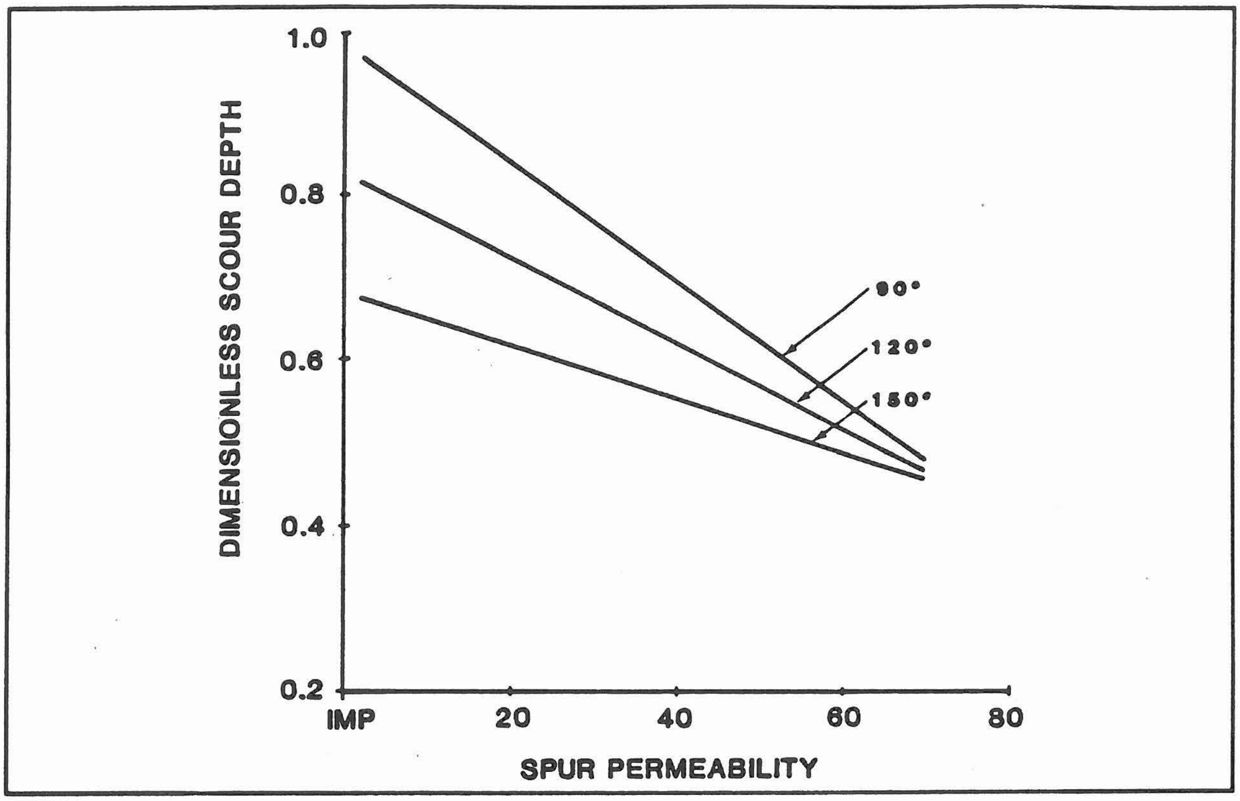


Figure 33. Plot of spur permeability and orientation vs. relative scour depth at the spur tip (After [44]).

From the above discussion, it is apparent that spurs of varying permeability will provide protection against meander migration. Impermeable spurs provide more positive flow control but cause more scour at the toe of the spur and, when submerged, cause erosion of the stream bank. High permeability spurs are suitable for use where only small reductions in flow velocities are necessary as on mild bends, but can be used for more positive flow control where it can be assumed that clogging with small debris will occur and bed load transport is large. Spurs with permeability up to about 35 percent can be used in severe conditions but permeable spurs may be susceptible to damage from large debris and ice.

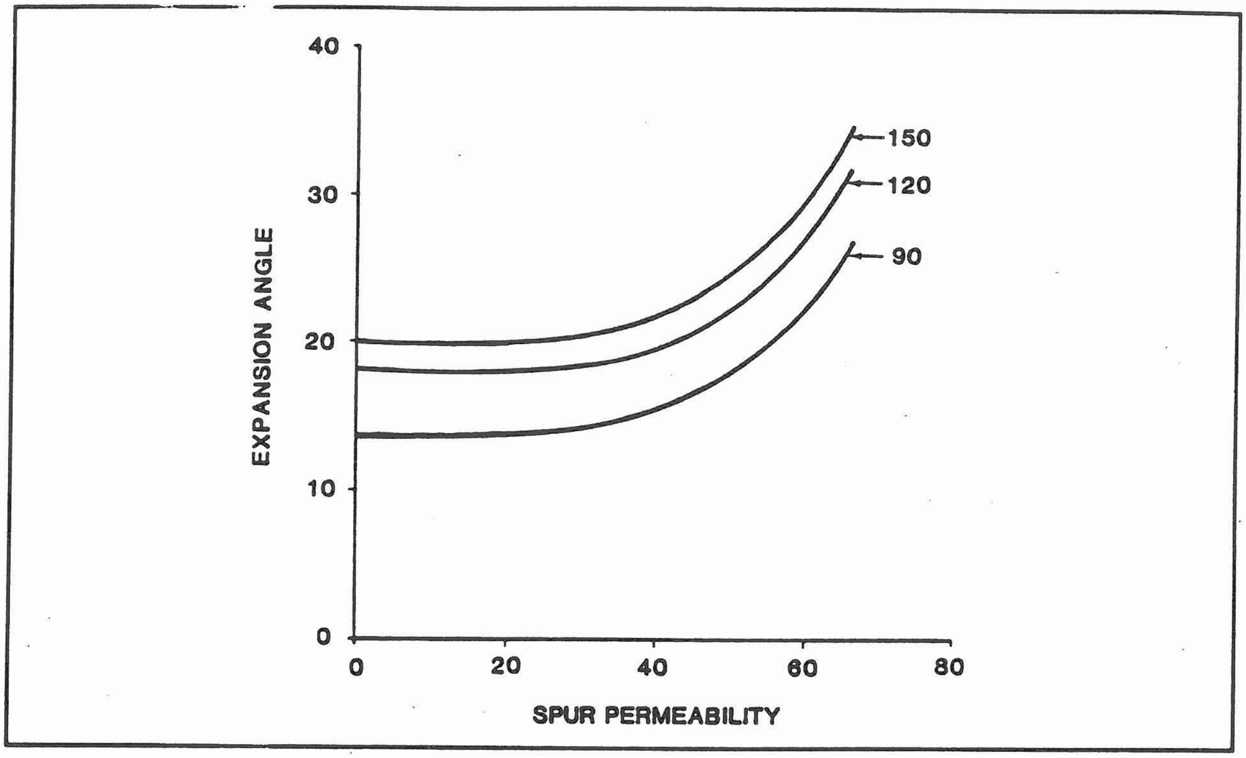


Figure 34. Spur permeability and spur orientation vs. expansion angle (After [44]).

Spur Height and Crest Profile

Impermeable spurs are generally designed not to exceed the bank height because erosion at the end of the spur in the overbank area could increase the probability of outflanking at high stream stages. Where stream stages are greater than or equal to the bank height, impermeable spurs should be equal to the bank height. If flood stages are lower than the bank height, impermeable spurs should be designed so that overtopping will not occur at the bank. Bank erosion is more severe if the spur is oriented in the downstream direction.

The crest of impermeable spurs should slope downward away from the bank line, because it is difficult to construct and maintain a level spur of rock or gabions. Use of a sloping crest will avoid the possibility of overtopping at a low point in the spur profile, which could cause damage by particle erosion or damage to the stream bank.

Permeable spurs, and in particular those constructed of light wire fence, should be designed to a height that will allow heavy debris to pass over the top. However, highly permeable spurs consisting of jacks or tetrahedrons are dependent on light debris collecting on the spur to make them less permeable. The crest profile of permeable spurs is generally level except where bank height requires the use of a sloping profile.

Bed and Bank Contact

The most common causes of spur failure are undermining and outflanking by the stream. These problems occur primarily in highly alluvial streams that experience wide fluctuations in the channel bed. Impermeable rock riprap spurs and gabion spurs can be designed to counter erosion at the toe by providing excess material on the streambed as illustrated in Figures 35 and 36. As scour occurs, excess material is launched into the scour hole, thus protecting the end of the spur. Gabion spurs are not as flexible as riprap spurs and may fail in very dynamic alluvial streams.

Permeable spurs can be similarly protected as illustrated in Figure 37. The necessity for using riprap on the full length of the spur or any riprap at all is dependent on the erodibility of the stream bed, the distance between the slats and the stream bed, and the depth to which the piling are driven. The measure illustrated would also be appropriate as a retrofit measure at a spur that has been severely undermined, and as a design for locations at which severe erosion of the toe of the stream bank is occurring.

Piles supporting permeable structures can also be protected against undermining by driving piling to depths below the estimated scour. Round piling are recommended because they minimize scour at their base.

Extending the facing material of permeable spurs below the stream bed also significantly reduces scour. If the retarder spur or retarder/deflector spur performs as designed, retardance and diversion of the flow within the length of the structure should make it unnecessary to extend the facing material the full depth of anticipated scour except at the toe.

A patented Henson spur, as illustrated in Figure 38, and marketed by Hold That River, Inc. of Houston, Texas maintains contact with the stream bed by vertical wood slats mounted on pipes which are driven to depths secure from scour. The units slide down the pipes where undermining occurs. Additional units can be added on top as necessary.

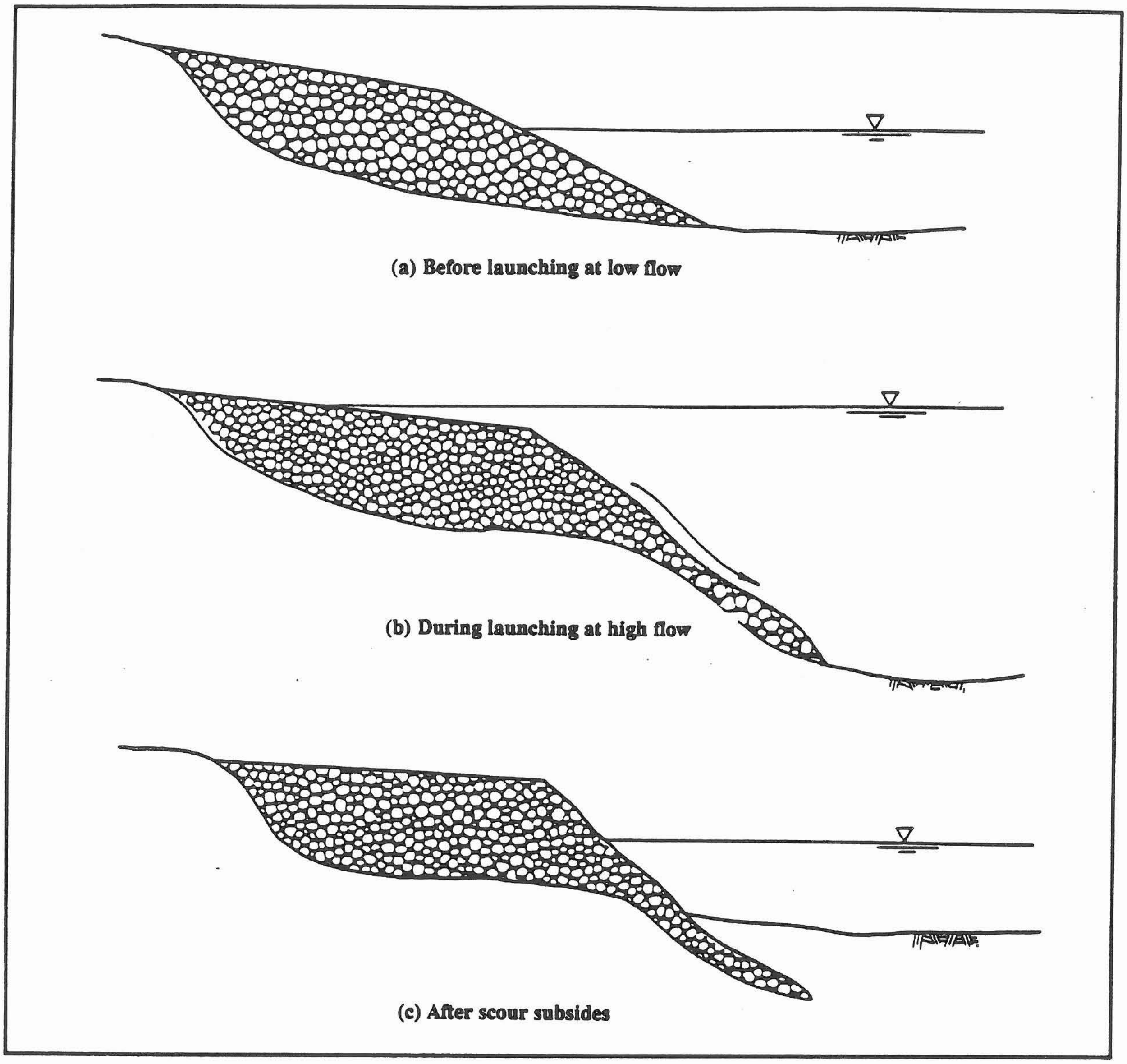


Figure 35. Launching of stone toe protection on a riprap spur: (a) before launching at low flow, (b) during launching at high flow, and (c) after scour subsides (After [44]).

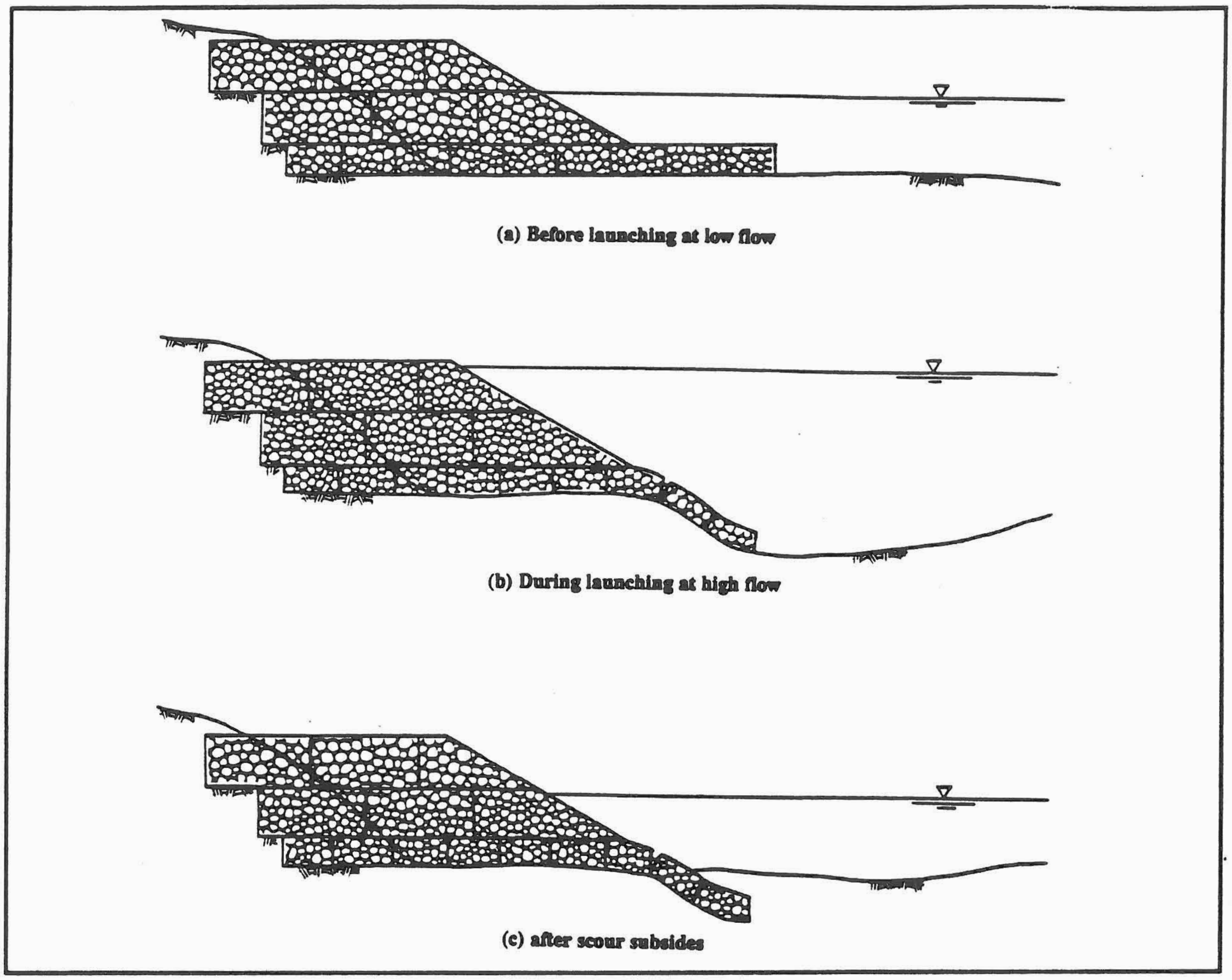


Figure 36. Gabion spur illustrating flexible mat tip protection: (a) before launching at low flow, (b) during launching at high flow, and (c) after scour subsides (After [44]).

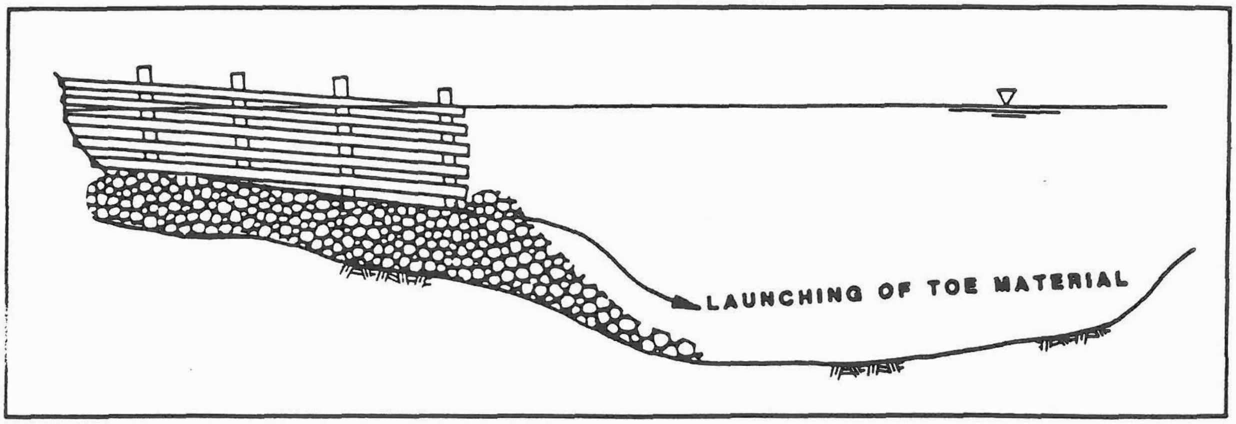


Figure 37. Permeable wood-slat fence spur showing launching of stone toe material (After [44]).

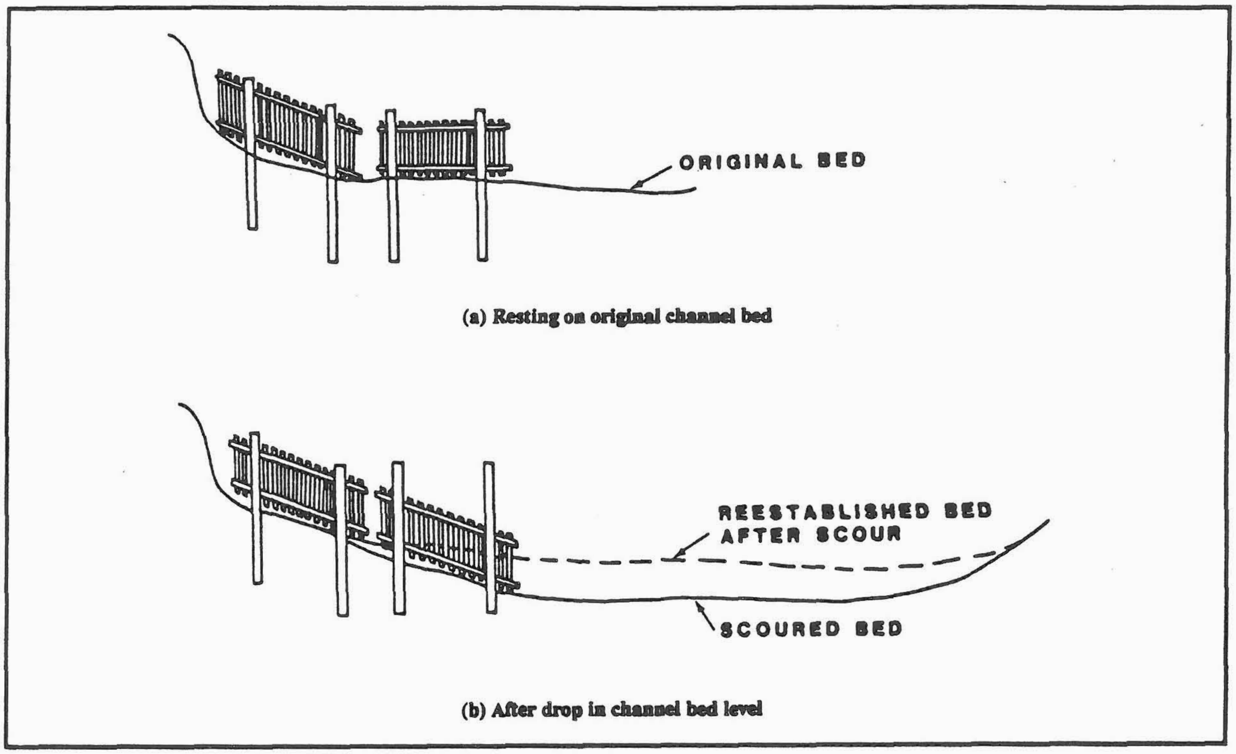


Figure 38. Henson spurs (a) resting on original channel bed, and (b) after drop in channel bed level (After [44]).

Spur Spacing

Spur spacing is a function of spur length, spur angle, permeability, and the degree of curvature of the bend. 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. This ratio is susceptible to alteration by excavation on the inside of the bend or by scour caused by the spur installation. Figure 39 indicates that the expansion angle for impermeable spurs is an almost constant 17 degrees. Spurs with 35 percent permeability have almost the same expansion angle except where the spur length is greater than about 18 percent of the channel width.

As permeability increases, the expansion angle increases, and as the length of spurs relative to channel width increases, the expansion angle increases exponentially. The expansion angle varies with the spur angle, but not significantly.

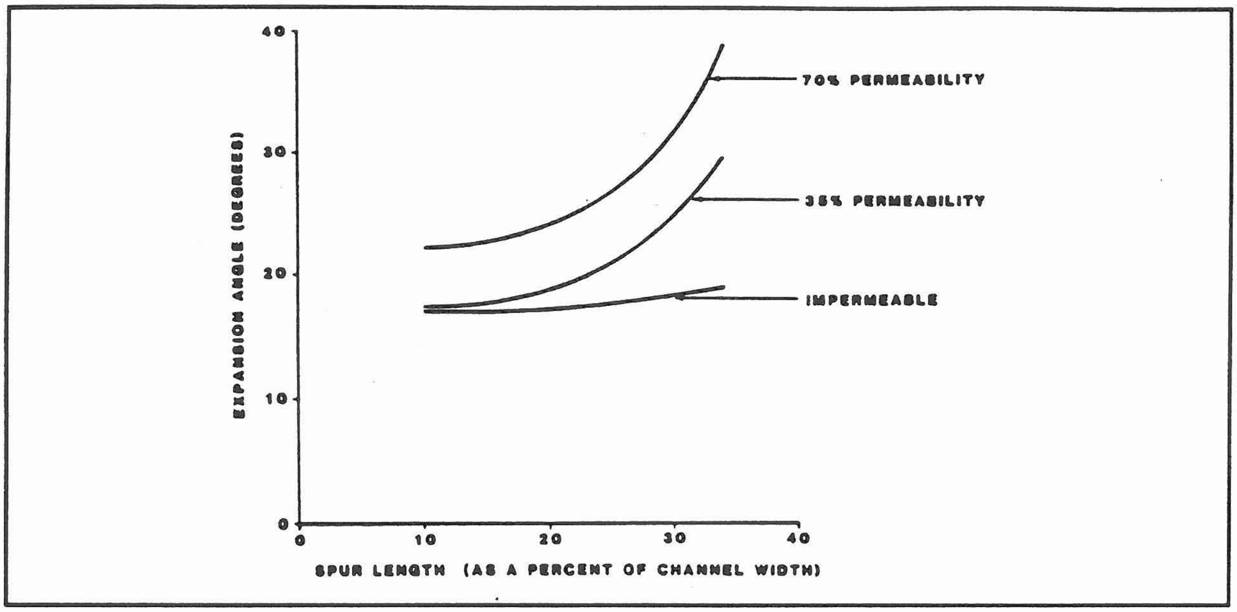


Figure 39. Relationship between spur length and expansion angle for several spur permeabilities (After [44]).

Spur spacing in a bend can be established by first drawing an arc representing the desired flow alignment (see Figure 40). This arc will represent the desired extreme location of the thalweg nearest the outside bank in the bend. The desired flow alignment may differ from existing conditions or represent no change in conditions, depending on whether there is a need to arrest erosion of the concave bank or reverse erosion that has already occurred. If the need is to arrest erosion, permeable retarder spurs or retarder structures may be appropriate. If the flow alignment must be altered in order to reverse erosion of the bank or to alter the flow alignment significantly, deflector spurs or retarder/deflector spurs are appropriate. The arc representing the desired flow alignment may be a compound circular curve or any curve which forms a smooth transition in flow directions.

The second step is to draw an arc representing the desired bank line. This may approximately describe the existing concave bank or a new theoretical bank line which protects the existing bank from further erosion. Also, draw an arc connecting the toe of spurs in the installation. The distance from this arc to the arc describing the desired bank line, along with the expansion angle, fixes the spacing between spurs. The arc describing the ends of spurs projecting into the channel will be essentially concentric with the arc describing the desired flow alignment.

The third step consists of establishing the location of the spur to be located at the downstream end of the installation. This can be done by first multiplying the distance between the arcs established in steps two and three by the cotangent of the expansion angle. This is the distance that the toe of the first spur will be located upstream of the point on the bank which does not required protection. This can also be done graphically on a scale drawing.

The fourth step is to establish the spacing between each of the remaining spurs in the installation (see Figure 40). The distance between the spur located in step four and the next spur upstream is the length of the first spur between the arc describing the desired bank line and the toe of the spur multiplied by the cotangent of the flow expansion angle. This is the distance between the toe of spurs measured along a chord of the arc describing the spur toes. Remaining spurs in the installation will be at the same spacing if the arcs are concentric. The procedure is illustrated by Figure 40 and expressed in Equation 14.

At less than bank full flow rates, flow currents may approach the concave bank at angles greater than those estimated from Figure 39. Therefore, spurs should be well-anchored into the existing bank, especially the spur at the upstream end of the installation, to prevent outflanking.

The above procedure is expressed in Equation 14:

$$S = L \cot\theta \tag{14}$$

where:

S is the spacing between spurs at the toe, feet

L is the effective length of spur, or the distance between arcs describing the toe of spurs and the desired bank line, feet, and

θ is the expansion angle downstream of spur tips, degrees

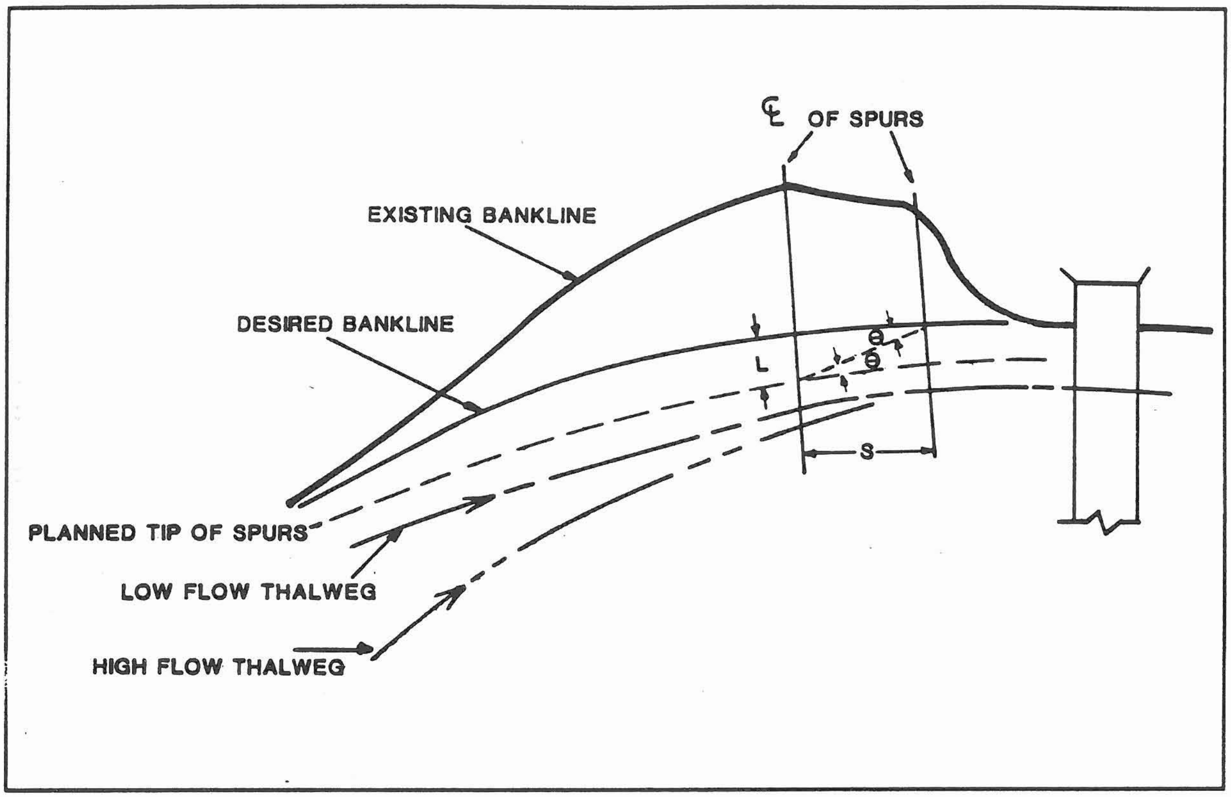


Figure 40. Spur spacing in a meander bend (After [44]).

Shape and Size of Spurs

In general, straight spurs should be used for most bank protection. Straight spurs are more easily installed and maintained and require less material. For permeable spurs, the width depends on the type of permeable spur being used. Less permeable retarder/deflector spurs which consist of a soil or sand embankment should be straight with a round nose as shown in Figure 41.

The top width of embankment spurs should be a minimum of 3 feet. However, in many cases the top width will be dictated by the width of any earth moving equipment used to construct the spur. In general a top width equal to the width of a dump truck can be used. The side slopes of the spur should be 2:1 or flatter.

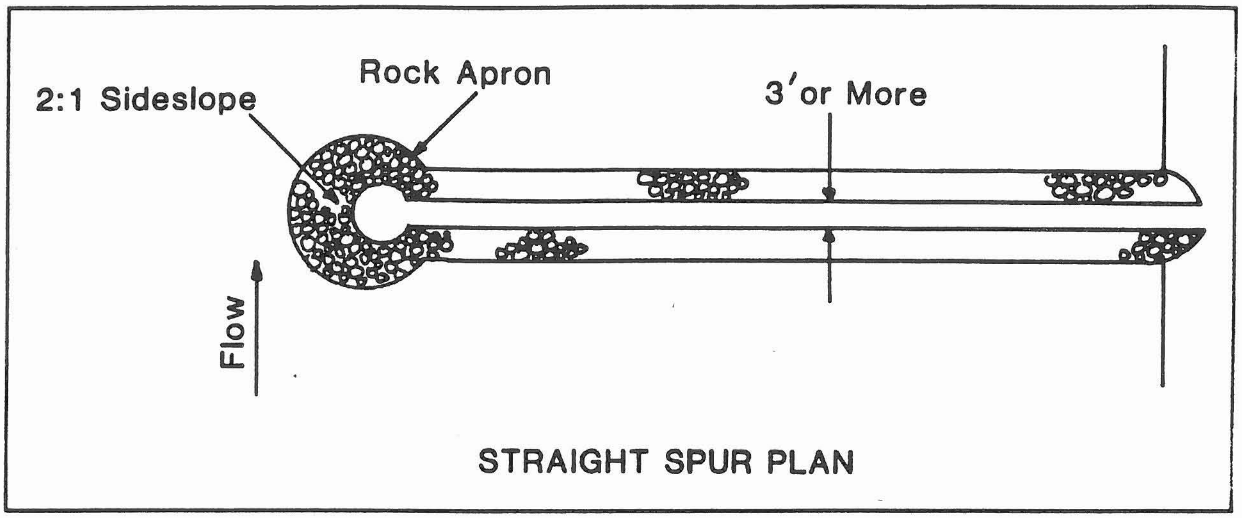


Figure 41. Typical straight, round nose spur.

Riprap

Rock riprap should be placed on the upstream and downstream faces as well as on the nose of the spur to inhibit erosion of the spur. Depending on the embankment material being used, a gravel, sand, or fabric filter may be required. The designer is referred to Highways in the River Environment [4] for design procedures for sizing riprap at spurs.

It is recommended that riprap be extended below the bed elevation to a minimum depth of five feet. Riprap should also extend to the crest of the spur, in cases where the spur would be submerged at design flow, or to two feet above the design flow, if the spur crest is higher than the design flow depth. Additional riprap should be placed around the nose of the spur, so that spur will be protected from scour.

6.3.2 Design Example of Spur Installation

Figure 42 illustrates a location at which a migrating bend threatens an existing bridge (existing conditions are shown with a solid line). Ultimately, based upon the following design example, seven spurs will be required. Although the number of spurs is not known a-priori, the spurs (and other design steps) are shown as dashed lines on Figure 42 as they will be specified after completing the following design example.

For this example, it is desirable to establish a different flow alignment and to reverse erosion of the concave (outside) bank. The spur installation has two objectives: (1) to stop migration of the meander before it damages the highway stream crossing, and (2) to reduce scour at the bridge abutment and piers by aligning flow in the channel with the bridge opening.

Permeable retarder/deflector spurs or impermeable deflector spurs are suitable to accomplish the objectives and the stream regime is favorable for the use of these types of countermeasure. The expansion angle for either of these spur types is approximately 17 degrees for a spur length of about 20% of the desired channel width, as indicated in Figure 39.

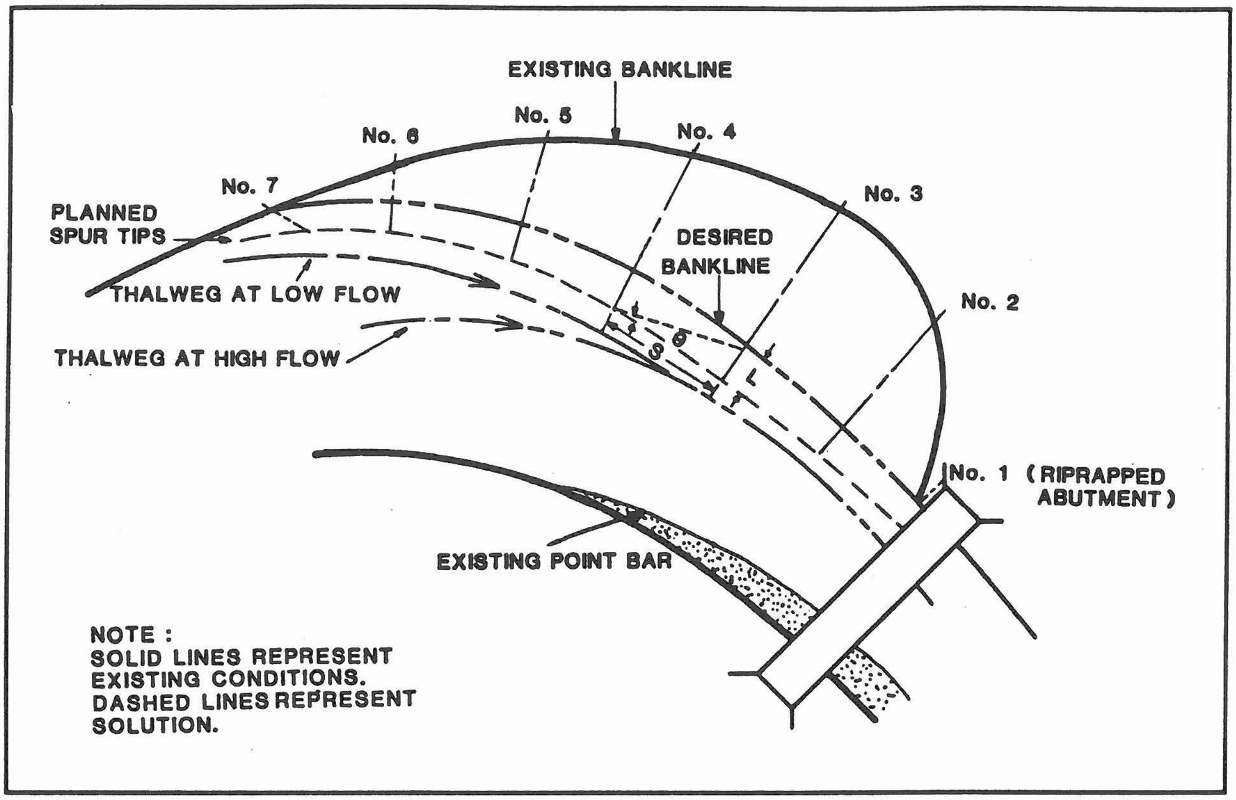


Figure 42. Example of spur design.

Step 1. Sketch Desired Thalweg

The first step is to sketch the desired thalweg location with a smooth transition from the upstream flow direction through the curve to an approach straight through the bridge waterway (see Figure 42). For an actual location, it would be necessary to examine a greater length of stream to establish the most desirable flow alignment.

Step 2. Sketch Alignment of Spur Toes

The second step is to sketch a smooth curve through the toe locations of the spurs, concentric with the desired flow alignment. The theoretical or desired left bank line is established as a continuation of the bridge abutment and left bank downstream through the curve, smoothly joining the left bank at the upstream extremity of eroded bank.

Step 3. Locate First Spur

Step number three is to locate spur number 1 so that flow expansion from the toe of the spur will intersect the stream bank downstream of the abutment. This is accomplished by projecting an angle of 17 degrees from the abutment alignment to an intersection with the arc describing the toe of spurs in the installation or by use of Equation 14. Spurs are set at 90 degrees to a tangent with the arc for economy of construction. Alternatively, the first spur could be considered to be either the upstream end of the abutment or guide bank.

It may be desirable to place riprap on the stream bank at the abutment. Furthermore, the lateral size of the scour hole at the spur directly upstream of the bridge should be estimated using the procedures described in Figures 30, 31, and 33. If the extent of scour at this spur, overlaps local scour at the pier, total scour depth at the pier may be increased. This can be determined by extending the maximum scour depth at the spur tip, up to the existing bed elevation at the pier at the angle of repose.

Step 4. Locate Remaining Spurs

Spurs upstream of spur number 1 are then located by use of Equation 14, using dimensions as illustrated in Figure 40. Using this spur spacing, deposition will be encouraged between the desired bank line and the existing eroded bank.

The seventh and last spur upstream is shown oriented in a downstream direction to provide a smooth transition of the flow approaching the spur field. This spur could have been oriented normal to the existing bank, and been shorter and more economical, but might have created excessive scour. Angling the furthest upstream spur in the downstream direction provides a smoother transition into the spur field, and decreases scour at the toe of the spur. Subsequent spurs downstream can be oriented normal to the intended bank line for economy.

Note that spur number 7 is somewhat downstream of the beginning of the eroded bank. This area could be protected in one of two ways. The first would be to orient spur number seven perpendicular to the planned bank and install an eighth spur, angled downstream which begins upstream of the eroded bank. The second method would be to install a hard point where the bank is beginning to erode. Hard points are discussed in the section entitled "Other Countermeasures". In this case the hard point can be considered as a very short spur which is located at the intersection of the actual and planned bank lines. In either case, spurs or hard points should be anchored well into the bank to prevent outflanking.

Spur lengths of less than 20 percent of channel width protect a greater length of channel bank relative to the projected length of spur. In the above example, spur lengths are obviously greater than 20 percent of the total channel width. However, if projected spur lengths are measured from the desired bank line, the guidance developed from laboratory tests is followed. In instances where excessive constriction of the channel would result from spur installation, excavation on the inside of the channel bend may be advisable.

6.4 Guide Banks

6.4.1 Design Considerations

When embankments encroach on wide flood plains, the flows from these areas must flow parallel to the approach embankment to the bridge opening. These flows can erode the approach embankment. At the abutment severe flow contraction can reduce the effective bridge opening, possibly increasing the severity of abutment and pier scour.

Guide banks (also referred to as spur dikes) can be used in these cases to prevent erosion of the approach embankments by cutting off the flow adjacent to the embankment, guiding stream flow through a bridge opening, and transferring scour away from abutments to prevent damage caused by abutment scour. The two major enhancements guide banks bring to bridge design are 1) reduce the separation of flow at the upstream abutment face and thereby maximize the use of the total bridge waterway area and 2) reduce the abutment scour due to lessening turbulence at the abutment face. Guide banks can be used on both sand and gravel bed streams.

Principal factors to be considered when designing guide banks, are their orientation to the bridge opening, plan shape, upstream and downstream length, cross-sectional shape, and crest elevation. Reference [19] is used as the principal design reference for this section.

Figure 43 presents a typical guide bank plan view. It is apparent from the figure that without this guide bank overbank flows would return to the channel at the bridge opening, which can increase the severity of contraction and scour at the abutment. Note, that with installation of guide banks the scour holes which normally would occur at the abutments of the bridge are moved upstream away from the abutments. Guide banks may be designed at each abutment, as shown, or singly, depending on the amount of overbank or floodplain flow directed to the bridge by each approach embankment.

The goal in the design of guide banks is to provide a smooth transition and contraction of the stream flow through the bridge opening. Ideally, the flow lines through the bridge opening should be straight and parallel. As in the case with other countermeasures, the designer should consider the principles of river hydraulics and morphology, and exercise sound engineering judgment.

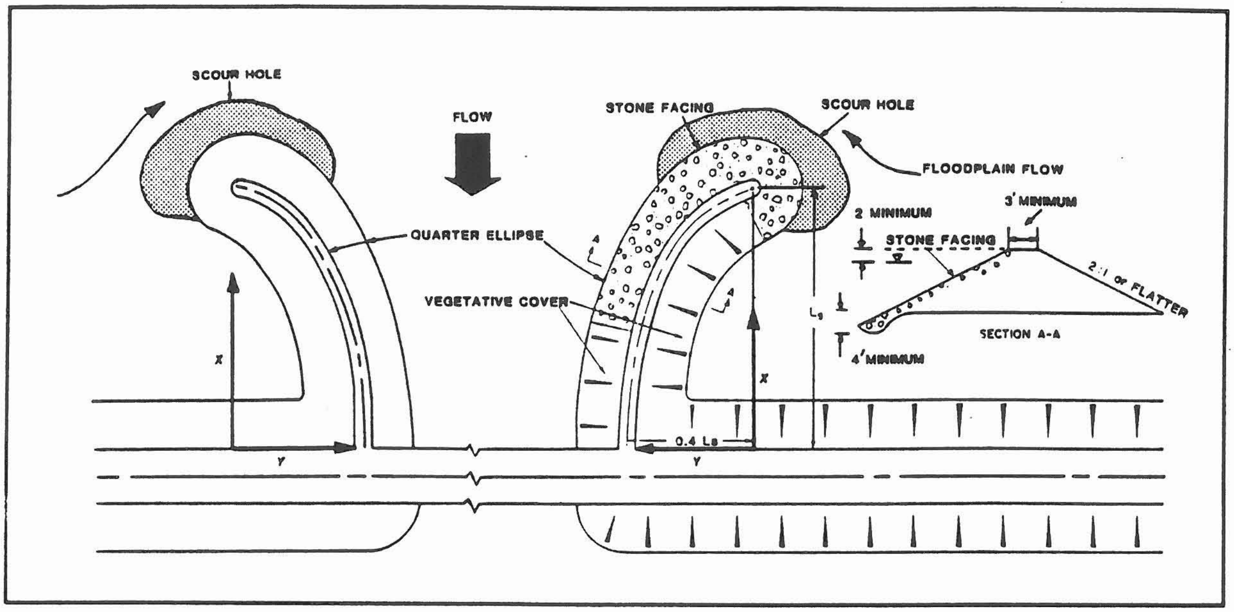


Figure 43. Typical guide bank (Modified from [19]).

Orientation

Guide banks should start at and be set parallel to the abutment and extend upstream from the bridge opening. The distance between the guide banks at the bridge opening should be equal to the distance between bridge abutments. Best results are obtained by using guide banks with a plan form shape in the form of a quarter of an ellipse, with the ratio of the major axis (length L_s) to the minor axis (offset) of 2.5 : 1. This allows for a gradual constriction of the flow. Thus, if for design purposes the length of the guide bank, measured perpendicularly from the approach embankment to the upstream nose of the guide bank is denoted as L_S, the amount of expansion of each guide bank (offset), measured from the abutment parallel to the approach roadway, should be 0.4 L_S.

The plan view orientation can be determined using Equation 15, which is the equation of an ellipse with origin at the nose of the guide bank. For this equation, X is the distance measured perpendicularly from the bridge approach and Y is the offset measured parallel to the approach embankment, as shown on Figure 43.

$$\frac{X^2}{L_s^2} + \frac{Y^2}{(0.4L_s)^2} = 1 \tag{15}$$

It is important that the face of the guide bank match the abutment so that the flow is not disturbed where the guide bank meets the abutment. For new bridge construction, abutments can be sloped to the channel bed at the same angle as the guide bank. For retrofitting existing bridges modification of the abutments or wing walls may be necessary.

Length

For design of guide banks, the length of the guide bank, L_s must first be determined. This can be easily determined using a nomograph which was developed from laboratory tests performed at Colorado State University [45] and [46] and from field data compiled by the U.S. Geological Survey [47]. For design purposes the utilization of the nomograph involves the following parameters:

Q	=Total discharge of the stream (cfs)
Q_f	=Lateral or flood plain discharge of either flood plain (cfs)
Q_{100}	=Discharge in 100 feet of stream adjacent to the abutment (cfs)
b	=Length of the bridge opening (ft)
A_{n2}	=Cross-sectional flow area at the bridge opening at normal stage (ft^2)
V_{n2}	$= \frac{Q}{A_{n2}}$=Average velocity through the bridge opening (cfs)
$\frac{Q_f}{Q_{100}}$	=Guide bank discharge ratio
L_s	=Projected length of guide bank.

A nomograph is presented in Figure 44 to determine the projected length of guide banks. This nomograph should be used to determine the guide bank length for designs greater than 50 feet and less than 250 feet. If the nomograph indicates the length required to be greater than 250 feet the design should be set at 250 feet. It is recommended that the minimum length of guide banks be 50 feet. An example of how to use this nomograph is presented in the next section.

FHWA practice has shown that many guide banks have performed well using a standardized length of 150 feet. Based on this experience, guide banks of 150 feet in length should perform very well in most locations. Even shorter guide banks have been successful if the guide bank intersects the tree line.

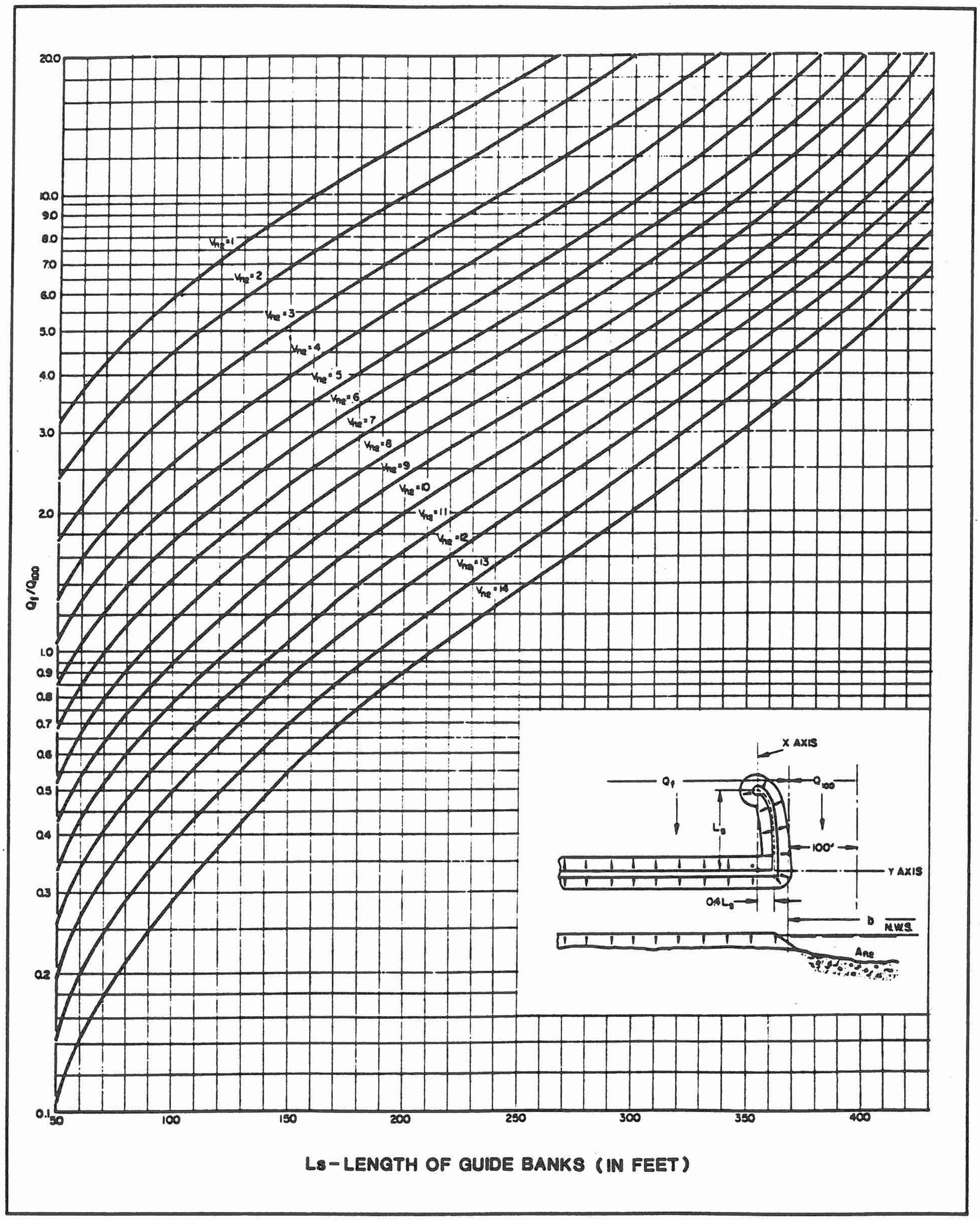


Figure 44. Nomograph to determine guide bank length (After [19]).

Crest Height

As with deflection spurs, guide banks should be designed so that they will not be overtopped at the design discharge. If this were allowed to occur, unpredictable cross flows and eddies might be generated, which could scour and undermine abutments and piers. In general, a minimum of 2 feet of freeboard, above the design water surface elevation should be maintained.

Shape and Size

The cross-sectional shape and size of guide banks should be similar to deflector, or deflector/retarder spurs discussed previously. Generally, the top width is 10 to 12 feet, but the minimum width is 3 feet when construction is by drag line. The upstream end of the guide bank should be round nosed. Side slopes should be 2:1 or less.

Downstream Extent

In some states, highway departments extend guide banks downstream of the abutments to minimize scour due to rapid expansion of the flow at the downstream end of the abutments. These downstream guide banks are sometimes called "heels". If the expansion of the flow is too abrupt, a shorter guide bank, which usually is less than 50 feet long, can be used downstream. Downstream guide banks should also start at and start parallel to the abutment and the distance between them should enlarge as the distance from the abutment of the bridge increases.

In general downstream guide banks are a shorter version of the upstream guide banks. Riprap protection, crest height and width should be designed in the same manner as for upstream guide banks.

Riprap

Guide banks are constructed by forming an embankment of soil or sand extending upstream from the abutment of the bridge. To inhibit erosion of the embankment materials, guide banks must be adequately protected with riprap or stone facing.

Rock riprap should be placed on the stream side face as well around the end of the guide bank. It is not necessary to riprap the side of the guide bank adjacent to the highway approach embankment. As in the case of spurs, a gravel sand or fabric filter may be required to protect the underlying embankment material. The designer is referred to Highways in the River Environment [4] for design procedures for sizing riprap. Riprap should be extended below the bed elevation to a minimum depth of five feet and extend up the face of the guide bank to two feet above the design flow. Additional riprap should be placed around the upstream end of the guide bank so to protect the embankment from scour.

As in the case of spurs, it is important to adequately tie guide banks into the approach embankment.

Guide banks on Non-symmetrical Highway Crossings

Hydraulics of Bridge Waterways [19] states:

> "From meager testing done to date, there is not sufficient evidence to warrant using longer dikes (guide banks) at either abutment on skewed bridges. Lengths obtained from [the nomograph] should be adequate for either normal or skewed crossings."

Therefore, for skewed crossings, the length of guide banks should be set using the nomograph for whichever side of the bridge crossing which yields the largest guide bank length.

Other Design Concerns

In some cases, where the cost of stone riprap facing is prohibitive, the guide bank can be covered with sod or other minimal protection. If this approach is selected, the design should allow for and stipulate the repair or replacement of the guide bank after each high water occurrence.[19] Other measures which will minimize damage to approach embankments, and guide banks during high water are:

* Keep trees as close to the toe of guide bank embankments as construction will permit. Trees will increase the resistance to flow near and around the toe of the embankment, thus reducing velocities and scour potential.

* Do not allow the cutting of channels or the digging of borrow pits along the upstream side of approach embankments and near guide banks. Such practices encourage flow concentration and increases velocities and erosion rates of the embankments.

* In some cases the area behind the guide bank may be too low to drain properly after a period of flooding. This can be a problem, especially when the guide bank is relatively impervious. Small drain pipes can be installed in the guide bank to drain this ponded water.

* In some cases, only one approach will cut off the overbank flow. This is common when one of the banks is high and well defined. In these cases, only one guide bank may be necessary.

6.4.2 Design Example of Guide Bank Installation

For the example design of a guide bank, Figure 45 will be used. This figure shows the cross-section of the channel and floodplain before the bridge is constructed and the plan view of the approach, guide banks, and embankments after the design steps outlined below are completed.

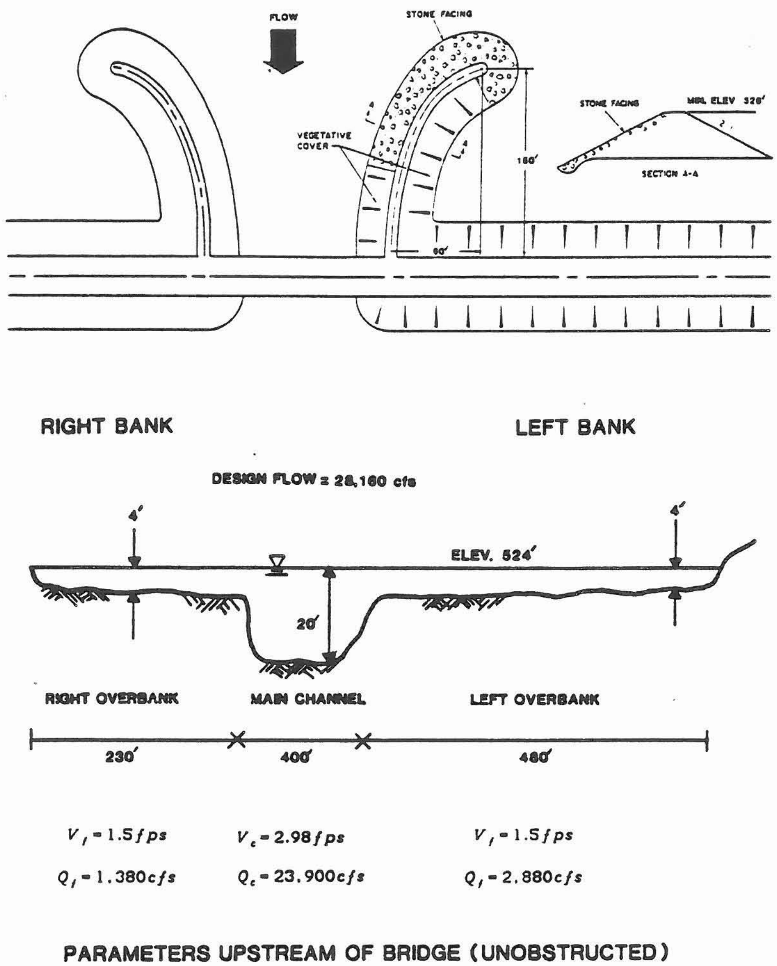


Figure 45. Example guide bank design.

Step 1. Hydraulic Design Parameters

The first step in the design of guide banks requires the computation of the depth and velocity of the design flood in the main channel and in the adjacent overbank areas. These studies are performed by using step backwater computations upstream and through the bridge opening. The computer programs WSPRO or HEC-2 are suitable for these computations. Using these programs or by using conveyance curves developed from actual data, the discharges and depths in the channel and overbank areas can be determined.

To use the conveyance curve approach, the designer is referred to example problem Number 4 (page 71) in Hydraulics of Bridge Waterways [19] for methods to determine these discharges and areas. That publication also contains another example of the design of a guide bank.

For this example, the total, overbank, and channel discharges, as well as the flow area are given. We also assume that a bridge will span a channel with a bottom width of 400 feet and that the abutments will be set back 100 feet from each bank of the main channel. The abutments of this bridge are spill-through with a side slope of 2:1. The design discharge is 28,160 cfs, which after backwater computations, results in a mean depth of 20 feet in the main channel and a mean channel velocity of 2.98 fps.

Step 2. Determine Q_f in the left and right overbank

The depth in both overbank areas are 4 feet and the width of the left and right overbank areas are 480 and 230 feet respectively. Velocity in the overbank areas (assuming no highway approach embankment) is 1.5 fps. Therefore, by noting that the discharge is the product of the velocity and the cross-sectional area in the overbank areas, Q_f will be equal to 2,880 cfs for the left overbank and 1,380 cfs for the right overbank.

Step 3. Determine Q_{100} and Q_f / Q_{100} for the left and right overbank

The overbank discharge in the first 100 feet of stream adjacent to the left and right abutments needs to be determined next. Since for this case the flow is of uniform depth (4 feet) and velocity (1.5 fps) over the entire width of the floodplain, and the abutments will be set back 100 feet from the main channel banks, the value of Q_{100} will be 600 cfs for both sides.

For the left and right overbanks the reference values of Q_f / Q_{100} can be determined by simple division of the discharges determined in previous steps. For this example, these values are 4.8 and 2.3 for the left and right overbank respectively. For design purposes, the largest value will result in the more conservative determination of the length of the guide bank. Therefore for this example, a value of Q_f / Q_{100} equal to 4.8 will be used.

Step 4. Determine The length of the guide bank, L_s

The average channel velocity through the bridge opening can be determined by dividing the total discharge of the stream (Q) by the cross sectional flow area at the bridge opening (A_{n2}), which in this case includes the main channel ($8000\ ft^2$) plus 100 feet of the left and right overbank areas adjacent to the abutments at the bridge opening ($800\ ft^2$). Thus, the average channel velocity (V_{n2}) is 3.2 fps. For Q_f / Q_{100} equal to 4.8 and an average channel velocity of 3.2 fps, the length of the guide bank is determined using the nomograph presented in Figure 44. For this example the length, L_s determined from the nomograph is approximately 150 feet. The offset of the guide bank is determined by multiplying L_s by 0.4. These dimensions locate the end of the guide bank. The shape of the guide bank from this location to the abutment is simply an ellipse as described by Equation 15.

Step 5. Miscellaneous Specifications

The crest of the guide bank must be a minimum of 2 feet above the water surface. Therefore, the crest elevation for this example should be greater than or equal to 526 feet. The crest width should be at least 3 feet. For this example, a crest width of 10 feet will be specified so that the guide bank can be easily constructed with dump trucks.

Stone or rock riprap should be placed in the locations shown on Figure 45. This riprap should extend a minimum of two feet above the design water surface (Elevation 526 feet) and at least four feet below the intersection of the toe of the guide bank and the existing ground surface.

6.5 Check Dams

Check dams or channel drop structures are used downstream of highway crossings to arrest head cutting and maintain a stable stream bed elevation in the vicinity of the bridge. Check dams are usually built of rock riprap, concrete, sheet piles, gabions, or treated timber piles. The material used to construct the structure depends on the availability of materials, the height of drop required, and the width of the channel. Rock riprap and timber pile construction have been most successful on channels having small drops and widths less than 100 feet. Sheet piles, gabions, and concrete structures are generally used for larger drops on channels with widths ranging up to 300 feet.

Check dams can initiate erosion of banks and the channel bed downstream of the structure as a result of energy dissipation and turbulence at the drop. This local scour can undermine the check dam and cause failure. The use of energy dissipators downstream of check dams can reduce the energy available to erode the channel bed and banks. In some cases it may be better to construct several consecutive drops of shorter height to minimize extensive erosion.

Lateral erosion of channel banks just downstream of drop structures is another adverse result of check dams and is caused by turbulence produced by energy dissipation at the drop, bank slumping from local channel bed erosion, or eddy action at the banks. Bank erosion downstream of check dams can lead to erosion of bridge approach embankments and abutment foundations if lateral bank erosion causes the formation of flow channels around the ends of check dams. The usual solution to these problems is to place riprap revetments on the stream bank. Riprap of the bank downstream of the check dam will be needed. The design of riprap is given in references [4], [38], and [39].

Erosion of the stream bed can also be reduced by placing rock riprap in a preformed scour hole downstream of the drop structure. A row of sheet piling with top set at or below stream bed elevation can keep the riprap from moving downstream. Because of the problems associated with check dams, the design of these countermeasures requires designing the check dams to resist scour by providing for dissipation of excess energy and protection of areas in the bed and the bank which are susceptible to erosive forces.

6.5.1 Bed Scour For Vertical Drop Structures

The most conservative estimate of scour downstream of channel drop structures is for vertical drops with unsubmerged flow conditions. For the purposes of design the maximum expected scour will be assumed to be equal to the scour for a vertical, unsubmerged drop, regardless of whether the drop is actually sloped or is submerged.

A typical vertical drop structure is diagramed in Figure 46. The Veronese equation [48] is recommended to estimate the depth of scour downstream of a vertical drop:

$$d_s = K H_t^{0.225} q^{0.54} - d_m$$

(16)

Where:

d_s = Local scour depth for a free overfall, measured from the stream bed downstream of the drop, ft

q = Discharge per unit width, cfs per foot of width

H_t = Total drop in head, measured from the upstream to the downstream energy grade line, ft

d_m = Tail water depth, ft

K = 1.32

It should be noted that H_t is the difference in the total head from upstream to downstream. This can be computed using Bernouli's equation for steady uniform flow:

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

(17)

Where:

Y = Depth, ft

V = Velocity, ft/s

Z = Bed elevation referenced to a common datum, ft

g = Acceleration due to gravity 32.2 f/s^2

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

The depth of scour as estimated by the above equation is independent of the grain size of the bed material. This concept acknowledges that the bed will scour regardless of the type of material composing the bed, but the rate of scour depends on the composition of the bed.

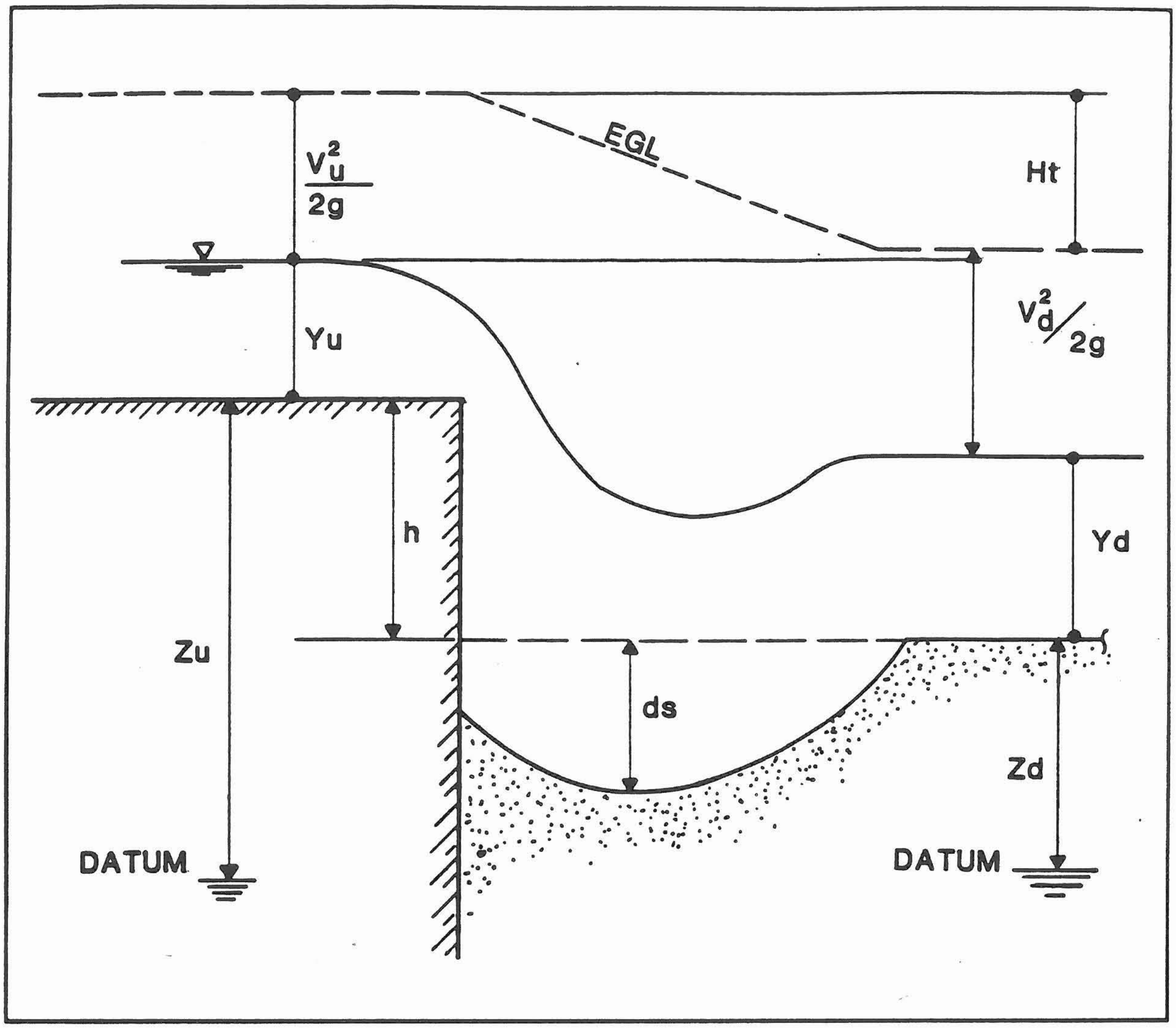


Figure 46. Schematic of a vertical drop caused by a check dam.

In some cases, with large or resistant material, it may take years or decades to develop the maximum scour hole. In these cases, the design life of the bridge may need to be considered when designing the check dam.

The drop structure must be designed structurally to withstand the forces of water and soil assuming that the scour hole is as deep as estimated using the equation above. Therefore, the designer should consult soils and structural engineers so that the drop structure will be stable under the full scour condition. 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 depth of scour (see for example [49]).

6.5.2 Design Example

The following design example is based upon a comparison of scour equations presented by the USBR [48]. For this example, as illustrated by Figure 47, the following hydraulic parameters are used:

Design Discharge	Q	= 110,000 ft^3/s
Channel Width	B	= 990 ft
Tail Water Depth	d_m, y_d	= 12.4 ft
Unit Discharge	q	= 111 $ft^3/s/ft$
Mean Velocity	V	= 8.95 ft/s
Drop Height	h	= 5 ft

For this example $H_t = 5ft$ if the drop height is 5 feet and the depth and velocity upstream and downstream of the drop are the same. Using the recommended equation the estimated depth of scour below the bed level downstream will be:

$$d_s = 1.32(5)^{0.225}(111)^{0.54} - 12.4$$

$$d_s = 11.7ft$$

If for structural reasons, the scour depth was to be limited to a maximum of 7 feet, for example, then either riprap to limit the depth of scour [49] or a series of check dams could be constructed. For this case three drops of 1.7 feet would be required. Using the recommended equation, this drop height would result in an estimated depth of scour of 6.4 feet per drop. It should be noted that if a series of drops are required, adequate distance between each drop must be maintained.

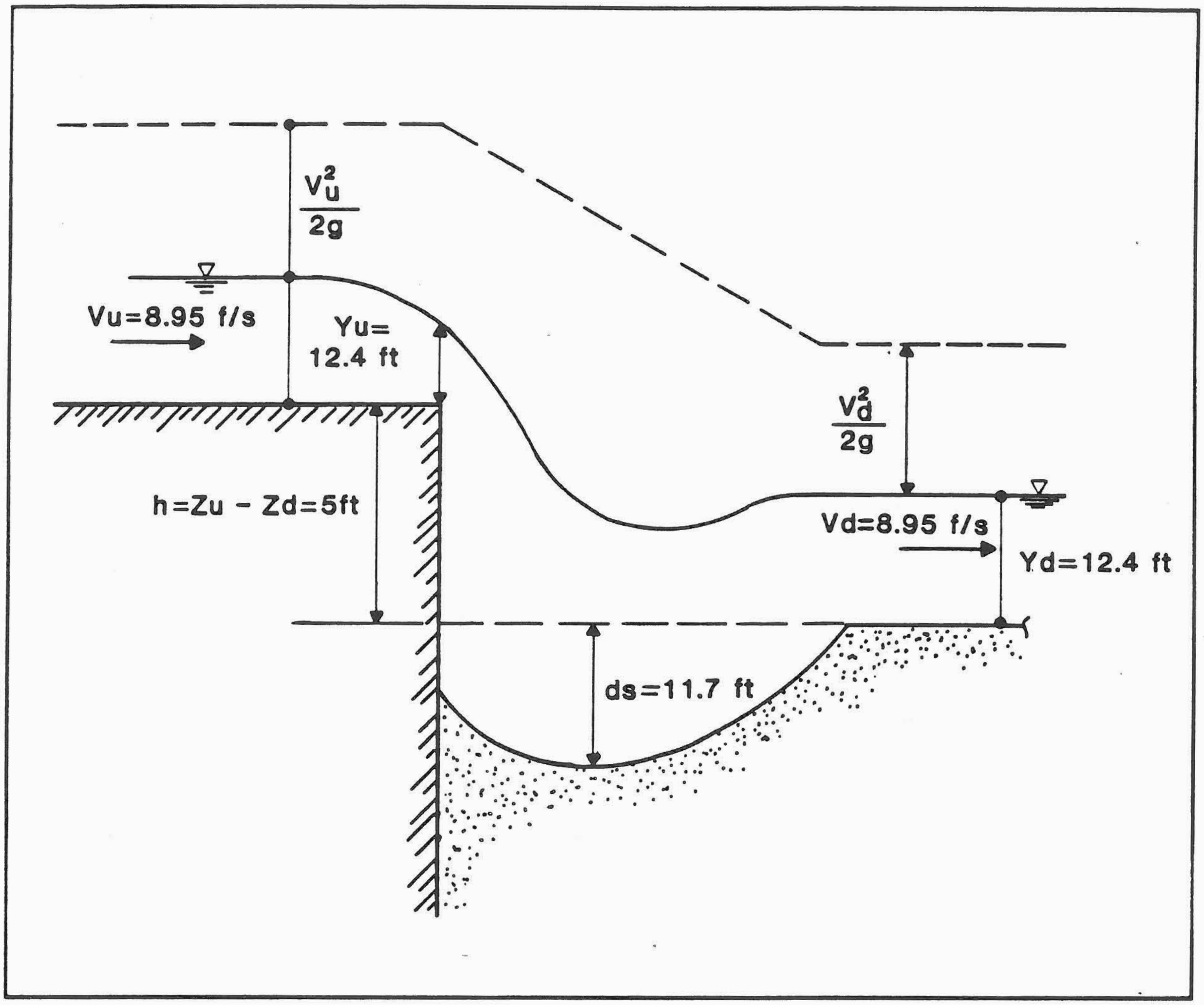


Figure 47. Design example of scour downstream of a drop structure.

6.5.3 Lateral Scour Downstream of Check Dams

As was mentioned, lateral scour of the banks of a stream downstream of check dams can cause the stream flow to divert around the check dam. If this occurs, a head cut may move upstream and endanger the highway crossing. To prevent this the banks of the stream must be adequately protected using riprap or revetments. Riprap should be sized and placed in a similar fashion as for spurs and guide banks. The designer is referred to [4] or [38] for proper sizing, and placement of riprap on the banks. Revetments are discussed in the section entitled "Other Countermeasures" in this text.

6.6 Other Countermeasures

6.6.1 Revetments

Revetments may be flexible or rigid and can be used to counter all erosion mechanisms. They may be used to provide protection for embankments, streambanks, and streambeds. They do not significantly constrict channels or alter flow patterns. Revetments have been unsuccessful in resisting slumps in saturated streambanks and embankments and relatively unsuccessful in stabilizing streambanks and streambeds in degrading streams. Special precautions must be observed in the design of revetments for degrading channels.

Flexible Revetments

Flexible revetments include rock riprap, rock-and-wire mattresses, gabions, precast concrete blocks, rock-fill trenches, windrow revetments, used tire revetments, and vegetation. Rock riprap adjusts to distortions and local displacement of materials without complete failure of the revetment installation. However, flexible rock-and-wire mattress and gabions may sometimes span the displacement of underlying materials, but usually can adjust to most local distortions. Used tire mattress and precast concrete block mattresses are generally stiffer than rock riprap and gabions, and therefore, do not adjust to local displacement of underlying materials as well. References for design guidelines of flexible revetments depend on the type of flexible revetment being used and are discussed separately in the following sections.

Rock Riprap, Rock-and Wire Mattress, Gabions, and Precast Concrete Blocks. Design guidelines, design procedures, and suggested specifications for rock riprap, wire enclosed rock, stacked block gabions, and precast concrete blocks are included in HEC No. 11, "Use of riprap for Bank Protection."[38]

Rock-Fill Trenches and Windrow Revetment. Rock-fill trenches are structures used to protect banks from caving caused by erosion at the toe. A trench is excavated along the toe of the bank and filled with rocks as shown in Figure 48. The size of trench to hold the rock fill depends on expected depths of scour.

As the streambed adjacent to the toe is eroded, the toe trench is undermined and the rock fill slides downward to pave the bank. It is advantageous to grade the banks before paving the slope with riprap and placing rock in the toe trench. The slope should be at such an angle that the saturated bank is stable while the stream stage is falling.

An alternative to a rock-fill trench at the toe of the bank is to excavate a trench above the water line along the top of the bank and fill the trench with rocks. As the bank erodes, stone material in the trench is added on an as-needed basis until equilibrium is established. This method is applicable in areas of rapidly eroding banks of medium to large size streams.

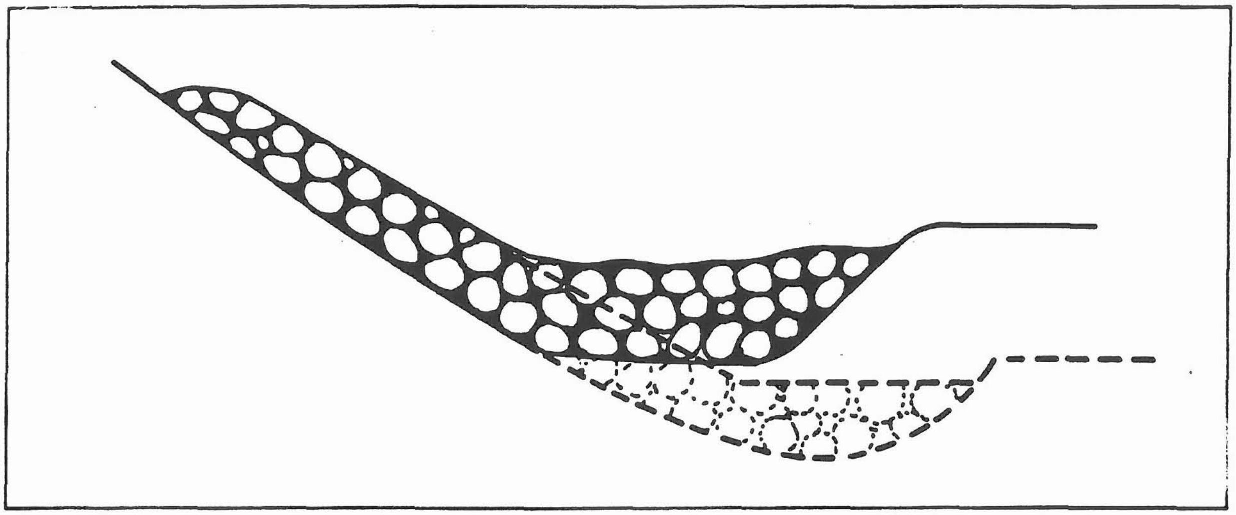


Figure 48. Rock-fill trench (After [4]).

Windrow revetment (Figure 49) consists of a supply of rock deposited along an existing bank line at a location beyond which additional erosion is to be prevented. When bank erosion reaches and undercuts the supply of rock, it falls onto the eroding area, thus giving protection against further undercutting. The resulting bank line remains in a near natural state with an irregular appearance due to intermittent lateral erosion in the windrow location. The treatment particularly lends itself to the protection of adjacent wooded areas, or placement along stretches of presently eroding, irregular bank line.

The effect of windrow revetment on the interchange of flow between the channel and overbank areas and flood flow distribution in the flood plain should be carefully evaluated. Windrow installations will perform as guide banks or levees and may adversely affect flow distribution at bridges or cause local scour. Tying the windrow to the highway embankment at an abutment would be contrary to the purpose of the windrow since the rock is intended to fall into the channel as the bank erodes. The abutment is not intended to fall into the channel.

The following observations and conclusions from model investigations of windrow revetments and rock-fill trenches may be used as design guidance. More definitive guidance is not presently available.[39]

* The application rate of stone is a function of channel depth, bank height, material size, and estimated bed scour.

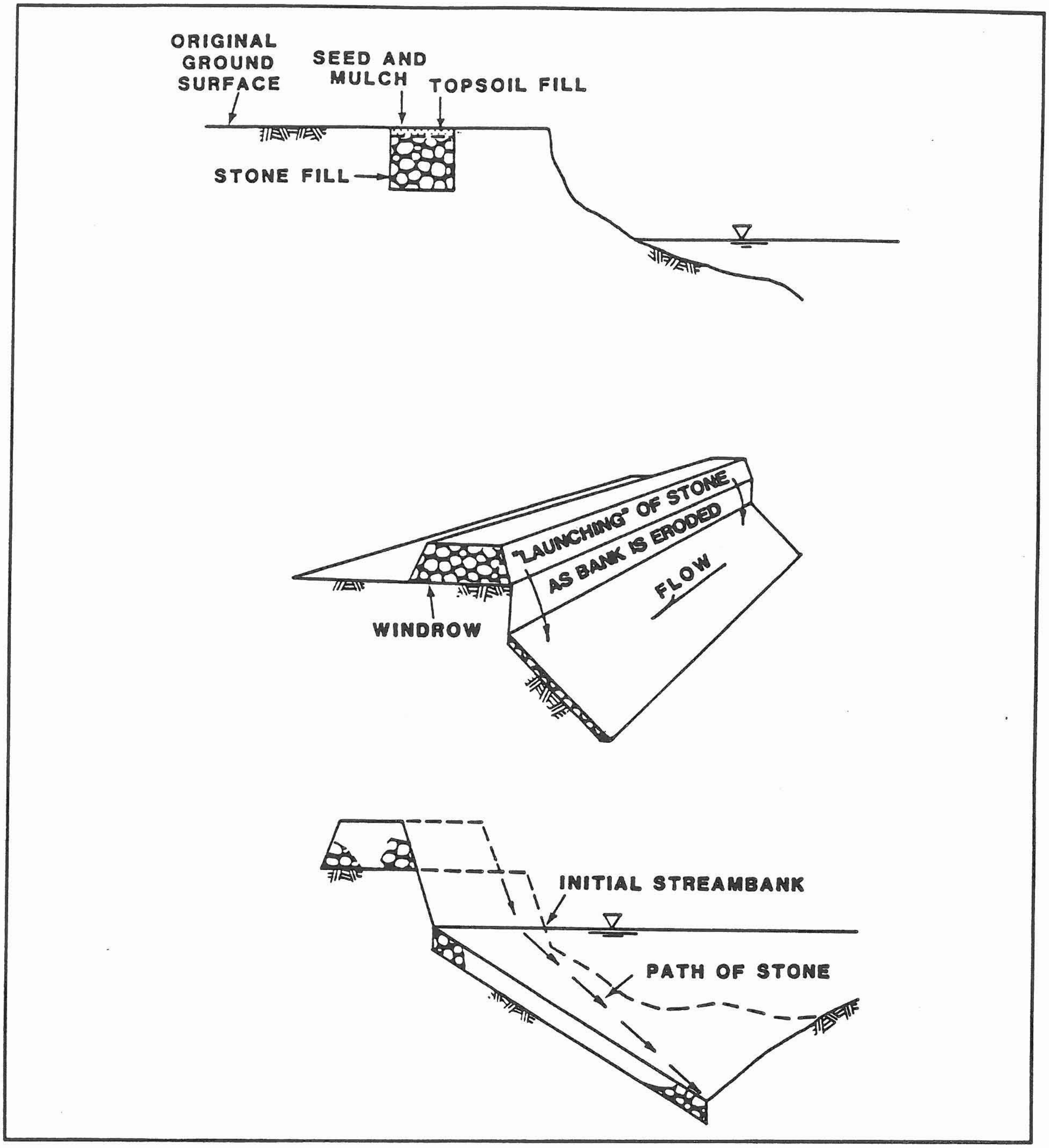


Figure 49. Windrow revetment, definition sketch (After [39]).

* A triangular windrow is the least desirable shape, a trapezoidal shape provides a uniform blanket of rock on an eroding bank, and a rectangular shape provides the best coverage. A rectangular shape is most easily placed in an excavated trench.

* Bank height does not significantly affect the final revetment; however, high banks tend to produce a nonuniform revetment alignment. Large segments of bank tend to break loose and rotate slightly on high banks, whereas low banks simply "melt" or slough into the stream.

* Stone size influences the thickness of the final revetment, and a smaller gradation of stone forms a more dense, closely chinked protective layer. Stones must be large enough to resist being transported by the stream, and a well-graded stone should be used to ensure that the revetment does not fail from leaching of the underlying bank material. Large stone sizes require more material than smaller stone sizes to produce the same relative thickness of revetment. In general, the greater the stream velocity, the steeper the side slope of the final revetment. The final revetment slope will be about 15 percent flatter than the initial bank slope.

* A windrow segment should be extended landward from the upstream end to reduce the possibility of outflanking of the windrow.

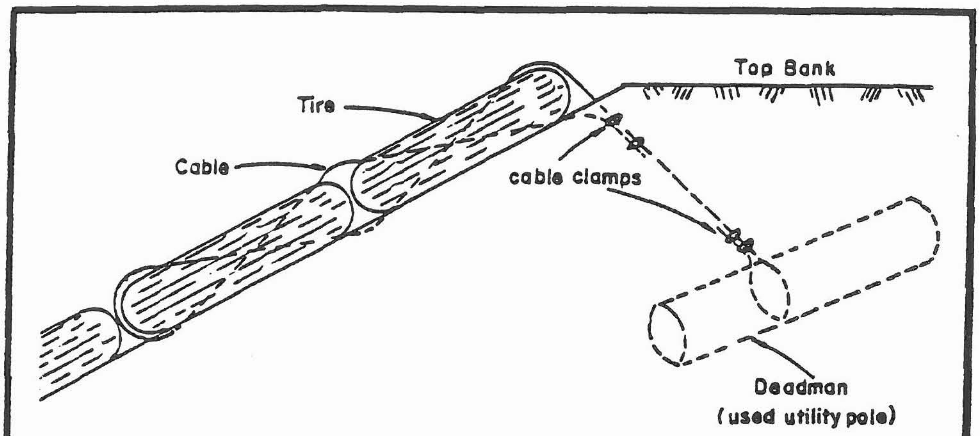
Used Tire Revetments. Used tire revetments have been successfully used for velocities up to 10 feet per second on mild bends. They will accommodate a limited amount of bank subsidence, but usually will be damaged where substantial subsidence occurs. They are not well-suited for use where scour at the toe of the installation would undermine the revetment, but a riprap launching apron or toe trench will alleviate this problem to some extent. Used tire revetments are somewhat unsightly and vandalism has proved to be more of a problem than for other schemes of bank protection. Construction is labor-intensive and is therefore expensive.

The following precautions should be followed to ensure that the mattress will stay in place on an eroding bank:

* The tires must be banded together; alternatively, cable running the length and width of the mattress can be woven through the tires.

* The top, toe and the upstream and downstream ends of the mattress must be tied to the bank (Figure 50). Riprap should be placed at the toe of the mattress for protection against scour.

While the above precautions are essential to a stable mattress, other measures can also help to ensure stability. They are:



Deadman and cable used to tie in top of mattress to bank. Piles, posts, or screw-type anchors are other types of materials that can be used as deadmen.

Figure 50. Used tire mattress (After [50]).

* Cut, drill or burn holes in the tire sidewalls to prevent flotation.
* Sort the tires by size to help in fitting them together.
* Fasten the mattress to the bank at intervals with earth screw anchors (or some other type of anchor).
* Pack the tires with stone or rubble.
* Plant willows or other fast growing, thick brush inside the tires. Once established, the root system will strengthen the bank and the willows will obscure the somewhat unsightly mattress and decrease flow velocities near the bank. If willows are not readily available, other species should be planted. Possible species for use are discussed under vegetation.

If the mattress effectively controls the streambank erosion and remains intact, sediment may gradually cover the revetment. If willows have not been planted, volunteer vegetation may become established.

Vegetation. Vegetation is the most natural method for protecting streambanks because it is relatively easy to establish and maintain and is visually attractive. However, vegetation should not be seriously considered as a countermeasure against severe bank erosion where a highway facility is at risk. At such locations, vegetation can best serve to supplement other countermeasures.

Vegetation can effectively protect a bank below the water line in two ways. First, the root system helps to hold the soil together and increases overall bank stability by forming a binding network. Second, the exposed stalks, stems, branches and foliage provide resistance to flow, causing the flow to lose energy by deforming the plants rather than by removing soil particles. Above the water line, vegetation prevents surface erosion by absorbing the impact of falling raindrops and reducing the velocity of overbank flow and rainfall runoff. Further, vegetation provides additional capacity for infiltration by taking water from the soil, and may improve bank stability by water withdrawal.

Vegetation is generally divided into two broad categories: grasses and woody plants (trees and shrubs). A major factor affecting species selection is the length of time required for the plant to become established on the slope. Grasses are less costly to plant on an eroding bank and require a shorter period of time to become established. Woody plants offer greater protection against erosion because of more extensive root systems; however, under some conditions the weight of the plant will offset the advantage of the root system. On high banks, tree root systems may not penetrate to the toe of the bank. If the toe becomes eroded, the weight of the tree and its root mass may cause a bank failure.

Water-tolerant grasses such as canarygrass (Phalaris), reedgrass (Calamagrostis), cordgrass (Spartina), and fescue (Festuca) are effective in preventing erosion on upper banks which are inundated from time to time and are subject to erosion due primarily to rainfall, overland flow, and minor wave action. Along the lower bank, where erosive forces are high, vegetation is generally not effective as a protective measure; however, cattails (Typha), bulrushes (Scripus), reeds (Phragmites), knotweed and smartweed (Plygonum), rushes (Juncus), and mannagrass (Glyceria) are helpful in inducing deposition and reducing velocities in shallow water or wet areas at the bank toe and in protecting the bank in some locations. Willows (Salix) are among the most effective woody plants in protecting low banks because they are resilient, are sufficiently dense to promote deposition of sediment, can withstand inundation, and easily become established.

Rigid Revetments

Rigid revetments include portland-cement paving, concrete filled mats, sand and cement bags, grouted riprap, and soil cement. Rigid revetments are generally smoother than flexible revetments and thus improve hydraulic efficiency and are generally highly resistant to erosion and impact damage. They are susceptible to damage from the removal of foundation support by subsidence, undermining, hydrostatic pressures, slides, and erosion at the perimeter. They are also among the most expensive streambank protection countermeasures.

Concrete Pavement. Concrete paving should be used only where the toe can be adequately protected from undermining and where hydrostatic pressures behind the paving will not cause failure. This might include impermeable bank materials and portions of banks which are continuously under water. Sections intermittently above water should be provided with weep holes. Refer to HEC 11 [38] for design of concrete pavement revetment.

Soil Cement. In areas where riprap is scarce, use of in-place soil combined with cement can sometimes provide a practical alternative. Figure 51 shows a detail of typical soil-cement construction for bank protection. For use in soil cement, soils should be easily pulverized and contain at least five percent, but not more than 35 percent, silt and clay (material passing the No. 200 sieve). Finer textured soils usually are difficult to pulverize and require more cement as do 100 percent granular soils which have no material passing the No. 200 sieve. Soil cement can be placed and compacted on slopes as steep as two horizontal to one vertical. Best results have been achieved on slopes no steeper than 3:1. However, in the arid Southwest a 1:1 slope is generally used for stair-stepped soil cement. Where velocities exceed 6 to 8 feet per second and the flow carries sufficient bed load to be abrasive, aggregates should contain at least 30 percent gravel particles retained on a No. 4 (4.75 mm) sieve.

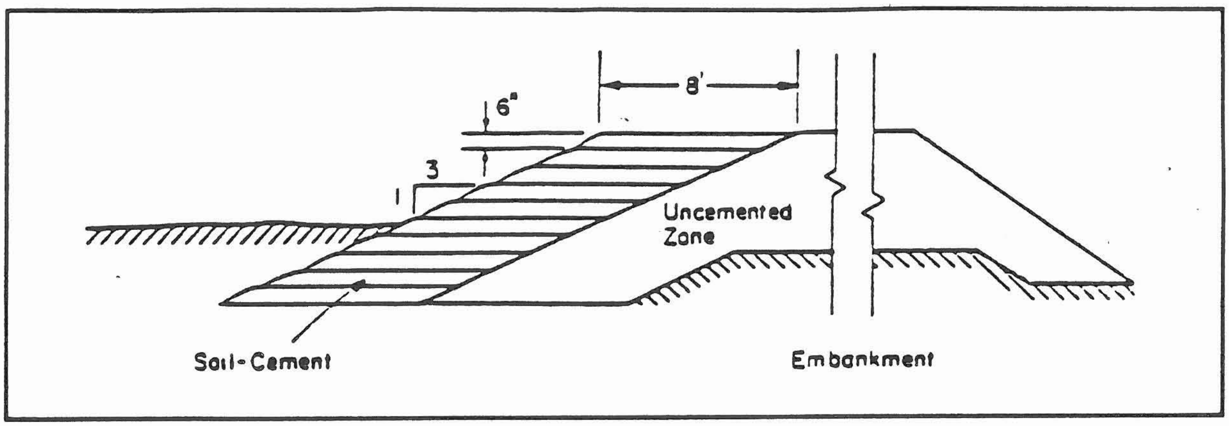


Figure 51. Typical soil-cement bank protection (After [4]).

A stair-step construction is recommended on channel banks with relatively steep slopes. Placement of small quantities of soil-cement for each layer (six inch layers) can progress more rapidly than a large quantity of fill material. Special care should be exercised to prevent raw soil seams between successive layers of soil cement. A sheepsfoot 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 seven 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 [51] and [52].

A soil-cement blanket with 8 to 15 percent cement may be an economical and effective streambank protection method for use in areas where vegetation is difficult to establish and the bank material is predominantly sand. The sand can be mixed with cement by hand or mechanically to a depth of at least 4 inches. The mixture should then be wet down and allowed to set up. This method has the advantage of low cost. However, a soil-cement blanket has three major deficiencies: impermeability, low strength, and susceptibility to temperature variations. If the bank behind the blanket becomes saturated and cannot drain, failure may occur. Also, because a sand-cement blanket is relatively brittle, very little if any vehicular, pedestrian, or livestock traffic can be sustained without cracking the thin protective veneer. In northern climates the blanket can break up during freeze-thaw cycles.

Precautions must be taken to prevent undermining at the toe and ends. 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 for relief of hydrostatic pressure are required for many situations.

Sacks. Burlap sacks filled with soil or sand-cement mixtures have long been used for emergency work along levees and streambanks during floods (Figure 52). Commercially manufactured sacks (burlap, paper, plastics, etc.) have been used to protect streambanks in areas where riprap of suitable size and quality is not available at a reasonable cost. Sacks filled with sand-cement mixtures can provide long-term protection if the mixture has set up properly, even though most types of sacks are easily damaged and will eventually deteriorate. Sand-cement sack revetment construction is not economically competitive in areas where good stone is available. However, where quality riprap must be transported over long distances, sack revetment can often be placed at a lesser cost than riprap.

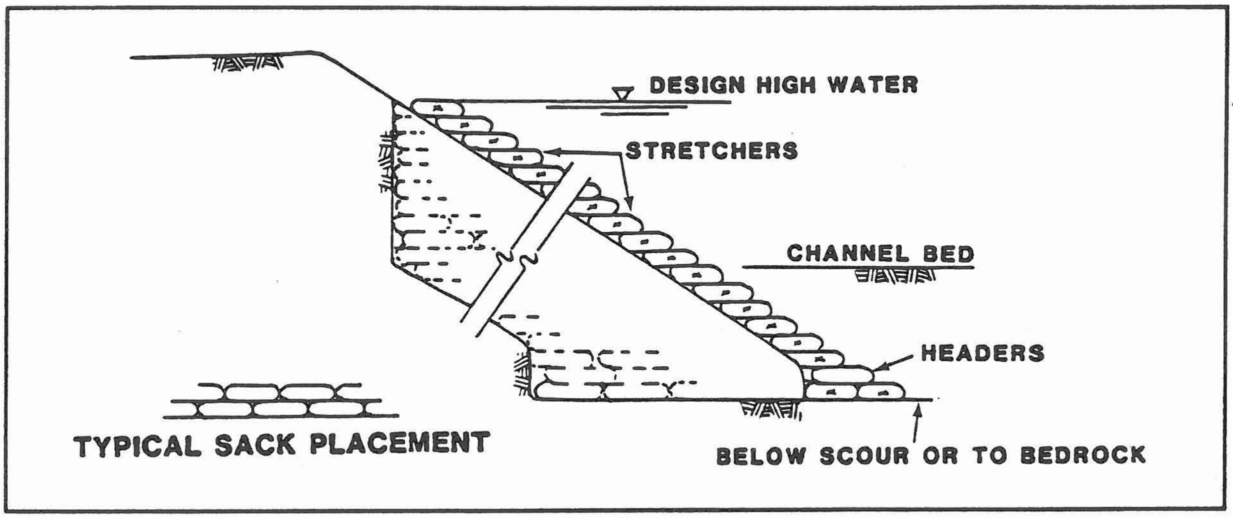


Figure 52. Typical sand-cement bag revetment (Modified from California Department of Public Works, 1970; After [37]).

If a permanent revetment is to be constructed, the sacks should be filled with a mixture of 15 percent cement (minimum) and 85 percent dry sand (by weight). The filled sacks should be placed in horizontal rows like common house brick beginning at an elevation below any toe scour (alternatively, riprap can be placed at the toe to prevent undermining of the bank slope). The successive rows should be stepped back approximately 1/2-bag width to a height on the bank above which no protection is needed. The slope of the completed revetment should not be steeper than 1:1. After the sacks have been placed on the bank, they can be wetted down for a quick set or the sand-cement mixture can be allowed to set up naturally through rainfall, seepage or condensation. If cement leaches through the sack material, a bond will form between the sacks and prevent free drainage. For this reason, weepholes should be included in the revetment design. The installation of weepholes will allow drainage of groundwater from behind the revetment thus helping to prevent pressure buildup that could cause revetment failure. This revetment requires the same types of toe protection as other types of rigid revetment.

Grouted Riprap. Grouted riprap consists of rock slope protection with voids filled with concrete grout to form a monolithic armor. It is generally used where rock of sufficient size, gradation, or quantity is not economically available to install riprap revetment. Grouted riprap is rigid but not extremely strong; therefore, support by the streambank is essential. Precautions against failure from hydrostatic pressure behind the riprap are appropriate, as well as provisions to prevent undermining at the toe of the bank and at the termini of the installation. Refer to HEC No. 11 for design guidance.[38]

Concrete-Filled Mats. Concrete filled mats consist of fabric envelopes pumped full with sand and cement grout. This product is marketed under the names "Fabriform," "Fabricast," and "Enkamat" and are protected under various U. S. and foreign patents or patents pending.

Concrete-filled mats have not performed well in high-velocity environments. More experience with the use of these measures is advisable before they are used at high-risk locations.

6.6.2 Hardpoints

Hardpoints consist of stone fills spaced along an eroding bank line, protruding only short distances into the channel. A root section extends landward to preclude flanking. The crown elevation of hardpoints used by the U. S. Army Corps of Engineers at demonstration sites on the Missouri River was generally at the normal water surface elevation at the toe, sloping up at a rate of about 1 foot in 10 feet toward the bank. Hardpoints are most effective along straight or relatively flat convex banks where the streamlines are parallel to the bank lines and velocities are not greater than 10 ft/s within 50 feet of the bank line. Hardpoints may be appropriate for use in long, straight reaches where bank erosion occurs mainly from a wandering thalweg at lower flow rates. They would not be effective in halting or reversing bank erosion in a meander bend unless they were closely spaced, in which case spurs, retarder structures, or bank revetment would probably cost less. Figure 53 is a perspective of a hardpoint installation.

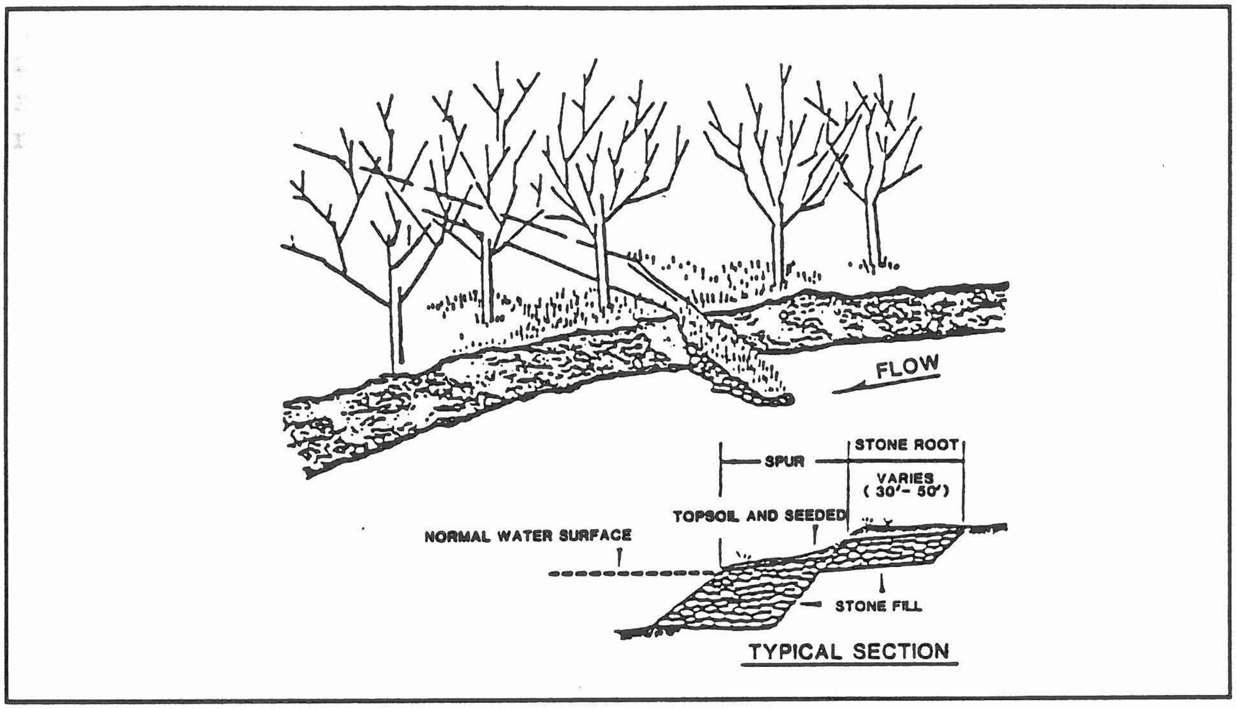


Figure 53. Perspective view of hardpoint installation with section detail (After [37]).

6.6.3 Retarder Structures

Retarder structures are permeable devices generally placed parallel to streambanks to reduce velocities and cause deposition near the bank. They are best suited for protecting low banks or the lower portions of streambanks. Retarder structures can be used to protect an existing bank line or to establish a different flow path or alignment. Retards do not require grading of the streambank, and they create an environment which is favorable to the establishment of vegetation.

Jacks and Tetrahedrons

Jacks most commonly consist of three linear members fixed together at their midpoints so that each member is perpendicular to the other two. Wires are strung on the members to resist distortion and to collect debris. Cables are used to tie individual jacks together and for anchoring key units to deadmen. Tetrahedrons consist of six members of equal length fixed together so as to form three faces, each of which is an equilateral triangle, i.e, a tetrahedron. The tetrahedron unit may be braced as shown in Figure 54 and wire mesh added to enhance flow retardance. Tetrahedrons are not as widely used as are jacks.

Jacks and tetrahedrons are effective in protecting banks from erosion only if light debris collects on the structures thereby enhancing their performance in retarding flow. However, heavy debris and ice can damage the structures severely. They are most effective on mild bends and in wide, shallow streams which carry a large sediment load.

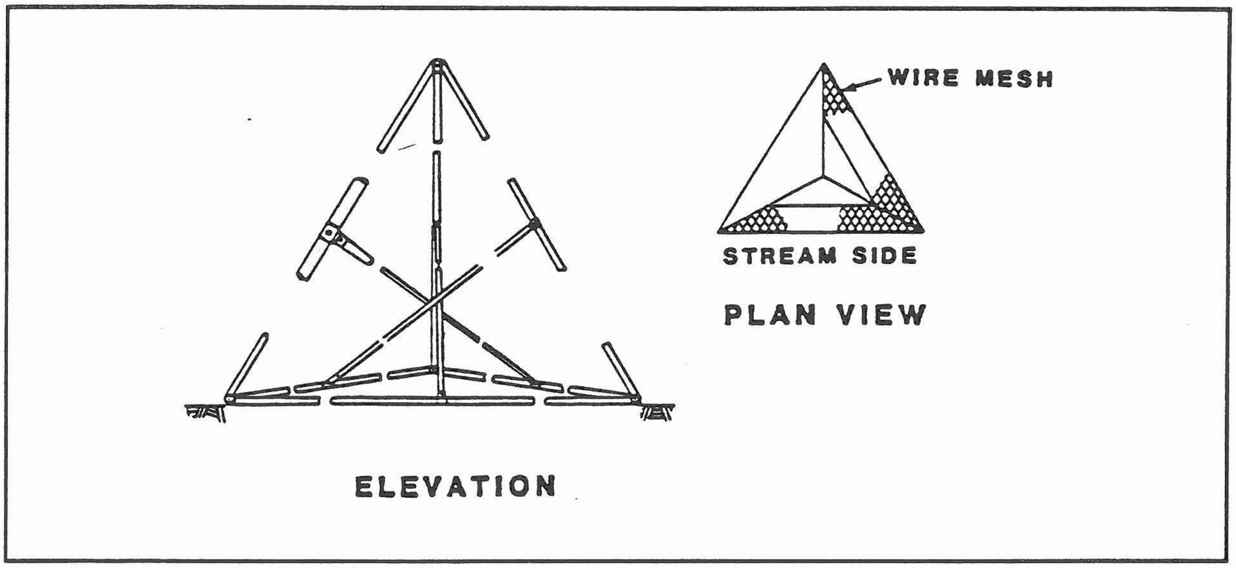


Figure 54. Typical tetrahedron design (After [37]).

Where jacks are used to stabilize meandering streams, both lateral and longitudinal rows are often installed to form an area retarder structure rather than a linear structure. Lateral rows of jacks are usually oriented in a downstream direction from 45 to 70 degrees. Spacing of the lateral rows of jacks may be 50 to 250 feet depending on the debris and sediment load carried by the stream. A typical jack unit is shown in Figure 55 and a typical area installation is shown in Figure 56.

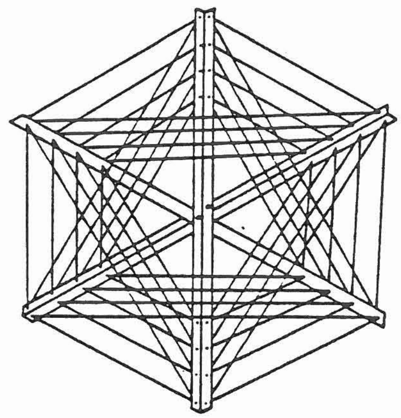


Figure 55. Typical jack unit (After [37]).

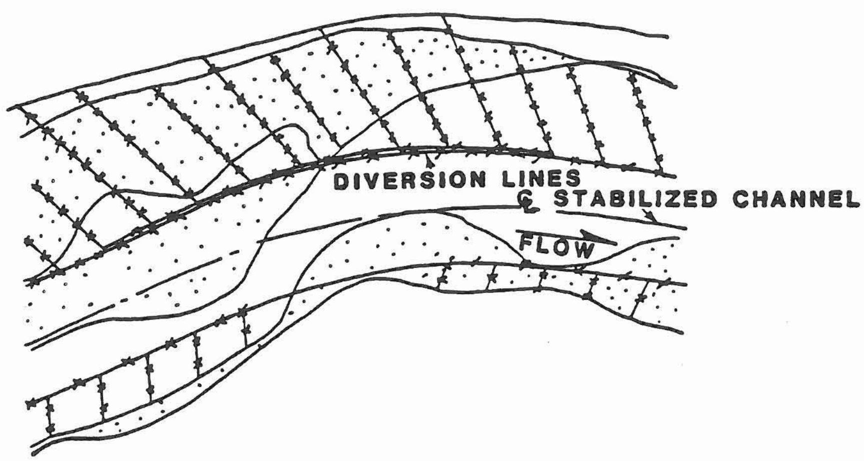


Figure 56. Retarder field schematic (After [4]).

Outflanking of jack installations is a common problem. Adequate transitions should be provided between the upstream bank and the structure, and the jack field should be extended to the overbank area to retard flow velocities and provide additional anchorage. Jacks are not recommended for use in corrosive environments or at locations where they would constitute a hazard to recreational use of the stream.

Fence Retarder Structures

Fence retarder structures provide protection to the lower portions of banks of relatively small streams. Posts may be of wood, steel, or concrete and fencing may be composed of wood planks or wire.

Scour and the development of flow channels behind linear structures are common causes of failure of longitudinal fences. Scour at the supporting members of the structure can be reduced by placing rock along the fence or the effects of scour can be overcome by driving supporting members to depths below expected scour. Tiebacks can be used to retard velocities between the linear structure and the streambank, thus reducing the ability of the stream to develop flow channels behind the structure.

Timber Pile. Timber pile retarder structures may be of a single, double, or triple row of piles with the outside of the upstream row faced with wire mesh or other fencing material. They have been found to be effective at sharp bends in the channel and where flows are directly attacking a bank. They are effective in streams which carry heavy debris and ice loads and where barges or other shipping vessels could damage other countermeasures or a bridge. As with other retarder structures, protection against scour failure is essential. Figure 57 illustrates a design.

Wood Fence. Wood fence retarder structures have been found to provide a more positive action in maintaining an existing flow alignment and to be more effective in preventing lateral erosion at sharp bends than other retarder structures. Figure 58 is an end view of a typical wood fence design with rock provided to protect against scour.

Wire fence retarder structures may be of linear or area configuration, and linear configurations may be of single or multiple fence rows. Double-row fence retards are sometimes filled with brush to increase the flow retardance. Figures 59 and 60 illustrate two types of wire fence retarder structures.

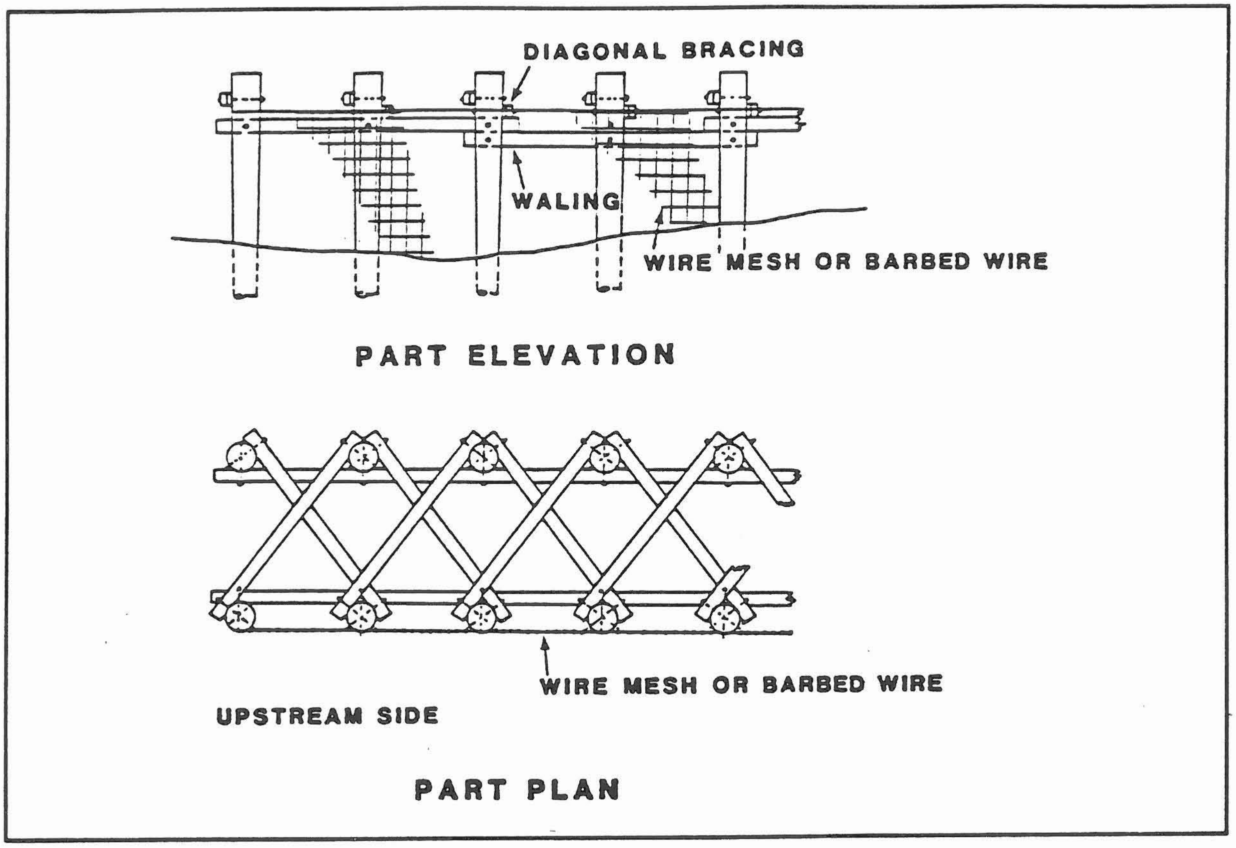


Figure 57. Timber pile bent retarder structure (Modified from California Department of Public Works, 1970; After [37]).

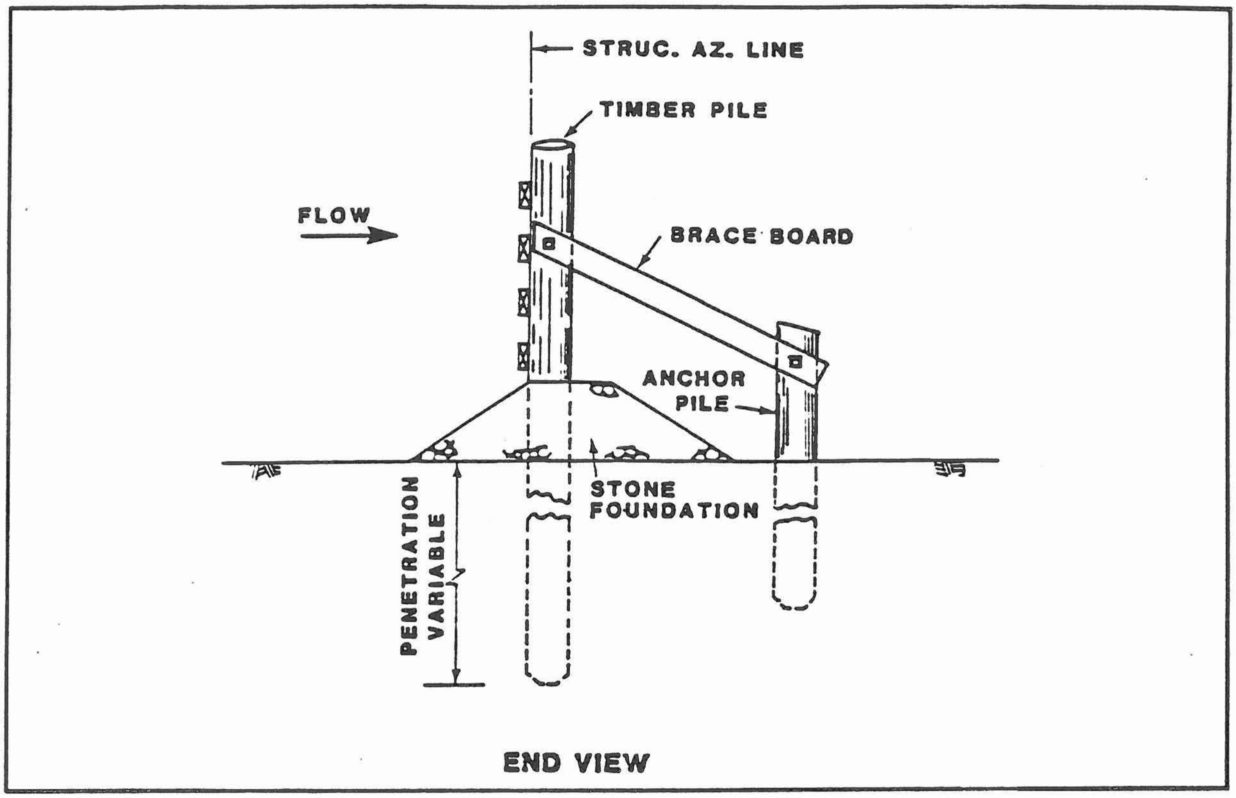


Figure 58. Typical wood fence retarder structure (Modified from U. S. Army Corps of Engineers, 1981; After [37]).

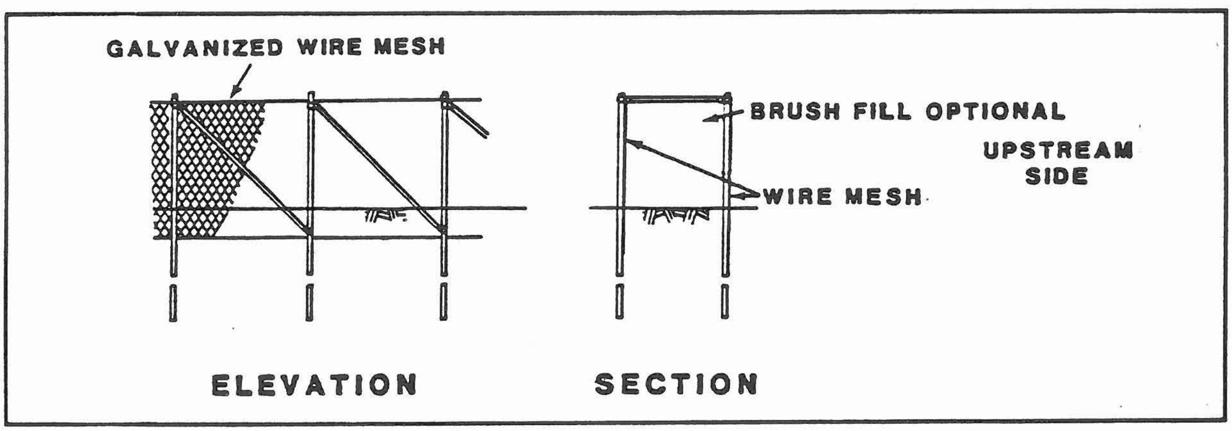


Figure 59. Light double row wire fence retarder structure (After [37]).

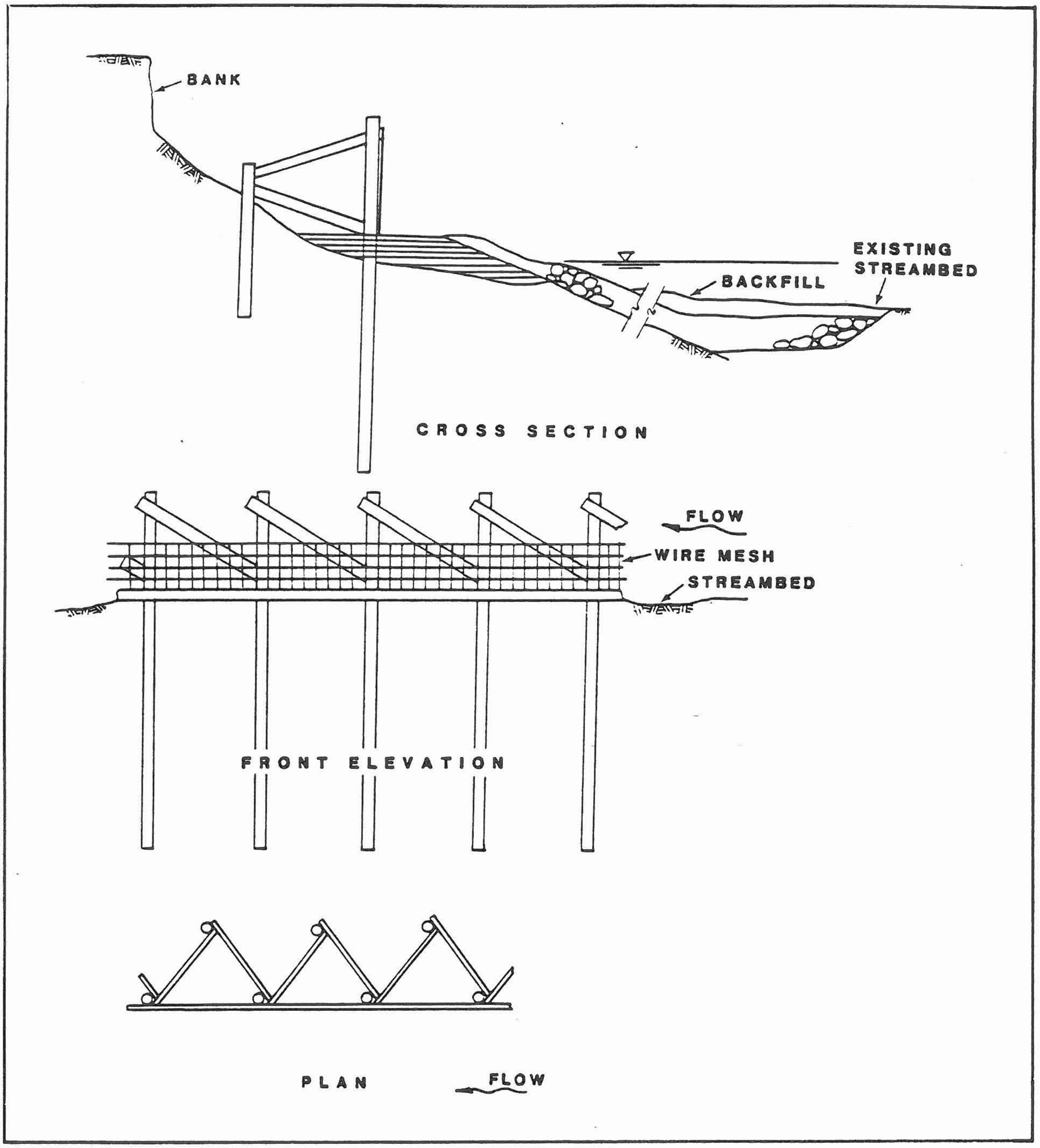


Figure 60. Heavy timber-pile and wire fence retarder structures (Modified from U. S. Army Corps of Engineers 1981; After [37]).

6.6.4 Longitudinal Dikes

Longitudinal dikes are essentially impermeable linear structures constructed parallel with the streambank or along the desired flow path. They protect the streambank in a bend by moving the flow current away from the bank. Longitudinal dikes may be classified as earth or rock embankment dikes, crib dikes, or rock toe-dikes.

Earth or Rock Embankments

As the name implies, these dikes are constructed of earth with rock revetment or of rock. They are usually as high or higher than the original bank. Because of their size and cost, they are useful only for large-scale channel realignment projects.

Rock Toe-Dikes

Rock toe-dikes are low structures of rock riprap placed along the toe of a channel bank. They are useful where erosion of the toe of the channel bank is the primary cause of the loss of bank material. The Corps of Engineers has found that longitudinal stone dikes provide the most successful bank stabilization measure studied for channels which are actively degrading and for those having very dynamic beds. Where protection of higher portions of the channel bank is necessary, rock toe-dikes have been used in combination with other measures such as vegetative cover and retarder structures.

Figure 61 shows the typical placement and sections of rock toe-dikes. The volume of material required is 1-1/2 to 2 times the volume of material that would be required to armor the sides of the anticipated scour to a thickness of 1-1/2 times the diameter of the largest stone specified. Rock sizes should be similar to those specified for riprap revetments. Tiebacks are often used with rock toe-dikes to prevent flanking, as illustrated in Figure 62. Tiebacks should be used if the toe-dike is not constructed at the toe of the channel bank.

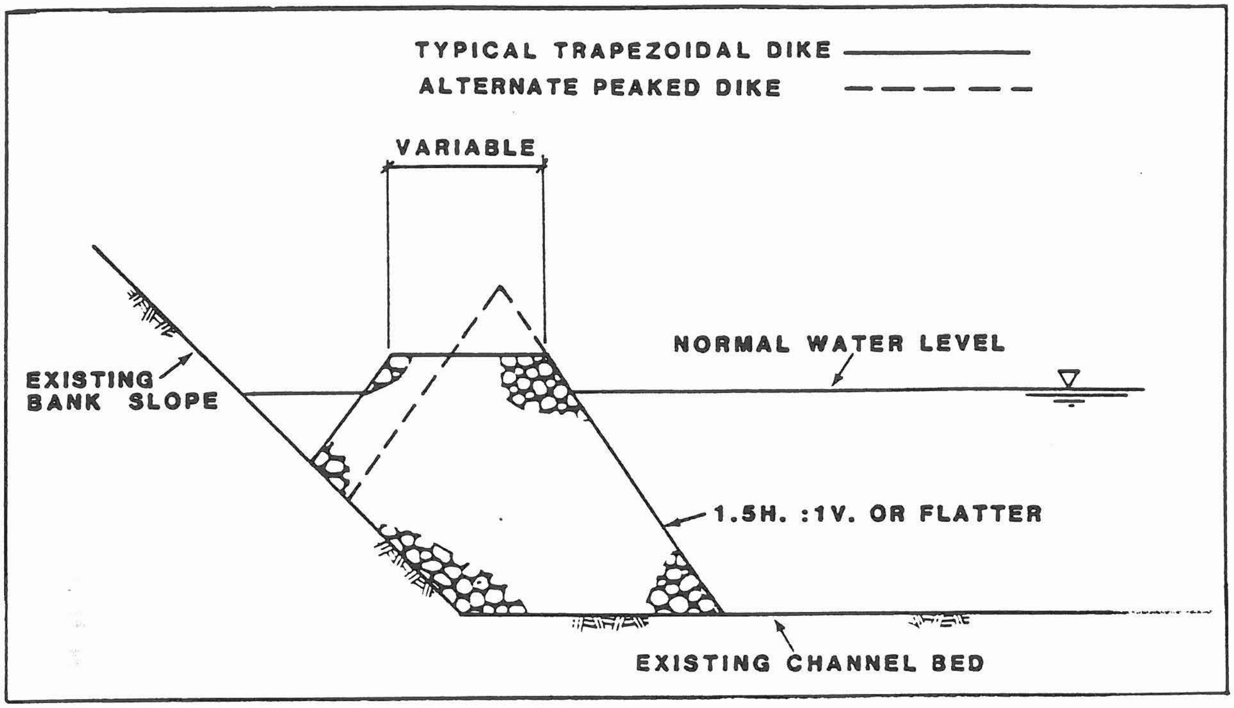


Figure 61. Typical longitudinal rock toe-dike geometries (After [37]).

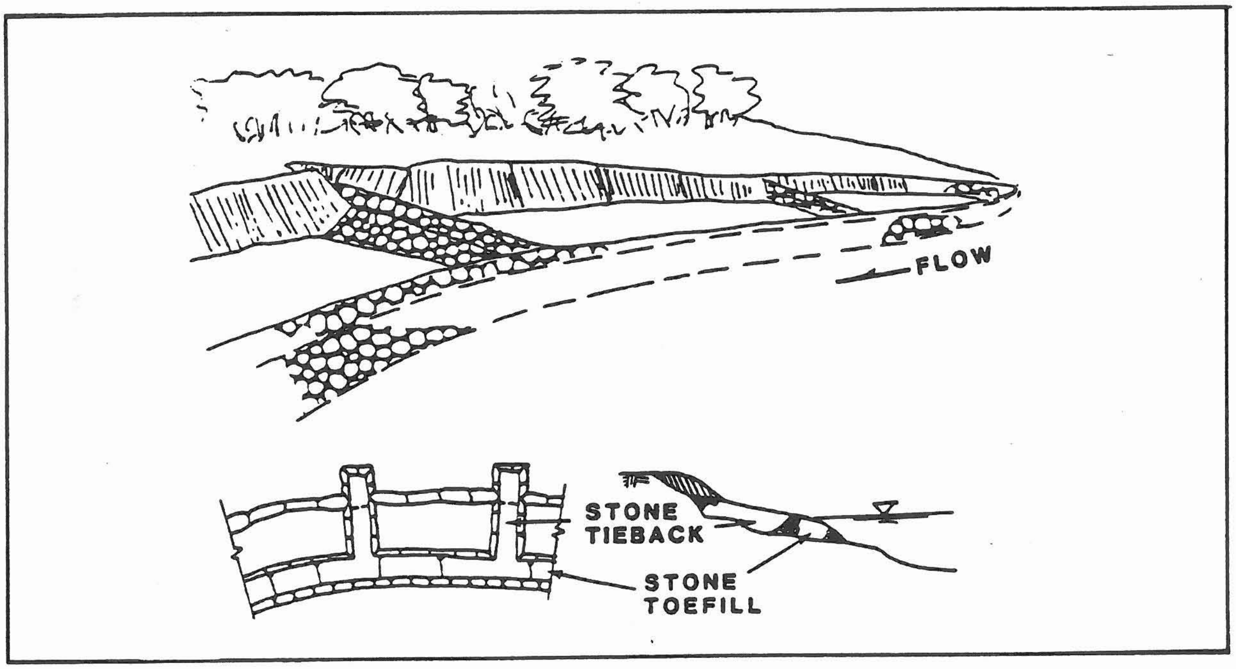


Figure 62. Longitudinal rock toe-dike tiebacks (After [37]).

Rock toe-dikes are useful on channels where it is necessary to maintain as wide a conveyance channel as possible. Where this is not important, spurs could be more economical since scour is a problem only at the end projected into the channel. However, spurs may not be a viable alternative in actively degrading streams.

Crib Dikes

Longitudinal crib dikes consist of a linear crib structure filled with rock, straw, brush, automobile tires or other materials. They are usually used to protect low banks or the lower portions of high banks. At sharp bends, high banks would need additional protection against erosion and outflanking of the crib dike. Tiebacks can be used to counter outflanking.

Crib dikes are susceptible to undermining, causing loss of material inside the crib, thereby reducing the effectiveness of the dike in retarding flow. Figure 63 illustrates a crib dike with tiebacks and a rock toe on the stream side to prevent undermining.

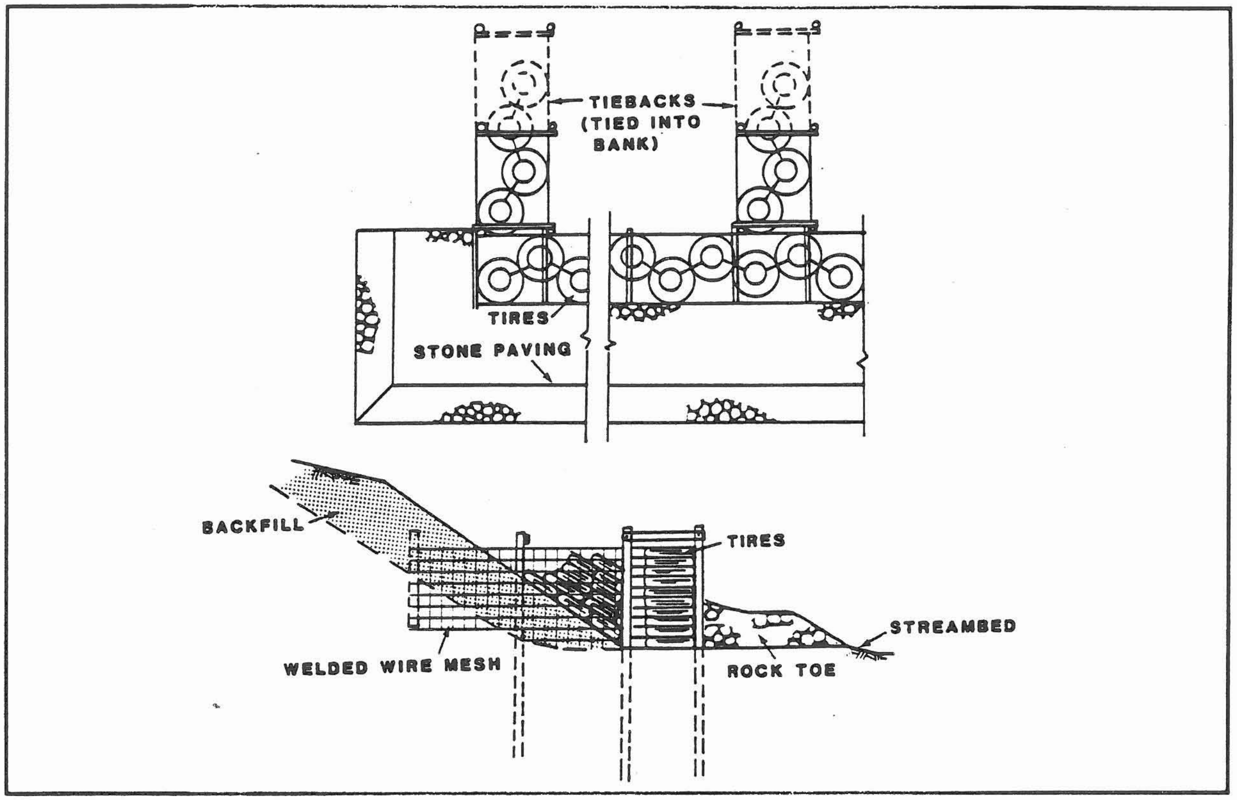


Figure 63. Timber pile, wire mesh crib dike with tiebacks (Modified from U. S. Army Corps of Engineers, 1981; After [37]).

6.6.5 Vane Dikes

Vane dikes are a series of low elevation structures designed to guide flow away from an eroding bank line. The structures can be constructed of rock or other erosion resistant material. The crests are below the design water elevation and flow can pass over or around the structures, with the main thread of flow directed away from the eroding bank.

Figure 64 is a layout of an "Iowa Vane" system installed at a bend in the East Nishnabota River at U. S. Highway 34 in Iowa. The system functions by eliminating or reducing secondary currents which dive at the concave bank and cause bank erosion. Prior to installation of the vane system, the river bend was moving toward U. S. 34 at a rate of 20 to 30 feet per year. Developers of the system who designed the prototype installation are confident that the system would halt the bend migration with relatively minor modifications, although the bank did recede about four feet during a four month period in which a moderately large flood occurred.[50] According to the above report, the effectiveness of the system was reduced, to some extent, by changing stream conditions. This needs further study since conditions continually change in alluvial streams. Figure 65 presents a perspective view of a typical vane dike layout.

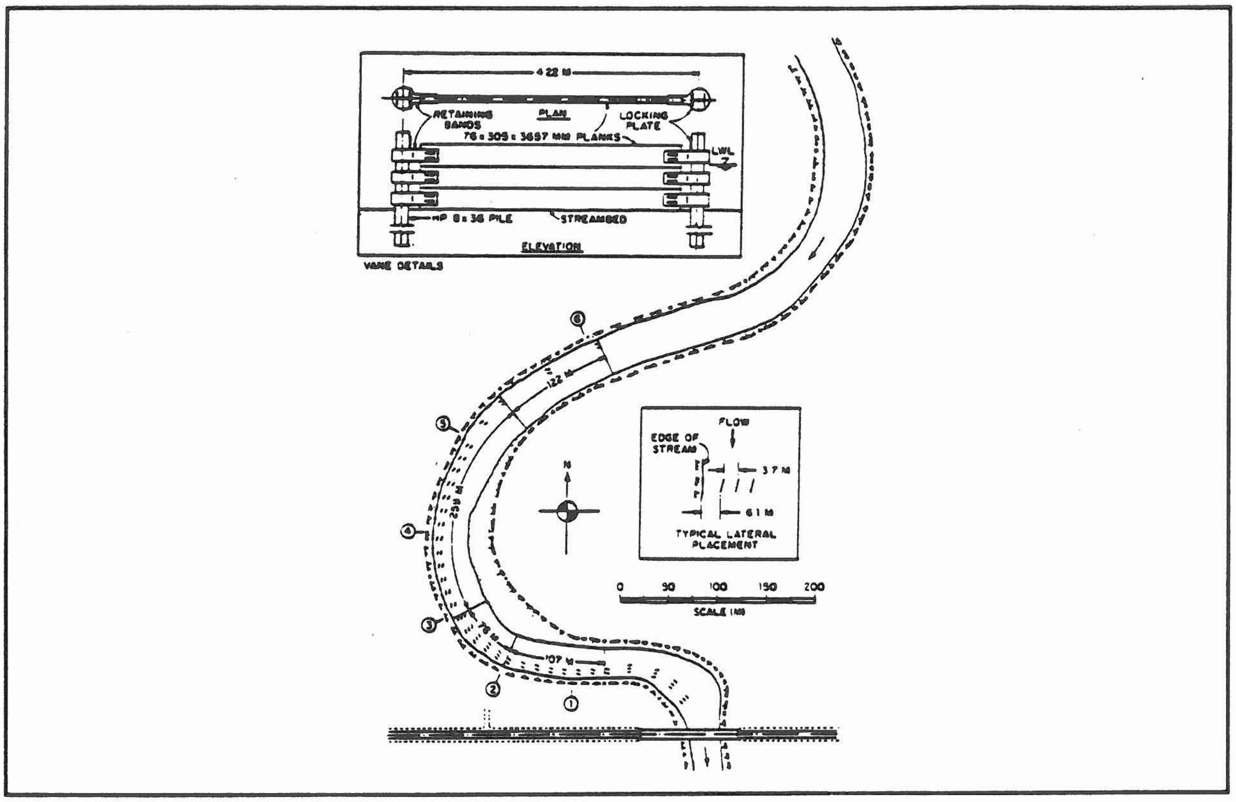


Figure 64. Layout of Iowa Vane system in East Nishnabota River bend (After [53]).

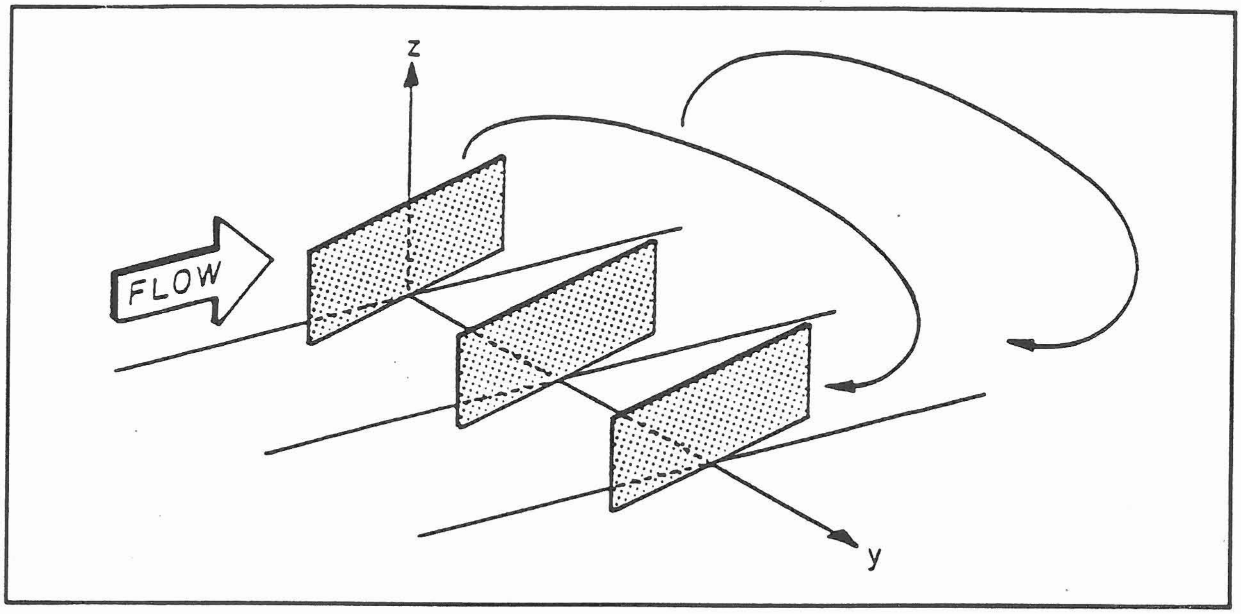


Figure 65. Perspective of an Iowa Vane layout.[54]

6.6.6 Bulkheads

Bulkheads are used for purposes of supporting the channel bank and protecting it from erosion. They are generally used as protection for the lower bank and toe, often in combination with other countermeasures that provide protection for higher portions of the bank. Bulkheads are most frequently used at bridge abutments as protection against slumping and undermining at locations where there is insufficient space for the use of other types of bank stabilization measures, and where saturated fill slopes or channel banks cannot otherwise be stabilized.

Bulkheads are classified on the basis of construction methods and materials. They may be constructed of concrete, masonry, cribs, sheet metal, piling, reinforced earth, used tires, gabions, or other materials. They must be protected against scour or supported at elevations below anticipated scour, and where sections of the installation are intermittently above water, provisions must be made for seepage through the wall. Some bulkhead types, such as crib walls and gabions, should be provided with safeguards against leaching of materials from behind the wall.

Bulkheads must be designed to resist the forces of overturning, bending and sliding, either by their mass or by structural design. Figure 66 illustrates anchorage schemes for a sheetpile bulkhead. Because of costs, they should be used as countermeasures against meander migration only where space is not available to construct other types of measures.

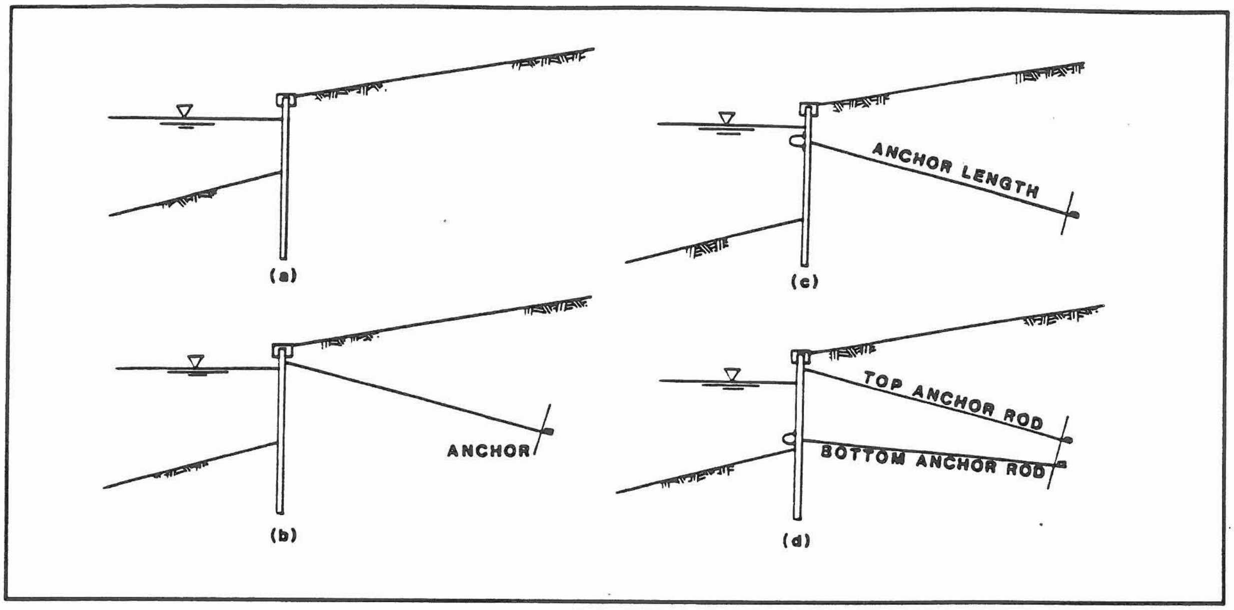


Figure 66. Anchorage schemes for a sheetpile bulkhead (After [37]).

6.6.7 Channel Relocation

At some locations, it may be advantageous to realign a stream channel, either in combination with the use of other countermeasures against meander migration or in lieu of other countermeasures.

Figure 67 illustrates hypothetical highway locations fixed by considerations other than stream stability. To create better flow alignment with the bridge, consideration could be given to channel realignment as shown in this figure (parts a and b). Similarly, consideration for realignment of the channel would also be advisable for a hypothetical lateral encroachment of a highway as depicted in part c of the figure. In either case, criteria are needed to establish the cross sectional dimensions.

Prior to realigning a stream channel, the stability of the existing channel must be examined. The stream classification, recent and older aerial photographs, and field surveys are necessary. The realigned channel may be made straight without curves, or may include one or more curves. If curves are included, decisions regarding the radius of curvature, the number of bends, the limits of realignment (hence the length and slope of the channel) and the cross sectional area have to be made. Different streams have different historical backgrounds and characteristics with regard to bend migration, discharge, stage, geometry, and sediment transport, and an understanding and appreciation of river hydraulics and morphology

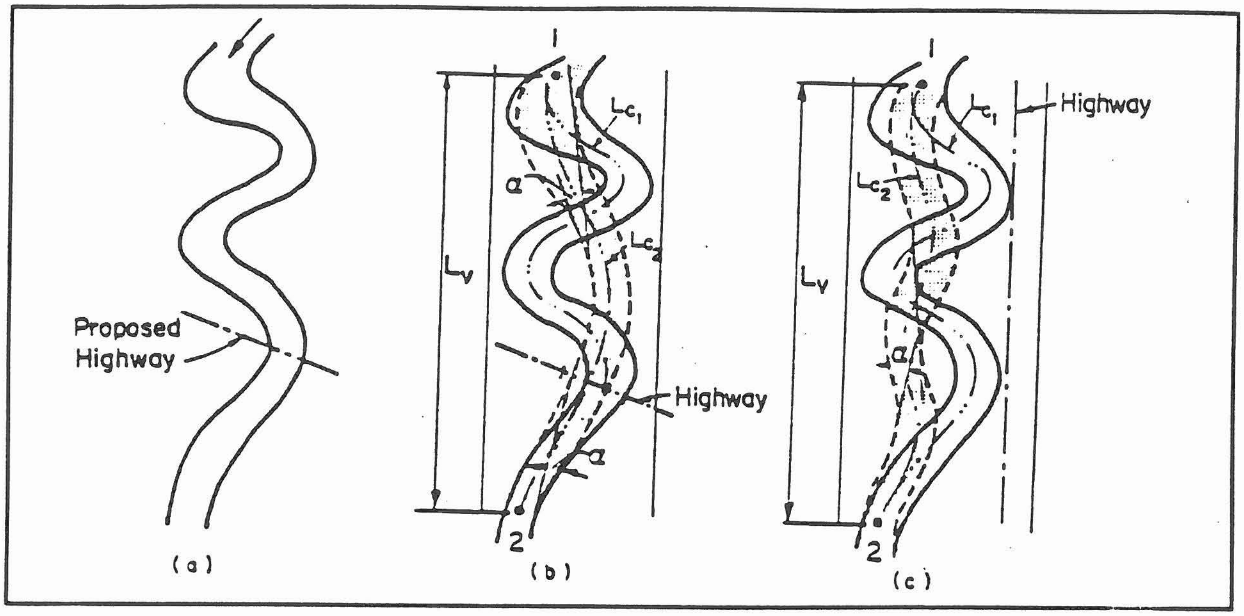


Figure 67. Encroachments on meandering streams (After [4]).

is important to decision making. It is difficult to state generalized criteria for channel relocation applicable to all streams. Knowledge about stream systems has not yet advanced to such a state as to make this possible. Nevertheless, some principles and guidelines can be provided. (See Chapters 2.0 through 4.0).

As the general rule, bend radii in realigned channels should be made about equal to the mean radius of bends, r_c, in extended reaches of the stream. Where the angle defined in Figure 68 exceeds about 40 degrees, there is a sufficient crossing length for the thalweg to shift from one side of the channel to the other. Generally, it may be necessary to stabilize the outside banks of curves in order to hold the new alignment and, depending upon crossing length, some maintenance may be necessary to remove sandbars after large floods so that the channel does not develop new meander patterns.

Sinuosity and channel bed slope are related in that the total drop in bed elevation for the old channel and the relocated channel are the same. Thus, the mean slope of the channel bed after relocation (subscript 2) is greater than the mean slope of the original channel bed (subscript 1), ($S_2 > S_1$). If the larger slope, S_2, will not satisfy the equation, ($SQ^{1/4} \leq 0.0010$), the possibility of the stream changing to a braided channel because of the steeper slope should be carefully evaluated (see Section 2.3). With the steeper slope, there could be an increase in sediment transport which could cause degradation, and the effect would be extended both upstream and downstream of the relocated reach. Also, meander patterns could change. Considerable bank protection might be necessary to contain lateral migration which is characteristic of a braided channel, and if the slope is sufficiently steep, head cuts could develop

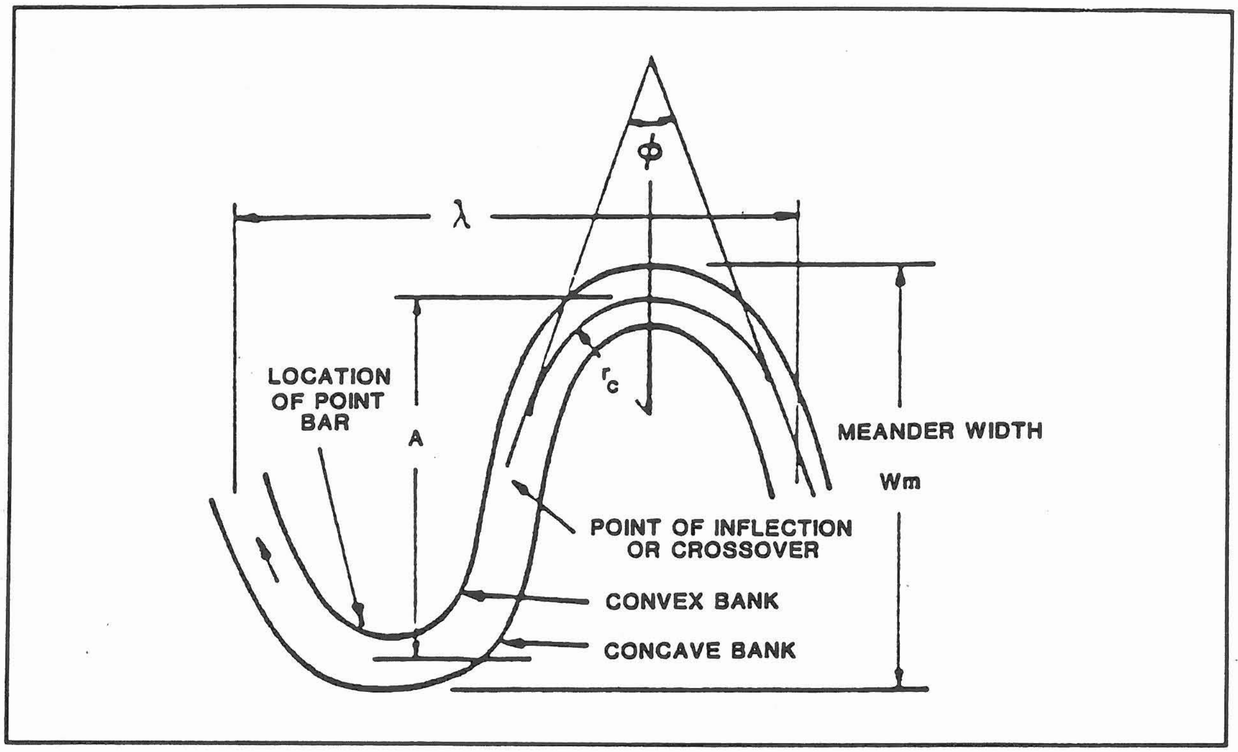


Figure 68. Definition sketch for meanders (After [4]).

which migrate upstream with attendant effects on the plan geometry of the channel. Even where changes in slope are not very large, a short-term adjustment of the average stream slope occurs beyond the upstream and downstream limits of channel realignment, consistent with the sediment transport rate, flow velocities and roughness.

Based on a study of the stability of relocated channels, Brice presented the following recommendations and conclusions regarding specific aspects of planning and construction of channel realignment:[55]

* <u>Channel Stability Prior to Realignment</u>. Assessment of the stability of a channel prior to realignment is needed to assess the risk of instability. An unstable channel is likely to respond unfavorably. Bank stability is assessed by field study and by stereoscopic examination of aerial photographs. The most useful indicators of bank instability are cut or slumped banks, fallen trees along the bank line, and exposed wide point bars. Bank recession rates are measured by comparison of time-sequential aerial photographs. Vertical instability is equally important but more difficult to determine. It is indicated by changes in channel elevation at bridges and gaging stations. Serious degradation is usually accompanied by generally cut or slumped banks along a channel and by increased debris transport.

* Erosional Resistance of Channel Boundary Materials. The stability of a channel, whether natural or relocated, is partly determined by the erosional resistance of materials that form the wetted perimeter of the channel. Resistant bedrock outcrops in the channel bottom or that lie at shallow depths will provide protection against degradation, but not all bedrock is resistant. Erosion of shale, or of other sedimentary rock types interbedded with shale, has been observed. Degradation is not a problem at most sites where bed sediment is of cobble and boulder size. However, degradation may result from the relocation of any alluvial channel, whatever the size of bed material, but the incidence of serious degradation of channels relocated by highway agencies is small in number. The erosional resistance of channel beds tends to increase with clay content. Banks of weakly cohesive sand or silt are clearly subject to rapid erosion, unless protected with vegetation. No consistent relation has been found between channel stability and the cohesion of bank materials, probably because of the effects of vegetation.

* Length of Realignment. The length of realignment contributes significantly to channel instability at sites where its value exceeds 250 channel widths. When the value is below 100 channel widths, the effects of length of relocation are dominated by other factors. The probability of local bank erosion at some point along a channel increases with the length of the channel. The importance of vegetation, both in appearance and in erosion control, would seem to justify a serious and possibly sustained effort to establish it as soon as possible on graded banks.

* Bank Revetment. Revetment makes a critical contribution to stability at many sites where it is placed at bends and along roadway embankments. Rock riprap is by far the most commonly used and effective revetment. Concrete slope paving is prone to failure. Articulated concrete block is effective where vegetation can establish in the interstices between blocks.

* Check Dams (drop structures). In general, check dams are effective in preventing channel degradation. The potential for erosion at a check dam depends on its design and construction, its height and the use of revetment on adjoining banks. A series of low check dams, less than about 1.5 feet in height, is probably preferable to a single higher structure, because the potential for erosion and failure is reduced (see Section 6.5). By simulating rapids, low check dams may add visual interest to the flow in a channel. One critical problem arising with check dams relates to improper design for large flows. Higher flows have worked around the ends of may installations to produce failure.

* Maintenance. Problems which could be resolved by routine maintenance were observed along relocated channels. These were problems with the growth of annual vegetation, reduction of channel conveyance by overhanging trees, local bank cutting, and bank

slumping. The expense of routine maintenance or inspection of relocated channels beyond the highway right-of-way may be prohibitive; however, most of the serious problems could be detected by periodic inspection, perhaps by aerial photography, during the first 5 to 10 years after construction. Hydraulic engineers responsible for the design of relocated stream channels should monitor their performance to gain experience and expertise.

6.6.8 Scour at Bridges

For a discussion of selection of countermeasures for scour at bridges, see Section 5.4. For information on countermeasures for stream instability because of scour at bridges, see [23,24,26].

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HEC-20 - APPENDIX A

ILLUSTRATIVE EXAMPLE - LEVEL 1 AND LEVEL 2 ANALYSES

INTRODUCTION

This example is taken from HIRE [4] and was chosen to be incorporated into this text to illustrate the three level approach to river stability problems. The example has been edited to correspond with the format of this text.

The Rillito River System in Tucson, Arizona provides an example of the problems encountered in bridge crossing design. The objective of this example is to illustrate to the designer the methodologies used in a stream stability analysis in support of bridge crossing design.

Two bridge sites are considered which provide insight into several problems characteristic of the Rillito system. These are the Sabino Canyon Road site with an existing bridge crossing (constructed 1936) and the Craycroft Road site with a dip crossing (where the roadway is at the same elevation as the channel bed). Design improvements are being considered in the Sabino Canyon bridge, and it is proposed that the Craycroft dip crossing be replaced with a bridge. The study reach of Rillito River included approximately 11.5 miles of channel extending from Dodge Boulevard to Agua Caliente Wash (see Fig. 69). This included two miles on the Rillito River, six and one-half miles upstream of Craycroft Road on the Tanque Verde Creek, two miles upstream of Craycroft Road on Pantano Wash, and one mile on Sabino Creek upstream of the confluence with Tanque Verde Creek.

LEVEL 1 ANALYSIS

Level 1 - Step 1: Stream Characteristics

The history of flood events and the recent geomorphology of the Rillito system has shown that it is very dynamic and illustrates the characteristics of a braided river. The channel is steep, dropping at the rate of 21 feet per mile. The bed material is predominantly in the medium to coarse sand sizes. The natural sinuosity of the river is low. Additionally, the river is generally unstable, changes alignment rapidly, carries large quantities of sediment, and is difficult to predict.

Level 1 - Step 2: Land Use Changes

A large portion of the river system is in the metropolitan area of Tucson where man's activities in, and adjacent to, the river environment have induced a number of changes in the system. Primary impacts on the system have occurred due to encroachment by urban development and channelization of segments of the river. Uncontrolled sand and gravel extraction has also led to even more rapid and significant changes in the river system. A secondary effect of urbanization is a reduction in sediment supply and an increase in water runoff from tributaries draining urban areas. Undeveloped land in the study area generally has little protective cover and supplies large quantities of sediment to the river system. However, extensive erosion control measures established for urban development and the creation of impermeable areas in these tributaries has reduced the sediment supply to the river system and increased runoff. As urbanization continues there will be a long-term decrease in sediment supply and an increase in runoff that will have a significant influence on the geomorphology of the river system.

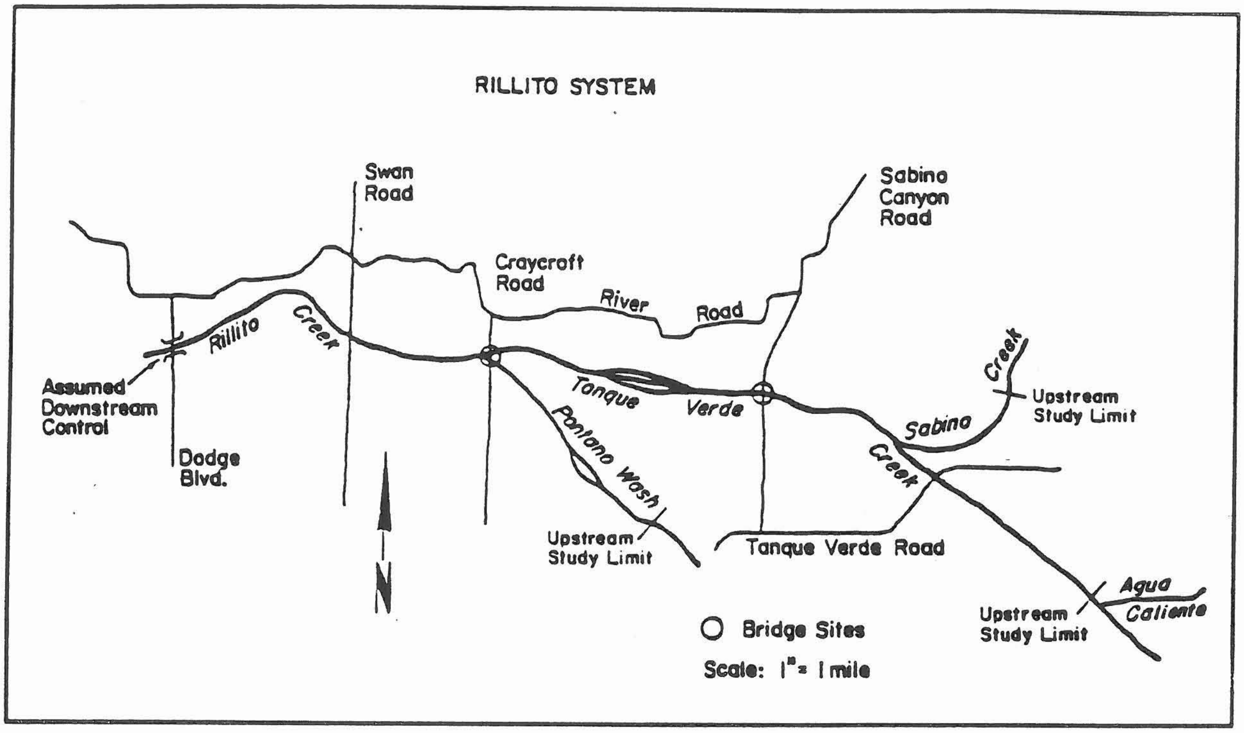

Fig. 69. Rillito system vicinity map.

Level 1 - Step 3: Overall Stability

Much of the system has been significantly disturbed by human activities. Observed activities include channelization, sand and gravel mining, construction of bridges, construction of grade controls, road crossings, and encroachment by urbanization. Much of the system's shape and form, then, is dictated by man's activities rather than natural processes. This is especially true for the portions of the Rillito River and Pantano Wash within the study area. The Rillito River has been subjected to major channelization upstream and downstream of Swan Road. There is also a large instream gravel pit below Swan Road. The Pantano Wash system has a large instream gravel pit below, and bank stabilization works in the vicinity of Tanque Verde Boulevard. The results of these activities have been to change these systems from their natural braided forms to defined channels. Pantano Wash still possesses a stretch of over 3,000 feet which is braided. Tanque Verde Creek, however, has experienced less impact from man's activities than the other two systems. The islands, bends, and natural channel alignment observed in the 1941 aerial photographs of Tanque Verde Wash are still intact.

Level 1 - Step 4: Lateral Stability

In order to understand the lateral migration processes more thoroughly, 13 miles of Tanque Verde Wash above the confluence with Pantano Wash were studied closely to determine the range of channel plan geometry. Meander amplitudes, wave lengths, and radii of curvature were measured.

Meander wave length, channel width and radius of curvature (L, B, and r_c) pairs were plotted for each meander loop. These points were plotted on logarithmic scales and on linear scales. Straight lines were fitted and the following equations were obtained and adopted for the lower 13 miles of the Tanque Verde Creek:

$$L = 2.6 r_c$$

$$L = 2.75 B^{1.35}$$

$$r_c = 1.06 B^{1.35}$$

These relationships are used to determine appropriate channel widths and bend shapes at the Sabino Canyon Road Bridge site.

Sabino Canyon Road Bridge is currently located on a bend with a curvature that creates several problems. At low flows, scour occurs at the outside of the bend (south side) because of high velocity and secondary currents. This phenomena is evident in the present channel cross section under the bridge. At flood flows, the problem is nearly the opposite. The north side of the bridge is attacked because of the tendency of the thalweg to straighten out the bend. The amplitude of the meander bend is 300 feet. Therefore, the lateral migration tendency is on the order of 300 feet.

These facts point to the necessity for engineering control measures to be taken at Sabino Canyon Road Bridge in order to prevent future failure of the structure from lateral migration.

Level 1 - Step 5: Vertical Stability

Four bridge sites exist in the study area at Dodge Boulevard, Swan Road, Sabino Canyon Road, and Tanque Verde Road. Measures have been taken at Craycroft Crossing to stabilize the crossing during the low flows and as a result the crossing is acting as a grade control on the Pantano Wash. Stabilization measures have not been successful on the north side of Craycroft and no grade control has formed. Complex hydraulic conditions exist at the confluence of Tanque Verde Creek and Pantano Wash during the 100-year flood. Divided flow occurs with flood water spilling laterally into Pantano Wash from Tanque Verde Creek during a flood from that watershed, or flood water spills to Tanque Verde Creek from Pantano Wash during a flood from the Pantano watershed.

The bridges across the Rillito River and Tanque Verde Creek span a variety of channel conditions. The sedimentation and erosion processes due to the proposed bridges will depend on the extent to which the bridge influences the hydraulic conditions in the river (primarily the velocity and depth). Conversely, the changing form of the channel due to lateral migration or long-term changes in the channel profile can alter the hydraulic conditions at the bridge. The Dodge Boulevard bridge is assumed to be the downstream control for the study reach.

This assumption is valid because the bridge crosses a channelized section of the Rillito River which has little influence on the water surface elevation in the channel for the 100-year flood. The downstream boundary hydraulic condition is assumed to be uniform flow.

Level 1 - Step 6: Stream Response

A braided river can be identified by the equation

$$\boxed{SQ^{1/4} \geq 0.01}$$

in which S is the average bed slope and Q is the dominant discharge (cfs). The mean annual flood of 5,000 cfs is assumed to represent dominant conditions in the system. The average slope of 13.1 miles of the Tanque Verde Creek and Rillito River is 0.0044. This slope and discharge give a value of 0.037, which is well within the braided range (see Chapter 2.0, Figure 8). Pantano Wash has a slightly steeper grade, which would place it even further into the braided range. Even though much of the river has been channelized, it should be recognized that the river is in the braided range and, hence, is very dynamic.

Often, the general response of a river system to a flood event can be qualitatively assessed by studying its profile and plan view. This is especially true of a system which has been altered by man. This type of analysis is based on estimating the relative velocity along the system. In locations where a channel is constricted or the profile steepens, the velocity would be expected to increase. Since velocity is the dominant factor in determining sediment transport rate (when the sediment size does not change greatly), areas with large increases in velocity should degrade; areas where velocities are slowed considerably should experience aggradation. This is expressed in Lane's relationship, which can be written

$$\boxed{Q_s D_{50} \alpha Q S}$$

In this relationship, Q_s is the sediment transport rate, D_{50} is the median sediment size, Q is the flow rate of water and S is the slope of the bed (see Section 2.3).

For example, the instream gravel pit below Swan Road will trap sediment and reduce sediment supply to the downstream reach. This can be expressed using the Lane relationship as:

$$\boxed{Q_s^- D_{50} \alpha Q S^-}$$

From this, one would expect an overall response of possible degradation in the reach of the Rillito River below the gravel pit to Dodge Boulevard. A similar application of the Lane relationship to various reaches in the study area indicates that over half of the channel reaches are well balanced for sediment transport. None of the reaches has a great potential for either aggradation or degradation. In all, the system should not experience large bed elevation changes except for those related to increased development by man and localized flow conditions.

Potential Local Problems at the Proposed Craycroft Road Bridge

Each bridge site has possible problems associated with local erosion and sedimentation processes. These problems are identified below.

Location of the bridge at the confluence of Tanque Verde and Pantano Wash (Fig. 70) can cause several problems. First, the confluence of two sand-bed rivers is usually very dynamic and can shift upstream or downstream and laterally quite quickly. This is especially true when an abnormal sequence of events results in a shift in the relative balance of flows between the two rivers. To compound this problem, the grade control structure has created a situation in which Pantano Wash has a bed elevation several feet higher than Tanque Verde Creek at the same location. This provides an additional tendency for flows from Pantano Wash to migrate toward Tanque Verde Creek, creating a situation in which the flow could attack the bridge piers and abutments at angles other than designed. As a result, local scour around piers and abutments could be significantly increased.

Neither Pantano Wash nor Tanque Verde Creek can contain a 100-year flood within its own channel. Since the two usually do not reach peak flows at the same time, the flow spills out of the flooding channel across the floodplain area between the two channels and into the opposite channel. In the process, the overflow deposits most of its sediment in the floodplain and the clear water entering the opposite channel causes degradation. There is also the problem of poor flow alignment past piers and abutments.

Potential Local Problems of the Sabino Canyon Road Bridge Site

The Sabino Canyon Road Bridge site (see Fig. 71) has several potential erosion and sedimentation problems that should be considered in the bridge design. The present bridge has already experienced several such problems. The flow area of the bridge appears to be inadequate for the 100-year flood event. Over four feet of scour has occurred around the bridge piers and abutments. In addition, the channel is located on a reach that is migrating to the left (looking downstream). This is causing the left abutment to be attacked. The migration tendency of Tanque Verde Creek is largely due to its braided nature and its lack of confinement by bank stabilization or channelization works.

Considerable scour is occurring on the left side of the channel under the bridge since it is located on the outside of the slight bend. This is the usual case with a river bend because high-velocity flow and secondary currents scour sediment from the outside of the slight bend.

The final consideration is the gravel mining from the river. Currently, there is a mine approximately 3,000 feet upstream of the bridge. The pit could act as a sediment trap and cause scour downstream of the pit near the bridge site as the water removes sediment from the bed to regain an equilibrium sediment transport rate. Because of the distance, the threat is not large from the present activity, considering the passage of the 100-year flood; however, gravel mining operations located closer to the bridge site could cause problems if not properly managed. In addition, over a long period of time the overextraction of sand and gravel can cause significant degradation for the entire reach downstream of the operating site, and possible headcuts upstream of the mining.

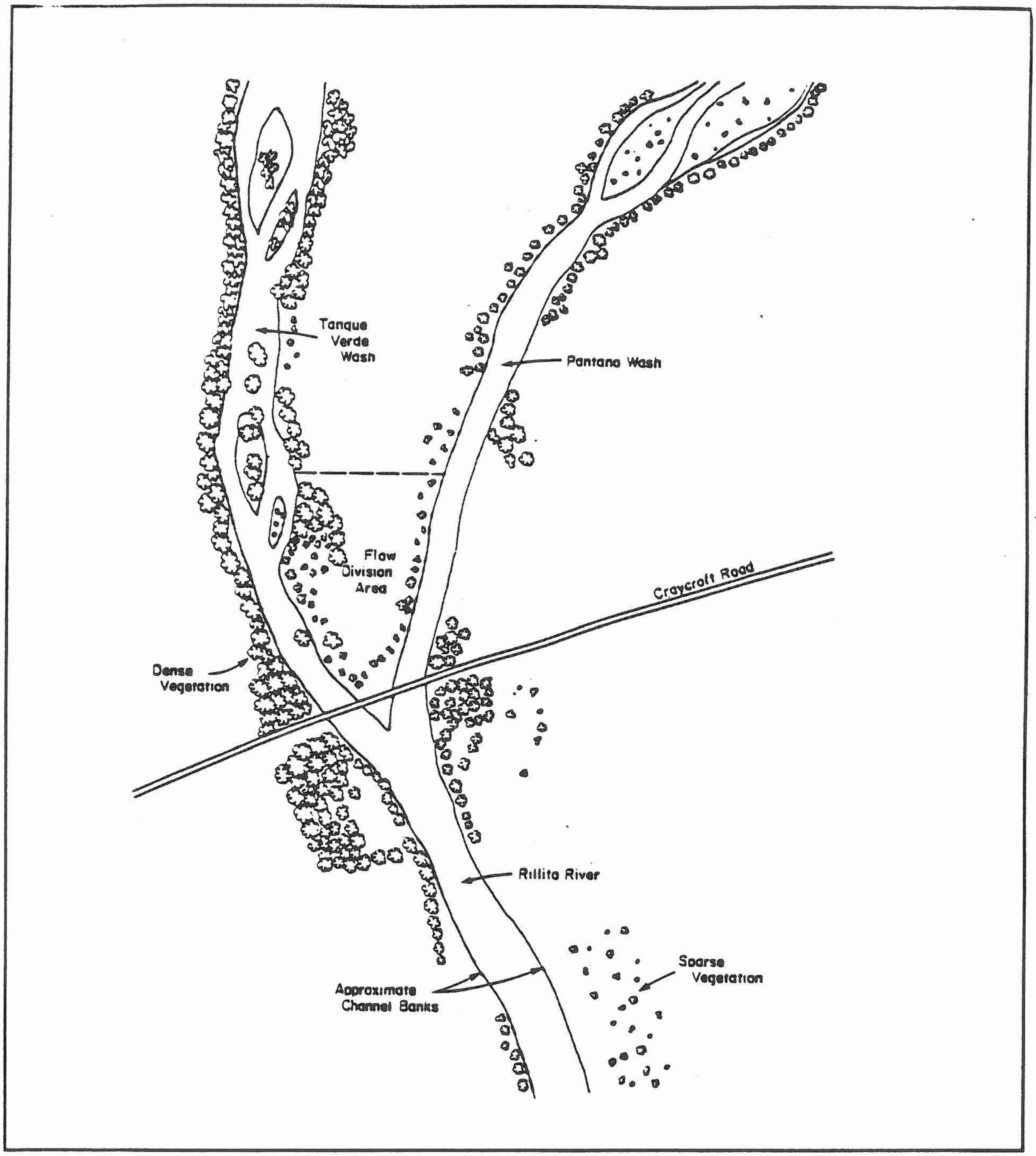

Fig. 70. Sketch of Craycroft Road Bridge crossing.

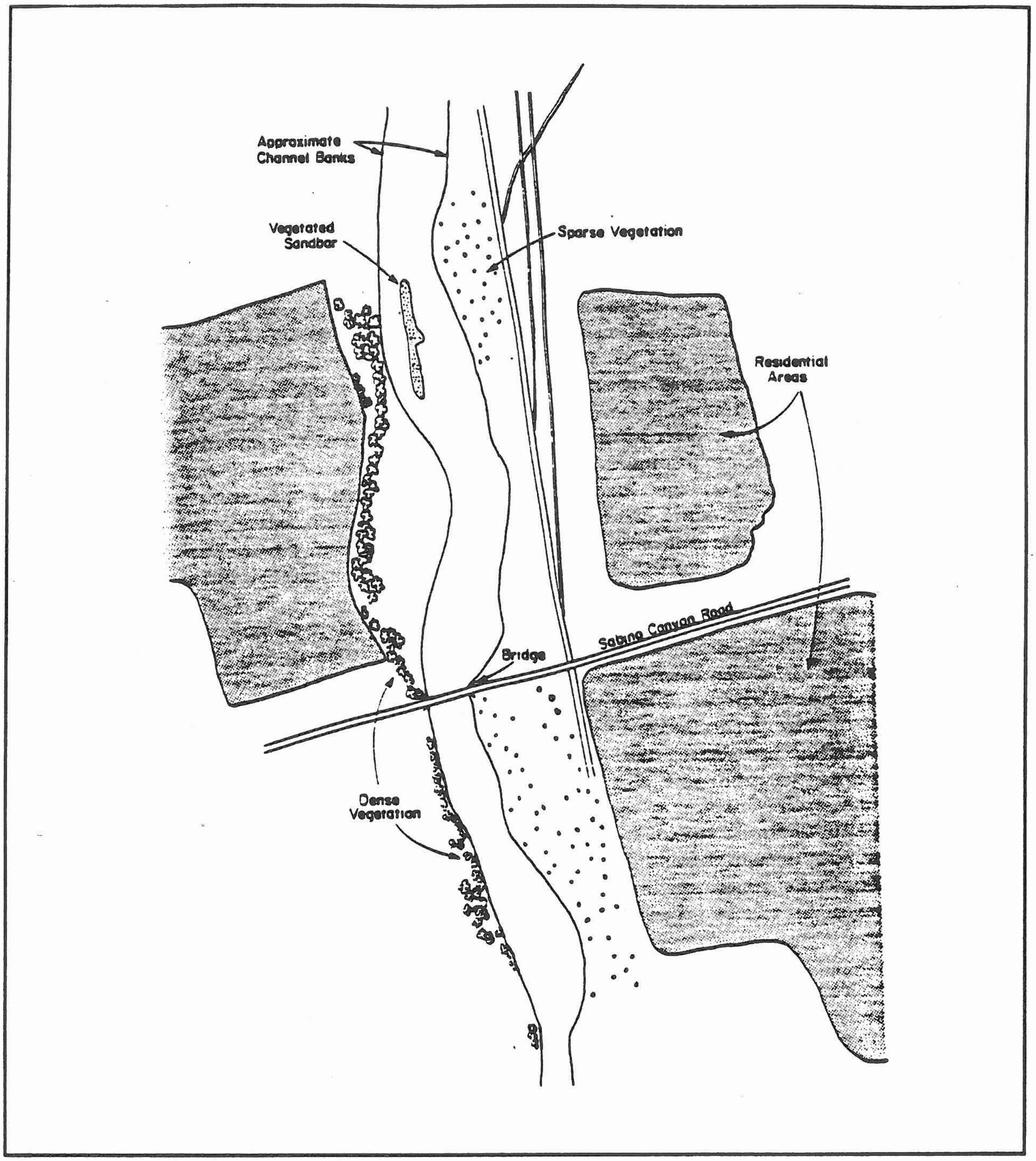

Fig. 71. Sabino Canyon Road crossing site.

LEVEL 2 ANALYSIS

Level 2 - Step 1: Flood History

The Rillito River is formed by the confluence of Pantano Wash and Tanque Verde Creek (see Figure 69) northeast of Tucson and flows west-northwest about 12 miles to its confluence with the Santa Cruz River.

Precipitation in the Rillito River watershed is produced by three types of storms: general winter storms, general summer storms, and local thunderstorms. The general winter storms usually last for several days and result in widespread precipitation. General summer storms are often accompanied by relatively heavy precipitation over large areas for periods of up to 24 hours. Local thunderstorms can occur at any time of the year; however they cover comparatively small areas and cause high-intensity precipitation for a few hours.

The flow in the Rillito River is intermittent; the creek is almost always dry, other than during or immediately after rain. The USGS gaging station on the Rillito River near Tucson kept daily discharge records from October, 1908 to September, 1975, after which it was converted to a crest-stage partial-record station.

Utilizing the USGS records at Rillito Station, all of the extreme events since 1915 are plotted in Fig. 72. Based on these flood data the flood frequency curves are plotted on log-normal paper (Fig. 73). The USGS log-Pearson Type III analysis is shown in Table 8. By reviewing historical floods which occurred in the Rillito River system, and judging from the physical characteristics of Tanque Verde Creek watershed and Pantano watershed, one may conclude that the flood peaks in Tanque Verde Creek are almost independent of those from Pantano Wash. The chance of simultaneous occurrence of both peaks is very small.

The hydrograph of the 1965 flood observed at the Rillito River gage near Tucson (Fig. 74) was used to establish the 100-year flood hydrographs for Tanque Verde Creek and Pantano Wash. The design hydrographs for the 100-year flood for Tanque Verde Creek Sabino Creek, Pantano Wash, Ventana Wash, and Alamo Wash are given in Fig. 75.

Level 2 - Step 2: Hydraulic Conditions

During the December 1965 flood, the Rillito River was in upper regime, having antidunes with breaking waves (see Chapter 3.0, Fig. 13a). The bed forms of the channels in the study system will be antidunes or standing waves during floods. Resistance to flow associated with antidunes depends on how often the antidunes form, the area of the reach they occupy, and the violence and frequency of their breaking. If many antidunes break, resistance to flow can be large because breaking waves dissipate a considerable amount of energy. With breaking waves, Manning's coefficient n could range from about 0.019 to 0.038 for the flow depths being considered.

The existing channels will not contain all of the 100-year flood flows. Some overbank flow will occur. Sparse vegetation, brush, trees and houses are in the floodplain. These elements increase the resistance to flow. For a conservative erosion and sedimentation analysis (high channel velocity), a Manning's roughness of 0.025 for the main channels was assumed for this study. For overbank flows, a higher Manning's n value of 0.05 was used from Dodge Boulevard to Sabino Creek and an n value of 0.06 was used from Sabino Creek to Agua Caliente Wash.

Exceedance Probability	Return Period (year)	Expected Discharge (cfs)	95% Confidence Limit (One-Sided Test)	
			Lower (cfs)	Upper (cfs)
0.5000	2	5,000	4,240	5,800
0.2000	5	9,300	7,670	11,100
0.1000	10	12,500	10,100	15,600
0.0400	25	17,200	13,300	22,000
0.0200	50	21,100	15,800	27,300
0.0100	100	25,200	18,400	33,000
0.0050	200	29,800	20,900	39,200
0.0020	500	35,700	24,400	47,800

Table 8. Rillito River near Tucson, Arizona log-Pearson type III frequency analysis by USGS.

Water surface profile calculation from Dodge Boulevard to Agua Caliente Wash was conducted using the Corps of Engineers HEC-2 program. The main channel roughness was reduced to near the lower limit of the river flow regime expected during the 100-year flood. The hydraulic conditions are predominantly subcritical up to Sabino Creek. The reach from Sabino Creek to Agua Caliente Wash increases its gradient and a mix of subcritical and supercritical hydraulic conditions is possible.

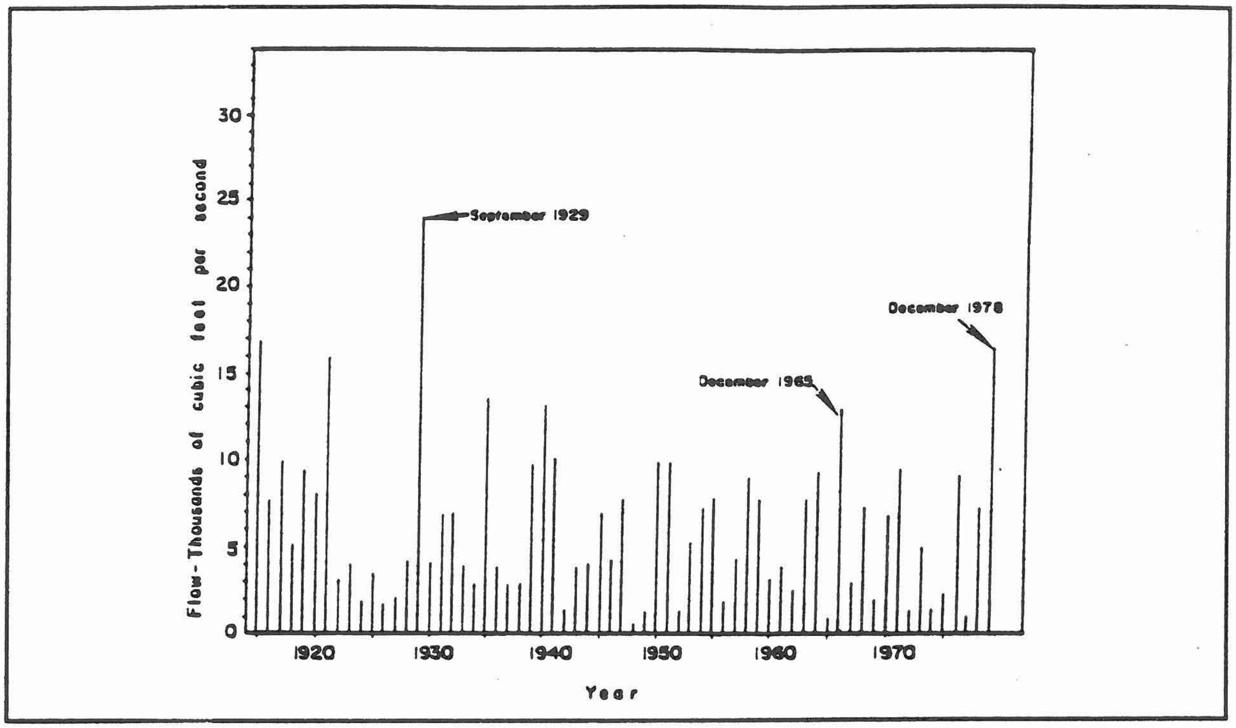

Fig. 72. Flood event at Rillito River near Tucson, Arizona (drainage area 915 square miles).

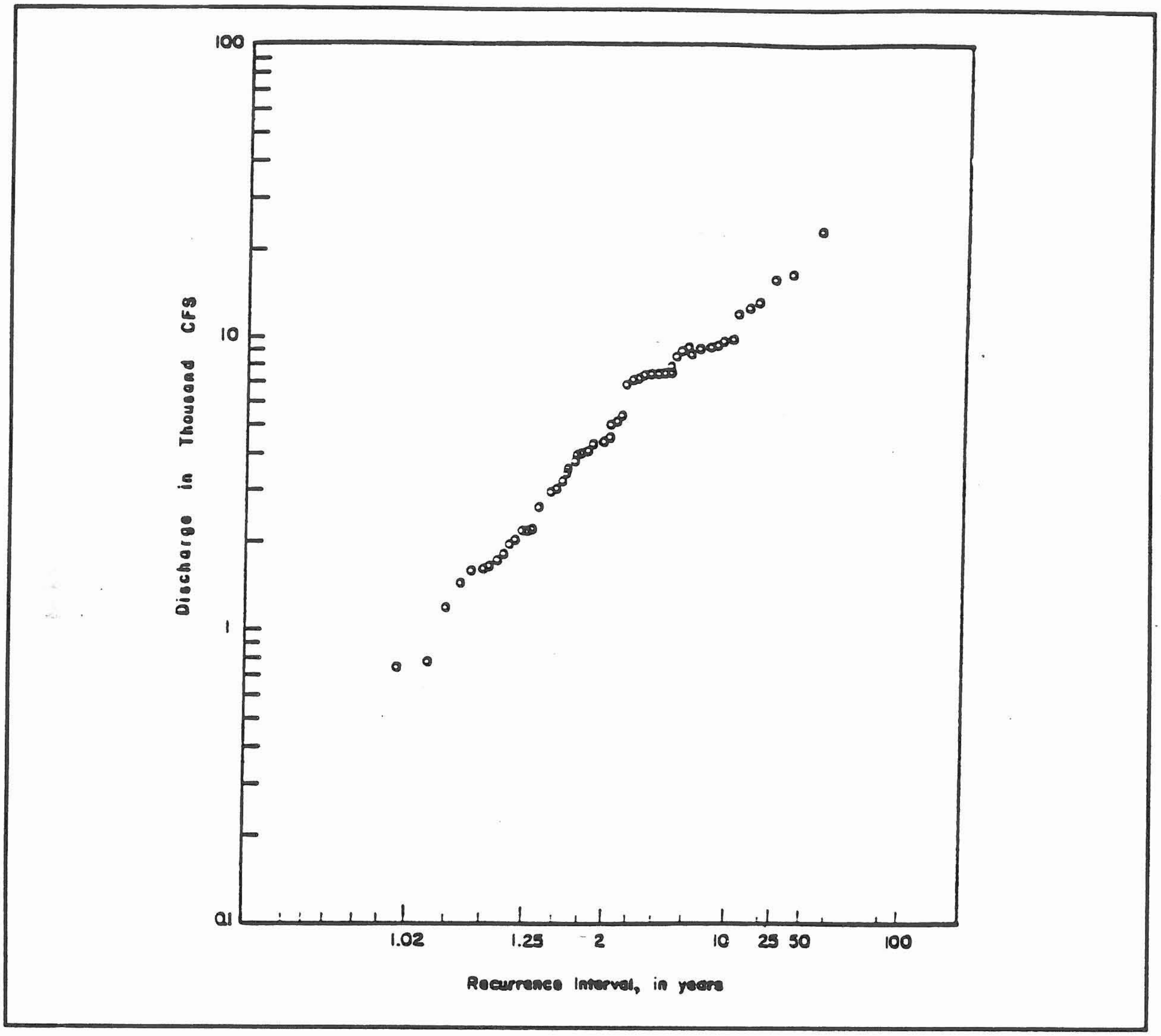

Fig. 73. Log-normal frequency analysis for Rillito River near Tucson.

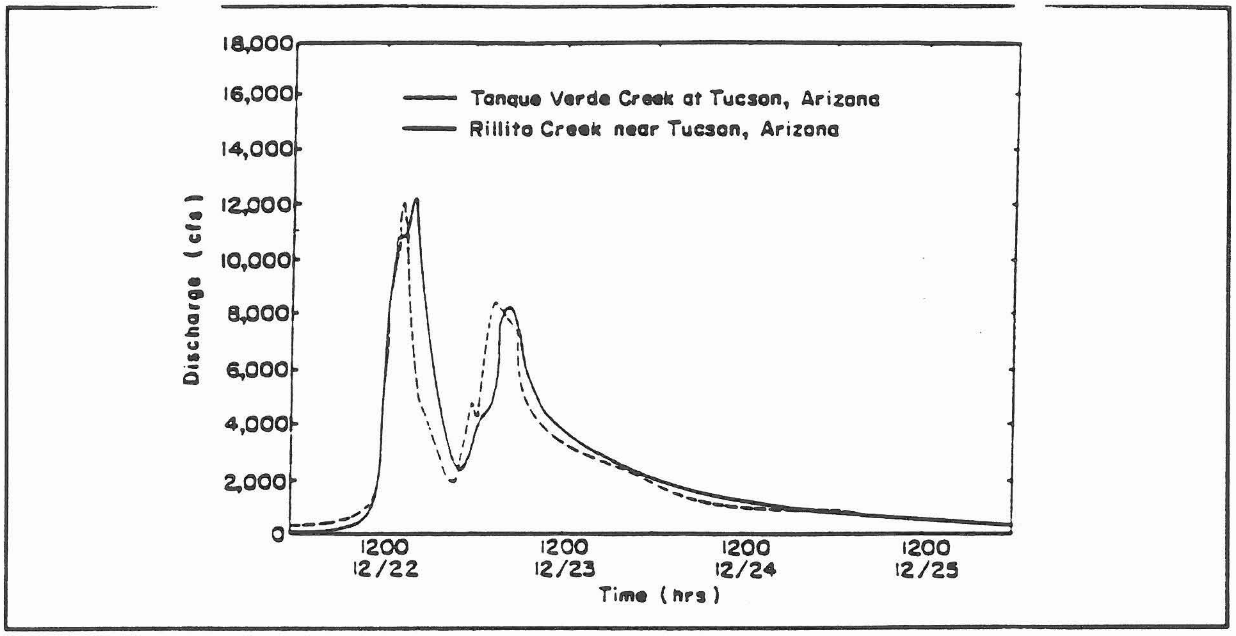

Fig. 74. December, 1965 flood in Rillito River and Tanque Verde Creek.

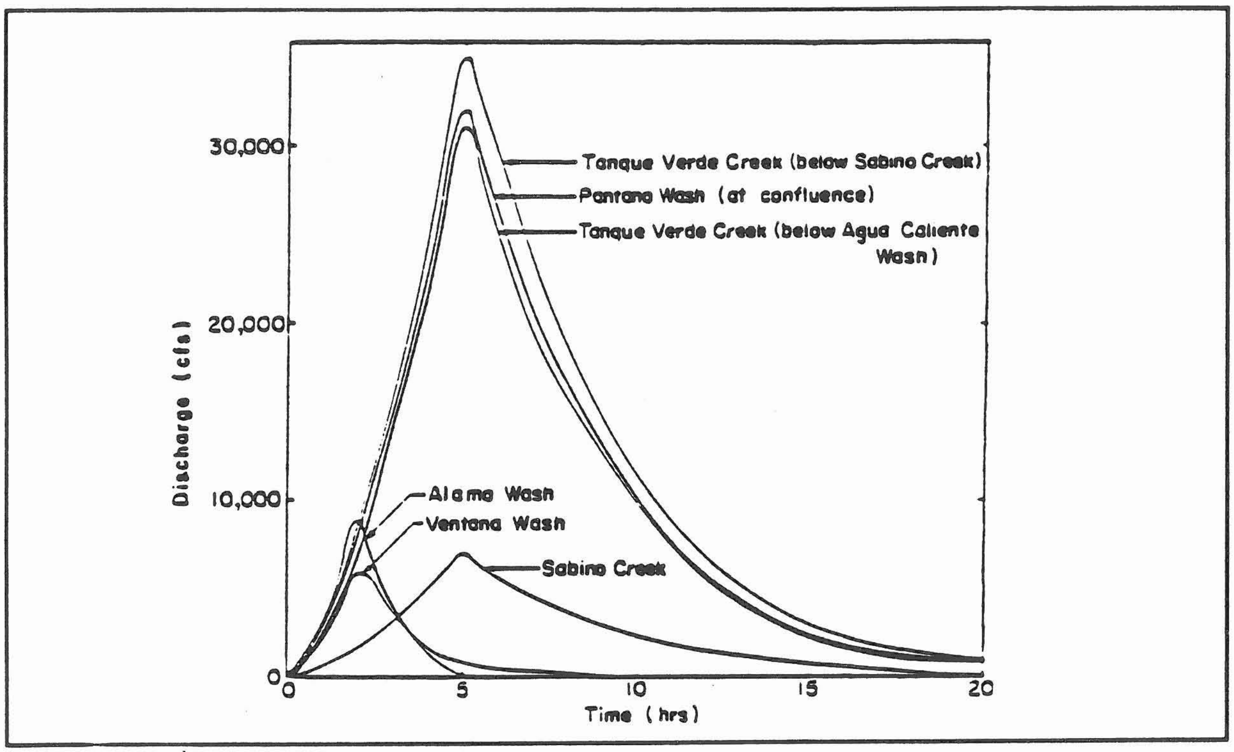

Fig. 75. 100-year flood design hydrographs.

Level 2 - Step 3: Bed and Bank Material

Sediment size is one of the most important parameters used in evaluating sediment transport. A thorough sediment sampling survey was conducted on the river system, consisting of 41 bed material samples. Variation of the size distribution within these segments of the river did not follow an identifiable trend, and therefore an average size distribution was used and the variation from the average size distribution was assumed to be sampling error. Three size distributions were used to cover the river segments from Dodge Boulevard to Pantano Wash (including Pantano Wash), Pantano Wash to Sabino Creek, and Sabino Creek to Agua Caliente Wash. Fig. 76 shows the size distribution used for design on various segments of the river system. The size distribution of Pantano Wash is included to illustrate its similarity to the Rillito River size distribution.

Subsurface bed material samples and bank material samples were also taken. The subsurface bed material is slightly coarser in most cases, but still lacked sizes in the non-transporting range (see Level 2 - Step 5). Bed material samples had more fine material and these distributions varied substantially from one location to the next.

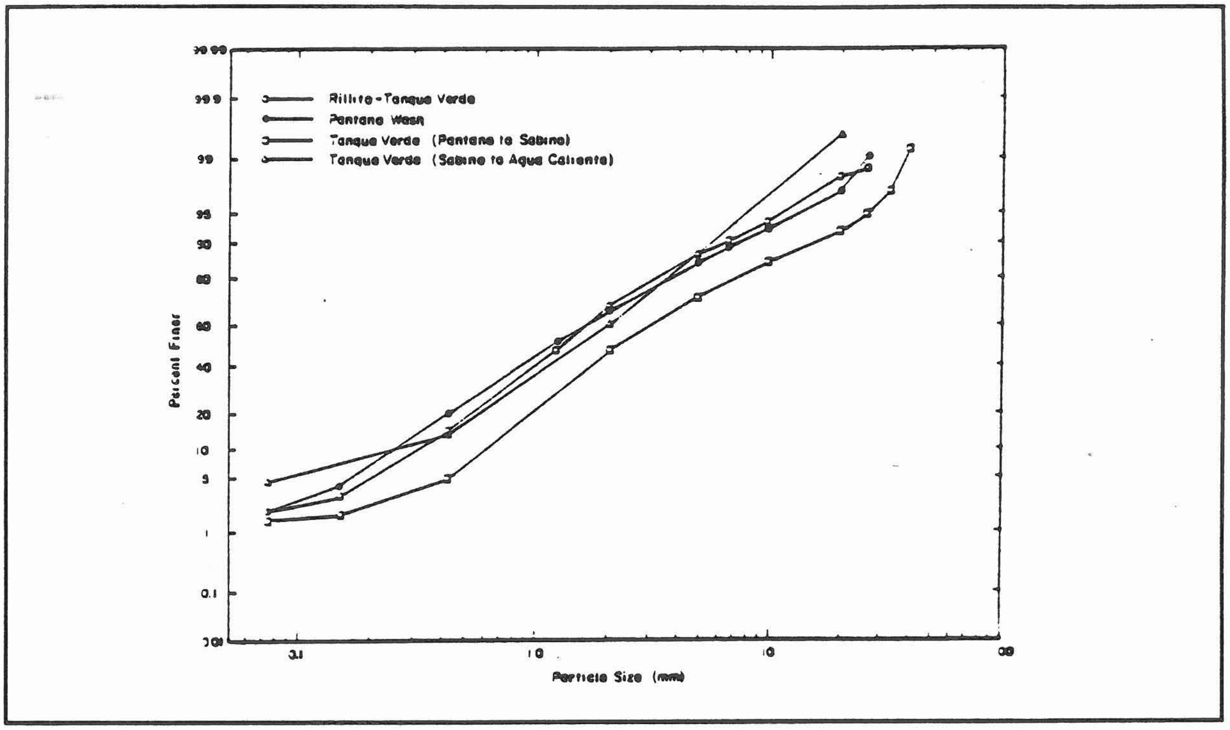

Fig. 76. Rillito-Pantano-Tanque Verde bed sediment distribution.

Level 2 - Step 4: Watershed Sediment

In arid region sand bed channels with significant undeveloped watersheds, it is generally reasonable to assume that equilibrium conditions exist. That is, the watershed sediment supply/yield is equal to the sediment transport capacity of the channel.

An indirect check of the sediment supply/sediment transport rate determinations was available on Tanque Verde Creek. This area has undergone the least change in river form of all the locations in the study area. The 1941 and present aerial photographs show this portion of the system to have remained nearly unchanged. Therefore, it is expected that these reaches must have sediment transporting capacities near equilibrium.

Another factor that helped provide a reliable determination of sediment supply was the grade control structure on Pantano Wash at the Craycroft Road Bridge site. A channel will quickly come to equilibrium behind such a structure since the results of an excess or imbalance in sediment transport rate to the structure are corrected by removal or storage of material behind the structure. This process allows the channel to quickly reach equilibrium behind the structure by producing a channel bed slope that will result in a sediment transport rate equal to the incoming supply.

Level 2 - Step 5: Incipient Motion and Step 6: Armoring Potential

A large percentage of sediment in the study reach falls in the coarse sand and fine gravel range with less than 10 percent classified as medium gravel. Very coarse gravels are not present. From an analysis based on Shields' criteria (see Chapter 4.0, Equation 12), all sizes present can be easily transported by the mean annual storm. Formation of an armoring layer on the bed is unlikely since coarse, nontransportable particle sizes are missing from the distribution.

Level 2 - Step 7: Rating Curves

The stage-discharge plot for the USGS gaging station on Rillito River near Tucson is shown in Fig. 77. In this plot, the gage height values from the flood observations have been converted to equivalent stages at the present sites, based on the gage datum information given below. The shifting of the stage-discharge relationships from early years to 1970 is also indicated in this plot. For a given discharge the water surface elevation drops about three feet from the 1956-1965 curve to the 1966 curve and drops another two feet to the 1974-1978 curve. The decrease of the water surface elevation at this station is a result of the channel degradation since 1956.

Level 2 - Step 8: Scour Analysis

Each of the bridges has its own unique problems that must be considered in the formulation of alternative designs. Analysis of the design alternative is broken into three areas: (1) low chord criteria, (2) total scour criteria, and (3) other additional considerations.

When designing a bridge foundation, proper consideration of scour must be made to determine the required safe depth of piles or other supports. A design which gives adequate support for the structure when the channel bed is at its initial elevation may be inadequate after scour occurs and lowers the channel bed. The physical processes that must be considered are long-term changes in bed elevation, local scour, contraction scour, and passage of sand waves. The total scour is the sum of these, and must be subtracted from the initial design elevation to establish the design depth for all supports. The supports must have a depth of burial below this elevation sufficient to support the structure (see HEC-18 [23]).

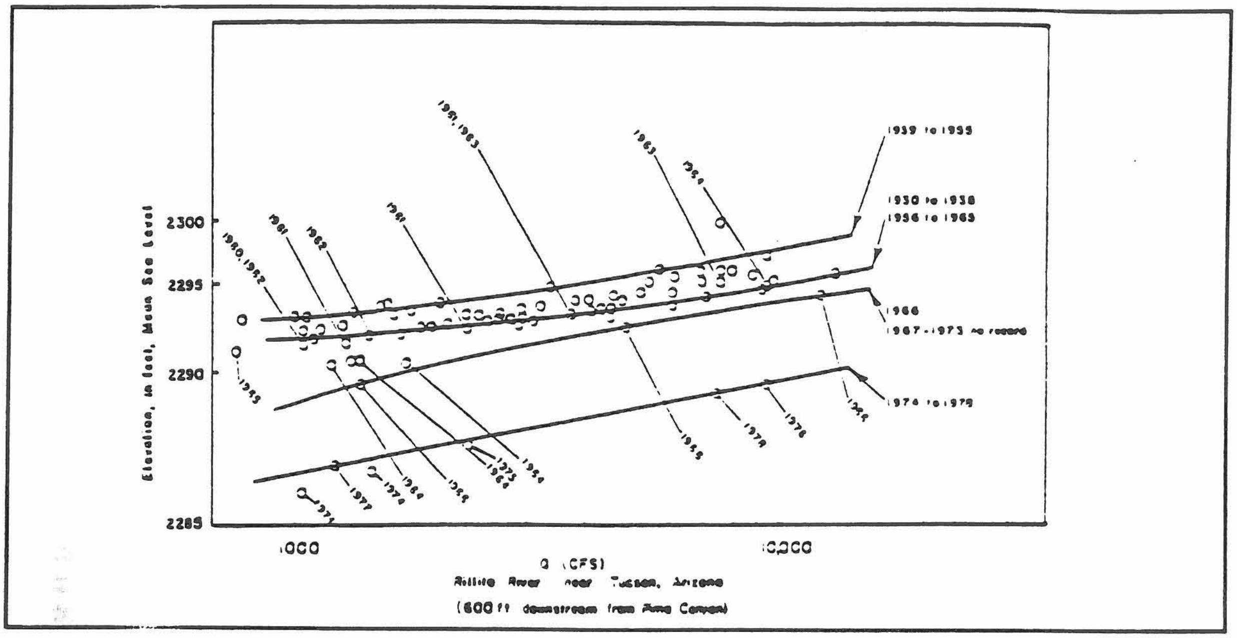

Fig. 77. Stage-discharge plot for Rillito River near Tucson.

CONCLUSIONS

The results of Level 1 and 2 analyses of the Rillito River system have demonstrated that scour will be a significant factor in bridge design. Due to the complexity of channel response in this region, a Level 3 analysis was undertaken. For details of that analysis and the resulting bridge design alternatives, the reader is referred to HIRE [4] (pp. VII-112 to VII-122).

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