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Hydraulic Engineering Circular No. 18

Evaluating Scour at Bridges

Second Edition

Office of Research and Development
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16. Abstract

This is the second edition of HEC-18. It contains updated material not included in the first edition dated February 1991. This document presents the state of knowledge and practice for the design, evaluation and inspection of bridges for scour. This document is a revision to HEC-18 dated February 1991 which, in turn, was an update of the publication, "Interim Procedures for Evaluating Scour at Bridges," issued in September 1988 as part of the FHWA Technical Advisory TA 5140.20, "Scour at Bridges." TA 5140.20 has since been superseded by TA 5140.23,"Evaluating Scour at Bridges." dated October 28,1991. This circular contains revisions as a result of further scour related developments and use of the 1991 edition of HEC-18 by the highway community.

The principal changes from the 1991 edition of HEC-18 are: the inclusion of a section on tidal scour with example problems; a comparison between Neill's equation for beginning of motion for coarse-bed material and an equation that results from Laursen's clear-water scour equation; clarification and simplification of the use of the clear-water and live-bed contraction scour equations; replacing the total scour example problem in Chapter 4 with a problem based on the results of a WSPRO analysis of a highway crossing; elimination of the computation of guide bank length in the appendices (the complete procedure is contained in HEC-20);[8] inclusion of an updated version of North Carolina's scour evaluation procedures in Appendix D; replacing the scour analysis for Great Pee Dee River, South Carolina with the scour analysis for the South Platte River in Colorado in Appendix F; updating the information of scour detection equipment in the appendix G and correction of editorial and minor errors in the text and figures.

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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

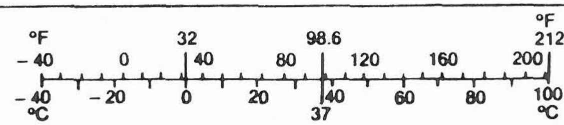
Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimetres	mm
ft	feet	0.305	metres	m
yd	yards	0.914	metres	m
mi	miles	1.61	kilometres	km
		AREA		
in^2	square inches	645.2	millimetres squared	mm^2
ft^2	square feet	0.093	metres squared	m^2
yd^2	square yards	0.836	metres squared	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

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PREFACE

This is the second edition of HEC-18. It contains updated material not included in the first edition dated February 1991 and should be used as the primary reference.

This Federal Highway Administration (FHWA) publication, Hydraulic Engineering Circular No. 18 (HEC-18), "Evaluating Scour at Bridges," provides procedures for the design, evaluation and inspection of bridges for scour. This document is a revision to HEC-18 dated February 1991 which, in turn, was an update of the publication, "Interim Procedures for Evaluating Scour at Bridges," issued in September 1988 as part of the FHWA Technical Advisory TA 5140.20, "Scour at Bridges." TA 5140.20 has since been superseded by TA 5140.23, "Evaluating Scour at Bridges" October 28, 1991.[5] This circular contains revisions as a result of further scour related developments and the use of the 1991 edition of HEC-18 by the highway community.

The principle changes from the 1991 edition of HEC-18 are:

1. The inclusion of a section on tidal scour with example problems in Chapter 4;

2. A comparison between Neill's equation for beginning of motion for coarse bed material and an equation that results from Laursen's clear-water scour equation in Chapter 2;

3. Clarification and simplification of the use of the clear-water and live-bed contraction scour equations in Chapter 4;

4. The inclusion of Melville's 1988 pier scour equation in Figure 4;

5. A change in the maximum expected value of y_s/a in Figure 5;

6. Replacing the total scour example problem in Chapter 4 with a problem based on the results of a WSPRO analysis of a highway crossing;

7. Elimination of the computation of guide bank length in the appendices (the complete procedure is contained in HEC-20) [8];

8. Inclusion of an updated version of North Carolina's scour evaluation procedures in the Appendix D;

9. Replacing the scour analysis for Great Pee Dee River, South Carolina with the scour analysis for the South Platte River in Colorado in Appendix F;

10. Updating the information of scour detection equipment in the Appendix G; and

11. Figure 2 has been revised and a more complete discussion of this figure has been provided.

12. Correction of editorial and minor errors in the text and figures.

CHAPTER 1

INTRODUCTION

1.1 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.

1.2 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 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 and evaluating total scour depths at piers and abutments for both riverine and tidal waterways. 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 procedures from several states to assess and evaluate scour problems are presented.

1.3 Background

The most common cause of bridge failures is floods with the scouring of bridge foundations being the most common cause of flood damage to bridges. **The hydraulic design of bridge waterways 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 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 Record 950.[3]

1.4 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.[4] Approximately 84 percent of the 577,000 bridges in the National Bridge Inventory are built over waterways. Statistically, we can expect hundreds of these bridges to experience floods in the 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 from future floods may have to be accepted. **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 to ten times the cost of the bridge itself. Moreover, the need to ensure public safety and 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 TA 5140.23 entitled, "Evaluating Scour at Bridges."[5] 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 is the identification of scour-critical bridges which will be entered into the National Bridge Inventory using the FHWA document "Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges."[6]

1.5. Improving the State-of-Practice of Estimating Scour at Bridges

Some of the problems associated with estimating scour and providing cost-effective and safe designs are being addressed in research and development programs of the FHWA and individual State highway agencies. The following sections detail the most pressing research needs.

1. **Field Measurements of Scour**. The current equations and methods for estimating scour at bridges are based primarily on laboratory research. Very little field data have been collected to verify the applicability and accuracy of the various design procedures for the range of soil conditions, streamflow 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. In excess of 20 States have 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 guidance "Interim Procedures for Evaluating Scour at Bridges," [7] which accompanied the September 1988 FHWA Technical Advisory.[5]

2. **Scour Monitoring and Measurement Equipment**. There is a need for the development of instrumentation 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 foundations which are vulnerable to scour. It is not economically feasible to repair or replace all of these bridges. 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. Instrumentation is also needed to determine unknown bridge foundations (See Appendix G).

   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. Research has also been initiated to develop techniques and instruments to identify unknown foundations of existing bridges.

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, to be discussed later, for the analysis of flow through bridges and for computing scour. There currently is a contract for the development of software to determine total scour at bridge crossings. This effort should continue. In addition, the maintenance, support and improvement of existing and future software should be continually updated and enhanced.

4. **Laboratory Studies of Scour**. There is a need for laboratory studies to better understand certain elements of the scour processes and to develop alternate and improved scour countermeasures. Only through controlled experiments can the effect of the variables and parameters associated with scour be determined. Through these efforts, scour prediction equations can be improved and additional design methods for countermeasures can be developed. Results from these laboratory experiments must be verified by ongoing field measurements of scour.

   Laboratory research is needed for:

   a. Determining methods to predict scour depths associated with pressure flow,

   b. Determining more applicable coefficients for the abutment scour equations to replace the simplistic use of abutment length,

   c. Improving methods for estimating contraction scour for abutments which are set back from the channel when there is overbank flow,

   d. Fundamental research on the mechanics of tidal scour.

e. Determining methods to predict scour depths when there is ice or debris buildup at a pier or abutment,

f. Determining the influence of graded, armored, or cohesive bed material on maximum local scour at piers and abutments,

g. Determining the effect of pile caps or footings on pier scour depth,

h. Improving methods for determining the size and placement of riprap (elevation, width and location) in the scour hole to protect piers and abutments,

i. Determining the width of scour hole as a function of scour depth and bed material size,

j. Fundamental research on the mechanics of riverine scour,

k. Improved knowledge of the effect of flow depth and velocity on scour depths,

l. Improved understanding of the bridge scour failure mechanism which would combine the various scour components (pier, abutment, contraction, lateral migration, degradation) into an estimate of the scoured cross section under the bridge,

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

n. Effect of wide and variable pier widths on scour depths, and

o. Determining the impact of overlapping scour holes.

CHAPTER 2

BASIC CONCEPTS AND DEFINITIONS OF SCOUR

2.1 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 sand-bed streams.** Under constant flow conditions, scour will reach 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. Under flow conditions more typical of actual bridge crossings, several floods will be needed to attain maximum scour.

Designers and inspectors need to carefully study site-specific subsurface information in evaluating scour potential at bridges, giving particular attention to foundations on rock. Massive rock formations with few discontinuities are highly resistant to scour during the lifetime of a typical bridge.

All of the equations for estimating contraction and local scour are based on laboratory experiments with limited field verification. The equations recommended in this document are considered to be the most applicable for estimating scour depths.

A factor in scour at highway crossings and encroachments is whether the scour is clear-water or live-bed scour. Clear-water scour occurs where there is no transport of bed material upstream of the crossing or encroachment and live-bed scour occurs where there is transport of bed material from the upstream reach into the crossing or encroachment. This subject is discuss in detail in Section 2.6.

This document presents procedures, equations, and methods to analyze scour in both riverine and coastal areas. In riverine environments scour results from flow in one direction (downstream). In coastal areas, highways that cross streams and/or encroach longitudinally on them are subject to tidal fluctuation and scour results from flow in two directions. In waterways influenced by tidal fluctuations, flow velocities do not necessarily decrease as scour occurs and the waterway area increases. This is in sharp contrast to riverine waterways where the principle of flow continuity requires that velocity be inversely proportional to the waterway area. **However, the methods and equations for determining stream instability, scour and associated countermeasures apply for both riverine and coastal streams.** The difficulty in tidal streams is in determining the hydraulic parameters (such as discharge, velocity, and depth) that are to be used in the scour equations.

2.2 Total Scour

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

1. Long-term aggradation and degradation,
2. Contraction scour, and
3. Local scour.

In addition, lateral migration of the stream must be assessed when evaluating total scour at piers and abutments of highway crossings.

2.2.1 Aggradation and Degradation

These are long-term streambed elevation changes due to natural or man-induced causes which can affect the reach of the river on which the bridge is located. Aggradation involves the deposition of material eroded from the channel or watershed upstream of the bridge, whereas degradation involves the lowering or scouring of the bed of a stream due to a deficit in sediment supply from upstream.

2.2.2 Contraction Scour

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 area or change in downstream control of the water surface elevation. The scour is the result of increased velocities and shear stress on the bed of the channel.

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. Contraction scour can be either clear-water or live-bed. Live-bed contraction scour typically occurs during the rising stage of a runoff event, while refilling of the scour hole occurs during the falling stage. Also, clear-water scour at low or moderate flows can change to live-bed scour at high flows. This cyclic nature creates difficulties in measuring contraction scour after a flood event.

2.2.3 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, and is usually cyclic in nature. Local scour can also be either clear-water or live-bed scour.

2.2.4 Lateral Stream Migration

In addition to the types of scour mentioned above, naturally occurring lateral migration of the main channel of a stream within a floodplain may increase pier scour, erode abutments or the approach roadway, or change the total scour by changing the flow angle of attack at piers. Factors that affect lateral stream 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 Hydraulic Engineering Circular No. 20, [8] and "Highways in the River Environment"[9]).

The following paragraphs provide a more detailed discussion of the various components of total scour.

2.3 Aggradation and Degradation - Long-Term Streambed 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. The streambed may be aggrading, degrading or in relative equilibrium in the vicinity of the bridge crossing. In this section long-term trends are considered. Long-term aggradation and degradation do not include the localized cutting and filling of the bed of the stream that might occur during a runoff event (contraction and local scour). A stream may cut and fill at specific locations during a runoff event and also have a long-term trend of an increase or decrease in bed elevation over a reach of a stream. The problem for the engineer is to estimate the long-term bed elevation changes that will occur during the life of the structure.

A long-term trend may change during the life of the bridge. These long-term changes are the result of modifications to the stream or watershed. Such changes may be the result of natural processes or human 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 long-term streambed changes, must be estimated.

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 streambed, 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. Tidal ebb and flood may degrade a coastal stream, whereas littoral drift may result in aggradation of a stream.

The Corps of Engineers and other agencies should be contacted concerning documented long-term streambed variations. If no documented data exist or if such data require further evaluation, an assessment of long-term streambed elevation changes for riverine streams should be made using the principles of river mechanics. With coastal streams the principals of both river and coastal engineering mechanics are needed. 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 (watershed erosion), the sediment transport capacity of a stream (hydraulics) and the response of a stream to these factors (geomorphology and river mechanics). In coastal streams, in addition to the above, consideration must be made of tidal conditions; i.e., the magnitude and period of the storm surge, the sediment delivery to the channel by the ebb and flow of the tide, littoral drift, the sediment transport capacity of the tidal flows and the response of the stream to these tidal and coastal engineering factors.

Significant morphologic impacts can result from human activities. The assessment of the impact of human activities requires a study of the history of the river, estuary, or tidal inlet, as well as a study of present water and land use and stream control activities. All agencies involved with the river or coastal area should be contacted to determine possible future changes in the river.

To organize such an assessment, a three-level fluvial system approach can be used comprising of (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 models or physical process computer modeling using mathematical models such as BRI-STARS [10] and the U.S. Army Corps of Engineers HEC-6 [11] to make predictions of quantitative changes in streambed elevation due to changes in the stream and watershed. Methods to be used in Levels 1 and 2 are presented in HEC-20, "Stream Stability at Highway Structures," [8] and HIRE.[9] Additional discussion of this subject is presented in Chapter 4 of this document.

For coastal areas, where highway crossings (bridges) and/or longitudinal stream encroachments are subject to tidal influences, the three-level fluvial system approach is also appropriate. The approach for tidal waterways is described in Chapter 4 of this document.

2.4 Contraction Scour

2.4.1 General

Contraction scour occurs when the flow area of a stream at flood stage is reduced, either by a natural contraction or by a bridge. From continuity, a decrease in flow area results in 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, in the riverine situation, 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.

In coastal streams which are affected by tides, as the cross-section area increases the discharge from the ocean may increase and thus the velocity and shear stress may not decrease. Consequently, relative equilibrium may not be reached. Thus, at tidal inlets which experience clear-water or live-bed scour, contraction scour may result in a continual lowering of the bed (long-term degradation).

Contraction scour can also be caused by short-term (daily, weekly, yearly or seasonal) changes in the downstream water surface elevation that control 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 aggradation/degradation.

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 either a natural decrease in flow area of the stream channel or by abutments projecting into the channel and/or the piers blocking a large portion of the flow area. Contraction can also be caused by the approaches to a bridge cutting off the floodplain flow. This can cause clear water scour on a setback portion of a bridge section and/or a relief bridge because the floodplain flow does not normally transport significant concentrations of bed material sediments. The difference between clear-water and live-bed scour is discussed in detail in Section 2.6. 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 returning to the main channel at the end of the abutment.

Other factors that can cause contraction scour are (1) natural stream constrictions, (2) long highway approaches over the floodplain to the bridge, (3) ice formation or jams, (4) natural berms 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 floodplain.

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 at the point bar. 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.

Contraction Scour Equations. 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 streambed sediment being transported into the contracted section from upstream. In this case, the scour hole reaches equilibrium when the transport of bed material out of the scour hole is equal to that transported into the scour hole from upstream. Clear-water scour occurs when the stream bed sediment transport in the uncontracted approach flow is negligible. In this case, the scour hole reaches equilibrium when the average bed shear stress is less than that required for incipient motion of the bed material. Clear-water and live-bed scour are discussed further in Section 2.6.

Contraction scour equations are based on the principle of conservation of sediment transport. In the case of live-bed scour, this 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. As scour develops, the shear stress in the contracted section decreases as a result of a larger flow area and decreasing average velocity. For live-bed scour, maximum scour occurs when the shear stress reduces to the point that sediment transported in equals the bed sediment transported out and the conditions for sediment continuity are in balance. For clear-water scour, the transport into the contracted section is essentially zero and maximum scour occurs when the shear stress reduces to the critical shear stress of the bed material.

2.4.2 Live-Bed Contraction Scour Equation

Laursen [12] derived the following live-bed contraction scour equation based on a simplified transport function and other simplifying assumptions. The application of this equation is presented in Section 4.3.4.

$$\frac{y_2}{y_1} = \left(\frac{Q_2}{Q_1}\right)^{\frac{6}{7}} \left(\frac{W_1}{W_2}\right)^{k_1} \left(\frac{n_2}{n_1}\right)^{k_2} \qquad \textbf{(1)}$$

$$y_s = y_2 - y_1 = \text{(Average scour depth, ft)}$$

where

- y_1 = average depth in the upstream main channel, ft
- y_2 = average depth in the contracted section, ft
- W_1 = bottom width of the upstream main channel, ft
- W_2 = bottom width of main channel in the contracted section, ft
- Q_1 = flow in the upstream channel transporting sediment, cfs
- Q_2 = flow in the contracted channel, cfs. Often this is equal to the total discharge unless the total flood flow is reduced by relief bridges or water overtopping the approach roadway
- n_2 = Manning's n for contracted section
- n_1 = Manning's n for upstream main channel
- k_1 & k_2 = exponents determined below depending on the mode of bed material transport

V_*/w	k_1	k_2	Mode of Bed Material Transport
<0.50	0.59	0.066	Mostly contact bed material
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_* = $(gy_1S_1)^{1/2}$ shear velocity in the upstream section, ft/s
w = median fall velocity of the bed material based on the D_{50} (see Figure 3)
g = acceleration of gravity (32.2 ft/s^2)
S_1 = slope of energy grade line of main channel, ft/ft
D_{50} = median diameter of the bed material, ft

2.4.3 Clear-Water Contraction Scour Equation

Laursen's [13] clear-water contraction scour equation has a much simpler derivation because it does not involve a transport function. It simply recognizes that the shear stress in the contracted section must equal the critical shear stress.

$$\tau_2 = \tau_c \tag{2}$$

where

τ_2 = average bed shear stress, contracted section
τ_c = critical bed shear stress at incipient motion

For noncohesive bed materials and for fully developed clear-water scour, Laursen used Equation 3 to estimate the critical shear stress.

$$\tau_c = 4\ D_{50} \tag{3}$$

The bed shear stress can be expressed as:

$$\tau_2 = \gamma y_2 S_f = \frac{\gamma V_2^2 n^2}{(1.49)^2\ y_2^{\frac{1}{3}}} \tag{4}$$

where

γ = the unit weight of water (62.4 lb/ft^3)
y_2 = average depth in the contracted section, ft
S_f = slope of the energy grade line, ft/ft
V_2 = average velocity in the contracted section, ft/s

Using Strickler's approximation for Manning's n:

$$n = 0.034\ D_{50}^{1/6} \tag{5}$$

Rearranging Equation 2:

$$\frac{\tau_2}{\tau_c} = 1.0 \tag{6}$$

By substituting Equations 3 and 4 into Equation 6 and solving for y_2, Laursen's clear-water contraction scour equation can be derived:

$$y_2 = \left[\frac{V_2^2}{120\ D_{50}^{\frac{2}{3}}}\right]^3 \tag{7}$$

In terms of discharge (using continuity), the equation is:

$$y_2 = \left[\frac{Q_2^2}{120\ D_{50}^{\frac{2}{3}}\ W_2^2}\right]^{\frac{3}{7}} \tag{8}$$

The velocity and depth given in Equations 7 and 8 are associated with initiation of motion of the indicated D_{50} size. Equation 7 can be rearranged to give the critical velocity V_c as follows:

$$V_c = 10.95\ y^{\frac{1}{6}}\ D_{50}^{\frac{1}{3}} \tag{9}$$

A dimensionless form of Equation 8 can be written if flow continuity can be assumed for the approach and contracted segments of the floodplain being analyzed. That is:

$$Q_2 = Q_1 = V_1\ W_1\ y_1 \tag{10}$$

then

$$\frac{y_2}{y_1} = \left(\frac{W_1}{W_2}\right)^{\frac{6}{7}} \left[\frac{V_1^2}{120\ y_1^{\frac{1}{3}}\ D_{50}^{\frac{2}{3}}}\right]^{\frac{3}{7}} \tag{11}$$

Note that the term in brackets in Equation 11 should not exceed a value of 1.0. If this term is greater than 1.0, then live-bed conditions would control.

Laursen's clear-water contraction scour equations are based on rather limiting assumptions. For example they assume homogeneous bed materials. However, with clear-water scour in stratified materials, assuming the layer with the finest D_{50} would result in the most conservative estimate of contraction scour. Alternatively, the clear-water contraction scour equations could be used sequentially for stratified bed materials. An example problem illustrating the use of the contraction scour equations is presented in Chapter 4.

Both the live-bed and clear-water contraction scour equations are the best that are available and should be regarded as a first level of analysis. If a more detailed analysis is warranted, a sediment transport model like BRI-STARS [10] could be used.

2.5 Local Scour

The basic mechanism causing local scour at piers or abutments is the formation of vortices (known as the horseshoe vortex) at their base (**Figure 1**). The horseshoe vortex results from the pileup of water on the upstream surface of the obstruction and subsequent acceleration of the flow around the nose of the pier or embankment. The action of the vortex removes bed material from around the base of the obstruction. The transport rate of sediment away from the base region is greater than the transport rate into the region, and, consequently, a scour hole develops. As the depth of scour increases, the strength of the horseshoe vortex is reduced, thereby reducing the transport rate from the base region. Eventually, for live-bed local scour, equilibrium is reestablished and scouring ceases. For clear-water scour, scouring ceases when the shear stress caused by the horseshoe vortex equals the critical shear stress of the sediment particles at the bottom of the scour hole.

In addition to the horseshoe vortex around the base of a pier, there are vertical vortices downstream of the pier called the wake vortex (Figure 1). Both the horseshoe and wake vortices remove material from the pier base region. However, the intensity of 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.

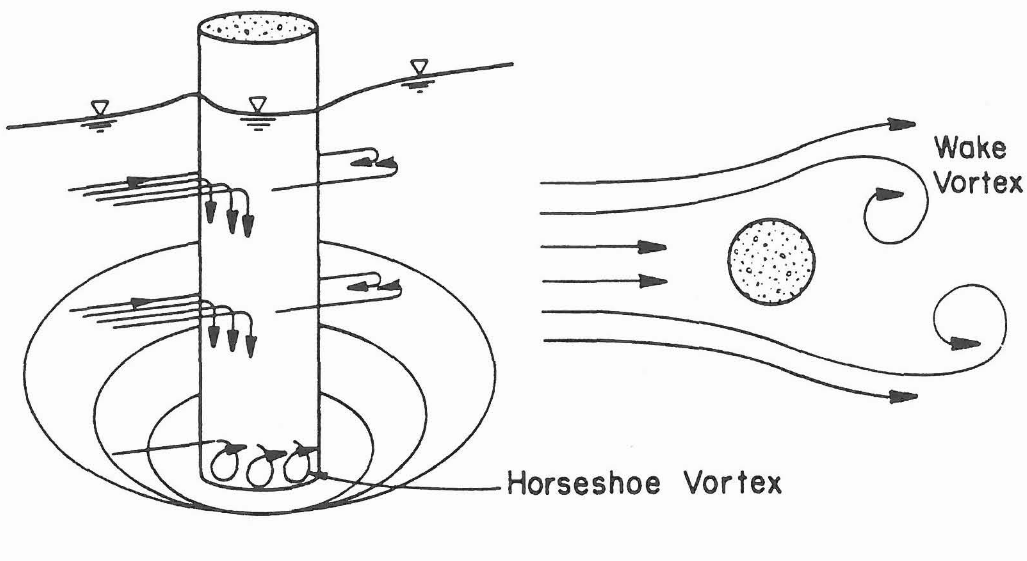

Figure 1. Schematic Representation of Scour at a Cylindrical Pier.

Factors which affect the magnitude of local scour at piers and abutments are (1) width of the pier, (2) discharge intercepted by the abutment and returned to the main channel at the abutment (in laboratory flumes this discharge is a function of 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. In laboratory flume studies, an increase in the projected length of an abutment (or embankment) into the flow increased scour, whereas this is not the case in the field. This result for flumes is caused by the fact that the discharge intercepted by the abutment and returned to the main channel is a function of the abutment length. However, in the field case with a non-uniform distribution of flow, the discharge returned to the main channel is not simply a function of the abutment length. Because of this, abutment scour equations, which are based on laboratory experiments, give very large depths. These depths would only occur in the field for conditions that duplicate the conditions under which the flume experiments were conducted.

3. Pier length has no appreciable effect on local scour depth as long as the pier is aligned with the flow. When the pier is skewed to the flow, the pier length has a significant influence on scour depth. For example, with the same angle of attack, doubling the length of the pier increases scour depth by 33 percent.

4. Flow depth also has an influence 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. 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 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 little effect on local scour depth. Likewise, 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, by the Washington State Department of Transportation, and by other researchers [14, 15, 16, 17] developed equations that take into account the decrease in scour due to the armoring of the scour hole. Richardson and Richardson [18] combined these equations into a simplified equation, which accounted for bed material

size. However, field data are inadequate to support these equations at this time. As such, 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 in Section 2.6.

Fine bed material (silts and clays) will have scour depths as deep as sand-bed streams. This is true even if bonded together by cohesion. The effect of cohesion is to influence the time it takes to reach the maximum scour. With sand bed material, the time to reach maximum depth of scour is measured in hours and can result from a single flood event. With cohesive bed materials it will take much longer 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 effect 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 [19], 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 can have up to a 20 percent influence on scour depth. 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. The shape effect is neglected for flow angles in excess of five degrees. Full retaining abutments with vertical walls on the streamside (parallel to the flow) will produce scour depths about double that of spill-through (sloping) abutments.

9. Bed configuration of sand-bed channels affects the magnitude of local scour. In streams with sand-bed material, the shape of the bed (bed configuration) as described by Richardson et al. [20] may be ripples, dunes, plane bed or antidunes. The bed configuration depends on the size distribution of the sand-bed material, hydraulic characteristics, and fluid viscosity. 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 affect flow velocity, sediment transport, and scour. Richardson et al. [9] discusses bed configuration in detail.

10. Ice and debris can 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 effects 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 to 20 feet.

2.6 Clear-Water and Live-Bed Scour

There are two conditions for contraction and local scour. These are clear-water and live-bed scour. Clear-water scour occurs when there is no movement of the bed material in the flow 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) coarse 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 streambeds 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 that 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). This is because clear-water scour occurs mainly in coarse bed material streams. In fact, local clear-water scour may not reach a maximum until after several floods. Maximum local clear-water pier scour is about 10 percent greater than the equilibrium local live-bed pier scour.

The following equation suggested by Neill [21] for determining the velocity associated with initiation of motion can be used as an indicator for clear-water or live-bed scour.

$$V_c = 1.58[(S_s - 1)\, g\, D_{50}]^{\frac{1}{2}}\, (y/D_{50})^{\frac{1}{6}} \tag{12}$$

where

- V_c = critical velocity above which bed material of size D_{50} and smaller will be transported, ft/s
- S_s = specific gravity of bed material
- y = depth of flow, ft

For most bed material, the value of S_s is approximately 2.65. Substituting this into Equation 12 and consolidating the variables results in the following:

$$V_c = 11.52\, y^{\frac{1}{6}} D_{50}^{\frac{1}{3}} \tag{13}$$

Comparing Equation 13 and Laursen's equation (Equation 9), indicates that these two equations differ only by their respective coefficient (11.52 vs. 10.95). For practical considerations either equation can be used (when S_s is 2.65) for the determination of the critical velocity V_c associated with the initiation of motion.

Equations 13 or 9 can be applied to the unobstructed flow to determine whether or not the flow condition is live-bed or clear-water. If the average velocities in the cross section are greater than V_c the scour will be live-bed. The preceding technique can be applied to any unvegetated channel to determine whether a clear-water or live-bed condition is likely. This procedure should be used with caution for assessing whether or not scour in the overbank will be clear-water or live-bed. For most cases, the presence of vegetation on the overbank will effectively bind and protect the overbank from erosive velocities. As such, most overbank situations will experience clear-water scour.

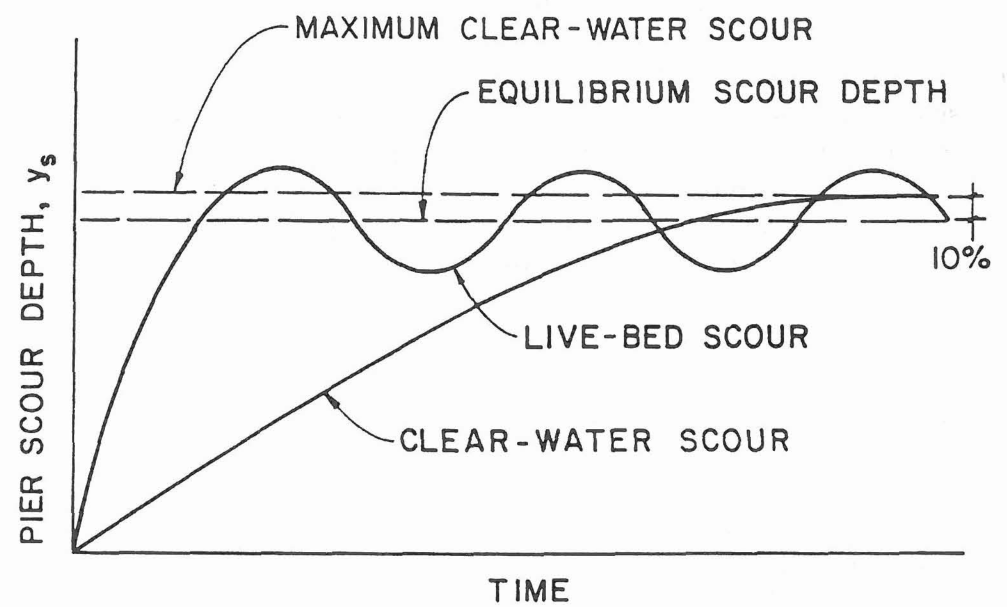

Figure 2. Illustrative Pier Scour Depth in a Sand-Bed Stream as a Function of Time. (not to scale)

Live-bed pier scour in sand-bed streams with a dune bed configuration fluctuates about the equilibrium scour depth (Figure 2). This is due to 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 and through the bridge), maximum depth of pier scour is about 30 percent larger than equilibrium depth of scour.

However, with the exception of crossings over large rivers (i.e., the Mississippi, Columbia, etc.), the bed configuration will plane out during flood flows due to the increase in velocity and shear stress. For general practices, the maximum depth of pier scour is approximately 10 percent greater than equilibrium scour. This is not illustrated in Figure 2.

For a discussion of bedforms in alluvial channel flow, the reader is referred to Chapter 3 of "Highways in the River Environment."[9] Equations for estimating local scour at abutments or piers are given in Chapter 4 of this document. These equations were developed from laboratory experiments and limited field data for both clear-water and live-bed scour.

2.7 Lateral Shifting of a Stream

Streams are dynamic. Areas of flow concentration continually shift bank lines. In meandering stream having an "S-shaped" planform, the channel moves both laterally and downstream. A braided stream has numerous channels which are continually changing. In a braided stream, the deepest natural scour occurs when two channels come together or when the flow comes together downstream of an island or bar. This scour depth has been observed to be 1 to 2 times the average flow depth.

A bridge is static. It fixes the stream at one place in time and space. A meandering stream whose channel moves laterally and downstream into the bridge reach can erode the approach embankment and affects contraction and local scour because of changes in flow direction. A braided stream can shift under a bridge and have two channels come together at a pier or abutment, increasing scour. Descriptions of stream morphology are given in "Highways in the River Environment" [9] and HEC-20.[8]

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

It is difficult to anticipate when a change in planform 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 are not easily determined. It is difficult to properly evaluate the vulnerability of a bridge due to changes in planform. It is important to incorporate potential planform changes into the design of new bridges and design of countermeasures for existing bridges.

Countermeasures for lateral shifting and instability of the stream may include changes in the bridge design, construction of river control works, protection of abutments with riprap, or careful monitoring of the river in a bridge inspection program. **Serious consideration should be given to placing footings/foundations located on floodplains 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 [8, 9, 22, 23, 28], U.S. Army Corps of Engineers [24, 25] and AASHTO.[26] Of particular importance are "Hydraulic Analyses for the Location and Design of Bridges," Volume VII-Highway Drainage Guidelines, 1992 [26], "Highways in the River Environment" [9]; "Use of Spurs and Guidebanks for Highway Crossings" [27], "Stream Stability at Highway Structures" HEC-20 [8], and "Design of Riprap Revetments" (HEC-11).[28]

2.8 Pressure Scour

When bridges are overtopped, the flow hydraulics at the bridge are dramatically altered, and local and contraction scour can be increased. This topic is discussed in greater detail in Section 4.3.5.

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CHAPTER 3

DESIGNING BRIDGES TO RESIST SCOUR

3.1 Design Philosophy and Concepts

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:

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

2. 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.

3. 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.

4. 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.

3.2 General Design Procedure

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

Step 1. Select the flood event(s) that are expected to produce the most severe scour conditions. Experience indicates that this is likely to be the 100-year flood or the overtopping flood when it is less than the 100-year flood. Check the 100-year flood or 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. Overtopping refers to flow over the approach embankment(s), the bridge itself or both.

Step 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 for Mainframe and Microcomputer" [29], or the Corps of Engineers HEC-2 [30], are recommended for this task.

Step 3. Using the 7-step Specific Design Approach in Chapter 4, estimate total scour for the worst condition from Steps 1 and 2 above. All foundations should be designed with a geotechnical safety factor ranging from 1.5 to 2, common geotechnical practice, for the 100-year or overtopping flood.

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

Step 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 judgment.

Step 6. Evaluate the bridge TS&L on the basis of the scour analysis performed in Steps 3 through 5. Modify the TS&L as necessary.

a. 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.

b. The extent of protection to be provided should be determined by:

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

Step 7. Perform the bridge foundation analysis on the basis that all streambed 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.[31] 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. Spread Footings On Soil

- Place the bottom of the footing below the total scour line from Step 4.

- Insure that the top of the footing is below the sum of the long-term degradation, contraction scour, and lateral migration.
local scour

b. Spread Footings On Rock Highly Resistant To Scour

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 clean 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. Guidance on scourability of rock formations is given in FHWA memorandum "Scourability of Rock Formations" dated July 19, 1991.

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

- Place the bottom of the footing below the total scour line from Step 4.

- Insure that the top of the footing is below the sum of the long-term degradation, contraction scour, and lateral migration.

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 long-term degradation and 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. Stub Abutments on Piling

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 bed material around the piling scours to the thalweg elevation.

Step 8. Repeat the procedure in Steps 2 through 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. If the magnitude of the 500-year flood is not available from a published source, use a discharge equal to 1.7 x Q_{100}. 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 less than the 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.

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

b. **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.

3.3 Checklist of Design Considerations

3.3.1 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.

   It is recommended that the elevation of the lower cord of the bridge be increased a minimum of 2 feet above the normal freeboard for the 100-year flood for streams that carry a large amount of debris.

2. Superstructures should be securely anchored to the substructure if buoyant, or if 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 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 is indeterminate and is deeper. The topwidth of a local scour hole ranges from 1.0 to 2.8 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. Section 4.3.5 has a discussion on pressure scour for these cases.

3.3.2 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 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 piers 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.

3.3.3 Abutments

1. Recognizing that abutment scour equations lack field verification, it is recommended that rock riprap and/or guide banks be considered for abutment protection. Properly designed, these two protective measures make it unnecessary to design abutments to resist the computed abutment scour depths. The design of rock riprap and guide banks is discussed in Section 7.5.

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

3. Where ice build-up is likely to be a problem, set the toe of spill-through slopes or vertical abutments back from the edge of the channel bank to facilitate passage of the ice.

4. Wherever possible, use spill-through (sloping) abutments. Scour at spill-through abutments is about 50 percent of that of vertical wall abutments.

CHAPTER 4

ESTIMATING SCOUR AT BRIDGES

4.1 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 for both riverine and tidal conditions at the end of the chapter. While the scour equations presented are based on riverine conditions, they are also recommended for tidal waterways. Section 4.6 discusses hydrodynamics and scour methodologies for tidal waterways.

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

4.2 Specific 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 piers.

Step 6: Compute the magnitude of local scour at abutments.

Step 7: Plot and evaluate the total scour depths as outlined in Steps 4 through 6 of the General Design Procedure in Chapter 3.

The engineer should evaluate how reasonable the individual estimates of contraction and local scour depths are in Steps 4 and 5 as well as evaluating the reasonableness of the total scour in Step 7. The results from this Specific Design Approach completes Steps 1 through 6 of Chapter 3. The design must now proceed to Steps 7 and 8 of the General Design Procedure in Chapter 3.

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

4.3 Detailed Procedures

4.3.1 Step 1: Determine Scour Analysis Variables

1. Determine the magnitude of the discharges for the floods in Steps 1 and 8 of the General Design Procedure in Chapter 3, including the overtopping flood when applicable. If the magnitude of the 500-year flood is not available from a published source, use a discharge equal to 1.7 times the 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 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 contraction and expansion of the flow in the bridge waterway. Consider present conditions and anticipated future changes in the river.

3. Determine the water-surface profiles for the discharges judged to produce the most scour from Step 1, using WSPRO [29] or HEC-2.[30] In some instances, the designer may wish to use BRI-STARS.[10] Hydraulic studies by the Corps of Engineers, U.S. Geological Survey (USGS), the Federal Emergency Management Agency (FEMA), etc. are potentially useful sources of hydraulic data to calibrate, verify, and evaluate results from WSPRO or HEC-2. The engineer should anticipate future conditions at the bridge, in the stream's watershed, and at downstream water-surface elevation controls as outlined in HEC-20.[8] From computer analysis and from other hydraulic studies, determine the discharge, velocity and depth input variables needed for the scour calculations.

4. Collect and summarize the following information as appropriate (see HEC-20 for a step-wise analysis procedure).

   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 floodplain cross section through the reach.

   d. Stream planform.

   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. History of flooding.

i. 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.

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

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

l. Erosion history of the stream.

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

n. Sand and gravel mining from the streambed upstream and downstream from site.

o. Other factors that could affect the bridge.

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

4.3.2 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 streambed elevation. The Corps of Engineers and other agencies may have information on historic and current streambed elevations. 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 sediment routing or sediment continuity computer programs such as BRI-STARS [10] and the Corps of Engineers HEC-6 [11],

   b. Straight line extrapolation of present trends,

   c. Engineering judgment,

d. The worst-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 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 low cord of the bridge. With an aggrading stream, use the present streambed elevation as the baseline for scour estimates because a major flood can occur prior to aggradation.

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

4.3.3 Step 3: Evaluate the Scour Analysis Method

The recommended 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.

1. Estimate the natural channel hydraulics for a fixed-bed condition based on existing conditions,

2. Assess the expected profile and planform changes,

3. Adjust the fixed-bed hydraulics to reflect any expected long-term profile or planform changes,

4. Estimate contraction scour using the empirical contraction formula and the adjusted fixed-bed hydraulics (see Step 4 below),

5. Estimate local scour using the adjusted fixed-bed channel and bridge hydraulics (see Steps 5 and 6 below), and

6. Add the local scour to the contraction scour to obtain the total scour. (see Chapter 3, General Design Procedure, Step 4 or, Chapter 4, Step 7 of the Specific Design Procedure).

4.3.4 Step 4: Compute the Magnitude of Contraction Scour

General. In the previous edition of this circular, and in the Interim Procedures [7], contraction scour at bridge sites was broken down into four conditions (cases) depending on the type of contraction, overbank flow, or relief bridges. Then specific equations were presented for the different cases. However, all conditions of contraction scour can be evaluated using two basic equations: (1) an equation for live-bed scour, and (2) an equation for clear-water scour. For any case or condition, it is only necessary to determine if the flow in the main channel or overbank area upstream of the bridge, or approaching a relief bridge, is transporting bed material (live-bed) or is not (clear-water), and then apply the appropriate equation with the variables defined according to the location of contraction scour (channel or overbank).

To determine if the flow upstream of the bridge is transporting bed material, calculate the critical velocity for beginning of motion V_c and compare it with the mean velocity V of the flow in the main channel or overbank area upstream of the bridge opening. If the critical velocity of the bed material is larger than the mean velocity ($V_c > V$), then clear-water contraction scour will exist. If the critical velocity is less than the mean velocity ($V_c < V$), then live-bed contraction scour will exist. To calculate the critical velocity use either Neill's [21] or Laursen's [13] equation given in Chapter 2. These equations are reiterated as follows:

Neill's equation with S_s equal to 2.65

$$V_c = 11.52 \, y_1^{\frac{1}{6}} \, D_{50}^{\frac{1}{3}} \tag{14}$$

where

- V_c = critical velocity which will transport bed materials of size D_{50} and smaller, ft/s
- S_s = specific gravity of bed material
- y_1 = depth of upstream flow, ft

Laursen's equation with S_s equal to 2.65

$$V_c = 10.95 \, y_1^{\frac{1}{6}} \, D_{50}^{\frac{1}{3}} \tag{15}$$

Contraction Scour Conditions. Four conditions (cases) of contraction scour (see illustrations in Appendix H) are commonly encountered:

Case 1. Involves overbank flow on a floodplain being forced back to the main channel by the approaches to the bridge. Case 1 conditions include:

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;

b. No contraction of the main channel, but the overbank flow area is completely obstructed by an embankment; or

c. Abutments are set back from the stream channel.

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 is 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).

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

Notes:

1. **Cases 1, 2, and 4** may either be live-bed or clear-water scour depending on whether there is bed material transport from the upstream reach into the bridge reach during flood flows. To determine if there is bed material transport compute the critical velocity for the D_{50} of the bed material using either Neill's or Laursen's equation given above (Equations 14 or 15) and compare to the critical velocity.

2. **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, and (5) the distribution of the flow in the bridge section, and (6) other factors.

   The main channel under the bridge may be live-bed scour, whereas the set-back overbank area may be clear-water scour.

   A water surface model like WSPRO [29] can be used to determine the distribution of flow between the main channel and the set-back overbank areas in the contracted bridge opening.

   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. Consideration should be given to using a guide bank and/or rock riprapping the bank and bed under the bridge in the overflow area.

3. **Case 3** may be clear-water scour even though the floodplain bed material is composed of fine sediments with a critical velocity that is less than the flow velocity in the overbank area. The reasons for this are (1) there may be vegetation growing part of the year, and (2) the fine bed material may go into suspension (wash load) at the bridge and not influence the contraction scour.

4. **Case 4** is similar to Case 3, but there is sediment transport into the relief bridge opening (live-bed scour). This case can occur when a relief bridge is over a secondary channel on the floodplain. Hydraulically this is no different from Case 1, but analysis is required to determine the floodplain width associated with the relief opening and

the flow distribution going to and through the relief bridge. This information could be obtained from WSPRO.[29]

Live-Bed Contraction Scour. A modified version of Laursen's 1960 equation [12] for live-bed scour at a long contraction is recommended to predict the depth of scour in a contracted section. The original equation is given in Chapter 2. The modification is to eliminate the ratio of Manning's n. The equation assumes that bed material is being transported in the upstream section.

$$\frac{y_2}{y_1} = \left(\frac{Q_2}{Q_1}\right)^{\frac{6}{7}} \left(\frac{W_1}{W_2}\right)^{k_1} \tag{16}$$

$$y_s = y_2 - y_1 = (\textit{average scour depth}) \tag{17}$$

where

- y_1 = average depth in the upstream main channel, ft
- y_2 = average depth in the contracted section, ft
- W_1 = bottom width of the upstream main channel, ft
- W_2 = bottom width of the main channel in the contracted section, ft
- Q_1 = flow in the upstream channel transporting sediment, cfs
- Q_2 = flow in the contracted channel, cfs
- k_1 = exponent determined below

V_*/w	k_1	Mode of Bed Material Transport
<0.50	0.59	mostly contact bed material discharge
0.50 to 2.0	0.64	some suspended bed material discharge
>2.0	0.69	mostly suspended bed material discharge

- V_* = $(\tau/\rho)^{1/2} = (gy_1\, S_1)^{1/2}$, shear velocity in the upstream section, ft/s
- w = fall velocity of bed material based on the D_{50}, ft/s (see **Figure 3**)
- g = acceleration of gravity (32.2 ft/s^2)
- S_1 = slope of energy grade line of main channel, ft/ft
- τ = shear stress on the bed, lb/ft^2
- ρ = density of water (1.94 slugs/ft^3)

Notes:

1. Q_2 may be the total flow going through the bridge opening as in Cases 1a and 1b. It is not the total for Case 1c.

2. Q_1 is the flow in the main channel upstream of the bridge, not including overbank flows.

3. The Manning's n ratio can be significant for a condition of dune bed in the main channel and a corresponding plane bed, washed out dunes or antidunes in the contracted channel. 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, increases the 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 would be an increase in scour depth. In addition, at flood flows, a plane bedform will usually exist upstream and through the bridge waterway, and the values of Manning's n will be equal. Consequently, the n value ratio is not recommended or presented in the recommended Equation 16.

4. W_1 and W_2 are not always easily defined. In some cases, it is acceptable to use the top width of the main channel to define these widths. Whether top width or bottom width is used, it is important to be consistent so that W_1 and W_2 refer to either bottom widths or top widths.

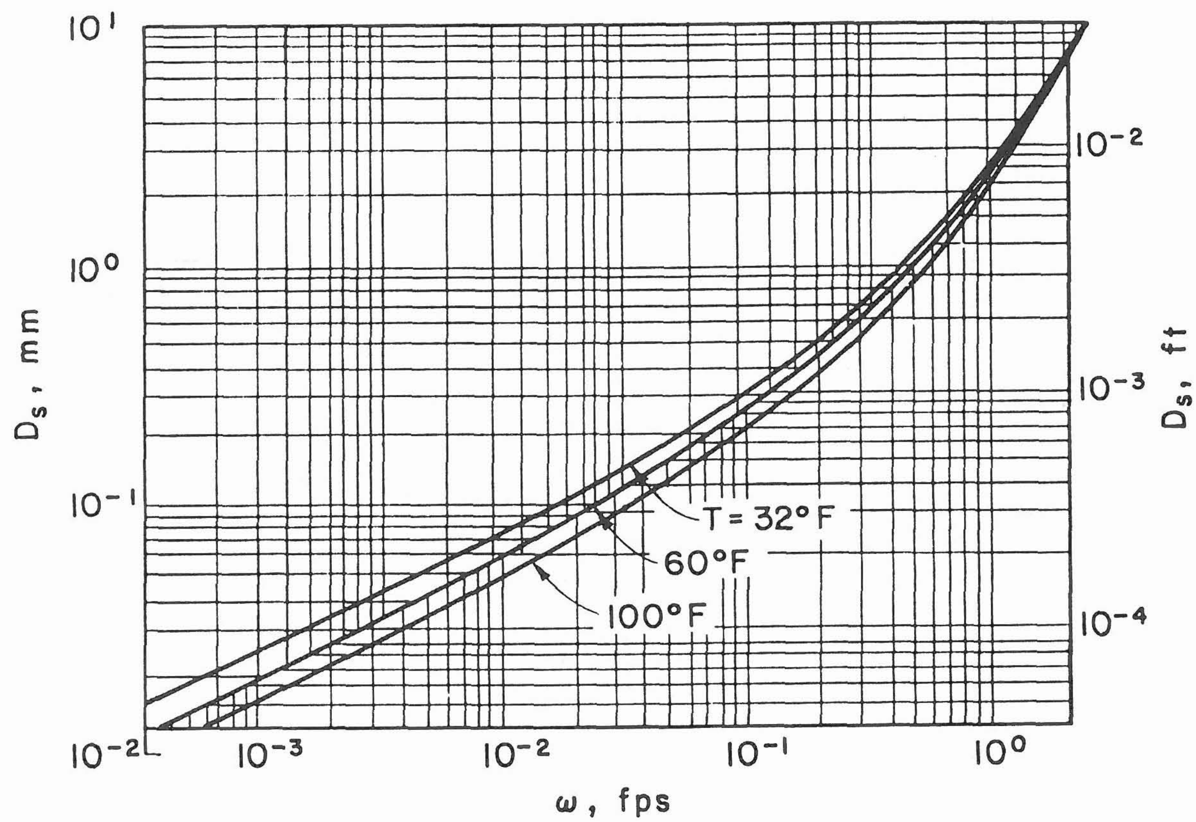


Figure 3. Fall Velocity of Sand-Sized Particles.

4. The average width of the bridge opening (W_2) 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 a natural contraction or if the contraction is the result of the bridge abutments and piers. At this time, however, it is the best equation available.

Clear-Water Contraction Scour. The recommended clear-water contraction scour equation is based on Laursen.[13] This was presented as Equation 8 in Chapter 2:

$$y_2 = \left[\frac{Q^2}{120\ D_m^{\frac{2}{3}}\ W^2}\right]^{\frac{3}{7}} \tag{18}$$

$$y_s = y_2 - y_1 = (\textit{average scour depth}) \tag{19}$$

where

y_1 = Depth of flow in the channel or on the floodplain prior to scour, ft
y_2 = Depth of flow in the bridge opening or on the overbank at the bridge, ft
y_s = Depth of scour, ft
Q = Discharge through the bridge or on the overbank at the bridge, cfs
D_m = Effective mean diameter (feet) of the bed material (1.25 D_{50}) in the bridge, opening or on the floodplain, ft
D_{50} = Median diameter (feet) of bed material in the bridge opening, or on the floodplain, ft
W = Bottom width of the bridge less pier widths, or overbank width (set back distance), ft

It should be noted that the recommended clear-water scour equation (Equation 18) differs from the original development by Laursen (Equation 8) in the use of the effective mean bed material, D_m instead of the D_{50}. This change is the result of subsequent research discussed in HIRE [9] and tends to reduce the computed clear-water contraction scour.

Equations 18 and 19 can be combined to form a single equation for computation of clear-water contraction scour:

$$\frac{y_s}{y_1} = 0.13\left[\frac{Q}{D_m^{\frac{1}{3}}\ y_1^{\frac{7}{6}}\ W}\right]^{\frac{6}{7}} - 1 \tag{20}$$

Note that for stratified bed material the depth of scour can be determined by using Equations 18 or 20 sequentially with successive D_m of the bed material layers.

Other Contraction Scour Conditions. 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 insofar as scour processes are concerned. Use the WSPRO [29] 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 toward the outside of the bend is analyzed by determining the super-elevation 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 site-specific conditions. Such studies should be undertaken by engineers experienced in the fields of hydraulics and river mechanics.

4.3.5 Step 5: Compute the Magnitude of Local Scour at Piers

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, which give similar results, are for live-bed scour in cohesionless sand-bed streams.

The FHWA [32] compared many of the more common equations in 1983. Comparison of these equations is given in **Figures 4 and 5**. An equation given by Melville and Sutherland [17] to calculate scour depths for live-bed scour in sand-bed streams has been added to the original figures. Some of the equations have velocity as a variable, normally in the form of a Froude Number. However, some equations, such as Laursen's [12] do not include velocity. A Froude Number of 0.3 was used in Figure 4 for purposes of comparing commonly used scour equations. In Figure 5, the equations are compared with some field data measurements. As can be seen from Figure 5, the Colorado State University (CSU) equation envelopes all the points, but gives lower values of scour than Jain and Fischer's [22], Laursen's [33], Melville and Sutherland's [17], and Neill's [21] equations. The CSU equation [9] includes the velocity of the flow just upstream of the pier by including the Froude Number in the equation. Chang [34] pointed out that Laursen's 1960 equation is essentially a special case of the CSU equation with the Fr = 0.4 (See **Figure 6**).

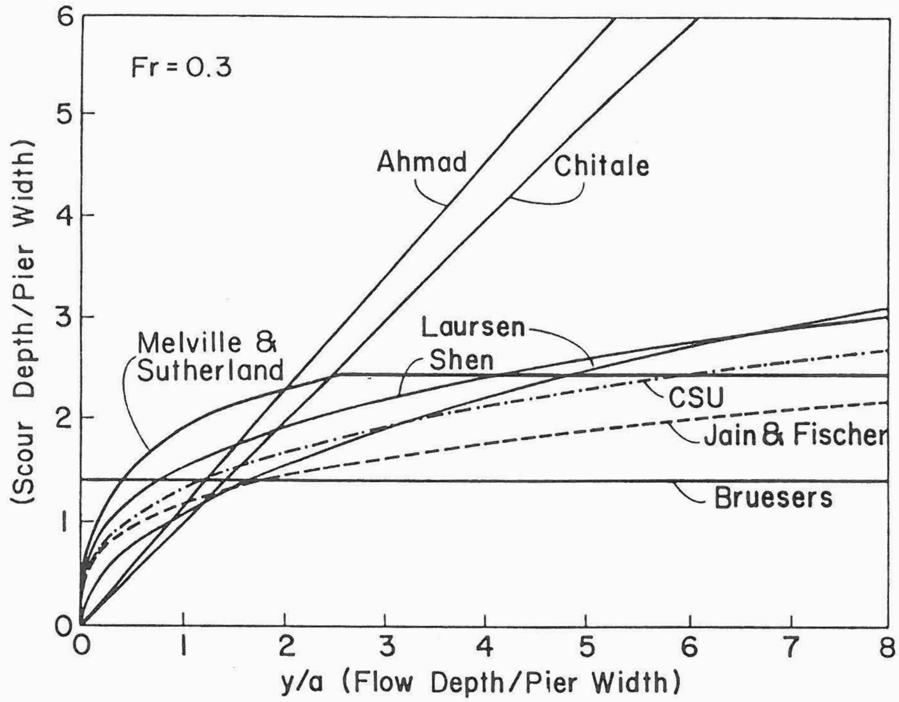


Figure 4. Comparison of Scour Formulas for Variable Depth Ratios (y/a) after Jones [32].

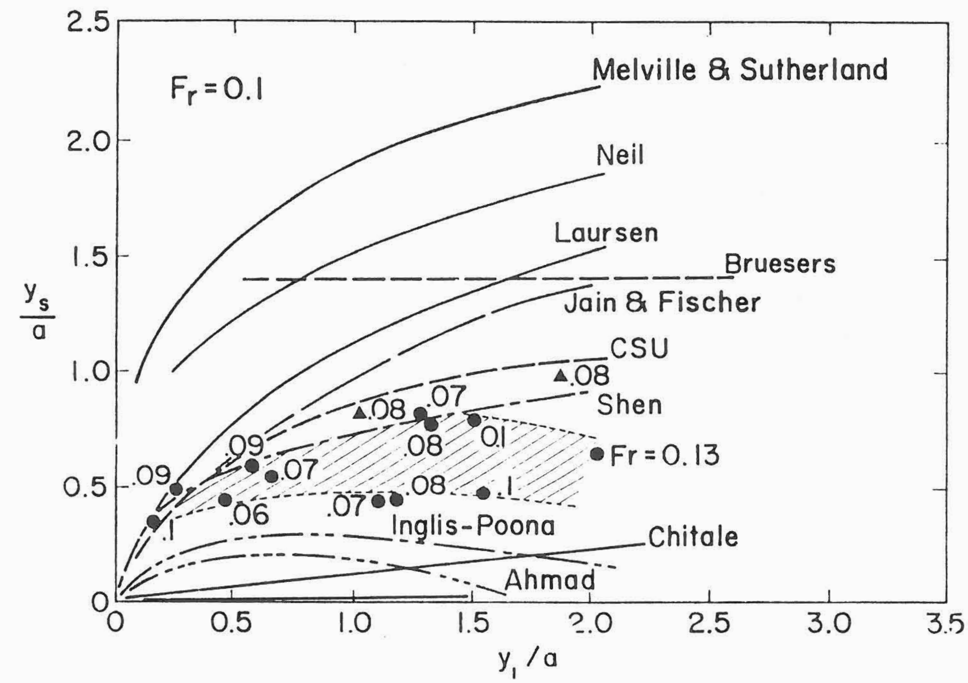


Figure 5. Comparison of Scour Formulas with Field Scour Measurements after Jones [32].

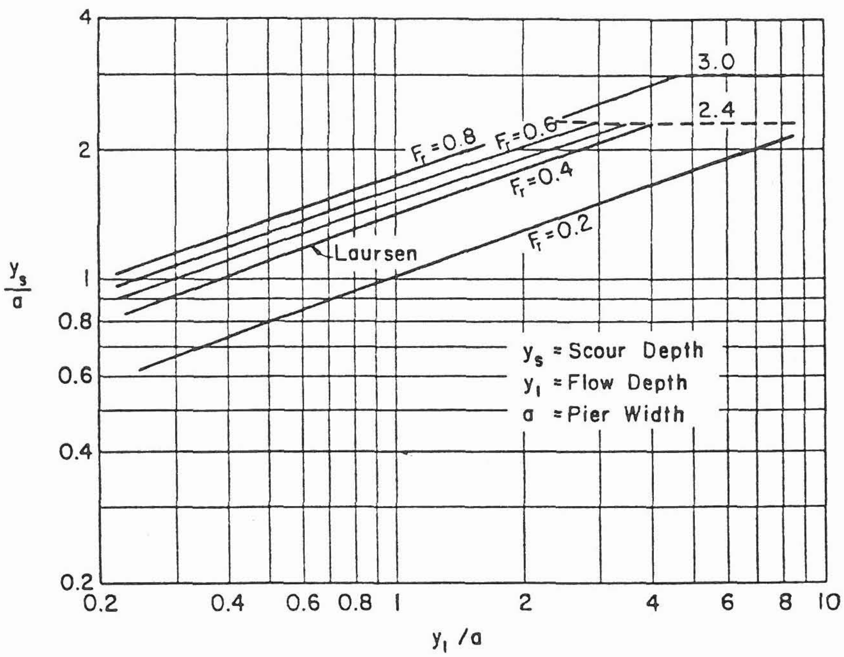


Figure 6. Values of y_s/a vs. y_1/a for CSU's Equation [34].

The equations illustrated in Figures 4, 5, and 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 may at some depth of scour limit the scour depth. Raudkivi [15], Melville and Sutherland [17], and others [14, 16] developed equations based on laboratory and limited field data which take into consideration large particles in the bed. Most of the field scour depths were measured after the flood had occurred and the depths were not representative of the flow conditions that caused them. 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.**

In Figure 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 Chang.[34] 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 around 3.0 were obtained by Jain and Fischer [22] 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 (plane bed, antidunes, and chutes and pools).

Chang [34] noted that in all the data he studied, there were no values of the ratio of scour depth to pier width (y_s/a) larger than 2.3. From laboratory data, Melville and Sutherland [17] reported 2.4 as an upper limit ratio for cylindrical piers. In these studies, the Froude Number was less than 1.0. These upper limits were derived for circular piers and were uncorrected for pier shape and for skew. Also, pressure flow or debris can increase the ratio.

From the above discussion, the ratio of y_s/a can be as large as 3 at large Froude Numbers. Therefore, it is recommended that the maximum value of the ratio is taken as

2.4 for Froude Numbers less than or equal to 0.8 and 3.0 for larger Froude Numbers. These limiting ratio values apply only to round nose piers which are aligned with the flow.

To determine pier scour, the CSU equation [9] is recommended for both live-bed and clear-water pier scour. The equation predicts equilibrium pier scour depths. For plane-bed conditions, which is typical of most bridge sites for the flood frequencies employed in scour design, the maximum scour may be 10 percent greater than computed with CSU's equation. In the unusual situation where a dune bed configuration with large dunes exists at a site during flood flow, the maximum pier scour may be 30 percent greater than the predicted equation value. This may occur on very large rivers, such as the Mississippi. For smaller streams that have a dune bed configuration at flood flow, the dunes will be smaller and the maximum scour may be only 10 to 20 percent larger than equilibrium scour. For antidune bed configuration the maximum scour depth may be 10 percent greater than the computed equilibrium pier scour depth. In **Table 1** values of the percent increase in equilibrium pier scour depths calculated with the CSU equation are given as a function of dune height H. These increases are tabulated as a correction (K_3) to the CSU equation.

Table 1. Increase in Equilibrium Pier Scour Depths (K_3) for Bed Condition.

Bed Condition	Dune Height H ft.	K_3
Clear-Water Scour	N/A	1.1
Plane bed and Antidune flow	N/A	1.1
Small Dunes	10> H < 2	1.1
Medium Dunes	30> H >10	1.1 to 1.2
Large Dunes	H >30	1.3

Computing Pier Scour. The CSU equation for pier scour is:

$$\frac{y_s}{y_1} = 2.0\ K_1\ K_2\ K_3 \left(\frac{a}{y_1}\right)^{0.65} Fr_1^{0.43} \qquad (21)$$

In terms of y_s/a, Equation 21 is:

$$\frac{y_s}{a} = 2.0\ K_1\ K_2\ K_3 \left(\frac{y_1}{a}\right)^{0.35} Fr_1^{0.43} \qquad (22)$$

where

y_s = scour depth, ft
y_1 = flow depth directly upstream of the pier, ft
K_1 = correction factor for pier nose shape from **Figure 7** and **Table 2**

K_2 = correction factor for angle of attack of flow from **Table 3**
K_3 = correction factor for bed condition from Table 1
a = pier width, ft
L = length of pier ft
Fr_1 = Froude Number = $V_1/(gy_1)^{1/2}$
V_1 = Mean velocity of flow directly upstream of the pier, ft/s

Table 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 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	2.75	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 2 for angles of attack up to 5 degrees. For greater angles, K_2 dominates and K_1 should be considered as 1.0. If L/a is larger than 12, use the values for L/a = 12 as a maximum.

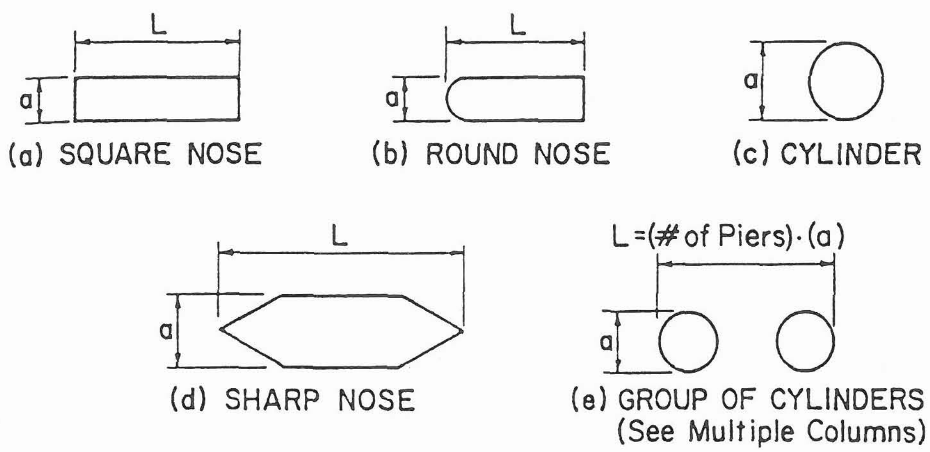


Figure 7. Common Pier Shapes.

Pier Scour for Exposed Footings. Pier footings and/or pile caps may become exposed to the flow by scour. This may occur either from long-term degradation, contraction 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 appears 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.[35] It appeared that the footing decreased the potential scour depth.

A model study of scour at the Acosta Bridge at Jacksonville, Florida, by Jones [36] found that when the top of the footing was flush with the streambed, 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 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.

> "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 (or pile cap) is at or below the streambed (after taking into account long-term degradation and contraction scour). If the pier footing extends above the streambed, make a second computation using the width of the footing for the value of "a" and 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."

If the top of the footing or pile cap is at the long-term degradation and/or contraction scour elevation then it is only necessary to compute the scour depth considering the pier width.

Determine the average velocity of flow at the exposed footing (V_f) using the following equation:

$$\frac{V_f}{V_1} = \frac{\ln\left(10.93\frac{y_f}{k_s}+1\right)}{\ln\left(10.93\frac{y_1}{k_s}+1\right)} \tag{23}$$

where

V_f = average velocity in the flow zone below the top of the footing, ft/s
y_f = distance from the bed to the top of the footing, ft
k_s = the grain roughness of the bed. Normally taken as the D_{84} of the bed material, ft
y_1 = depth of flow upstream of the pier, ft

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

Pier Scour for Exposed Pile Groups. Experiments were conducted by Jones [36] to determine guidelines for specifying the characteristic width of a pile group (**Figure 8**) that are or may be exposed to the flow (as the result of long-term degradation and/or contraction scour) when the piles are spaced laterally as well as longitudinally in the streamflow. The following was concluded:

> "**Pile groups that project above the streambed [as the result of long-term degradation and/or contraction scour] can be analyzed conservatively by representing them as a single width equal to the projected area of the piles ignoring the clear space between piles.** Good judgment 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."

If the pile group is exposed to the flow as the result of local scour then it is unnecessary to consider the piles in calculating pier scour.

For example, five 16-inch cylindrical piles spaced at 6 feet (Figure 8) would have an 'a' value of 6.67 feet. This composite pier width would be used in Equation 21 to determine depth of pier scour. The correction factor K_1 in Equation 21 for the multiple piles would be 1.0 regardless of shape. If the pile group is a square as in Figure 8 then K_2 would be 1.0. However, if the pile group is a rectangle use the dimensions as if they were a single pier and the appropriate L/a value for determining K_2.

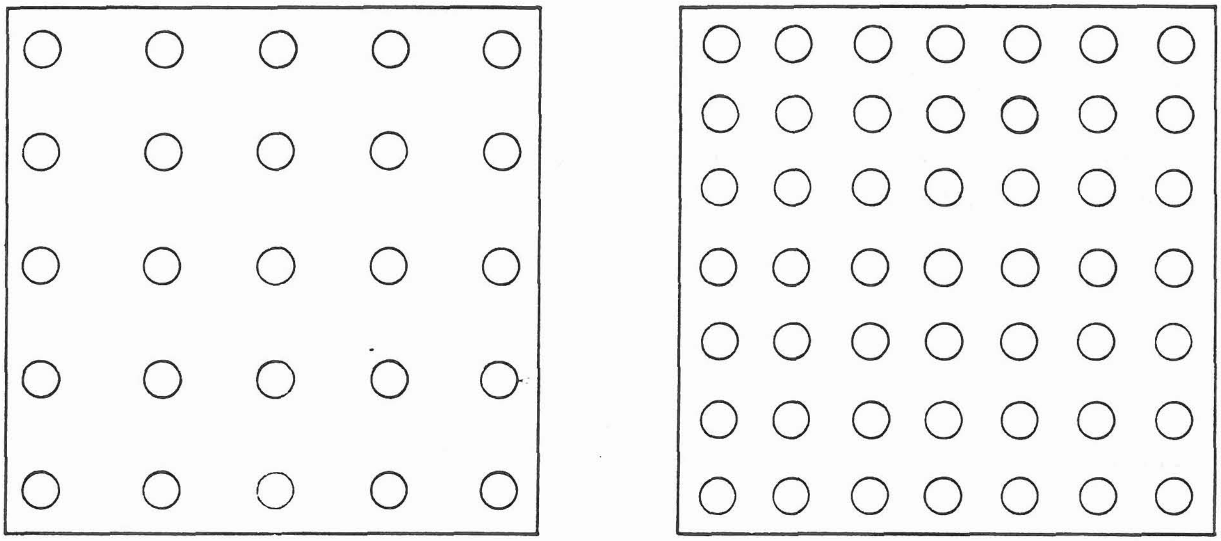


Figure 8. Pile Groups.

The depth of scour for exposed pile groups will be analyzed in this manner except when addressing the effect 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 3.

Pile Caps Placed at the Water Surface or in the Flow. For pile caps placed at or near the water surface or in the flow, it is recommended that the scour analysis include computation of scour caused by the exposed pile group, computation of the pier scour caused by the pile cap and pier scour caused by the pier if the pier is partially submerged in the flow. A conservative estimate of local scour will be the largest pier scour computed from these three scenarios.

When computing the pier scour caused by the pile cap, assume that the pile cap is resting on the bed and use the previously defined values of V_f and y_f in the CSU equation. Use the CSU equation for the pier shaft and exposed pile groups as recommended in the previous discussions.

Multiple Columns. For multiple columns (as illustrated as a group of cylinders in Figure 7) skewed to the flow, the scour depth depends on the spacing between the columns. The correction factor for angle of attack would be smaller than for a solid pier. How much smaller is not known. Raudkivi [15] 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 spaced less than 5 pier diameters apart, 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 21 to determine depth of pier scour.** The correction factor K_1 in Equation 21 for the multiple column would be 1.0 regardless of column shape. The coefficient K_2 would also be equal to 1.0 since the effect of skew would be accounted for by the projected area of the piers normal to the flow.

The depth of scour for a multiple column bent will be analyzed in this manner except when addressing the effect 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 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.

Pressure Flow Scour. Pressure flow, which is also denoted as orifice flow, occurs when the water surface elevation at the upstream face of the bridge is greater than or equal to the low chord of the bridge superstructure. Pressure flow under the bridge results from a pile up of water on the upstream bridge face, and a plunging of the flow downward and under the bridge. At higher approach flow depths, the bridge can be entirely submerged with the resulting flow being a complex combination of the plunging flow under the bridge (orifice flow) and flow over the bridge (weir flow).

In many cases, when a bridge is submerged, flow will also overtop adjacent approach embankments. This highway approach overtopping, is also weir flow. Hence, for any overtopping situation, the total weir flow can be subdivided into weir flow over the bridge and weir flow over the approach. Weir flow over approach embankments serves to reduce the discharge which must pass either under or over the bridge. In some cases, when the approach embankments are lower than the low chord of the bridge, the relief obtained from overtopping of the approach embankments will be sufficient to prevent the bridge from being submerged.

The hydraulic bridge routines of either WSPRO and HEC-2 are suitable for determination of the amount of flow which will flow over the roadway embankment, over the bridge as weir flow, and through the bridge opening as orifice flow, provided that the top of the highway is properly included in the input data. These models can be used to determine average flow depths and velocities over the road and bridge, as well as average velocities under the bridge.

With pressure flow, the local scour depths at a pier or abutment are larger than for free surface flow with similar depths and approach velocities. The increase in local scour at a pier subjected to pressure flow results from the flow being directed downward towards the bed by the superstructure (vertical contraction of the flow) and by increasing the intensity of the horseshoe vortex. The vertical contraction of the flow is a more significant cause of the increased scour depth. However, in many cases, when a bridge becomes submerged, the average velocity under the bridge is reduced due to a combination of additional backwater caused by the bridge superstructure impeding the flow, and a reduction of the discharge which must pass under the bridge due to weir flow over the bridge and approach embankments. As a consequence of this, **increases in local scour attributed to pressure flow scour at a particular site, may be offset to a degree by lesser velocities through the bridge opening due to increased backwater and a reduction in discharge under the bridge due to overtopping of the bridge and approach embankments.**

The average flow depth to be used to estimate scour depths should be measured to the highest portion of the bridge superstructure blocking the flow. Flow depths in excess of this elevation can be neglected in the scour computations because this excess depth is attributed to the weir flow over the bridge and not the orifice flow under the bridge. It should be noted that an open guardrail can be plugged with debris. If debris clogging is likely, the flow depth used in the scour equations should be measured from the stream bed to the top of the clogged guardrail.

The discharge through the bridge, approach velocity, and depths for calculating contraction and local scour can be obtained by using WSPRO or HEC-2 computer programs. Both programs have bridge routines with combined orifice (pressure) and weir flows. **It is highly recommended that WSPRO be used to analyze the scour problem when the bridge is overtopped with or without overtopping of the approach roadway.**

The worst case pressure scour problem normally occurs when all the flow must pass through the bridge and there is no relief from flow over the bridge or approach roadway and no backwater from downstream controls. This case was studied in a limited flume study at Colorado State University in Spring 1990. [37,38] In this study, a single pier with a simulated bridge deck was investigated in the flume. The height of the bridge deck above the bed was adjusted for each simulation so that the upstream face of the bridge deck was partially submerged (no flow over the top of the simulated bridge deck). The discharges used for this study provided a range of approach flow depths, and approach velocities. For all of the simulations, the underside of the bridge deck was, for the most part, in contact with the flow. There was no sediment transport upstream of the bridge (clear-water scour).

With the underside of the deck submerged, local pier scour depths calculated using the CSU equation were increased by a factor of 1 at an approach Froude Number of 0.13 to a factor of 1.6 at a Froude Number of 0.59. These results were obtained by comparison of scour depths for free surface and pressure flow simulations with similar hydraulic characteristics. The magnitude of the increase in local pier scour, as expected, depended on the velocity of the approach flow and the distance from the deck to the bed. For the same approach velocity, local pier scour increased as the distance from the bed to the deck decreased. Although not tested, it is possible that the local scour at a pier resulting from pressure flow would decrease if the flow overtops the bridge. Further analysis of the results of these experiments and additional laboratory studies will be necessary to define the impact of bridge submergence on local scour.

It is recommended that WSPRO or HEC-2 be used to determine the discharge through the bridge and the velocity of approach and depth upstream of the piers when flow impacts the bridge superstructure. These values should be used to calculate local pier scour. Engineering judgment would then be exercised to determine the appropriate multiplier times the calculated pier scour depth for the pressure flow scour depth. This ranges from 1.0 for low approach Froude Numbers (Fr = 0.1) to 1.6 for high approach Froude Numbers (Fr = 0.6). If the bridge is overtopped, the depth (y) to be used in the pier scour equations and for computing the Froude Number is the depth to the top of the bridge deck or guard rail obstructing the flow.

Scour from Debris on Piers. Debris lodged on a pier also increases local scour at a pier. This has the effect of increasing pier width with resultant increase in velocity and greater component of flow deflected downward. This increases the transport of sediment out of the scour hole. When floating debris is lodged on the pier, the scour depth is estimated by assuming that the pier width is larger than the actual width. The problem is in determining the increase in pier width to use in the pier scour equation. Furthermore, at large depths, the effect of the debris on the scour depths should diminish.

As with estimating local scour depths with pressure flow, only limited research has been done on local scour with debris. Melville and Dongol [39] have conducted a limited quantitative study of the effect of debris on local pier scour and have made some recommendations. However, additional laboratory studies will be necessary to better define the influence of debris on local scour.

Width of Scour Holes. The topwidth of a scour hole in cohesionless bed material from one side of a pier or footing can be estimated from the following equation:

$$W = y_s (K + \cot \theta) \tag{23}$$

where

W = topwidth of the scour hole from each side of the pier or footing, ft
y_s = scour depth, ft
K = bottom width of the scour hole as a fraction of scour depth
θ = Angle of repose of the bed material and ranges from about 30° to 44°

If the bottom width of the scour hole is equal to the depth of scour y_s (K = 1) the topwidth in cohesionless sand would vary from 2.07 to 2.80 y_s. At the other extreme if K = 0, the topwidth would vary from 1.07 to 1.8 y_s. Thus, the topwidth could range from 1.0 to 2.8 y_s and will depend on the bottom width of the scour hole and composition of the bed material. In general, the deeper the scour hole, the smaller the bottom width. A topwidth of 2.8 y_s is suggested for practical application.

4.3.6 Step 6: Local Scour at Abutments

General. Equations for predicting abutment scour depths are based entirely on laboratory data. For example, equations by Liu et al. [40], Laursen [33], Froehlich [41], and Melville [42] are based entirely on laboratory data. The problem is that little field data on abutment scour exist. Liu et al.'s equations were developed by dimensional analysis of the variables with a best-fit line 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 equation was derived from dimensional analysis and regression analysis of the available laboratory data. Melville's equations were derived from dimensional analysis and development of relations between dimensionless parameters using best-fit lines through laboratory data.

All equations in the literature were developed using the abutment and roadway approach length as one of the variables and result in excessively conservative estimates of scour depth. As Richardson and Richardson [43] point out in a discussion of Melville's (1992) paper,

> "The reason the equations in the literature predict excessively conservative abutment scour depths for the field situation is that, in the laboratory flume, the discharge intercepted by the abutment is directly related to the abutment length; whereas, in the field, this is rarely the case."

Figure 9 illustrates the difference. Thus, using the abutment length in the equations instead of the discharge returning to the main channel at the abutment results in a spurious correlation between abutment lengths and scour depth at the abutment end.

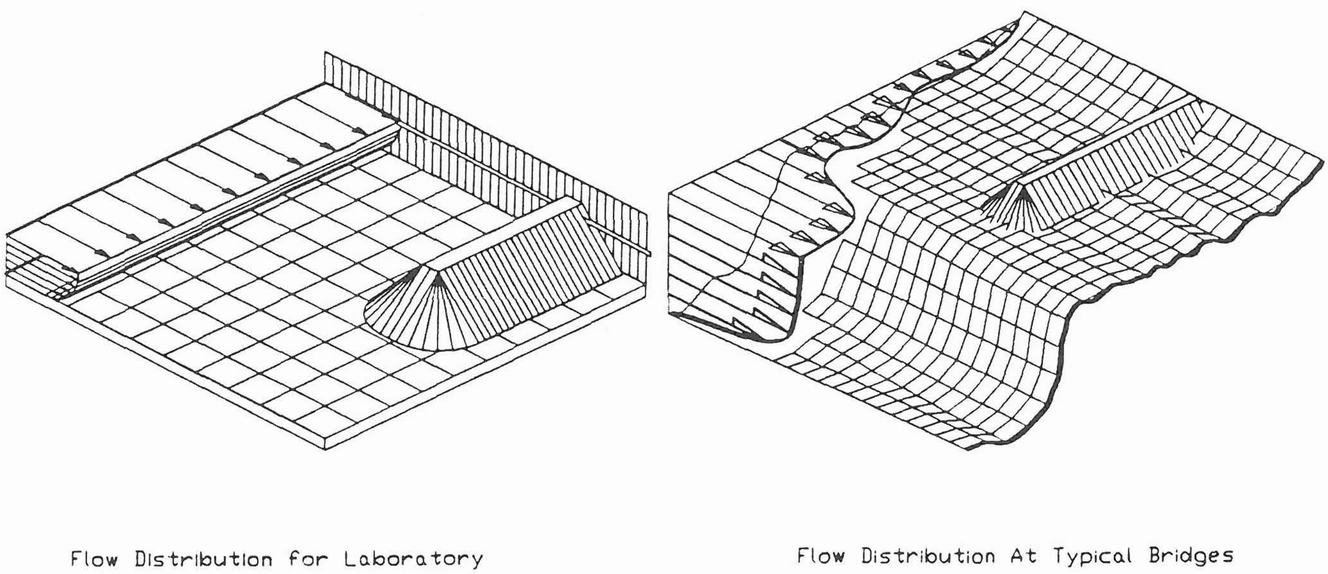


Figure 9. Comparison of Laboratory Flow Characteristics to Field Conditions.

Abutment scour depends on the interaction of the flow obstructed by the abutment and roadway approach and the flow in the main channel at the abutment. The discharge returned to the main channel at the abutment is not simply a function of the abutment and roadway length in the field case. Richardson and Richardson [43] noted that abutment scour depth depends on abutment shape, sediment characteristics, cross-sectional shape of the main channel at the abutment (especially the depth of flow in the main channel and depth of the overbank flow at the abutment), alignment, etc. In addition, field conditions may have tree lined or vegetated banks, low velocities, and shallow depths upstream of the abutment. **Research to date has failed to replicate these field conditions.**

Therefore, engineering judgment is required in designing foundations for abutments. In many cases, foundations can be designed with shallower depths than predicted by the equations when the foundations are protected with rock riprap placed below the streambed and/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 HEC-20.[8]

In the following sections, two equations are presented for use in estimating scour depths as a guide in designing abutment foundations. As stated above, these equations give excessively conservative estimates of scour depths.

Abutment Site Conditions. Abutments can be set back from the natural streambank or 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. More severe abutment scour will occur when the majority of overbank flow returns to the bridge opening directly upstream of the bridge crossing. Less severe abutment scour will occur when overbank flows gradually return to the main channel upstream of the bridge crossing.

Abutment Shape. There are three general shapes for abutments: (1) spill-through abutments, (2) vertical-wall abutments with wing walls (**Figure 10**), and (3) vertical walls without wing walls. Depth of scour is approximately double for vertical-wall abutments as compared with spill-through abutments.

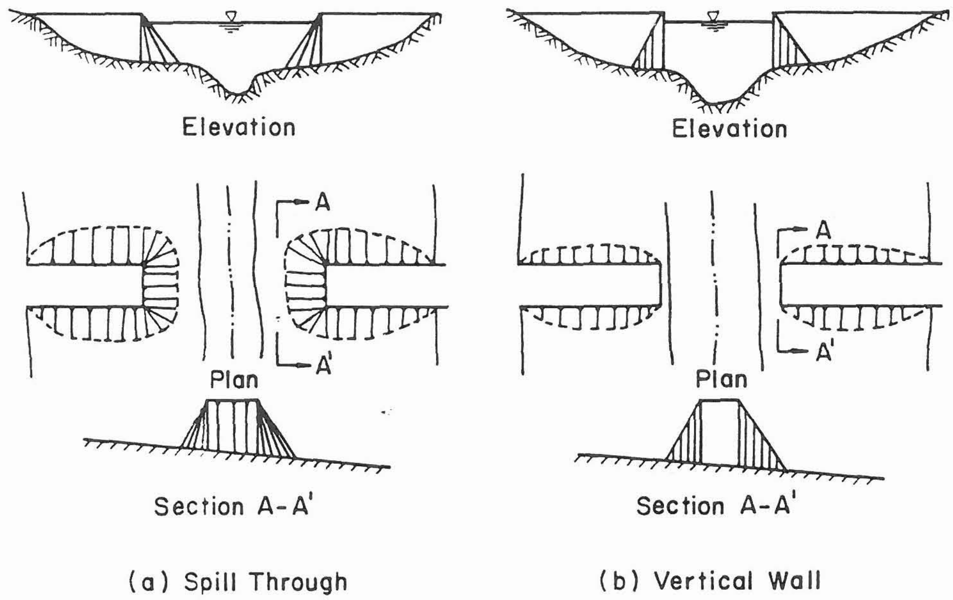


Figure 10. Abutment Shape.

Design for Scour at Abutments. The potential for lateral channel migration, long term degradation and contraction scour should be considered in setting abutment foundation depths near the main channel. It is recommended that foundation depths for abutments be set at least 6 feet below the streambed, including long-term degradation, contraction scour, and lateral stream migration. Normally, protection is provided using rock riprap with the guidance from Chapter 7 and/or guide banks designed as given in HEC-20.[8] **Engineering judgment is required in setting foundation depths for abutments.**

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 [42] live-bed scour equation or an equation from HIRE [9] can be used. Appendix B presents an alternate design approach, using material contained in the original FHWA Interim Procedures for Evaluating Scour at Bridges.[7] Froehlich analyzed 170 live-bed scour measurements in laboratory flumes to obtain the following equation:

$$\frac{y_s}{y_a} = 2.27\, K_1\, K_2 \left(\frac{a'}{y_a}\right)^{0.43} Fr^{0.61} + 1 \tag{24}$$

where

K_1 = coefficient for abutment shape (see **Table 4**)
K_2 = coefficient for angle of embankment to flow
K_2 = $(\theta/90)^{0.13}$ (see Figure 11 for definition of θ)
$\theta < 90^0$ if embankment points downstream
$\theta > 90^0$ if embankment points upstream
a' = the length of abutment projected normal to flow, ft

A_e = the flow area of the approach cross section obstructed by the embankment, ft^2
Fr = Froude Number of approach flow upstream of the abutment
= $V_e/(gy_a)^{1/2}$
V_e = Q_e/A_e, ft/s
Q_e = the flow obstructed by the abutment and approach embankment, cfs
y_a = average depth of flow on the floodplain, ft
y_s = scour depth, ft

Table 4. Abutment Shape Coefficients.

Description	K_1
Vertical-wall abutment	1.00
Vertical-wall abutment with wing walls	0.82
Spill-through abutment	0.55

An equation in HIRE [9] was developed from Corps of Engineers field data of scour at the end of spurs in the Mississippi River. This field situation closely resembles the laboratory experiments for abutment scour in that the discharge intercepted by the spurs was a function of the spur length. The HIRE equation is applicable when the ratio of projected abutment length (a) to the flow depth (y_1) is greater than 25. This equation can be used to estimate scour depth (y_1) at an abutment where conditions are similar to the field conditions from which the equation was derived:

$$\frac{y_s}{y_1} = 4\ Fr_1^{0.33} \tag{25}$$

where

y_s = scour depth, ft
y_1 = depth of flow at the abutment, on the overbank or in the main channel, ft
Fr_1 = the Froude Number based on the velocity and depth adjacent to and upstream of the abutment

To correct Equation 25 from HIRE [9] for abutments skewed to the stream use **Figure 11.**

The abutment scour depths determined from the HIRE equation (Equation 25) will need to be corrected for abutment type if this equation is used for any abutment shape other than spill-through shapes. This correction can be made by multiplying the abutment scour depth from Equation 25 by the factor $K_1/0.55$, where K_1 is determined from Table 4.

Clear-Water Scour at an Abutment. Use Equations 24 or 25 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, Froehlich's clear-water scour equation is not recommended.

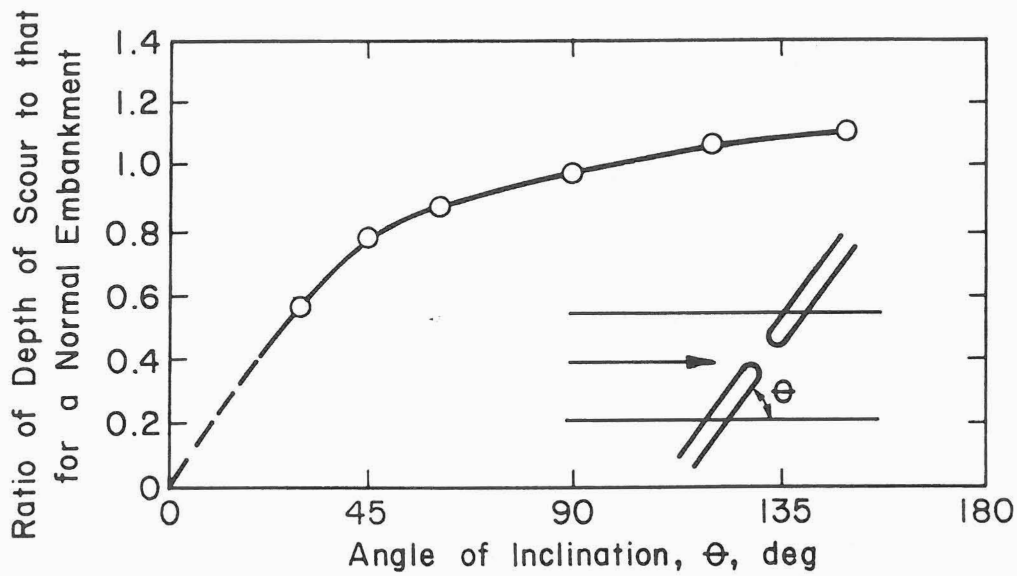


Figure 11. Adjustment of Abutment Scour Estimate for Skew.

4.3.7 Step 7: Plot and Evaluate the Total Scour Depths

Plot the Total Scour Depths. On the cross section of the stream channel and floodplain at the bridge crossing, plot the estimate of long-term bed elevation change, contraction scour, and 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 planform changes (lateral stream channel movement due to meander migration, etc.) that might be reasonably expected to occur.

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

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

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

4. Plot not only the depth of scour at each pier and abutment, but also the scour hole width. Use 2.8 y_s to estimate scour hole width on each side of the pier.

Evaluate the Total Scour Depths.

1. Evaluate whether the computed scour depths are reasonable and consistent with the design engineer's previous experience, and engineering judgment. If not, modify the depths to reflect sound engineering judgment.

2. Evaluate whether the local scour holes from the piers or abutments overlap between spans. If so, local scour depths can be larger though indeterminate. For new or replacement bridges, the length of the bridge opening should be reevaluated and the opening increased or the number of piers decreased as necessary to avoid overlapping scour holes.

3. Evaluate other factors such as lateral movement of the stream, streamflow hydrograph, velocity and discharge distribution, movement of the thalweg, shifting of the flow direction, channel changes, type of stream, or other factors.

4. Evaluate whether the calculated scour depths appear too deep for the conditions in the field, relative to the laboratory conditions (**Abutment scour equations are for the worst case conditions**). Rock riprap or a guide bank could be a more cost- effective solution than designing the abutment to resist the computed abutment scour depths.

5. Evaluate cost, safety, etc. Also, account for debris effects.

6. In the design of bridge foundations, the bottom foundation elevation(s) should be at or below the total scour elevation(s) as discussed in Chapter 3.

Reevaluate the Bridge Design. Reevaluate the bridge design on the basis of the foregoing scour computations and evaluation. Revise the design as necessary. This evaluation should consider the following questions:

1. Is the waterway area large enough (i.e., is contraction scour too large)?

2. Are the piers too close to each other or to the abutments (i.e., do the scour holes overlap)? The topwidth of a scour hole on each side of a pier is about 2.8 times the depth of scour. If scour holes overlap, local scour can be deeper.

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

4. Are bridge abutments properly aligned with the flow and located properly in regard to the stream channel and floodplain?

5. Is the bridge crossing of the stream and 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 bridge?

6. 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?

4.4 Computer Program HY-9 for Computing Scour Depths

The HY-9 computer program developed by Fraher (FHWA) [45] is a convenient tool for solving the equations presented in this chapter. The program is interactive (i.e., the user is prompted to enter the variables needed to solve the equations). The program parallels the manual by presenting the equation names and numbers and all variables just as they are in the manual. The following important features are provided:

1. Data are saved to a user named file and can be reopened for editing of values.
2. A hard copy report is available which includes the equation names and all variables.
3. An ASCII file output is available to allow transport to other word processing programs.
4. The program can handle data for up to 5 flow events and up to 10 pier solutions per flow.
5. The HY-9 program is available from the University of Florida McTRANS Center, Gainesville, Florida.

4.5 Scour Example Problem

4.5.1 General Description of Problem

This example problem is taken from a paper by Arneson.[46] A 650-foot long bridge (**Figure 12**) is to be constructed over a channel with spill-through abutments (slope of 2H:1V). The left abutment is set approximately 200 feet back from the channel bank. The right abutment is set at the channel bank. The bridge deck is set at elevation 22 and has a girder depth of 4 feet. Six round-nose piers are evenly spaced in the bridge opening. The piers are 5 feet thick, 40 feet long, and are aligned with the flow. The 100-year design discharge is 30,000 cfs. The 500-year flow of 51,000 cfs was estimated by multiplying the Q_{500} by 1.7 since no hydrologic records were available to predict the 500-year flow.

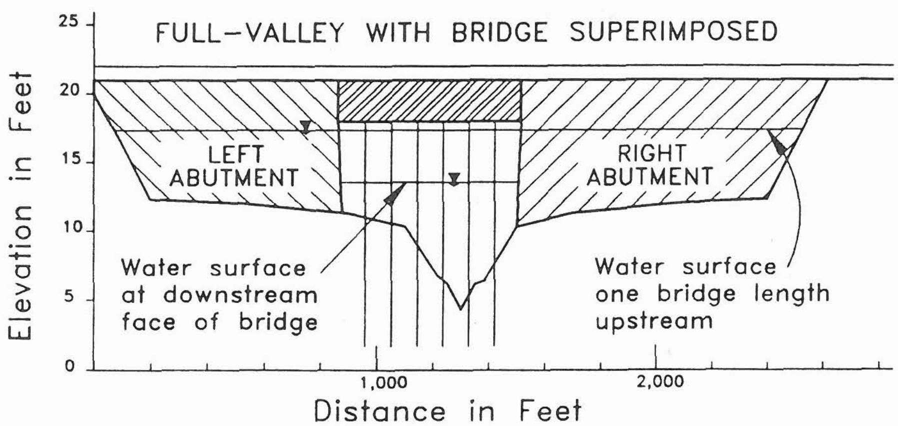


Figure 12. Cross Section of Proposed Bridge.

4.5.2 Step 1: Determine Scour Analysis Variables

From Level 1 and Level 2 analysis: a site investigation of the crossing was conducted to identify potential stream stability problems at this crossing. Evaluation of the site indicates that the river has a relatively wide floodplain. The floodplain is well vegetated with grass and trees. However, the presence of remnant channels indicates that there is a potential for lateral shifting of the channel.

The bridge crossing is located on a relatively straight reach of channel, The channel geometry is relatively the same for approximately 1,000 feet upstream and downstream of the bridge crossing. The D_{50} of the bed material, and overbank material is approximately 2 mm. The maximum grain size of the bed material is approximately 8 mm.

The river and crossing are located in a rural area with the primary land use consisting of agriculture and forest.

Rock outcrops have been identified in the valley bottom approximately 3,000 feet upstream and downstream of the bridge crossing; however, at the bridge site, bedrock is approximately 150 feet below the channel bed.

Since this is a sand-bed channel, no armoring potential is expected. Furthermore, the bed for this channel at low flow consists of dunes which are approximately 1 to 1.5 feet high. At higher flows, above the Q_5, the bed will be either plane bed or antidunes.

The left and right banks are relatively well vegetated and stable; however, there are isolated portions of the bank which appear to have been undercut and are eroding. Brush and trees grow to the edge of the banks. Banks will require riprap protection if disturbed. Riprap will be required upstream of the bridge and extend downstream of the bridge.

Hydraulic characteristics. Hydraulic characteristics at the bridge were determined using WSPRO.[29] Three cross sections were used for this analysis and are denoted as "EXIT" for the section downstream of the bridge, "FULLV" for the full-valley section at the bridge, and "APPR" for the approach section located one bridge length upstream of the bridge. The bridge geometry was superimposed on the full-valley section and is denoted "BRDG." Values used for this example problem are based on the output from the WSPRO model which is presented in Appendix C. Specific values for scour analysis variables are given for each computation separately and cross referenced to the line numbers of the WSPRO output.

Both the bridge and approach sections were coded to output 20 equal conveyance tubes. **Figure 13 and Figure 14** illustrate the location of these conveyance tubes for the approach and bridge cross section respectively. **Figure 15** illustrates the average velocities in each conveyance tube and the contraction of the flow from the approach section through the bridge. Figure 15 also identifies the equal conveyance tubes of the approach section which are cut off by the abutments.

Hydraulic variables for performing the various scour computations were determined from the WSPRO output (see Appendix C) and from Figures 13, 14, and 15. These variable which will be used to compute contraction scour and local scour are presented in **Tables 5 through 10**.

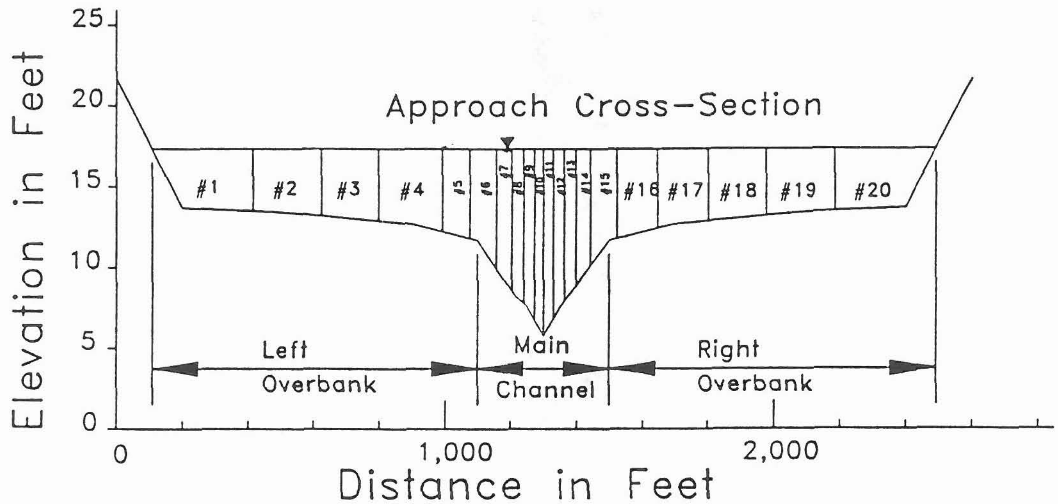


Figure 13. Equal Conveyance Tubes of Approach Section.

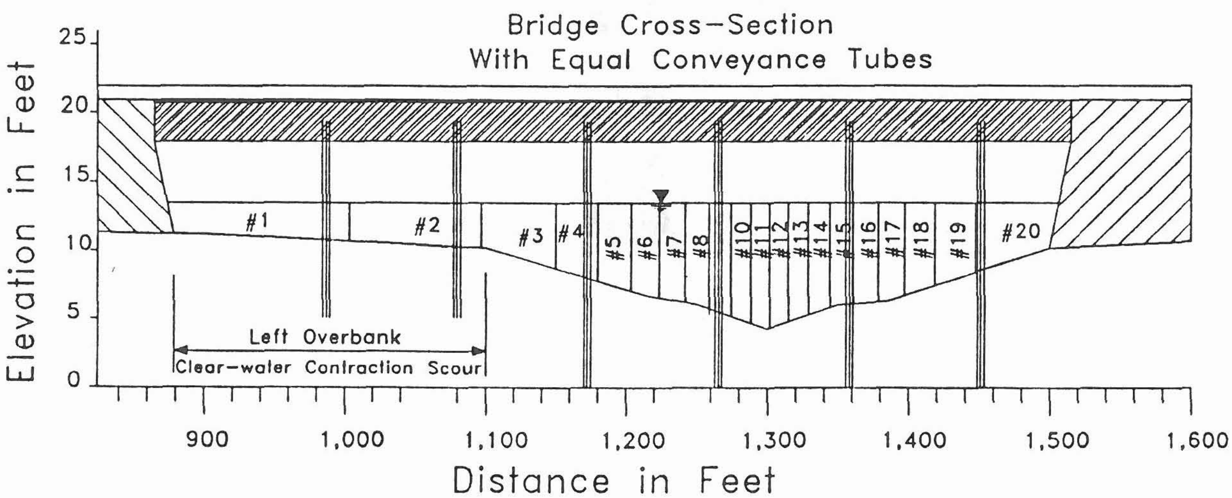


Figure 14. Equal Conveyance Tubes of Bridge Section.

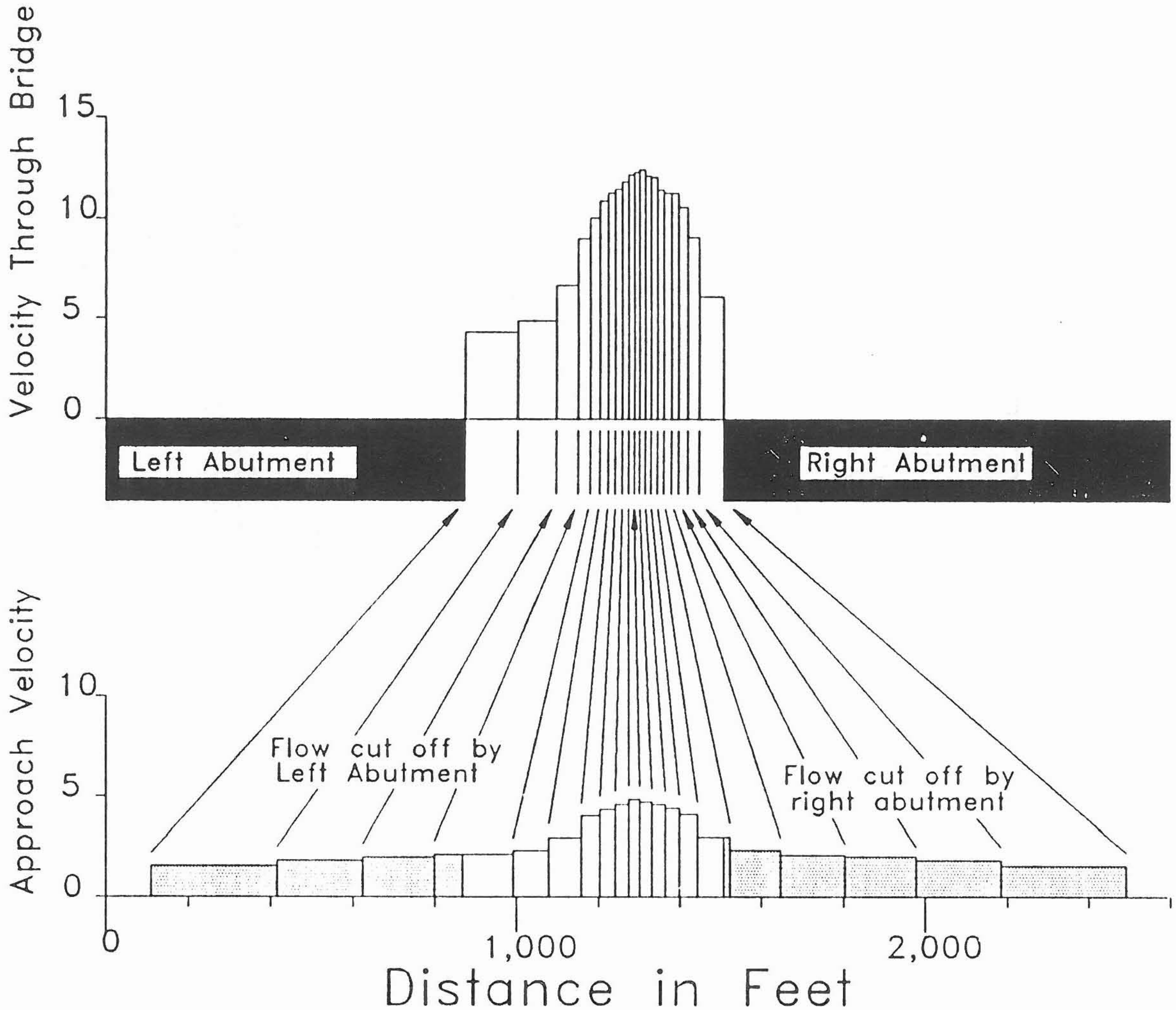


Figure 15. Plan View of Equal Conveyance Tubes Showing Velocity Distribution at Approach and Bridge Sections.

Table 5. Hydraulic Variables From WSPRO for Estimation of Live-bed Contraction Scour.

		Remarks
Q (cfs)	30,000	Total discharge input to WSPRO, line 11
K_c (Approach)	680,989	Conveyance of main channel of approach. Read directly from WSPRO, line 295, SA#2
K_{total} (Approach)	1,414,915	Total conveyance of approach section. Read directly from WSPRO, line 297
W_1 (Approach) (ft)	400	Taken as the top width of flow (TOPW) for this case. Assumed to represent active live bed width of approach. Read directly from WSPRO, line 295, SA#2
A_c (Approach) (ft^2)	3,467	Main channel area of approach section. Read directly from WSPRO, line 295, SA#2
TOPW (Approach) (ft)	400	Top width of main channel of approach section. Read directly from WSPRO, line 295, SA#2
WETP (Approach) (ft)	400	Wetted perimeter of main channel of approach section. Read directly from WSPRO, line 295, SA#2.
K_c (Bridge)	392,654	Conveyance of main channel through bridge. Read directly from WSPRO, line 244, SA#2
K_{total} (Bridge)	433,451	Total conveyance through bridge. Read directly from WSPRO, line 245
W_2 (Bridge) (ft)	380	Difference between subarea break points defining channel banks at the bridge. Read directly from WSPRO, line 93, less pier widths (20 ft.)
S_f (ft/ft)	0.002	Average unconstricted energy slope. Defined as the head loss (HF) listed on lines 318 or 322 of the WSPRO output divided by the distance between cross sections listed on lines 316, 319, and 323.

Table 6. Hydraulic Variables from WSPRO for Estimation of Clear-water Contraction Scour on Left Overbank.

		Remarks
Q (cfs)	30,000	Total discharge input to WSPRO, line 11
Q_{chan} (Bridge) (cfs)	27,176.4	Flow in main channel at bridge. Determined in live-bed computation of Step 5A
Q_2 (Bridge) (cfs)	2,823.6	Flow in left overbank through bridge. Determined by subtracting Q_{chan} from total discharge through bridge, or by multiplying total discharge by K_1/K_{total} (line 243, SA#1) for left overbank through bridge
D_{50} (Bridge Overbank) (ft)	0.0066	Median grain size of left overbank area. Note conversion from mm to feet
$W_{setback}$ (Bridge)(ft)	211	Distance from left bank to toe of left abutment less pier width. Determine by subtracting XLAB on line 335 and total pier width from left bank station on line 137
A_{left} (Approach) (ft^2)	4,049	Area of left overbank at approach. From WSPRO, line 294, SA #1
$TOPW_{left}$ (Approach) (ft)	992	Topwidth of left overbank at approach. From WSPRO, line 294, SA #1

Table 7. Hydraulic Variables from WSPRO for Estimation of Pier Scour.

		Remarks
Area (ft^2)	120.7	Read directly from WSPRO output
V_1 (fps)	12.43	Velocity in conveyance tube #12. Read directly from WSPRO output, line 224
Topwidth (ft)	13.1	Difference between left and right end stations of equal conveyance tube. Read from WSPRO output, line 222
Y_1 (ft)	9.21	Mean depth of Tube #12, computed as area divided by topwidth of conveyance tube

Table 8. Hydraulic Variables from WSPRO for Estimation of Abutment Scour Using Froehlich's [41] Equation for Left Abutment.

		Remarks
Q (cfs)	30,000	Total discharge, input to WSPRO, line 11
q_{tube} (cfs)	1,500	Discharge per equal conveyance tube, defined as total discharge divided by 20
#Tubes	3.5	Number of approach section conveyance tubes which are obstructed by left abutment. Determined by superimposing abutment geometry onto the approach section
Q_e (cfs)	5,250	Flow in left overbank obstructed by left abutment. Determined by multiplying #Tubes and q_{tube}
A_c (left abut.) (ft^2)	2,910	Area of conveyance tube 1, 2, 3, and half of tube 4. Determined from WSPRO output, line 266
a' (ft)	766.65	Length of abutment projected into flow, determined by adding topwidths of conveyance tube 1, 2, 3, and half of tube 4, determined from WSPRO output, line 265

Table 9. Hydraulic Variables from WSPRO for Estimation of Abutment Scour Using HIRE [9] Equation for Left Abutment.

		Remarks
Q (cfs)	30,000	Total discharge, input to WSPRO, line 11
q_{tube} (cfs)	1,500	Discharge per equal conveyance tube, defined as total discharge divided by 20
A_{tube} #1 (ft^2) (Bridge x-Section)	346.5	Area of conveyance tube #1, adjacent to left abutment. Read directly from WSPRO, line 215
V_{tube} (ft/s) (Bridge x-Section)	4.33	Mean velocity of conveyance tube #1, adjacent to left abutment. Read directly from WSPRO, line 216
$TOPW_{tube}$ #1 (ft) (Bridge x-Section)	129.5	Difference between left and right station of conveyance tube 1. From WSPRO, line 214
y_1 (ft) (Bridge x-Section)	2.68	Average depth of conveyance tube 1. Computed as: $A_{tube}/TOPW_{tube}$ of conveyance tube #1

Table 10. Hydraulic Variables from WSPRO for Estimation of Abutment Scour Using HIRE [9] Equation for Right Abutment.

		Remarks
Q (cfs)	30,000	Total discharge, input to WSPRO, line 11
q_{tube} (cfs)	1,500	Discharge per equal conveyance tube, defined as total discharge divided by 20
A_{tube} #20 (ft^2)	245.2	Area of conveyance tube #20, read directly from WSPRO, line 227
V_{tube} (ft/s)	6.12	Mean velocity of conveyance tube #20, read directly from WSPRO, line 228
$TOPW_{tube}$ #1 (ft)	59.7	Difference between left and right station of conveyance tube 1. From WSPRO, line 214
y_1 (ft)	4.11	Average depth of conveyance tube 20 ($A_{tube}/TOPW_{tube}$)

Contraction scour will occur both in the main channel and on the left overbank of the bridge opening. For the main channel, contraction scour will be live-bed because the channel is predominantly sand which is transported as both contact and suspended load in the main channel.

In the overbank area adjacent to the left abutment, clear-water scour will occur. This is because the overbank areas upstream of the bridge are vegetated, and because the velocities in these areas will be low. Thus, returning overbank flow which will pass under the bridge adjacent to the left abutment will not be transporting significant amounts of material to replenish the scour on the left overbank adjacent to the left abutment.

Because of this, two computations for contraction scour will be required. The first computation, which will be illustrated in Step 4-A will use Laursen's live-bed equation to determine the contraction scour in the main channel. The second computation, which is illustrated in Step 4-B will utilize Laursen's clear-water equation for the left overbank area. Hydraulic data for these two computations are presented in Tables 5 and 6 for the live-bed and clear-water computations respectively.

Table 7 lists the hydraulic variables which will be used to estimate the local scour at the piers (Step 5). These hydraulic variables were determined from a plot of the velocity distribution derived from the WSPRO output (**Figure 16**). For this example the highest velocities and flow depths in the bridge cross section will be used (at conveyance tube number 12). Only one pier scour computation will be computed because the possibility of thalweg shifting and lateral migration will require that all of the piers be set assuming that any pier could be subjected to the maximum scour producing variables.

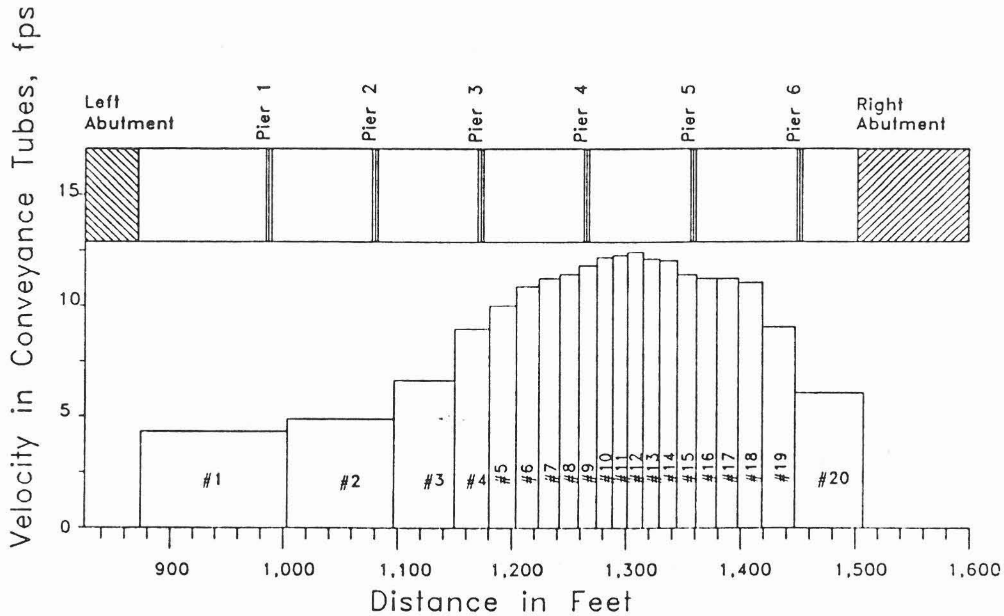


Figure 16. Velocity Distribution at Bridge Crossing.

Local scour at the left abutment will be illustrated in Step 6-A using the Froehlich [41] and HIRE [9] equations. Scour variables derived from the WSPRO output for these two computations are presented in Tables 8 and 9 for the Froehlich and HIRE equation respectively. Local scour at the right abutment will be computed in Step 6-B using the HIRE equation, and the hydraulic variables listed in Table 10.

4.5.3 STEP 2: Analyze Long-term Bed Elevation Changes

Evaluation of stage discharge relationships and cross sectional data obtained from other agencies do not indicate progressive aggradation or degradation. Furthermore, the presence of bed rock outcrops both upstream and downstream of the bridge crossing serve as grade control for the this reach of river. Based on these observations, the channel is relatively stable vertically at present.

Furthermore there are no plans to change the local land use in the watershed. The forested areas of the watershed are government owned and regulated to prevent wide spread fire damage, and in-stream gravel mining is prohibited. These observations indicate that future aggradation or degradation of the channel, due to changes in sediment delivery from the watershed, are minimal.

Based on these observations, and due to the lack of other possible impacts to the river reach, it is determined that the channel will be relatively stable vertically at the bridge crossing and long term aggradation or degradation potential is considered to be minimal. However, there is evidence that the channel is unstable laterally. This will need to be considered when assessing the total scour at the bridge.

4.5.4 Step 3: Evaluate the Scour Analysis Method

For this problem it is assumed that the components of scour will develop independently (Method 1). Therefore the contraction and local scour will be computed using the hydraulic characteristics determined from the WSPRO model. The fixed bed geometry will not be modified.

In some cases, when the contraction scour is large, (greater than approximately 5 feet), local velocities in the bridge opening can be measurably reduced as a result of contraction scour. In such cases, the fixed-bed hydraulic model can be modified to account for the contraction scour and the hydraulic characteristics of the bridge opening can be redetermined using WSPRO. Contraction scour can be recomputed followed by computation of the local scour. This method (denoted Method 2) is usually not necessary.

4.5.5 Step 4A: Compute the Magnitude of Contraction Scour

It was determined that the contraction scour in the main channel will be live-bed. The following computation determines the mode of bed material transport and the factor k_1. All hydraulic parameters which are needed for this computation are listed in Table 5. Neill's [47] equation was used to determine the critical velocity for D_{50} of the bed material and compared to the actual velocity to determine that for this situation, the flow will be live-bed.

The hydraulic radius of the approach channel is:

$$R = \frac{A_c}{WETP} = \frac{3467\ ft^2}{400\ ft} = 8.67\ ft \tag{46}$$

The average shear stress on the channel bed is:

$$\tau = \gamma RS = (62.4 lb/ft^3)\ (8.67\ ft)\ (0.002) = 1.08\ lb/ft^2 \tag{47}$$

The shear velocity in the approach channel is:

$$V_* = (\tau/\rho)^{0.5} = (1.08/1.94)^{0.5} = 0.75\ ft/s \tag{48}$$

Bed material is sand with D_{50} =2.0 mm. = 0.0066 ft

Fall velocity (w) = 0.9 ft/s from Figure 3

Therefore

$$\frac{V_*}{w} = \frac{0.75}{0.9} = = 0.83 \tag{49}$$

From the above, the coefficient k_1 is determined (from the discussion for Equation 16) to be equal to 0.64 which indicates that the mode of bed material transport is a mixture of suspended and contact load.

The discharge in the main channel of the approach section is determined from the ratio of the conveyance in the main channel to the total conveyance of the approach section. By multiplying this ratio by the total discharge, the discharge in the main channel at the approach section (Q_1) can be determined.

$$Q_1 = Q\ (K_1/K_{total}) = 30{,}000\ cfs \left(\frac{680{,}989}{1{,}414{,}915}\right) = 14{,}439\ cfs \tag{50}$$

Likewise, the discharge in the main channel at the bridge (Q_2) is also determined from the ratio of conveyance for the bridge section.

$$Q_2 = Q\ (K_2/K_{total}) = 30{,}000\ cfs \left(\frac{392{,}654}{433{,}451}\right) = 27{,}176\ cfs \tag{51}$$

For many wider natural channels, the hydraulic radius is equal to the depth. For this example the average depth, y_1, is equal to the hydraulic radius of the main channel at the approach section, therefore: y_1=8.67.

The channel widths at the approach and bridge section are given in Table 5. Therefore all parameters to determine contraction scour have been determined and Laursen's live-bed equation (Equation 16) can be employed.

$$\frac{y_2}{y_1} = \left(\frac{Q_2}{Q_1}\right)^{\frac{6}{7}}\left(\frac{W_1}{W_2}\right)^{k_1} \tag{52}$$

$$\frac{y_2}{8.67} = \left(\frac{27{,}176}{14{,}439}\right)^{\frac{6}{7}}\left(\frac{400}{380}\right)^{0.64} = 1.78 \tag{53}$$

By multiplying the above result by y_1, y_2 is determined to be equal to 15.4 feet. Therefore the depth of contraction scour in the main channel is:

$$y_s = y_2 - y_1 = 15.4\ ft - 8.67\ ft = 6.7\ ft \tag{54}$$

This amount of contraction scour is large and could be minimized by increasing the bridge opening, providing for relief bridges in the overbank or, in some cases, providing for highway approach overtopping.

Since the contraction scour is large, Method 2 which would require revising the WSPRO model to account for this amount of contraction scour may be warranted. However, for this example, this amount of contraction scour is accepted and subsequent computations of local scour will be illustrated.

4.5.6 Step 4B: Determine Contraction Scour for Left Overbank

Clear-water contraction scour will occur in the overbank area between the left abutment and the left bank of bridge opening. Although the bed material in the overbank area is soil, it is protected by vegetation. Therefore, there would be no bed-material transport into the set-back bridge opening (clear-water conditions). The subsequent computations are based on the discharge and depth of flow passing under the bridge in the left overbank. These hydraulic variables were determined from the WSPRO output and are tabulated in Table 6.

Computation of clear-water contraction scour (Equation 18)

$$y_2 = \left[\frac{Q^2}{(120\ D_m^{2/3}\ W_{setback}^{\ 2})}\right]^{3/7} \tag{55}$$

Computation of flow depth in contracted section at bridge, y_2:

$$y_2 = \left[\frac{(2823.6\ cfs)^2}{(120)\ (0.0083\ ft)^{2/3}\ (211\ ft)^2}\right]^{3/7} = 4.67\ ft \tag{56}$$

Computation of flow depth in left overbank approach section, y_1:

$$y_1 = \frac{A}{TOPW} = \frac{(4049\ ft^2)}{(992\ ft)} = 4.08\ ft \tag{57}$$

Therefore the clear-water contraction scour in the left overbank of the bridge opening is:

$$y_s = y_2 - y_1 = 4.67\ ft - 4.08\ ft = 0.59\ ft. \qquad (58)$$

4.5.7 Step 5: Compute the Magnitude of Local Scour at Piers

It is anticipated that any pier under the bridge could potentially be subject to the maximum flow depths and velocities derived from the WSPRO hydraulic model(Table 7). Therefore, only one computation for pier scour is conducted and assumed to apply to each of the six piers for the bridge. This assumption is appropriate based on the fact that the thalweg is prone to shifting and because there is a possibility of lateral channel migration.

Computation of Pier Scour. The Froude Number for the pier scour computation is based on the hydraulic characteristics of equal conveyance tube number 12. Therefore:

$$Fr_1 = \frac{V}{(g\ y_1)^{0.5}} = \frac{12.43\ fps}{[(32.2\ ft/s^2)\ (9.21\ ft)]^{0.5}} = 0.72 \qquad (59)$$

For a round nose pier aligned with the flow:

$$K_1 = K_2 = 1.0 \qquad (60)$$

For plane-bed condition:

$$K_3 = 1.1 \qquad (61)$$

Using CSU's equation (Equation 20):

$$\frac{y_s}{y_1} = 2.0\ K_1\ K_2\ K_3 \left(\frac{a}{y_1}\right)^{0.65} Fr_1^{0.43} \qquad (62)$$

$$\frac{y_s}{9.21} = 2.0\ (1.0)\ (1.0)\ (1.1) \left(\frac{5\ ft}{9.21\ ft}\right)^{0.65} (0.72)^{0.43} = 1.28 \qquad (63)$$

From the above computation the maximum scour depth accounting for bed condition will be approximately 11.8 ft.

Correction for Skew. The above computation assumes that the piers are aligned with the flow (skew angles are less than 5°). However, if the piers were skewed greater than 5°, the value of y_s/y_1, as computed above, would need to be adjusted using K_2. The following computations illustrates the adjustment for piers skewed 10°.

$$\frac{L}{a} = \frac{40\ ft}{5\ ft} = 8 \tag{64}$$

K_2 can then be interpolated using an L/a of 8 and a 10° skew angle from the correction values tabulated in Table 3. For this example, $K_2=1.67$. Applying this correction for skew:

$$\frac{y_s}{9.21\ ft} = 1.67\ (1.28) = 2.14 \tag{65}$$

Therefore, the maximum scour depth for a pier angled 10° to the flow is 19.7 feet.

<u>Discussion of Pier Scour Computations</u>. Although the estimated local pier scour would probably not occur at each pier, the possibility of thalweg shifting, which was identified in the Level 1 analysis, precludes setting the piers at different depths even if there were a substantial savings in cost. This is because any of the piers could be subjected to the worst case scour conditions.

It is also important to assess the possibility of lateral migration of the channel. This possibility can lead to directing the flow at an angle to the piers, thus increasing local scour. Countermeasures to minimize this problem could include riprap for the channel banks both upstream and downstream of the bridge, and installation of guide banks to align flow through the bridge opening.

The possibility of lateral migration precludes setting the foundations for the overbank piers at a higher elevation. For this example, the foundations for the overbank piers should be set at the same elevations as the main channel piers.

4.5.8 Step 6A: Compute the Magnitude of Local Scour at Left Abutment

<u>Computation of Abutment Scour Using Froehlich's [44] Equation</u>. For spill-through abutments, $K_1=0.55$. For this example, the abutments are set perpendicular to the flow, Therefore $K_2=1.0$. Abutment scour can be estimated using Froehlich's equation with data derived from the WSPRO output (Table 8).

The y_a value at the abutment is assumed to be the average flow depth in the overbank area. It is computed as the cross sectional area of the left overbank cut off by the left abutment divided by the distance the left abutment protrudes into the overbank flow.

$$y_a = \frac{A_e}{a'} = \frac{2{,}910\ ft^2}{766.65\ ft} = 3.79\ ft \tag{66}$$

The average velocity of the flow in the left overbank (Figure 15) which is cut off by the left abutment is computed as the discharge cutoff by the abutment divided by the area of the left overbank cut off by the left abutment.

$$V_e = \frac{Q_e}{A_e} = \frac{5{,}250\ cfs}{2{,}910\ ft^2} = 1.8\ ft/s \tag{67}$$

Using these parameters, the Froude Number of the overbank flow is:

$$Fr = \frac{V_e}{(g\ y_a)^{1/2}} = \frac{1.8\ ft/s}{[(32.2\ ft/s^2)\ (3.79\ ft)]^{0.5}} = 0.16 \tag{68}$$

Using Froehlich's equation:

$$\frac{y_s}{y_a} = 2.27\ K_1\ K_2 \left(\frac{a'}{y_a}\right)^{0.43} Fr^{0.61} + 1 \tag{69}$$

$$\frac{y_s}{3.79\ ft} = 2.27\ (0.55)\ (1.0)\left(\frac{766.65\ ft}{3.79\ ft}\right)^{0.43} (0.16)^{0.61} + 1 = 5.05 \tag{70}$$

Using Froehlich's equation, the abutment scour at the left abutment is computed to be 19.1 feet.

Computation of Abutment Scour Using the HIRE [9] Equation. The HIRE equation for abutment is also applicable for this situation because L/y_1, as represented by a'/y_a from the previous computation, is greater than 25.

The HIRE equation is based on the velocity and depth of the flow passing through the bridge opening adjacent to the abutment end which is listed in Table 9. Therefore, the Froude Number of this flow is:

$$Fr_1 = \frac{V_{abut}}{(g\ y_1)^{0.5}} = \frac{4.33\ ft/s}{[(32.2\ ft/s^2)\ (2.68\ ft)]^{0.5}} = 0.47 \tag{71}$$

Using the HIRE equation (Equation 25):

$$\frac{y_s}{y_1} = 4\ Fr_1^{0.33} = 4\ (0.47)^{0.33} = 3.12 \qquad (72)$$

From the above computation, the depth of scour at the left abutment as computed using the HIRE equation, is 8.4 feet.

4.5.9 Step 6B: Compute Magnitude of Local Scour at Right Abutment

The HIRE equation for abutment is also applicable for the right abutment since L/y_1 is greater than 25.

The HIRE equation is based on the velocity and depth of the flow passing through the bridge opening adjacent to the end of the right abutment and listed in Table 10. The Froude Number of this flow is:

$$Fr_1 = \frac{V_{abut}}{(g\ y_1)^{0.5}} = \frac{6.12\ ft/s}{[(32.2\ ft/s^2)\ (4.11\ ft)]^{0.5}} = 0.53 \qquad (73)$$

Using the HIRE equation:

$$\frac{y_s}{y_1} = 4\ Fr_1^{0.33} = 4\ (0.53)^{0.33} = 3.25 \qquad (74)$$

From the above computation, the depth of scour at the right abutment, as computed using the HIRE equation is 13.3 feet.

<u>Discussion of Abutment Scour Computations</u>. Abutment scour as computed using the Froehlich equation will result in scour predictions at the abutments which are deep. These scour depths could occur if the abutments protruded into the main channel flow, or when a uniform velocity field is cut off by the abutment in a manner that most of the returning overbank flow is forced to return to the main channel at the abutment end. For most cases however, when the overbank area, channel banks and area adjacent to the abutment are well vegetated, scour depths as predicted with the Froehlich equation will probably not occur.

All of the abutment scour computations (left and right abutments) assumed that the abutments were set perpendicular to the flow. If the abutments were angled to the flow, a correction utilizing K_2 would be applied to Froehlich's equation or, using Figure 11 would be applied to the equation from HIRE. However the adjustment for skewed abutments is minor when compared to the magnitude of the computed scour depths. For example if the abutments for this example problem were angled 30 degrees upstream ($\theta = 120^o$), the correction for skew would increase the computed depth of scour by approximately 3 to 4 percent for the Froehlich and HIRE equation, respectively.

4.5.10 Step 7: Plot and Evaluate Total Scour Depths

As a final step, the results of the scour computations are plotted on the bridge cross section and carefully evaluated (**Figure 17**). For this example, only the computations for pier scour which were aligned with the flow were plotted. Additionally, only the abutment scour computations reflecting the results from the HIRE equation were plotted. The topwidth of the local scour holes is suggested as 2.8 y_s.

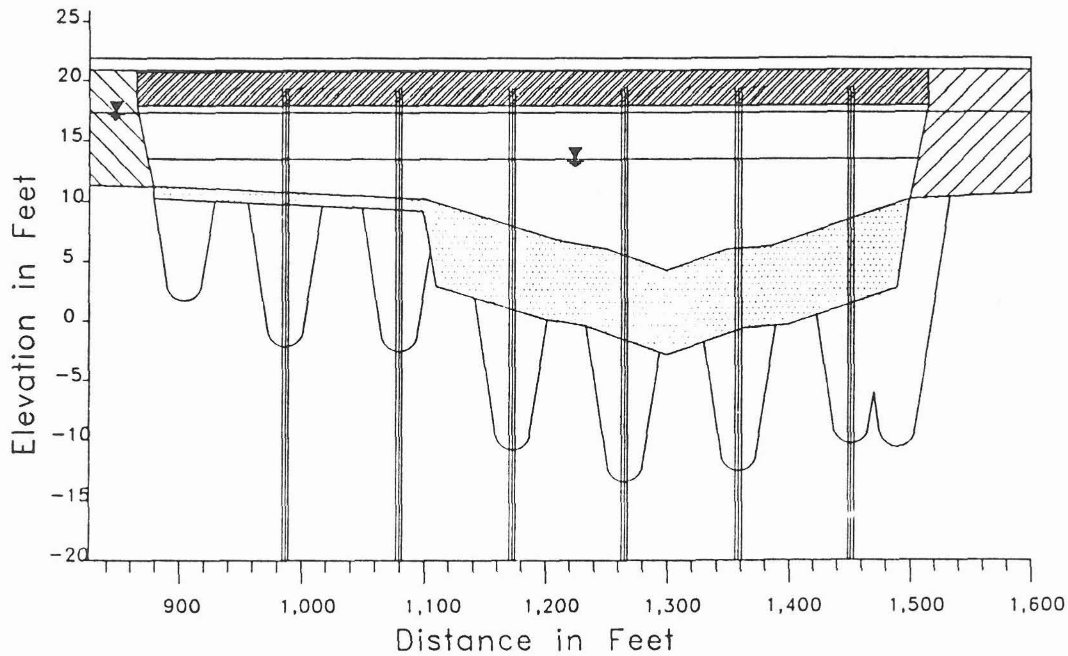


Figure 17. Plot of Total Scour for Example Problem.

It is important to carefully evaluate the results of the scour computations. For example, although the total scour plot indicates that the total scour at the overbank piers is less than for the channel piers, this does not indicate that the foundations for the overbank piers can be set at a higher elevation. Due to the possibility of channel and thalweg shifting, all of the piers should be set to account for the maximum total scour.

The plot of the total scour also indicates that there is a possibility of overlapping scour holes between the sixth pier and the right abutment. During the plotting process, it is unclear where the right abutment scour should be measured from, since the abutment is located at the channel bank. Both of these uncertainties should be avoided for replacement and new bridges whenever possible. As such, it would be advisable to set the right abutment back from the main channel. This would also tend to reduce the magnitude of contraction scour in the main channel.

The possibility of lateral migration of the channel will have an adverse effect on the magnitude of the pier scour. This is because lateral migration will most likely skew the flow to the piers. This problem could be minimized by using circular piers. An alternative approach would be to install guide banks to align the flow through the bridge opening. The usage of guide banks would also minimize abutment scour.

A final concern relates to the location and depth of contraction scour in the main channel near the second pier and near toe of the right abutment. At these locations, contraction scour in the main channel could increase the bank height to a point where bank failure and sloughing would occur. It is recommended that the existing bank lines be protected with revetment (i.e., riprap, gabions, etc.). Since the river has a history of channel migration, the bridge inspection and maintenance crews should be briefed on the potential nature of this problem so that any lateral migration can be identified.

4.5.11 Complete General Design Procedure

The results of this specific design problem completes Steps 1 through 6 of Chapter 3. The design must now proceed to Steps 7 and 8 of Chapter 3, which includes consideration of the check for superflood. This is not done for this example problem.

4.6 Scour Analysis for Tidal Areas

4.6.1 Introduction

This section presents methods and equations for determining stream stability and scour at tidal inlets, tidal estuaries, bridge crossings to islands and streams affected by tides (tidal waterways). Analysis of tidal waterways is very complex. The hydraulic analysis must consider the magnitude of the 100- and 500-year storm surge (storm tide), the characteristics (geometry) of the tidal inlet, estuary, bay or tidal stream and the effect of any constriction of the flow due to the bridge. In addition, the analysis must consider the long-term effects of the normal tidal cycles on long-term aggradation or degradation, contraction scour, local scour and stream instability.

A storm tide or storm surge in coastal waters results from astronomical tides, wind action, and rapid barometric pressure changes. In addition, the change in elevation resulting from the storm surge may be increased by resonance in harbors and inlets, whereby the tidal range in an estuary, bay or inlet is larger than on the adjacent coast.

The normal tidal cycle with reversal in flow direction and magnitude can increase long-term degradation, contraction scour and local scour. If sediment is being moved on the flood and ebb tide, there may be no net loss of sediment in a bridge reach because sediments are being moved back and forth. Consequently no net long-term degradation may occur. However, local scour at piers and abutments can occur at both the inland and ocean side of the piers and abutments and will alternate with the reversal in flow direction. If, however, there is a loss of sediment in one or both flow directions, then there will be long-term degradation in addition to local scour. Also, the tidal cycles may increase bank erosion, migration of the channel and thus, increase stream instability.

The complexity of the hydraulic analysis increases if the tidal inlet or the bridge constrict the flow and affect the amplitude of the storm surge in the bay or estuary so that there is a large change in elevation between the ocean and the estuary or bay. A constriction in the tidal inlet can increase the velocities in the constricted waterway opening, decrease interior wave heights and tidal range, and increase the phase difference (time lag) between exterior and interior water levels. Analysis of a constricted inlet or waterway may require the use of an orifice equation rather than tidal relationships.

For the analysis of bridge crossings of tidal waterways, a three level analysis approach similar to the approach outlined in HEC-20 [8] is suggested. **Level 1** includes a qualitative evaluation of the stability of the inlet or estuary, estimating the magnitude of the tides, storm surges, and flow in the tidal waterway, and attempting to determine whether the hydraulic analysis depends on tidal or river conditions, or both. **Level 2** represents the engineering analysis necessary to obtain the velocity, depths, and discharge for tidal waterways to be used in determining long-term aggradation, degradation, contraction scour and local scour. The hydraulic variables obtained from the Level 2 analysis are used in the riverine equations presented in previous sections to obtain total scour. Using these riverine scour equations, which are for steady state equilibrium conditions for unsteady, dynamic tidal flow will usually result in estimating deeper scour depths than will actually occur (conservative estimate), but this represents the state of knowledge at this time for this level of analysis.

For complex tidal situations, **Level 3** analysis using physical and 2-dimensional computer models may be required. This section will be limited to a discussion of Levels 1 and 2 analyses. In Level 2 analyses, unsteady 1-dimensional or quasi 2-dimensional computer models may be used to obtain the hydraulic variables needed for the scour equations. **The Level 1, 2, and 3 approaches are described in more detail in later sections.**

4.6.2 Overview Tidal Processes

Glossary.

Bay A body of water connected to the ocean with an inlet.

Estuary Tidal reach at the mouth of a river.

Flood or flood tide Flow of water from the ocean to the bay or estuary.

Ebb or ebb tide Flow of water from the bay or estuary to the ocean.

Littoral transport or drift Transport of beach material along a shoreline by wave action. Also, longshore sediment transport.

Run-up, wave Height to which water rises above still-water level when waves meet a beach, wall, etc.

Storm surge Oceanic tide-like phenomenon resulting from wind and barometric pressure changes. Hurricane surge, storm tide.

Tidal amplitude Generally, half of tidal range.

Tidal cycle One complete rise and fall of the tide.

Tidal inlet A channel connecting a bay or estuary to the ocean.

Tidal passage A tidal channel connected with the ocean at both ends.

Tidal period Duration of one complete tidal cycle.

Tidal prism Volume of water contained in a tidal bay, inlet or estuary between low and high tide levels.

Tidal range Vertical distance between specified low and high tide levels.

Tidal waterways A generic term which includes tidal inlets, estuaries, bridge crossings to islands or between islands,inlets to bays, crossings between bays, tidally affected streams, and etc.

Tides, astronomical Rhythmic diurnal or semi-diurnal variations in sea level that result from gravitational attraction of the moon and sun and other astronomical bodies acting on the rotating earth.

Tsunami Long-period ocean wave resulting from earthquake, other seismic disturbances or submarine land slides.

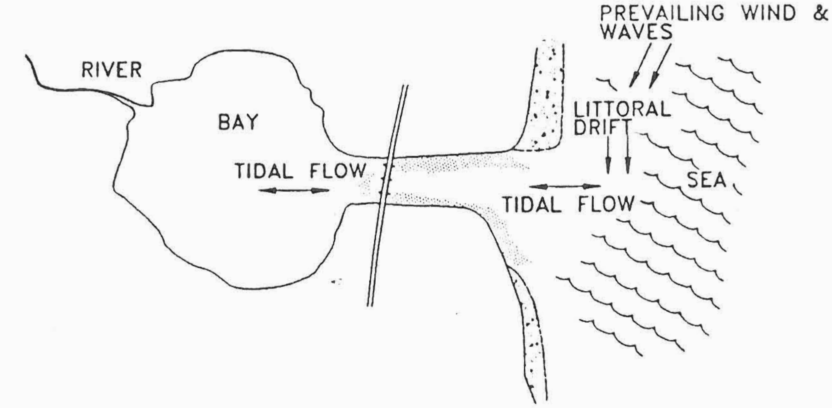
Waterway opening Width or area of bridge opening at a specific elevation, measured normal to principal direction of flow.

Wave period Time interval between arrivals of successive wave crests at a point.

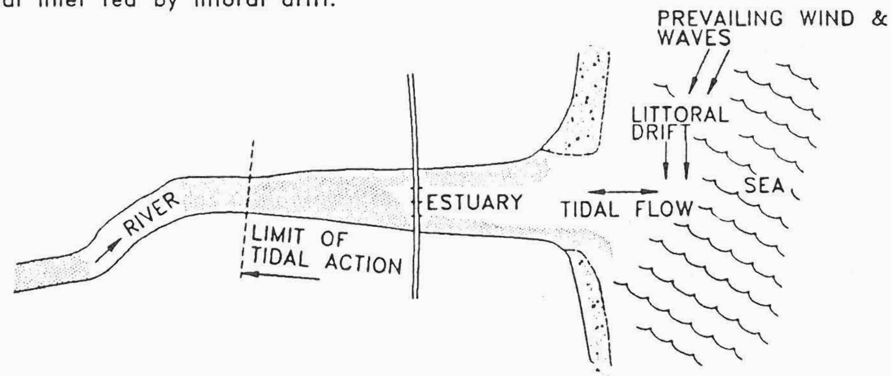
Definition of Tidal and Coastal Processes. Typical bridge crossings of tidal waterways are diagramed in **Figure 18**. From this figure, tidal flows can be defined as being between the ocean and a bay (or lagoon), from the ocean into an estuary, or through passages between islands.

Flow into (flood tide) and out of (ebb tide) a bay or estuary is driven by tides and by the discharge into the bay or estuary from upland areas. Assuming that the flow from upland areas is negligible, the ebb and flood in the bay or estuary will be driven solely by tidal fluctuations and storm surges as illustrated in **Figure 19**. With no inflow of water from rivers and streams, the net flow of water into and out of the bay or estuary will be nearly zero. Increasing the discharge from rivers and streams will lead to a net outflow of water to the ocean.

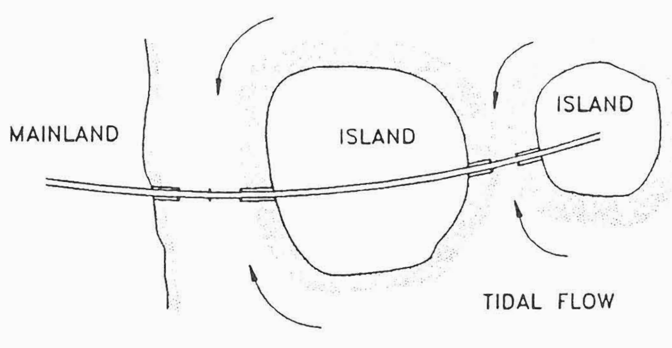
Hydraulically, the above discussion presents two limiting cases for evaluation of the flow velocities in the bridge reach. With negligible flow from the upland areas, the flow through the bridge opening is based solely on the ebb and flood resulting from tidal fluctuations or storm surges. Alternatively, when the flow from the streams and rivers draining into the bay or estuary is large in relationship to the tidal flows (ebb and flood tide), the effects of tidal fluctuations are negligible. For this latter case, the evaluation of the hydraulic characteristics and scour can be accomplished using the methods described previously in this chapter for inland rivers.



1. Inlets between the open sea and an enclosed lagoon or bay, where most of the discharge results from tidal flows. Tidal inlet fed by littoral drift.



2. River estuaries where the net discharge comprises river flow as well as tidal flow componenets.



3. Passages between islands, or between an island and the mainland, where a route to the open sea exists in both directions.

Figure 18. Types of Tidal Waterway Crossings (after Neill [47]).

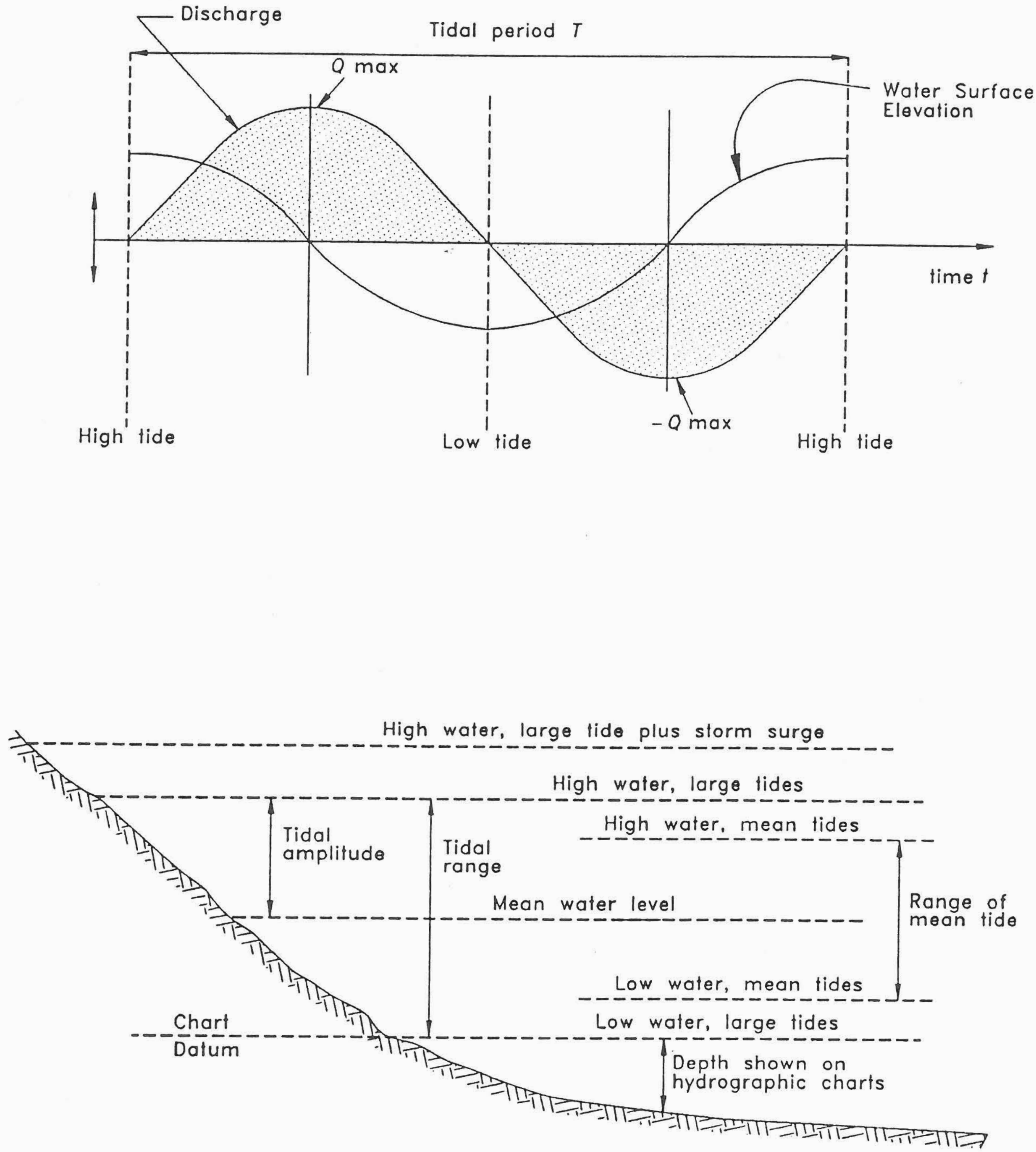


Figure 19. Principal Tidal Terms (after Neill [47]).

The forces which drive tidal fluctuations are, primarily, the result of the gravitational attraction of the sun and moon on the rotating earth (astronomical tides), wind and storm setup or seiching (storm surges), and geologic disturbances (tsunamis). These different forces which drive tides produce varying tidal periods and amplitudes. In general astronomical tides have a tidal period of approximately 12 hours. The continuous rise and fall of astronomical tides will usually influence long-term trends of aggradation and degradation, contraction and local scour. Conversely, when storm surges or tsunamis occur the short term contraction and local scour can be significant. These storm surges and tsunamis are infrequent events and have much longer tidal periods than astronomical tides. Storm surges and tsunamis are a single event phenomenon which, due to their magnitude, can present a significant threat to a bridge crossing in terms of scour.

Evaluation of total scour at bridges crossing tidal waterways requires the assessment of long-term aggradation or degradation, local scour and contraction scour. Long-term aggradation and degradation estimates can be derived from a geomorphic evaluation coupled with computations of scour based on the long contraction described by Laursen [12 and 13]. Such computations of long-term trends are usually driven by astronomical tide cycles. Worst case hydraulic conditions for contraction and local scour are usually the result of infrequent tidal events such as storm surges and tsunamis.

Although the hydraulics of flow for tidal waterways is complicated by the presence of two directional flow, the basic concept of sediment continuity is valid. Consequently, a clear understanding of the principle of sediment continuity is essential for evaluating scour at bridges spanning waterways influenced by tidal fluctuations. 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.

As with riverine scour, tidal scour can be characterized by either live-bed or clear-water conditions. In the case of live-bed conditions, sediment transported into the bridge reach will tend to reduce the magnitude of scour. Whereas, if no sediment is in transport to re-supply the bridge reach (clear-water), scour depths will be larger.

In addition to sediments being transported from upland areas, sediments are transported parallel to the coast by ocean currents and wave action (littoral transport). This littoral transport of sediment serves as a source of sediment supply to the inlet, bay or estuary, or tidal passage. During the flood tide, these sediments can be transported into the bay or estuary and deposited. During the ebb tide, these sediments can be re-mobilized and transported out of the inlet or estuary and either be deposited on shoals or moved further down the coast as littoral transport. (See **Figure 20**)

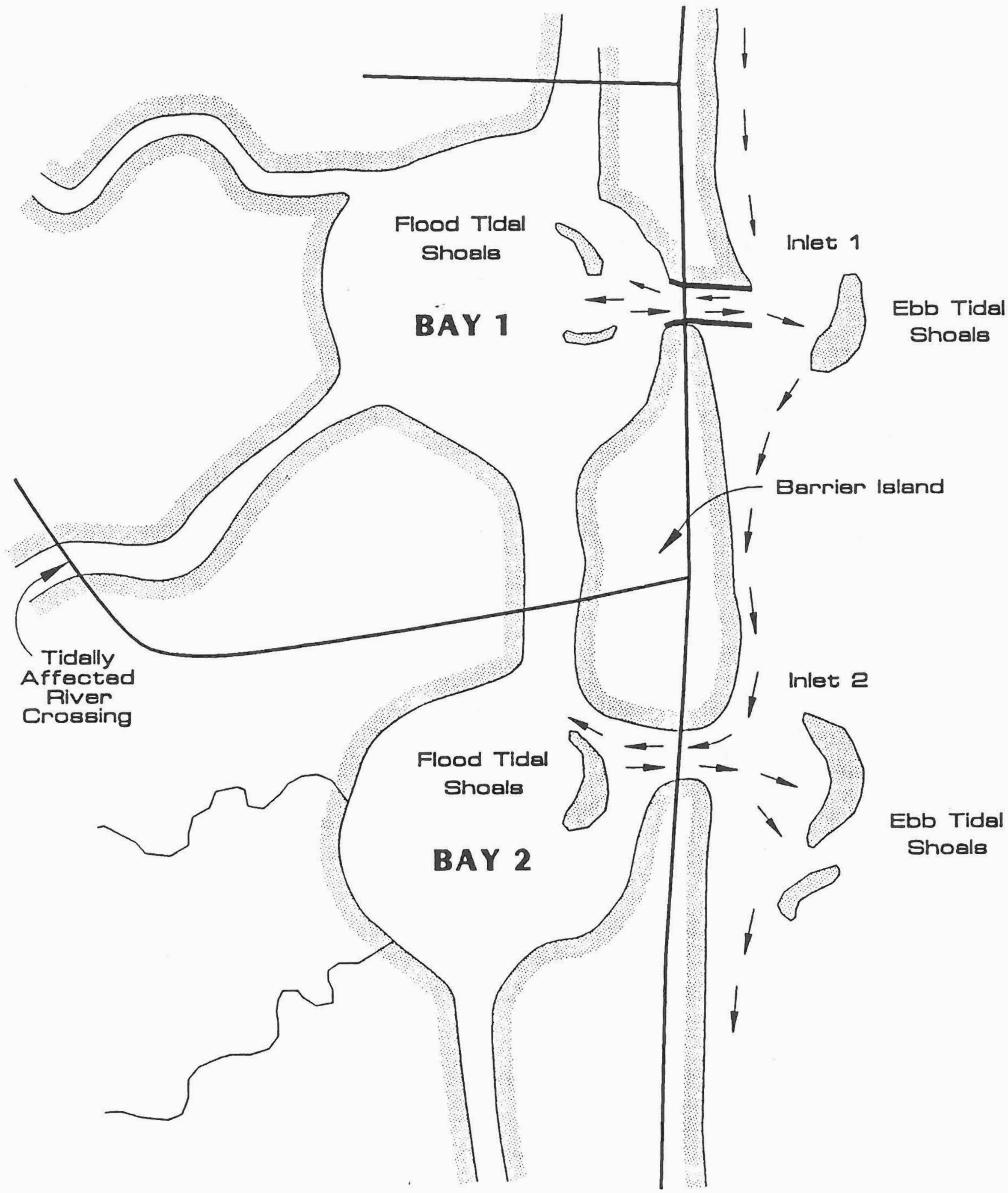


Figure 20. Sediment Transport in Tidal Inlets (after Sheppard [48]).

Sediment transported to the bay or estuary from the upland river system can also be deposited in the bay or estuary during the flood tide, and re-mobilized and transported through the inlet or estuary during the ebb tide. However, if the bay or estuary is large, sediments derived from the upland river system can deposit in the bay or estuary in areas where the velocities are low and may not contribute to the supply of sediment to the bridge crossing. The result is clear-water scour unless sediment transported on the flood tide (ocean shoals, littoral transport) is available on the ebb. Sediments transported from upland rivers into an estuary may be stored there on the flood and transported out during ebb tide. This would produce live-bed scour conditions unless the sediment source in the estuary was disrupted. Dredging, jetties or other coastal engineering activities can limit sediment supply to the reach and influence live-bed and clear-water conditions.

Application of sediment continuity involves understanding the hydraulics of flow and availability of sediment for transport. For example, a net loss of sediment in the inlet, bay or tidal estuary could be the result of cutting off littoral transport by means of a jetty projecting into the ocean (Figure 20). For this scenario, the flood tide would tend to erode sediment from the inlet and deposit sediment in the bay or estuary while the ensuing ebb tide would transport sediment out of the bay or estuary. Because the availability of sediment for transport into the bay is reduced, degradation of the inlet could result. As discussed later, as the cross sectional area of the inlet increases, the flow velocities during the flood tide increase, resulting in further degradation of the inlet.

From the above discussion, it is clear that the concept of sediment continuity provides a valuable tool for evaluation of aggradation and degradation trends of a tidal waterway. Although this principle is not easy to quantify without direct measurement or hydraulic and sediment continuity modeling, the principle can be applied in a qualitative sense to assess long-term trends in aggradation and degradation.

4.6.3 Level 1 Analysis

The objectives of a Level 1 qualitative analysis are to determine the magnitude of the tidal effects on the crossing, the overall long-term stability of the crossing (vertical and lateral stability) and the potential for waterway response to change.

The first step in evaluation of highway crossings is to determine whether the bridge crosses a river which is influenced by tidal fluctuations (tidally affected river crossing) or whether the highway crosses a tidal inlet, bay or estuary (tidally controlled). The flow in tidal inlets bays and estuaries is predominantly driven by tidal fluctuations (with flow reversal), whereas, the flow in tidally affected river crossings is driven by a combination of river flow and tidal fluctuations.

<u>Tidally Affected River Crossings</u>. Tidally affected river crossings are characterized by both river flow and tidal fluctuations. From a hydraulic standpoint the flow in the river is influenced by tidal fluctuations which result in a cyclic variation in the downstream control of the tailwater in the river estuary. The degree to which tidal fluctuations influence the discharge at the river crossing depends on such factors as the relative distance from the ocean to the crossing, riverbed slope, cross-sectional area, storage volume, and hydraulic

resistance. Although other factors are involved, distance of the river crossing from the ocean can be used as a qualitative indicator of tidal influence. At one extreme, where the crossing is located far upstream, the flow in the river may only be affected to a minor degree by changes in tailwater control due to tidal fluctuations. As such, the tidal fluctuation downstream will result in only minor fluctuations in the depth, velocity, and discharge through the bridge crossing.

As the distance from the crossing to the ocean is reduced, again assuming all other factors as equal, the influence of the tidal fluctuations increases. Consequently, the degree of tailwater influence on flow hydraulics at the crossing increases. A limiting case occurs when the magnitude of the tidal fluctuations is large enough to reduce the discharge through the bridge crossing to zero at high tide. River crossings located closer to the ocean than this limiting case have two directional flows at the bridge crossing, and because of the storage of the river flow at high tide, the ebb tide will have a larger discharge and velocities than the flood tide.

For the Level 1 analysis, it is important to evaluate whether the tidal fluctuations will significantly affect the hydraulics at the bridge crossing. If the influence of tidal fluctuations is considered to be negligible, then the bridge crossing can be evaluated based on the procedures outlined for inland river crossings presented previously in this document. If not, then the hydraulic flow variables must be determined using dynamic tidal flow relationships. This evaluation should include extreme events such as the influence of storm surges and design floods.

From historical records of the stream at the highway crossing determine, whether the worst case conditions of discharge, depths and velocity at the bridge are 100- and 500-year return period tide and storm surge, or the 100- and 500-year flood from upstream or a combination of the two. Historical records could consist of tidal and stream flow data from Federal Emergency Management Agency (FEMA), National Oceanic and Atmospheric Administration (NOAA), U.S. Army Corps of Engineers, and U.S. Geological Survey (USGS) records; aerial photographs of the area; maintenance records for the bridge or bridges in the area; newspaper accounts of previous high tides and/or flood flows; and interviews in the local area.

If the primary hazard to the bridge crossing is from upland flood events then scour can be evaluated using the methods given previously in this circular and in HEC-20.[8] If the primary hazard to the bridge is from tide and storm surge or tide, storm surge and flood runoff, then use the analyses presented in the following sections on tidal waterways. If it is unclear whether the worst hazard to the bridge will result from a storm surge, maximum tide, or from an upland flood, it may be necessary to evaluate scour considering each of these scenarios and compare the results.

<u>Tidal Inlets, Bays and Estuaries</u>. **For tidal inlets, bays and estuaries, the goal of the Level 1 analysis is to determine the stability of the inlet and identify and evaluate long-term trends at the location of the highway crossing.** This can be accomplished by careful evaluation of present and historical conditions of the tidal waterway and anticipating future conditions or trends.

Existing cross-sectional and sounding data can be used to evaluate the stability of the tidal waterway at the highway crossing in terms whether the inlet, bay or estuary is increasing or decreasing in size or is relatively stable. For this analysis it is important to evaluate these data based on past and current trends. The data for this analysis could consist of aerial photographs, cross section soundings, location of bars and shoals on both the ocean and bay sides of an inlet, magnitude and direction of littoral drift, and longitudinal elevations through the waterway. It is also important to consider the possible impacts (either past or future) of the construction of jetties, breakwaters, or dredging of navigation channels.

Sources of data would be Corps of Engineers, FEMA, USGS, U.S. Coast Guard (USCG), National Oceanic and Atmospheric Administration (NOAA), local Universities, oceanographic institutions and publications in local libraries. For example a publication by Bruun entitled "Tidal Inlets and Littoral Drift" [49] contains information on many tidal inlets on the east coast for the United States.

A site visit is recommended to gather such data as the conditions of the beaches (ocean and bay side); location and size of any shoals or bars; direction of ocean waves; magnitude of the currents in the bridge reach at mean water level (midway between high and low tides); and size of the sediments. Sounding the channel both longitudinally and in cross section using a conventional "fish finder" sonic fathometer is usually sufficiently accurate for this purpose.

Observation of the tidal inlet to identify whether the inlet restricts the flow of either the incoming or outgoing tide is also recommended. If the inlet or bridge restricts the flow, there will be a noticeable drop in head (change in water surface elevation) in the channel during either the ebb or flood tide. If the tidal inlet or bridge restricts the flow, an orifice equation may need to be used to determine the maximum discharge, velocities and depths (see the Level 2 analysis of this section).

Velocity measurements in the tidal inlet channel along several cross sections, several positions in the cross section and several locations in the vertical can also provide useful information for verifying computed velocities. Velocity measurements should be made at maximum discharge (Q_{max}). Maximum discharge usually occurs around the midpoint in the tidal cycle between high and low tide (see Figure 19).

The velocity measurements can be made from a boat or from a bridge located near the site of a new or replacement bridge. If a bridge exists over the channel, a recording velocity meter could be installed to obtain measurements over several tidal cycles. Currently, there are instruments available that make velocity data collection easier. For example, broad-band acoustic doppler current profilers and other emerging technologies will greatly improve the ability to obtain and use velocity data.

In order to develop adequate hydraulic data for the evaluation of scour, it is recommended that recording water level gages located at the inlet, at the proposed bridge site and in the bay or estuary upstream of the bridge be installed to record tide elevations at 15-minute intervals for at least one full tidal cycle. This measurement should be conducted during one of the spring tides where the amplitude of the tidal cycle will be largest. The gages should be referenced to the same datum and synchronized. The data from these recording gages are necessary for calibration of tidal hydraulic models such as ACES-INLET [50] or DYNLET1 [51] which are recommended in the Level 2 analysis, or other unsteady 2-dimensional or quasi 2-dimensional hydraulic flow models. A more complete description of data requirements for model application is given in Section 4.6.4.

The data and evaluations suggested above can be used to estimate whether present conditions are likely to continue into the foreseeable future and as a basis for evaluating the hydraulics and total scour for the Level 2 analysis. A stable inlet could change to one which is degrading if the channel is dredged or jetties are constructed on the ocean side to improve the entrance, since dredging or jetties could modify the supply of sediment to the inlet. In addition, plans or projects which might interrupt existing conditions of littoral drift should be evaluated.

It should be noted that in contrast to an upland river crossing, the discharge at a tidal inlet is not fixed. In inland rivers, the design discharge is fixed by the runoff and is virtually unaffected by the waterway opening. In contrast, the discharge at a tidal inlet can increase as the area of the tidal inlet increases, thus increasing long-term and local scour. Also, as Neill [48] points out, constriction of the natural waterway opening may modify the tidal regime and associated tidal discharge.

4.6.4 Level 2 Analysis

Introduction. **Level 2 analysis involves the basic engineering assessment of scour problems at highway crossings.** At the present time, there are no suitable scour equations which have been developed specifically for tidal flows. Because of this, it is recommended that the scour equations developed for inland rivers be used to estimate and evaluate scour. However, in contrast to the evaluation of scour at inland river crossings, the evaluation of the hydraulic conditions at the bridge crossing using either WSPRO [29] or HEC-2 [30] is not usually suitable for tidal flows.

Several methods to obtain hydraulic characteristics of tidal flows at the bridge crossing are recommended. These range from simple procedures to more complex 2-dimensional and quasi 2-dimensional unsteady flow models. Use of the simpler hydraulic procedures will be illustrated in example problems at the end of this section.

Evaluation of Hydraulic Characteristics. The velocity of flow, depth, and discharge at the bridge waterway are the most significant variables for evaluating bridge scour in tidal waterways. Direct measurements of the value of these variables for the design storm are seldom available. Therefore, it is usually necessary to develop the hydraulic and hydrographic characteristics of the tidal waterway, estuary or bay, and calculate the discharge, velocities, and depths in the crossing using coastal engineering equations. These values can then be used in the scour equations given in previous sections to calculate long-term aggradation or degradation, contraction scour, and local scour.

Unsteady quasi 2-dimensional mathematical models such as ACES-INLET [50] and DYNLET1 [51] can be used to model the hydraulic characteristics at the bridge. These computer models are available from the U.S. Army Corps of Engineers. ACES-INLET is restricted to analysis of tidal inlet with up to two inlets to a bay; whereas, DYNLET1 can be used for multiple tidal inlets, tidal estuaries, tidal affected streams and bridge crossings in passages to islands. Currently, research is being conducted to either adapt these or other existing models so that they can be better suited to the assessment of scour at bridges or to develop new models.

Although these unsteady flow models are suitable for determining the hydraulic conditions, their use requires careful application and calibration. The effort required to utilize these models may be more than is warranted for many tidal situations. As such, the use of these models may be more applicable under a Level 3 analysis. However, these models could be used in the context of a Level 2 analysis, if deemed necessary, to better define the hydraulic conditions at the bridge crossing.

Alternatively, either a procedure by Neill [47] or an orifice equation for constricted tidal inlets can be used to evaluate the hydraulic conditions at bridges influenced by tidal flows. A step-wise procedure for using these two methods to determine hydraulic conditions and scour is presented as a prelude to the example problems presented later.

The procedure developed by Neill [47] can be used for unconstricted tidal inlets. This method, which assumes that the water surface in the tidal prism is level, and the basin has vertical sides, can be used for locations where the boundaries of the tidal prism can be well defined and where heavily vegetated overbank areas or large mud flats represent only a small portion of the inundated area. Thick vegetation tends to attenuate tide levels due to friction loss, thereby violating the basic assumption of a level tidal prism. The discharges may be over estimated using this procedure if vegetation will attenuate tidal levels. In some complex cases, a simple tidal routing technique or 2-dimensional flow models may need to be used instead of this procedure.

The selection of which procedure to use depends on whether or not the inlet is constricted. In general, inlets to large bays as illustrated in Figure 18 can usually be classified as constricted, whereas estuaries, which are also depicted on Figure 18 can be classified as unconstricted. However, these guidelines cannot be construed as absolute.

Observation of an abrupt difference in water surface elevation during the normal ebb and flow (astronomical tide) at the inlet (during a Level 1 analysis) is a clear indication that the inlet is constricted. However, the observation of no abrupt change in water surface during astronomical tidal fluctuations does not necessarily indicate that the inlet will be unconstricted when extreme tides such as a storm surge occurs. In some cases, it may be necessary to compute the tidal hydraulics using both tidal prism and orifice procedures. Then, the worst-case hydraulic parameters would be used for the computation of scour.

Velocity measurements made at the bridge site (see Level 1) can be useful in determining whether or not the inlet is constricted as well as for calibration or verification of the tidal computation procedure. Using tidal data at the time that velocity measurements were collected, computed flow depths, velocities and discharge can be compared and verified to measured values. This procedure can form a basis for determining the most appropriate hydraulic computation procedure and for adjusting the parameters in these procedures to better model the tidal flows.

Design Storm and Storm Surge. Normally, long-term aggradation and degradation at a tidal inlet or estuary are influenced primarily by the periodic tidal fluctuations associated with astronomical tides. Therefore, flow hydraulics at the bridge should be determined considering the tidal range as depicted in Figure 19 for evaluation of long-term aggradation and degradation.

Extreme events associated with floods and storm surges should be used to determine the hydraulics at the bridge to evaluate local and contraction scour. Typically, events with a return period corresponding to the 100- and 500-year storm surge and flood need to be considered. Difficulty arises in determining whether the storm surge, flood or the combination of storm surge and flood should be considered controlling.

When inland flood discharges are small in relationship to the magnitude of the storm surge and are the result of the same storm event, then the flood discharge can be added to the discharge associated with the design tidal flow, or the volume of the runoff hydrograph can be added to the volume of the tidal prism. If the inland flood and the storm surge may result from different storm events, then, a joint probability approach may be warranted to determine the magnitude of the 100- and 500-year flows.

In some cases there may be a time lag between the storm surge discharge and the stream flow discharge at the highway crossing. For this case, stream flow routing methods such as the U.S. Army Corps of Engineers HEC-1 model [52] can be used.

For cases where the magnitude of the inland flood is much larger than the magnitude of the storm surge, evaluation of the hydraulics reduces to using the equations and procedures recommended for inland rivers. The selection of the method to use to combine flood and tidal surge flows is a matter of judgment and must consider the characteristics of the site and the storm events.

Scour Evaluation Concepts. The total scour at a bridge crossing can be evaluated using the scour equations recommended for inland rivers and the hydraulic characteristics determined using the procedures outlined in the previous sections. However, it should be emphasized that the scour equations and subsequent results need to be carefully evaluated considering other (Level 1) information from the existing site, other bridge crossings, or comparable tidal waterways or tidally affected streams in the area.

Evaluation of long-term aggradation and degradation at tidal highway crossings, as with inland river crossings, relies on a careful evaluation of the past, existing and possible future condition of the site. This evaluation is outlined under Level 1 and should consider the principles of sediment continuity. A longitudinal sonic sounder survey of a tide inlet is useful to determine if bed material sediments can be supplied to the tidal waterway from the bay, estuary or ocean. When available, historical sounding data should also be used in this evaluation. Factors which could limit the availability of sediment should also be considered.

Over the long-term in a stable tidal waterway, the quantity of sediment being supplied to the waterway by ocean currents, littoral transport and upland flows and being transported out of the tidal waterway are nearly the same. If the supply of sediment is reduced either from the ocean or from the bay or estuary, a stable waterway can be transformed into a degrading waterway. In some cases, the rate of long-term degradation has been observed to be large and deep. An estimate of the maximum depth that this long-term degradation can achieve can be made by employing Laursen's clear-water contraction scour equation (Equation 18) to the inlet. For this computation the flow hydraulics should be developed based on the range of mean tide as described in Figure 19. It should be noted that the use of this equation would provide an estimate of the worst case long-term degradation which could be expected assuming no sediments were available to be transported to the tidal waterway from the ocean or inland bay or estuary. As the waterway degrades, the flow conditions and storage of sediments in shoals will change, ultimately developing a new equilibrium. The presence of scour resistant rock would also limit the maximum long-term degradation.

Potential contraction scour for tidal waterways also needs to be carefully evaluated using hydraulic characteristics associated with the 100- and 500-year storm surge or inland flood as described in the previous section. For highway crossings of estuaries, where either the channel narrows naturally or where the channel is narrowed by the encroachment of the highway embankments, the live-bed or clear water contraction scour equations (Equations 16 or 18) can be utilized to estimate contraction scour.

Soil boring or sediment data are needed in the waterway upstream, downstream, and at the bridge crossing in order to determine if the scour is "clear-water" or "live-bed" and to support the scour calculations if Laursen's clear-water contraction scour equation is used. Equations 14 or 15 can be used to assess whether the scour is likely to be clear-water or live bed.

The live-bed contraction scour equation can be applied to estuaries to estimate contraction scour because the variables which are needed for these equations (i.e., the ratios of widths and discharges) can be determined based on the geometry of the estuary and highway crossing, and discharges passing through the bridge and in the channel upstream of the bridge. However, for inlets to bays the geometry of the bay and inlet differs significantly from the geometry and characteristics of flow for which the live-bed contraction scour equation was developed. Unless the bridge crosses a long inlet for which live-bed conditions can be fully developed on the inland or ocean side of the bridge, there is no live-bed contraction scour equation which can be recommended for estimating contraction scour at inlets to bays.

Although the clear-water contraction scour equation is of a form that it can be applied to inlets to bays for the assessment of contraction scour, the magnitude of the discharge associated with the storm surge will most likely result in an extremely conservative estimate of contraction scour. This is a result of one or more of the mitigating factors influencing contraction scour discussed below.

Mitigating factors concerning contraction scour relate to the assumptions for which the contraction scour equations were developed, and the resulting deviation from these assumptions when applying these equations for estimation of contraction scour at estuaries and bays. These mitigating factors apply to both clear-water and live-bed contraction scour.

The contraction scour equations were developed considering a long contraction and assuming either a constant discharge or a flood hydrograph with a long duration. The discharge hydrograph associated with tidal surges typically rise and recede more rapidly than flood hydrographs. As such, the duration of the peak flow in tidal inlets and estuaries due to a storm surge is significantly shorter than the duration of peak flows for an equivalent flood hydrograph. Because of this, maximum contraction scour as computed using the contraction scour equations will, in most cases, not fully develop.

Another mitigating factor which will tend to limit contraction scour concerns sediment delivery to the inlet or estuary from the ocean due to the storm surge and inland flood. A tidal surge can transport large quantities of sediment into the inlet or estuary during the flood tide. Likewise, upland floods can also transport sediment to the bay or estuary during extreme floods. Thus, contraction scour during extreme events will rarely be classified as clear-water because of the sediment being delivered to the inlet or estuary from the combined effects of the storm surge and flood tide.

From the above discussion, contraction scour equations as presented earlier in this chapter for inland river crossings can be applied for estuaries. However, the use of the live-bed equation for determining contraction scour at inlets to bays needs to be carefully evaluated. Whether the crossing is located at an inlet to a bay or at an estuary, the evaluation of contraction scour must be carefully evaluated using engineering judgment which considers the geometry of the crossing, estuary or bay, the magnitude and duration of the discharge associated with the storm surge or flood, the basic assumptions for which the contraction scour equations were developed, and mitigating factors which would tend to limit contraction scour.

Evaluation of the local scour at piers can be made by using the CSU equation as recommended for inland river crossings (Equation 21). This equation can be applied to piers in tidal flows in the same manner as given for inland bridge crossings. However, the flow velocity and depth will need to be determined considering the design flow event and hydraulic characteristics for tidal flows.

Scour Evaluation Procedure for an Unconstricted Waterway. This method applies only when the tidal waterway or the bridge opening does not significantly constrict the flow and uses the tidal prism method as discussed by Neill.[47]

STEP 1. Determine the net waterway area at the crossing as a function of elevation. Net area is the gross waterway area between abutments minus area of the piers. It is often useful to develop a plot of the area versus elevation.

STEP 2. Determine tidal prism volumes as a function of elevation. The volume of the tidal prism at successive elevations is obtained by planimetering successive sounding and contour lines and calculating volume by the average end area method. The tidal prism is the volume of water between low and high tide levels or between the high tide elevation and the bottom of the tidal waterway.

STEP 3. Determine the elevation versus time relation for the 100- and 500-year storm tides. The ebb and flood tide elevations can be approximated by either a sine or cosine curve. A sine curve starts at mean water level and a cosine curve starts at the maximum tide level. The equation for storm ebb tide that starts at the maximum elevation is:

$$y = A\ Cos\ \theta + Z \tag{75}$$

where

y = amplitude or elevation of the tide above mean water level, ft at time t
A = maximum amplitude of elevation of the tide or storm surge, ft. Defined as half the tidal range or half the height of the storm surge
θ = Angle in degrees subdividing the tidal cycle. One tidal cycle is equal to 360°.

$$\theta = 360 \left(\frac{t}{T}\right)$$

t = time in minutes from beginning of total cycle
T = total time for one complete tidal cycle, minutes
Z = vertical offset to datum, ft

The tidal range (difference in elevation between high and low tide) is equal to 2A. One-half the tidal period is equal to the time between high and low tide. These relations are shown in Figure 19. A figure similar to Figure 19, can be developed to illustrate quantitatively the tidal fluctuations and resultant discharges.

To determine the elevation versus time relation for the 100- and 500-year storm tides, two values must be known:

- the tidal range
- the tidal period

As stated earlier, FEMA, Corps of Engineers, NOAA and other federal or state agencies compile records which can be used to estimate the 100- and 500-year storm surge elevation, msl elevation, and low tide elevation. These agencies also are the source of data to determine the 100- and 500-year storm tide period.

Tides, and in particular storm tides, may have different periods than the major astronomical semi-diurnal and diurnal tides which have periods of approximately 12.5 and 25 hours, respectively. This is because storm tides are influenced by factors other than the gravitational forces of the sun, moon and other celestial bodies. Factors such as the wind, path of the hurricane or storm creating the storm tide, fresh water inflow, shape of the bay or estuary, etc. influence both the storm tide amplitude and period.

STEP 4. Determine the discharge, velocities and depth. Neill [47] has stated the maximum discharge in an ideal tidal estuary may be approximated by the following equation:

$$Q_{max} = \frac{3.14\ VOL}{T} \tag{76}$$

where

Q_{max} = maximum discharge in the tidal cycle, cfs
VOL = volume of water in the tidal prism between high and low tide levels, ft^3
T = tidal period, between successive high or low tides, s

In the idealized case, Q_{max} occurs in the estuary or bay at mean water elevation and at a time midway between high and low tides when the slope of the tidal energy gradient is steepest see Figure 19.

The corresponding maximum average velocity in the waterway is:

$$V_{max} = \frac{Q_{max}}{A'} \tag{77}$$

where

V_{max} = maximum average velocity in the cross section at Q_{max}, ft/s
A' = cross-sectional area of the waterway at mean tide elevation, halfway between high and low tide, ft^2

It should be noted that the velocity as determined in the above equations represents the average velocity in the cross section. This velocity will need to be adjusted to estimate velocities at individual piers to account for non-uniformity of velocity in the cross section. As for inland rivers, local velocities can range from 0.9 to approximately 1.7 times the average velocity depending on whether the location in the cross section was near the banks or near the thalweg of the flow.

Neill's studies indicate that the maximum velocity in estuaries is approximately 30 percent greater than the average velocity computed using Equation 76. If a detailed analysis of the horizontal velocity distribution is needed, the design discharge could be prorated based on the conveyance in subareas across the channel cross section.

Another useful equation from Neill [47] is:

$$Q_t = Q_{max} \sin\left(360\frac{t}{T}\right) \tag{78}$$

where

Q_t = discharge at any time t in the tidal cycle, cfs

The velocities calculated with this procedure can be plotted and compared with any measured velocities that are available for the bridge site or adjacent tidal waterways to evaluate the reasonableness of the results.

STEP 5. Evaluate the effect of flows derived from upland riverine flow on the values of discharge, depth and velocities obtained in Step 4. This evaluation may range from simply neglecting the upland flow into a bay (which is so large that the upland flow is insignificant in comparison to the tidal flows), to routing the upland flow into the bay or estuary. If an estuary is a continuation of the stream channel and the storage of water in it is small, the upland flow can simply be added to the Q_{max} obtained from the tidal analysis and the velocities then calculated from Equation 77. However, if the upland flow is large and the bay or estuary sufficiently small that the upland flow will increase the tidal prism, the upland flood hydrograph should be routed through the bay or estuary and added to the tidal prism. HEC-1 of the Corps of Engineers could be used to route the flows. In some instances, trial calculations will be needed to determine if and how the upland flow will be included in the discharge through the bridge opening.

STEP 6. Evaluate the discharge, velocities and depths that were determined in Steps 4 and 5 above (or the following section for constricted waterways). Use engineering judgment to evaluate the reasonableness of these hydraulic characteristics. Compare these values with values for other bridges over tidal waterways in the area with similar conditions. Compare the calculated values with any measured values for the site or similar sites. Even if the measured values are for tides much lower than the design storm tides they will give an appreciation of the magnitude of discharge to be expected.

STEP 7. Evaluate the scour for the bridge using the values of the discharge, velocity and depths determined from the above analysis using the scour equations recommended for inland bridge crossings presented previously. Care should be used in the application of these scour equations, using the guidance given previously for application of the scour equations to tidal situations.

Scour Evaluation Procedure for a Constricted Waterway.

a. The procedures given above except for Steps 2 and 4 (the determination of the tidal prism, discharge, velocity and depth for nonconstricted waterways) are followed. To determine these hydraulic variables when the constriction is caused by the channel and not the bridge, the following equation for tidal inlets taken from van de Kreeke [53] or Bruun [54] can be used.

$$V_{max} = C_d \, (2g \ \Delta h)^{1/2} \tag{79}$$

$$Q_{max} = A' \, V \tag{80}$$

where

V_{max} = maximum velocity in the inlet, fps
Q_{max} = maximum discharge in the inlet, cfs
C_d = coefficient of discharge ($C_d < 1.0$)
g = acceleration due to gravity, 32.2 ft/s^2
Δh = maximum difference in water surface elevation between the bay and ocean side of the inlet or channel, ft
A' = net cross-sectional area in the inlet at the crossing, at mean water surface elevation, ft^2

The coefficient of discharge (C_d) given by van de Kreeke [53] or Bruun [54] when the channel constricts the flow is:

$$C_d = (1/R)^{1/2} \tag{81}$$

where

$$R = K_o + K_b + \frac{2g \ n^2 \ L_c}{1.49^2 \ h_c^{4/3}} \tag{82}$$

and

R = coefficient of resistance
K_o = velocity head loss coefficient on the ocean side or downstream side of the waterway
taken as 1.0 if the velocity goes to 0
K_b = velocity head loss coefficient on the bay or upstream side of the waterway. Taken as 1.0 if the velocity goes to 0
n = Manning's roughness coefficient
L_c = length of the waterway, ft
h_c = average depth of flow in the waterway at mean water elevation, ft

Tidal Calculations Using ACES. ACES [50] is an acronym for the Automated Coastal Engineering System and was developed by the Corps of Engineers in an effort to incorporate many of the various computational procedures typically needed for coastal engineering analysis into an integrated, menu-driven user environment. As such there are separate computation modules for wave prediction, wave theory, littoral processes and other useful modules. One such module denoted as ACES-INLET is a spatially integrated numerical model for inlet hydraulics. This module can be used to determine discharges, depths and velocities in tidal inlets with up to two inlets connecting a bay to the ocean. This module can be used in place of or in addition to the procedures given in Steps 3 and 4, above, for tidal inlets. ACES-INLET is applicable only where the project site is at or very near the inlet throat (i.e., for the bridges in Figure 20 crossing inlets).

Other modules incorporated into ACES may be useful in evaluating tidal highway crossings. These modules can be used to estimate wave and tidal parameters, littoral drift, wave run-up and other aspects of tidal flow which could influence the design or evaluation of river crossings in tidal inlets connecting bays to the ocean.

Tidal Calculations Using DYNLET1. DYNLET1 [51] is a quasi 2-dimensional numerical computer model for determining the discharge, depths and velocities in multi-channel tidal inlets, tidally affected waterways, and tidal flows between islands or islands and the mainland. The Corps of Engineers report describes DYNLET1 as "a simple model for use in reconnaissance-level quantitative studies." The model is a 1-dimensional formulation of the dynamic (time-dependent) behavior of tidal flow at inlets and is based on the full 1-dimensional shallow-water equations employing an implicit finite difference technique. The model is intended for personal computer (PC) users and facilitates numerical grid generation and data entry. DYNLET1 can model very complicated systems all the way to the head of tide. Thus, it can handle not only the bridges in Figure 20, but also tidally affected river crossings.

Documentation from the Corps of Engineers states that DYNLET1 can predict flow conditions in channels with varied geometry, and it accepts varying friction factors across an inlet channel and geometric boundary conditions. Values of water surface elevation and average velocity are computed at locations across and along inlet channels and displayed on the PC monitor and written to output files for further analysis. The inlet to be modeled may consist of a single channel connecting the sea to the bay, or it can be a system of interconnected channels, with or without bays. The principal limitation of DYNLET1 is potential inaccuracy in situations where strong 2-dimensional flow fields such as gyres, exist perpendicular to the major axis of the channel comprising the modeled inlet.

The model is quasi 2-dimensional (in the same sense that WSPRO is quasi 2-dimensional). DYNLET1 gives a description of the flow across the channel by partitioning channel discharge proportional to cross section bathymetry. It should be sufficiently accurate for the analysis of the majority of bridge crossings of tidal waterways.

Data Requirements for Model Verification Using ACES-INLET or DYNLET1. Ideally, synoptic measurements of the following data are required to validate modeling using ACES-INLET or DYNLET1:

- Tidal elevations in the ocean and back bay locations. For DYNLET1, the extent of the grid network will determine the number of back bay and tributary gages required. For ACES-INLET, the only back-bay boundary conditions are the area of the bay (obtained by planimetering) and total river discharge impacting the bay.

- Velocity measurements are needed in the inlet throat as well as at proposed project sites.

- Boundary condition data for any back-bay, open-water boundaries; these data may be elevation, velocity, discharge, or any combination of these parameters. This information is especially critical for validating DYNLET1.

- Wind speed and direction if wind energy influences in the tidal system.

The above data may be available from previous studies of the tidal system (for example, Corps of Engineers or NOAA studies) or may be collected for a specific project.

4.6.5 Level 3 Analysis

As discussed in HEC-20 [8], Level 3 analysis involves the use of physical models or more sophisticated computer models for complex situations where Level 2 analysis techniques have proven inadequate. In general, crossings that require Level 3 analysis will also require the use of qualified hydraulic engineers. Level 3 analysis by its very nature is specialized and beyond the scope of this manual.

4.6.6 Example Problem Number 1

In this example problem the discharge, velocity, depths, and scour are to be determined for an existing bridge across a tidal estuary as part of an ongoing scour evaluation. The bridge is 2,685 feet long, has vertical wall abutments and 16- 12 foot diameter circular piers supported on piles. Neither the bridge or the tidal waterway constricts the flow.

For this evaluation, the bridge maintenance engineer has expressed concern about observed scour at one of the piers. This pier is located where the velocities at the pier are approximately 30 percent greater than the average velocities. The water depth at the pier referenced to mean sea level is 12.3 feet. The actual depth of flow at the pier will need to be increased to account for additional water depth caused by the storm surge for the computation of pier scour.

Level 1 Analysis

a. Level 1 analysis has determined that the 100- and 500-year return period tidal storm surge discharge,velocity and depths are much larger than those from upland runoff. There is minimal littoral drift and historical tides are low. From FEMA the storm surge tide for the 100-year return period is 7.2 ft and 500-year return period is 9.4 ft. Measured maximum velocity in the waterway at mean water level for a high tide of 2.2 ft was only 0.68 ft/sec.

   Sonic soundings in the waterway indicate there is storage of sediment in the estuary directly inland from the bridge crossing. This was determined by observing that the elevation of the bed of the waterway at the bridge site was lower than the elevation of the bottom of the estuary further inland. Although no littoral drift is evident, there is storage of sediment at the mouth of the estuary between the ocean and the bridge crossing.

b. Stability of the estuary and crossing was evaluated by examination of the periodic bridge inspection reports which included underwater inspections by divers, evaluation of historical aerial photography, and depth soundings in the estuary using sonic fathometers. From this evaluation it was determined that the planform of the estuary has not changed significantly in the past 30 years. These observations indicate that the estuary and bridge crossing has been laterally stable.

   Evaluation of sounding data at the bridge indicates that there has been approximately 5 feet of degradation at the bridge over the past 30 years; however, the rate of degradation in the past 5 years has been negligible. Underwater inspections indicted that local scour around the piers is evident.

c. A search of FEMA, Corps of Engineers, and other public agencies for flood and storm surge data was conducted. These data will be discussed under the Level 2 analysis.

d. Grain size analysis of the bed material indicates that the bed of the estuary is composed of fine sand with a D_{50} of approximately 0.27 mm (.00089 ft).

e. Velocities measured at Q_{max} during a large tide indicated that the maximum velocity in the bridge section was approximately 30 percent greater than the average velocity.

Level 2 Analysis

STEP 1. A plot of net waterway area as a function of elevation is given in **Figure 21.** Net waterway area is the average area at the bridge crossing less the area of the piers.

STEP 2. A plot of volume of the tidal prism as a function of elevation is also presented in Figure 21. It was developed by planimetering the area of successive sounding and contour lines and multiplying the average area by the vertical distance between them.

STEP 3. A synthesized storm surge for the 100- and 500-year return period was developed and is presented in Figure 21. It was obtained as follows:

An idealized tidal cycle for one half the tidal period, beginning at high tide was developed using the cosine equation (Equation 75). This plot can be used to develop an idealized tidal cycle for any waterway. Tidal range and period are needed to use the idealized tide cycle to develop a synthesized tidal cycle for this waterway.

The tidal ranges were obtained from a FEMA coastal flood insurance study during the Level 1 analysis (**Table 11**).

Table 11. Tidal Ranges Derived from FEMA Flood Study.

Return Period	High Tide	Low Tide
100-year	7.2 ft.	0
500-year	9.4 ft.	0

The tidal period is more difficult to determine because it is affected by more than the gravitational attraction of the moon and sun. At this waterway location, the direction of the storm and the characteristics of the estuary affected the tidal period. To determine the tidal period, major storm tides were plotted, as the fourth plot in Figure 21. From a study of these major storm tides a period of 12 hours was selected as being a conservative estimate of the time from flood (high) to ebb (low) tide. Tidal period T is then 24 hours.

STEP 4. Using the data developed in Steps 1 to 3 and the equations given previously the maximum tidal discharge (Q_{max}) and maximum average tidal velocity (V_{max}) are calculated. The values used in the calculations are given in **Table 12.**

STEP 5. The 100- and 500-year return period peak upland flow into the estuary was obtained from a USGS flood frequency study. These values are also given in Table 12.

Average flow depths can be determined by dividing the flow area as listed in Table 12 by the channel width (2,685 feet). Therefore the average flow depth for the 100- and 500-year event are 14.5 and 15.2 feet, respectively.

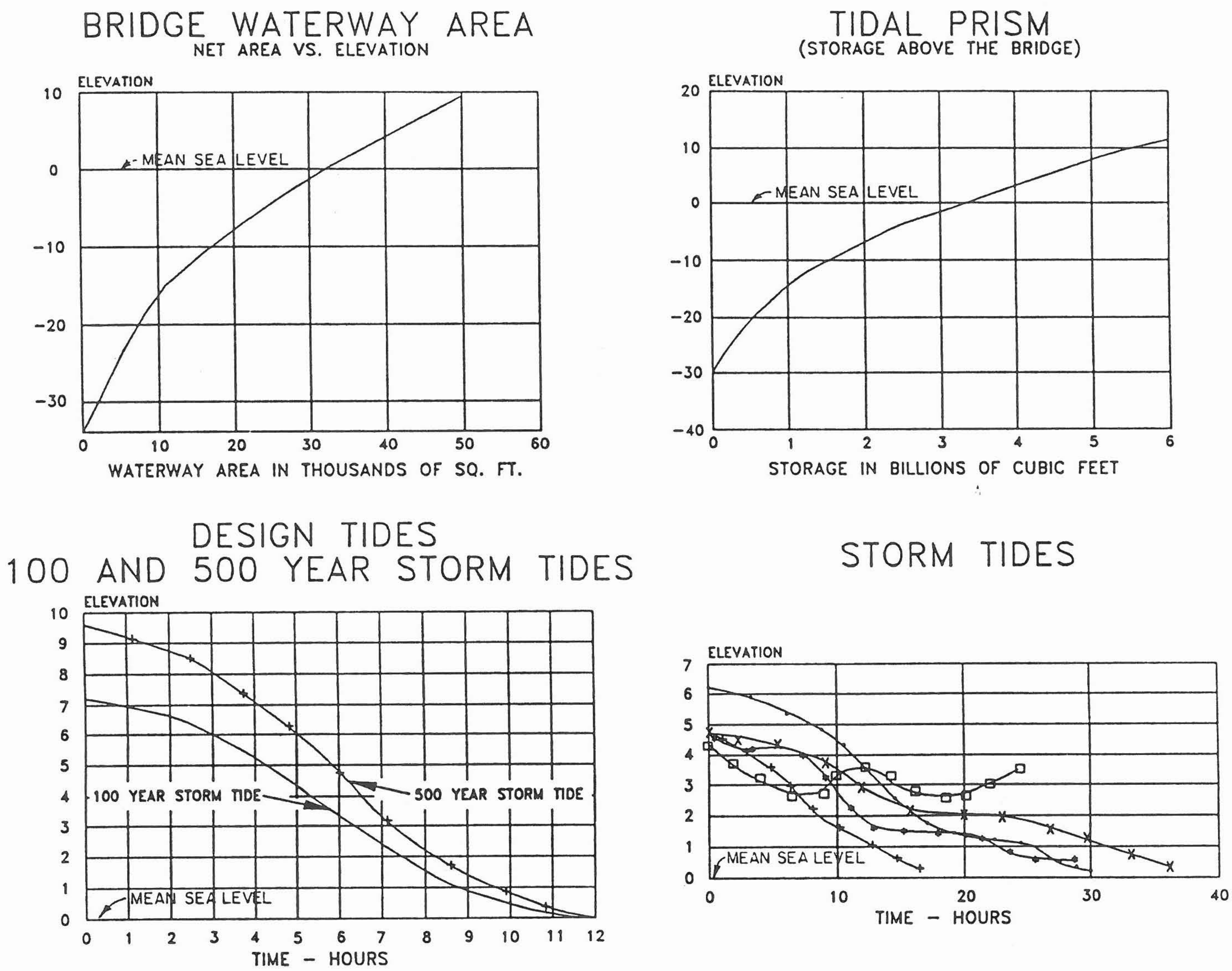


Figure 21. Tidal Parameters for Example Problem 1.

Table 12. Design Discharge and Velocities.

	100-Year Storm Tide	500-Year Storm Tide
Maximum storm tide elevation	7.2	9.4
Mean storm tide elevation	3.6	4.7
Low storm tide elevation	0.0	0.0
Tidal prism volume (millions of cubic feet) Figure 21	1,640	2,150
Net waterway area at mean storm tide elevation (A') square foot	39,000	41,000
Tidal period (T) hours	24.0	24.0
Q_{max} Tidal-cfs (Equation 76)	59,600	78,100
V_{max} Tidal-cfs (Equation 77)	1.53	1.91
Upland peak runoff cfs	4,980	7,920
Q_{max} (Tidal plus runoff) cfs	64,580	86,020
V_{max} (Tidal plus runoff) cfs ($V_{max} = Q_{max}/A'$)	1.66	2.1
Average flow depth - A'/width, ft	14.5	15.3

The volume of the runoff from the 100- and 500-year upland flow hydrograph is very small in comparison to the storage volume in the estuary. In this case, adding the peak discharge to the maximum tidal discharge will be a conservative estimate of the maximum discharge and maximum average velocity in the waterway. If the upland inflow into the estuary had been large, the flood could be routed through the estuary using standard hydrologic modeling techniques.

STEP 6. A comparison of the calculated velocities with the measured velocities indicate that they are reasonable. Simply adding the peak inflow from the upland runoff results in a conservative estimate of the average velocity. Therefore, the discharge and velocities given in Table 12 are acceptable for determining the scour depths. However, the average velocity will have to be adjusted for the nonuniformity of flow velocity in the vicinity of the bridge to obtain the velocities for determining local scour at the piers.

STEP 7. Calculate the components of total scour using the information collected in the Level 1 and Level 2 analyses.

Long-Term Aggradation/Degradation. The Level 1 analysis indicates that the channel is relatively stable at this time. However,there is an indication that over the past 30 years the channel has degraded approximately 5 feet. Therefore, for this evaluation, an estimate of long term degradation of approximately 5 feet for the future will be assumed.

Contraction Scour. Contraction scour depends on whether the flow will be clear-water or live- bed. Equation 15 is used to determine the critical velocity for the 100-year hydraulics.

$$V_c = 10.95\ (14.5)^{1/6}\ (0.00089)^{1/3} = 1.65\ ft/s \tag{83}$$

This indicates that the 100-year storm surge combined with the inland flow will result in velocities greater than the critical velocity. Therefore contraction scour will be live-bed.

Applying the modified live-bed contraction scour equation, it is noted that the ratio of discharges is equal to unity. Therefore the contraction scour will be influenced by the contraction resulting from the bridge piers reducing the flow width at the bridge crossing. Using Equation 16, and assuming that the mode of sediment transport is mostly suspended load, the estimate of live-bed contraction scour for the 100-year event is:

$$\frac{Y_2}{14.5} = \left[\frac{2685}{2493}\right]^{0.69} = 1.05 \tag{84}$$

Therefore, the contraction scour for the 100-year event is approximately 0.72 feet. Re-computation for the 500-year event with an average flow depth of 15.3 feet results in an estimate of contraction scour of approximately 0.77 feet.

<u>Local Scour at Piers</u>. The hydraulic analysis estimates average velocities in the bridge cross section only. Because of this, an estimate of the maximum velocity at the bridge pier is made to account for non-uniform velocity in the bridge cross section. The average velocity will be increased by 30 percent since velocities for normal flows (Level 1) indicated that the maximum velocity were observed to be approximately 30 percent greater than the average. Therefore the maximum velocity for the 100- and 500-year event are 2.16 and 2.73 ft/s respectively.

K_3 will be equal to 1.1 since the bed condition at the bridge is plane-bed. The depth of flow at the pier for the 100- and 500-year storm surge is determined by adding the mean storm tide elevation from Table 12 to the flow depth at the pier referenced to mean sea level. From this, y_1 will be equal to 15.9 and 17.0 feet for the 100- and 500-year storm surge, respectively.

Applying the CSU equation (Equation 21) for the 100-year event:

$$\frac{Y_s}{15.9} = 2.0\ (1.0)\ (1.0)\ (1.1) \left[\frac{12}{15.9}\right]^{0.65} (0.095)^{0.43} = 0.67 \tag{85}$$

From the above equation, the local scour at the piers is estimated to be approximately 10.7 feet. Considering the 500-year event, the estimate of local pier scour is 11.8 feet.

4.6.7 Example Problem 2

This problem presents a Level 2 analysis of a bridge over a tidal inlet where the waterway constricts the flow and illustrates how depletion of sediment supplied to the tidal inlet can result in a continual and severe long-term degradation. The length of the inlet is 1,500 ft, the width is 400 ft, Manning's n is 0.03, depth at mean water level is 20 ft. and area A' is 8,200 ft^2. The D_{50} of the bed material is 0.30 mm or 0.00098 ft. and the D_m (1.25 D_{50}) is 0.375 mm or 0.00123 ft.

From tidal records the long-term average difference in elevation from the ocean to the bay, through the waterway, averaged for both the flood and ebb tide is 0.6 ft. The difference in elevation for the 100-year storm surge is 1.8 ft and for the 500-year storm surge is 2.9 ft.

a. Determine the long-term potential degradation that may occur because construction of jetties has cut off the delivery of bed sediments from littoral drift to the inlet.

For this situation, long-term degradation can be approximated by assuming clear-water contraction scour and using the average difference in water surface between the ocean and bay for the hydraulic computation using the orifice equations (Equations 79 through 82).

Using Equation 82, determine R

$$R = 0.7 + 1.0 + \frac{2g\ (0.03)^2\ 1{,}500}{1.49^2\ (20)^{4/3}} \tag{86}$$

R = 2.42

From Equation 81, determine C_d

$$C_d = \left(\frac{1}{2.42}\right)^{1/2} \tag{87}$$

C_d = 0.643

Using Equation 79 determine V_{max}

$$V_{max} = (0.643)\ (2g\ \ 0.6)^{0.5} \tag{88}$$

V_{max} = 4.0 ft/s

Using Equation 80, determine Q_{max}

$$Q_{max} = V_{max}\, A' = 4.0\ (8{,}200) \tag{89}$$

Q_{max} = 32,800 cfs

Potential long-term degradation is determined using Equation 20:

$$\frac{y_s}{y_1} = 0.13 \left[\frac{Q}{D_m^{\frac{1}{3}} y_1^{\frac{7}{6}} W} \right]^{\frac{6}{7}} -1 \tag{90}$$

where

y_s = depth of scour, ft
y_1 = depth of flow in the waterway, ft
Q = discharge in the waterway, cfs
D_m = effective mean diameter of the bed material (1.25 D_{50}), ft
D_{50} = median diameter of bed material. Use a weighted average of the material in the scour zone, ft
W = bottom width of the waterway, ft

$$\frac{y_s}{20} = 0.13 \left[\frac{32{,}800}{0.00123^{\frac{1}{3}} 20^{\frac{7}{6}} 400} \right]^{\frac{6}{7}} -1 \tag{91}$$

y_s = 18.4 ft.

Discussion of Potential Long-Term Degradation

This amount of scour would occur in some time period that would depend on the amount of sediment that was available from the bay and ocean side of the waterway to satisfy the transport capacity of the back and forth movement of the water from the flood and ebb tide. Even if there was no sediment inflow into the waterway, the time it would take to reach this depth of scour is not known. To determine the length of time would require the use of a tidal model such as ACES-INLET or DYNLET1, and conducting a sediment continuity analysis.

Using a tidal model and sediment continuity analysis, calculate the amount of sediment eroded from the waterway during a tidal cycle and determine how much degradation this will cause. Then using this new average depth, recalculate the variables and repeat the process. Knowing the time period of the tidal cycle, then the time to reach a scour depth of 18.4 ft. could be calculated for the case of no sediment inflow into the waterway. Estimates of sediment inflow in a tidal cycle could be used to determine the time to reach the above estimated contraction scour depth when there is sediment inflow. **When the long-term degradation reaches 18.4 ft the scouring may not stop. The reason for this is that the discharge in the waterway is not limited, as in the case of inland rivers, but depends on the amount of flow that can enter the bay in a half tidal cycle. As the area of the waterway increases the flood tide discharge increases because, as an examination of Equations 86 and 87 show the velocity does not decrease. There may be a slight decrease**

in velocity because the difference in elevation from the ocean and the bay might decrease as the area increases. However, R in Equation 82 decreases with an increase in depth.

Although the above discussion would indicate that long-term degradation would increase indefinitely, this is not the case. As the scour depth increases there would be changes in the relationship between the incoming tide and the tide in the bay or estuary, and also between the tide in the bay and the ocean on the ebb tide. This could change the difference in elevation between the bay and ocean. At some level of degradation the incoming or out-going tides could pick up sediment from either the bay or ocean which would then satisfy the transport capacity of the flow. Also, there could be other changes as scour progressed, such as accumulation of larger bed material on the surface (armor) or scour resistance rock which would decrease or stop the scour.

In spite of these limiting factors, the above problem illustrates the fact that with tidal flow, in contrast to river flow, as the area of the cross section increases from degradation there is no decrease in velocity and discharge.

b. Determine V_{max}, Q_{max} for the 100-year storm surge and a depth of 20 ft.

The values of R and C_d do not change.

$$V_{max} = 0.643\ (2\ g\ \ 1.8)^{0.5} \tag{92}$$

$$V_{max} = 6.92 \text{ fps}$$

$$Q_{max} = 56{,}770 \text{ cfs}$$

These values or similar ones depending on the long-term scour depth, would be used to determine the local scour at piers and abutments using equations given previously.

These values could also be used to calculate contraction scour resulting from the storm surge. However, the contraction scour depth so calculated would be so large that it is unlikely it could occur in the short time period of the storm surge.

Currently, research is being conducted by the Corps of Engineers and others in support of FHWA and State highway agencies bridge scour assessment to provide improved techniques for determined hydraulics and scour at tidal bridge crossings. However, this research has not been completed at the time of this publication.

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CHAPTER 5

EVALUATING THE VULNERABILITY OF EXISTING BRIDGES TO SCOUR

5.1 Introduction

Existing bridges over streams subject to scour should be evaluated to determine their vulnerability to floods and whether they are scour vulnerable (Technical Advisories 5140.23, 1991).[5] This assessment or evaluation should be conducted by an interdisciplinary team of professional, experienced engineers who can make the necessary engineering judgments to determine:

1. Priorities for making bridge scour evaluations;
2. The scope of the scour evaluations to be performed in the office and in the field;
3. Whether or not a bridge is vulnerable to scour damage; i.e., whether the bridge is a scour-critical bridge;
4. Which alternative scour countermeasures would be applicable to make a bridge less vulnerable;
5. Which countermeasure is most suitable and cost-effective for a given bridge;
6. Priorities for installing scour countermeasures;
7. Monitoring and inspection schedules for scour-critical bridges; and
8. 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 immediately to eliminate or minimize the risk.

5.2 The Evaluation Process

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

STEP 1. Screen all bridges over waterways into three categories: (1) Low risk, (2) scour susceptible, or (3) unknown foundations. Bridges which are particularly vulnerable to scour failure should be identified immediately and the associated scour problem addressed. These particularly vulnerable "scour susceptible" bridges are:

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, etc.

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

- piers and abutments designed with spread footings or short pile foundations;
- superstructures with simple spans or nonredundant support systems that render them vulnerable to collapse in the event of foundation movement; and
- 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:

- active degradation or aggregation of the streambed;
- significant lateral movement or erosion of streambanks;
- steep slopes or high velocities;
- in-stream materials mining operations in the vicinity of the bridge; and
- histories of flood damaged highways and bridges.

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

- crossings near stream confluences, especially bridge crossings of tributary streams near their confluence with larger streams;
- crossings on sharp bends in a stream; and
- locations on alluvial fans.

STEP 2. Prioritize the scour susceptible bridges and bridges with unknown foundations, 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 Department of Transportation'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 in 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 the same as the check procedure set forth in Section 3.2, Step 8. 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. An overtopping flood will be used where applicable. 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 existing 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. As with the design of a new bridge, engineering judgment must be exercised in establishing the total scour depth for an existing bridge. The maximum scour depths that the existing foundation can withstand are compared with the total scour depth. 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 cannot withstand the total 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" [6] (see Appendix E). Update the list of the scour-critical bridges.

- Bridges assessed as "low risk" for Item 113 (scour critical bridges) should be coded as an "8". This is a modification of the definition of Code 8, Item 113 which states "...for calculated scour conditions..."

- Bridges with unknown foundations should be coded as a "6" in Item 113, indicating that a scour evaluation/calculation has not been made. It is recommended that only those bridges with unknown foundations, which have observed scour, receive scour evaluation prior to the deployment of instrumentation currently being developed to determine foundation type and depth.

- Bridges assessed to be "scour susceptible" are coded as "6" for Item 113 until such time that further scour evaluations determine foundation conditions.

STEP 4. 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:

- Timely installation of temporary scour countermeasures such as riprap.

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

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

b. Establishing a time table for Step 5 discussed below.

STEP 5. 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.

5.3 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 Q_{500} flood or a flow 1.7 times Q_{100}).

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, paragraph 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 are not available, it may have to be recalculated. For such instances, a "worst-case analysis" is suggested. If the bridge foundations are adequate for worst-case conditions, the bridge can be judged satisfactory. Where the worst-case analysis indicates that a scour problem may exist, further field and office analyses should be made.

5.4 Worst-Case Analysis

The following guide is offered for conducting a worst-case analysis:

5.4.1 Water-Surface Elevations

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 USGS 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.

5.4.2 Long-Term Aggradation and Degradation

Long-term streambed 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 streambed. 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 streambed elevation changes is included in Chapters 2, 3, and 4 and HEC-20.[8] 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.

5.4.3 Planform Changes

Assessing the significance of planform changes, such as the shifting location of meanders, the formation of islands, and the overall pattern of streams, usually cannot be accomplished in the office. Records and photographs taken by 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 judgment must be made as to whether possible future or existing planform changes represent a hazard to the bridge, and the extent of field work required to evaluate this condition.

5.4.4 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 up- and downstream of the bridge.

5.4.5 Local Pier Scour

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

5.4.6 Local Abutment Scour

Determination of local abutment scour using the procedures and 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.). It should be noted that the equations given in the literature are based on flume experiments and predict excessively conservative abutment scour depths.

5.5 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" [6] requires states to identify the current status of bridges regarding vulnerability to scour.

CHAPTER 6

INSPECTION OF BRIDGES FOR SCOUR

6.1 Introduction

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

1. To accurately record the present condition of the bridge and the stream; and

2. 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 interrelationship between the bridge, the stream, and the floodplain. Typically, a bridge spans the main channel of a stream and perhaps a portion of the floodplain. The road approaches to the bridge are typically on embankments which obstruct flow on the floodplain. 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 floodwater will expand back to the floodplain, 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 the use of 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."[6] (see Appendix E)

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" [6]) should be revised to reflect the effect of the scour on the substructure.

6.2 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:

1. Has an engineering scour evaluation study been made? If so, is the bridge scour critical?

2. If the bridge is scour critical, has a plan of action been made for monitoring the bridge and/or installing scour countermeasures?

3. What do comparisons of streambed cross sections taken during successive inspections reveal about the streambed? Is it stable? Degrading? Aggrading? Moving laterally? Are there scour holes around piers and abutments?

4. What equipment is needed (rods, poles, sounding lines, sonar, etc.) to obtain streambed cross sections?

5. Are there sketches and aerial photographs to indicate the planform location of the stream and whether the main channel is changing direction at the bridge?

6. What type of bridge foundation was constructed? (Spread footings, piles, drilled shafts, etc.) Do the foundations appear to be vulnerable to scour?

7. Do special conditions exist requiring particular methods and equipment (divers, boats, electronic gear for measuring stream bottom, etc.) for underwater inspections?

8. 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.)

6.3 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"[6] (see Appendix E) contains material for the following three items:

1. Item 60: Substructure,

2. Item 61: Channel and Channel Protection, and

3. 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.

6.3.1 Assessing the Substructure Condition

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."[6] 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:

1. Evidence of movement of piers and abutments;

    - 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.

2. Damage to scour countermeasures protecting the foundations (riprap, guide banks, sheet piling, sills, etc.),

3. Changes in streambed elevation at foundations (undermining of footings, exposure of piles), and

4. Changes in streambed cross section at the bridge, including location and depth of scour holes.

In order to evaluate the conditions of the foundations, the inspector should take cross sections of the stream, noting location and condition of streambanks. 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.

6.3.2 Assessing Scour Potential at Bridges

The items listed in **Table 13** 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 (see Appendix E).

Table 13. 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 jetties.

      UNSTABLE: Bank sloughing, undermining, evidence of lateral movement, damage to stream stabilization measures 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 streambed.
      - Evidence of movement of channel with respect to bridge (make sketches, take pictures).

   c. Floodplain

      - Evidence of significant flow on floodplain.
      - Floodplain 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 floodplain 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 13. Assessing the Scour Potential at Bridges (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 superstructure collect debris or present a large surface to the flow?)
- Design (Is superstructure vulnerable to collapse in the event of foundation movement, e.g., simple spans and nonredundant design for load transfer?)

c. Channel Protection and Scour Countermeasures

- Riprap (Is riprap adequately toed into the streambed 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 the stream and floodplain? Is there evidence of scour across a large portion of the streambed at the bridge? Do bars, islands, vegetation, and debris constrict the 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?

Table 13. Assessing the Scour Potential at Bridges (continued).

3. DOWNSTREAM CONDITIONS

a. Banks

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

UNSTABLE: Bank sloughing, undermining, evidence of lateral movement, damage to stream stabilization measures, 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 streambed.
- Evidence of movement of channel with respect to the bridge (make sketches and take pictures).

c. Floodplain

- Clear and open so that contracted flow at bridge will return smoothly to floodplain, or restricted and blocked by dikes, development, 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.)

6.3.3 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:

1. Use of divers; and

2. Use of electronic scour detection equipment (Appendix G).

For the purpose of evaluating resistance to scour of the substructure under Item 60 of the "Bridge Recording and Coding Guide," [6] the questions remain essentially the same for foundations in deep water as for foundations in shallow water:

1. What does the stream cross section look like at the bridge?

2. 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?

3. What are the shape and depths of scour holes?

4. Is the foundation footing (or the piling) exposed to the streamflow; and if so, what is the extent and probable consequences of this condition?

5. Has riprap around a pier been moved or removed?

6.3.4 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

7.1 Introduction

Scour countermeasures are those features incorporated after the initial construction of a bridge to make it less vulnerable to damage or failure from scour.

7.1.1 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:

1. Locating the bridge to avoid adverse flood flow patterns,
2. Streamlining bridge elements to minimize obstructions to the flow,
3. Design foundations safe from scour,
4. Founding bridges sufficiently deep to not require riprap or other countermeasures, and
5. Founding abutments above the estimated scour depth when the abutment is protected by well designed riprap or other suitable countermeasures.

7.1.2 Existing Bridges

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

1. Monitoring scour depths and closing bridge if excessive,
2. Providing riprap at piers and monitoring,
3. Providing riprap at abutments,
4. Constructing guide banks (spur dikes),
5. Constructing channel improvements,
6. Strengthening the bridge foundations,
7. Constructing sills or drop structures, and

8. 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:

1. Monitoring, inspecting and potentially closing a bridge until the countermeasures are installed,

2. Installing temporary scour countermeasures, such as riprap around a pier, along with monitoring a bridge during high flow,

3. Selecting and designing scour countermeasures, and

4. Scheduling construction of scour countermeasures.

These considerations are discussed in the following sections.

7.2 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 streambed degradation, aggradation, or lateral movement based on analysis of changes in stream cross sections taken during successive bridge inspections, sketches of the stream planform, aerial photographs, etc.

3. Recommended procedures and equipment for taking measurements of streambed 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. Develop a chain of command with authority to close bridges.

6. Instructions regarding the checking of streambed 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. Installation of scour depth warning devices.

7.3 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 combined with monitoring during and inspection after high flows could provide for the safety of the public without closing the bridge.

7.4 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.

Gradual river changes are common. 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.

7.5 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 (see also, HEC-20 [8]).

7.5.1 Rock Riprap at Piers and Abutments

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 does not 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 to insure that the riprap is stable.

Sizing Rock Riprap at Abutments. The FHWA conducted two research studies in a hydraulic flume to determine equations for sizing rock riprap for protecting abutments from scour.[55][56] One study investigated vertical wall and spill-through abutments which encroached 28 and 56 percent on the floodplain, respectively.[55] The second study investigated spill-through abutment which encroached on a floodplain with an adjacent main channel (see **Figure 22**). Encroachment varied from the largest encroachment used in the first study to a full encroachment to the edge of main channel bank. For spill-through abutments in both studies, the rock riprap consistently failed at the toe downstream of the abutment centerline (see **Figure 23**). For vertical wall abutments, the first study consistently indicated failure of the rock riprap at the toe upstream of the centerline of the abutment.

For Froude Numbers $V/(gy)^{1/2} \leq 0.80$, the recommended design equation for sizing rock riprap for spill-through and vertical wall abutments is in the form of the Isbash relationship:

$$\frac{D_{50}}{y} = \frac{K}{(S_s - 1)} \left[\frac{V^2}{gy} \right] \qquad (93)$$

where

D_{50} = median stone diameter, ft
V = characteristic average velocity in the contracted section (explained below), ft/s
S_s = specific gravity of rock riprap
g = gravitational acceleration, ft/s^2
y = depth of flow in the contracted bridge opening, ft
K = 0.89 for a spill-through abutment
1.02 for a vertical wall abutment

For Froude Numbers >0.80, Equation 94 is recommended:[57]

$$\frac{D_{50}}{y} = \frac{K}{(S_s-1)}\left[\frac{V^2}{gy}\right]^{.14} \tag{94}$$

where

K = 0.61 for spill-through abutments
= 0.69 for vertical wall abutments

In both equations, the coefficient K, is a velocity multiplier to account for the apparent local acceleration of flow at the point of rock riprap failure. Both of these equations are envelop relationships that were forced to overpredict 90 percent of the laboratory data.[55][56][57]

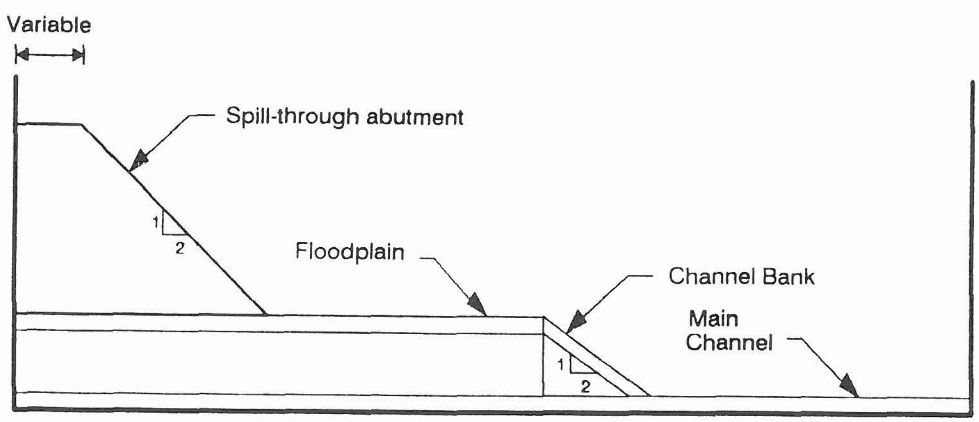

Figure 22. Section View of a Typical Setup of Spill-Through Abutment on a Floodplain with Adjacent Main Channel.

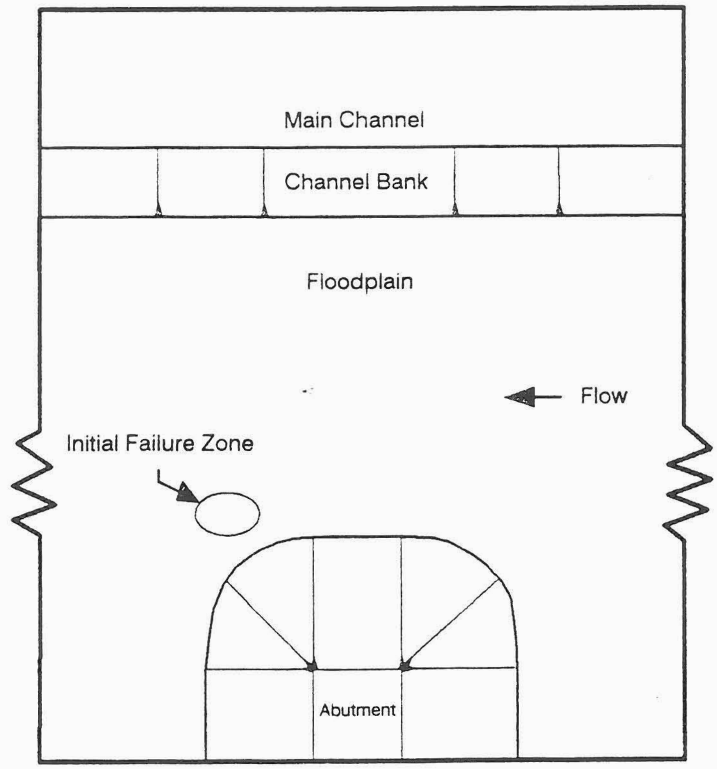

Figure 23. Plan View of the Location of Initial Failure Zone of Rock Riprap for Spill-Through Abutment.

A recommended procedure for selecting the characteristic average velocity is as follows:

1. Determine the set-back ratio (SBR) of each abutment. The set-back length is the distance from the near edge of the main channel to the toe of abutment.

   SBR = Set-back length/average channel flow depth

   a. If SBR is less than 5 for both abutments, compute a characteristic average velocity, Q/A, based on the entire contracted area through the bridge opening. This includes the total upstream flow, exclusive of that that overtops the roadway. The WSPRO average velocity through the bridge opening is also appropriate for this step.

   b. If SBR is greater than 5 for an abutment, compute a characteristic average velocity, Q/A, for the respective overbank flow only. Assume that the entire respective overbank flow stays in the overbank section through the bridge opening. This velocity can be approximated by a hand calculation using the cumulative flow areas in the overbank section from WSPRO, or from a special WSPRO run using an imaginary wall along the bank line.

c. If SBR for an abutment is less than 5 and SBR for the other abutment at the same site is more than 5, a characteristic average velocity determined from Step 1a for the abutment with SBR less than 5 may be unrealistically low. This would, of course, depend upon the opposite overbank discharge as well as how far the other abutment is set back. For this case, the characteristic average velocity for the abutment with SBR less than 5 should be based on the flow area limited by the boundary of that abutment and an imaginary wall located on the opposite channel bank. The appropriate discharge is bounded by this imaginary wall and the outer edge of the floodplain associated with that abutment.

2. Compute rock riprap size from Equations 93 and 94, based on the Froude Number limitation for these equations.

3. Determine extent of rock riprap

   a. The apron at the toe of the abutment slope 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 slopes.

   b. The apron should extend from the toe of the abutment into the bridge waterway a distance equal to twice the flow depth in the overbank area near the embankment, but need not exceed 25 feet (see **Figure 24**).[58]

   c. The abutment slope should be protected with rock riprap size computed from Equations 93 or 94. Coverage should agree with Step 3a.

   d. The rock riprap thickness should not be less than the larger of either 1.5 times D_{50} or D_{100}. The rock riprap thickness should be increased by 50 percent when it is placed under water to provide for the uncertainties associated with this type of placement.

   e. The rock riprap gradation and the potential need for underlying filter material must be considered.

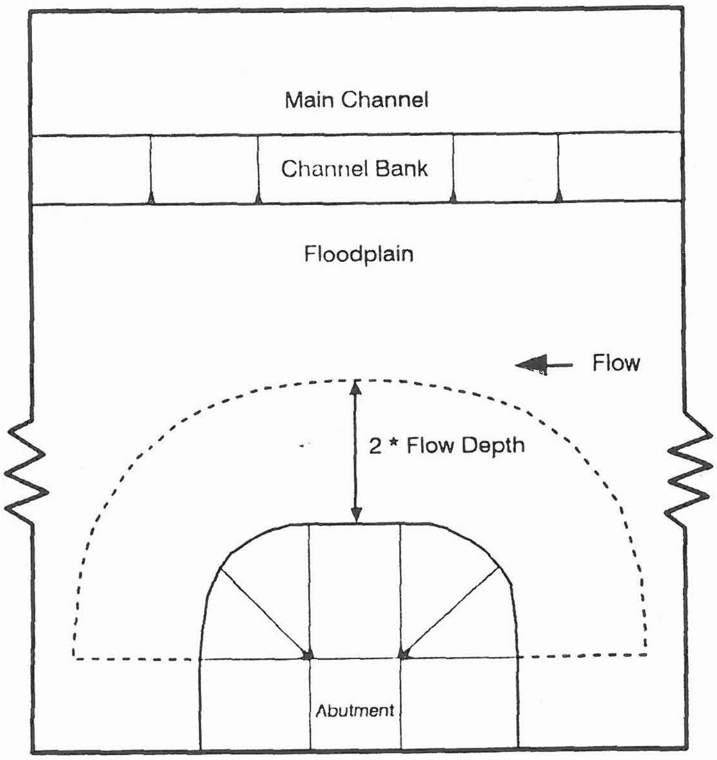

Figure 24. Plan View of the Extension of Rock Riprap Apron.

Sizing Riprap at Piers. **Riprap is not a permanent countermeasure for scour at piers for existing bridges and not to be used for new bridges.** Determine the D_{50} size of the riprap using the rearranged Ishbash equation (see HIRE [9]) to solve for stone diameter (in feet, for fresh water):

$$D_{50} = \frac{0.692(KV)^2}{(S_s-1)2g} \tag{95}$$

where

D_{50} = median stone diameter, ft
K = coefficient for pier shape
V = velocity on pier, ft
S_s = specific gravity of riprap (normally 2.65)
g = 32.2 ft/s^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.

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

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

   It is important to note that it is a disadvantage to bury riprap so that the top of the mat is below the streambed because inspectors have difficulty determining if some or all of the riprap has been removed. Therefore, it is recommended to place the top of a riprap mat at the same elevation as the streambed.

   a. The thickness of the riprap mat should be three stone diameters (D_{50}) or more.

   b. 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.

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

7.5.2 Guide Banks

Methods for designing guide banks are contained in the FHWA publication Hydraulic Design Series No. 1, "Hydraulics of Bridge Waterways" [59] and HEC-20.[8] The hydraulic effect of guide banks can be modeled through the use of the FHWA software, WSPRO.[29] The purpose of the guide bank is to provide a smooth transition for flows on the floodplain returning to the main channel at the bridge. The guide bank also serves to move the point of maximum scour upstream, away from the abutment and align flows through the bridge opening. Guide banks should be considered for protecting bridge abutments whenever there is a significant amount of flow on the floodplain that must return to the main channel at the bridge.

7.5.3 Channel Improvements

A wide variety of countermeasures are available for stabilizing and controlling flow patterns in streams.

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 approaches.

b. Countermeasures for degrading streams include the construction of sills and the strengthening of foundations as discussed in Item 5 (below).

c. Countermeasures for controlling lateral movement of a stream due to stream meanders include placement of dikes or jetties along the streambanks 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-20 [8] addresses this type of countermeasure in detail. Another useful reference is Transportation Research Board Record 950.[32]

7.5.4 Structural Scour Countermeasures

The use of structural designs to underpin existing foundations is discussed in the AASHTO Manual for Bridge Maintenance.[60] While structural measures may be more costly, they generally provide more positive protection against scour than countermeasures such as riprap.

7.5.5 Constructing Sills or Drop Structures

The use of sills and drop structures at bridges to stabilize the streambed and counteract the affects of degradation is discussed in FHWA publications.[8,9]

7.5.6 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 is applicable to implementation. In some locations with very unstable banks, additional spans may be more cost effective than attempting to stabilize the channel banks in the vicinity of the bridge.

7.6 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)," [2] is recommended as a guide for reviewing the performance of the countermeasures discussed above. This document is summarized in Chapter 5 of HEC-20.[8]

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APPENDICES

A. Alternate Scour Analysis Method

B. Equations for Abutment Scour

C. WSPRO Input and Output for Example Problem

D. North Carolina Scour Evaluation Procedures

E. FHWA 1988 Recording and Guide Coding for Structure Inventory

F. Scour Analysis for South Platte River, Colorado

G. Scour Detection Equipment

H. Ilustrations of the Four Cases of Contraction Scour

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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:

- estimate the natural channel's hydraulics for a fixed bed condition based on existing site conditions;
- estimate the expected profile and plan form changes based on the procedures in this manual and any historic data;
- adjust the natural channel's hydraulics based on the expected profile and plan form changes;
- select a trial bridge opening and compute the bridge hydraulics;
- estimate contraction scour;
- 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);
- using the foregoing revised bridge and channel hydraulic variables and parameters obtained considering the contraction scour, calculate the local scour; and
- extend the local scour depths below the predicted contraction scour depths in order to obtain the total scour.

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APPENDIX B

EQUATIONS FOR ABUTMENT SCOUR

In this appendix, scour at abutments is divided into its va cases and equations are given for each case (See Table B.1 Figures B.1 to B.3). These equations are given for the desi... 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.

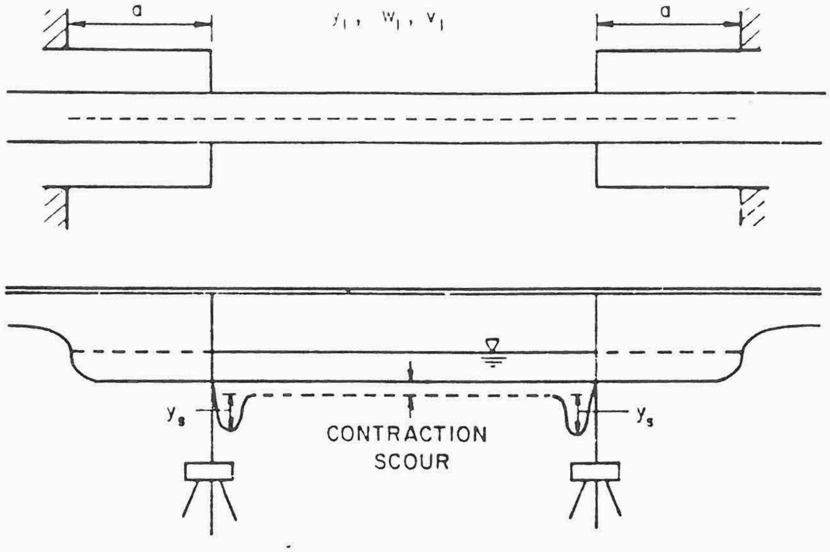
1. Equations for these cases (except for Case 6) are based on laboratory studies with little or no field data.
2. The factor a/y_1 = 25 as a limit for Cases 1-5 is rather arbitrary, but it is not practical to assume that scour depth, y_s, would continue to increase with an increase in abutment length "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.

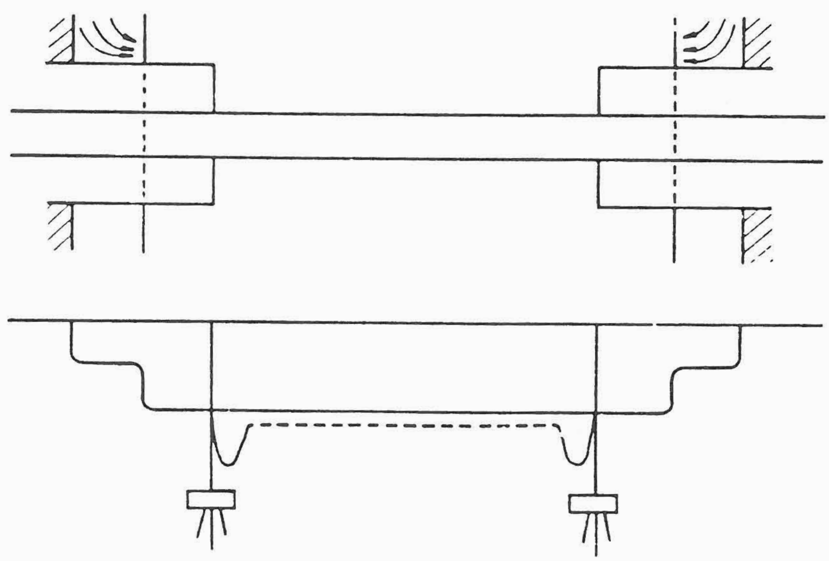
<u>IT IS MOST IMPORTANT THAT THE COMMENTARY ON EACH OF THE EQUATIONS BE READ AND UNDERSTOOD PRIOR TO ATTEMPTING TO USE THE EQUATIONS FOR DESIGN PURPOSES.</u> Engineering judgment must be used to select the depth of foundations. The designer should take into consideration the potential cost of repairs 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

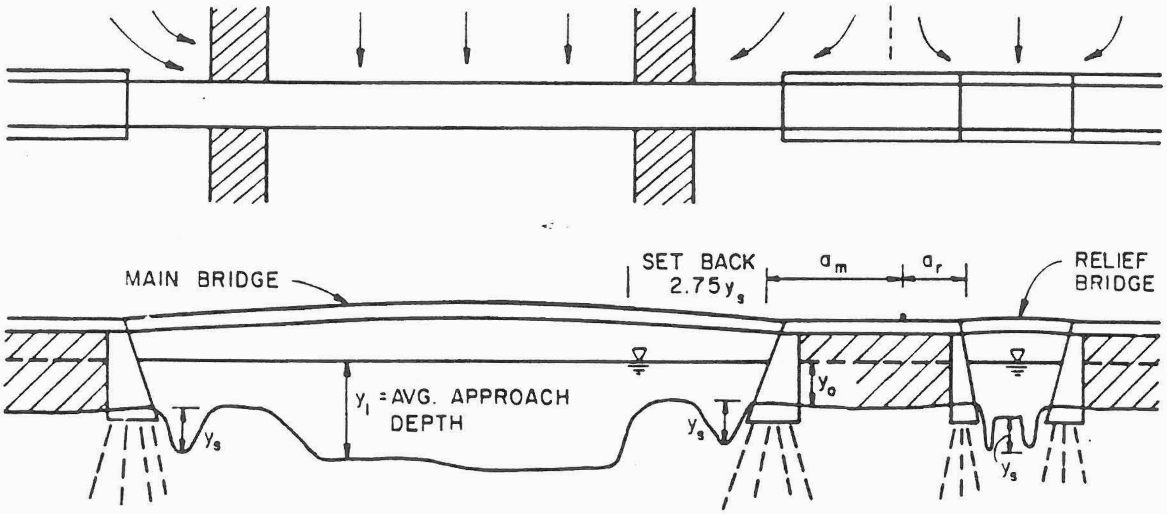


CASE 1 ABUTMENTS PROJECT INTO CHANNEL, NO OVERBANK FLOW

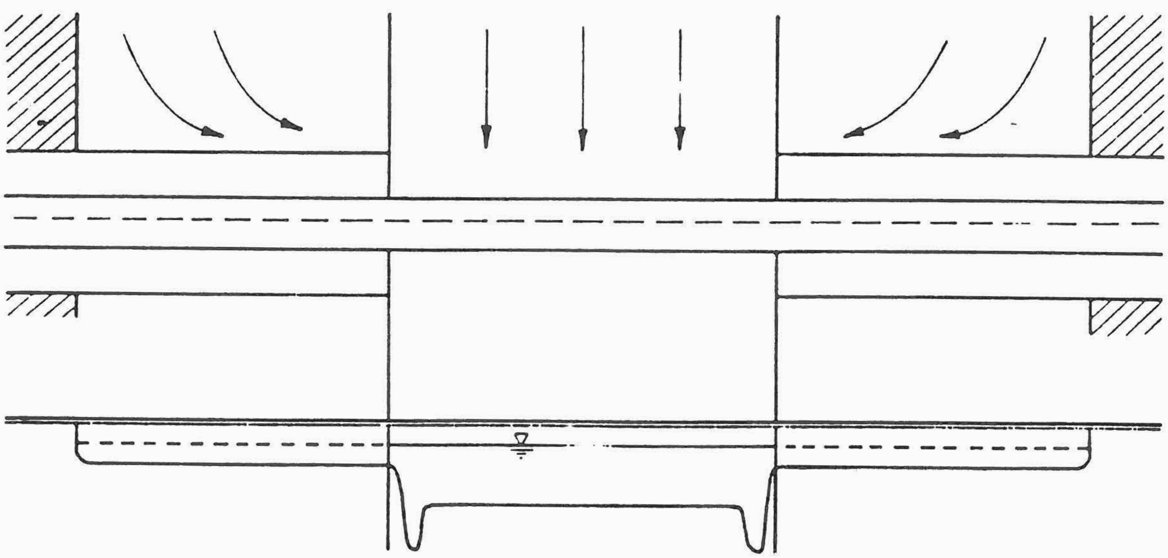


CASE 2 ABUTMENTS PROJECT INTO CHANNEL, OVERBANK FLOW

FIGURE B.1 ABUTMENT SCOUR CASES 1 AND 2.

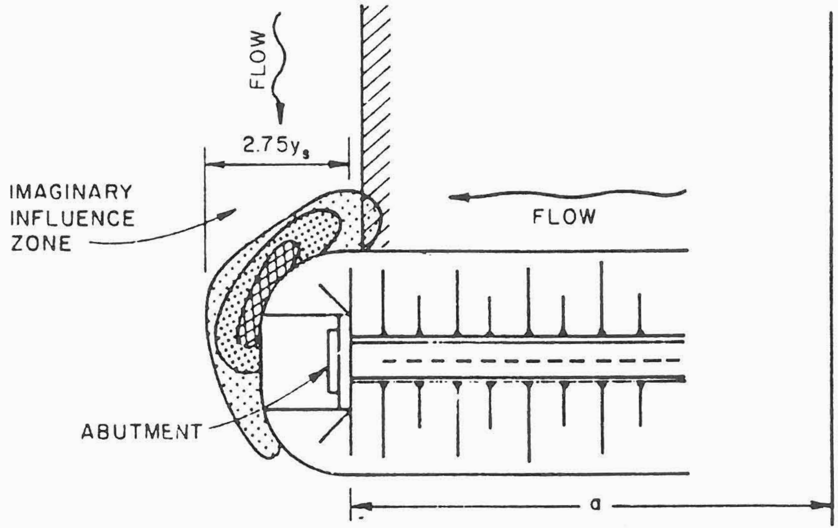


CASE 3 ABUTMENT SETBACK FROM THE CHANNEL MORE THAN 2.75 y_s.
CASE 4 RELIEF BRIDGE

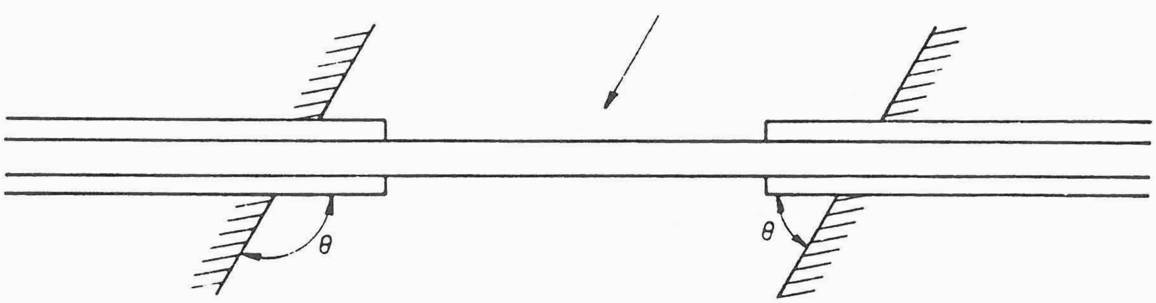


CASE 5 ABUTMENT SET AT EDGE OF CHANNEL, OVERBANK FLOW

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.

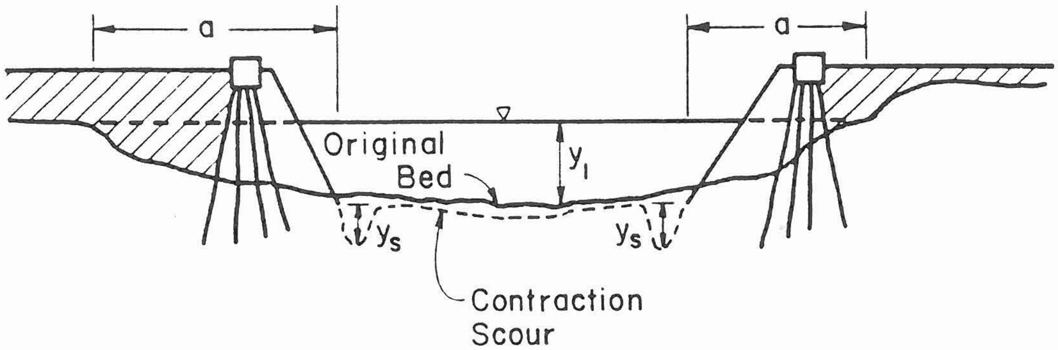


FIGURE B.4   DEFINITION SKETCH FOR CASE 1 ABUTMENT SCOUR

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 nc 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.

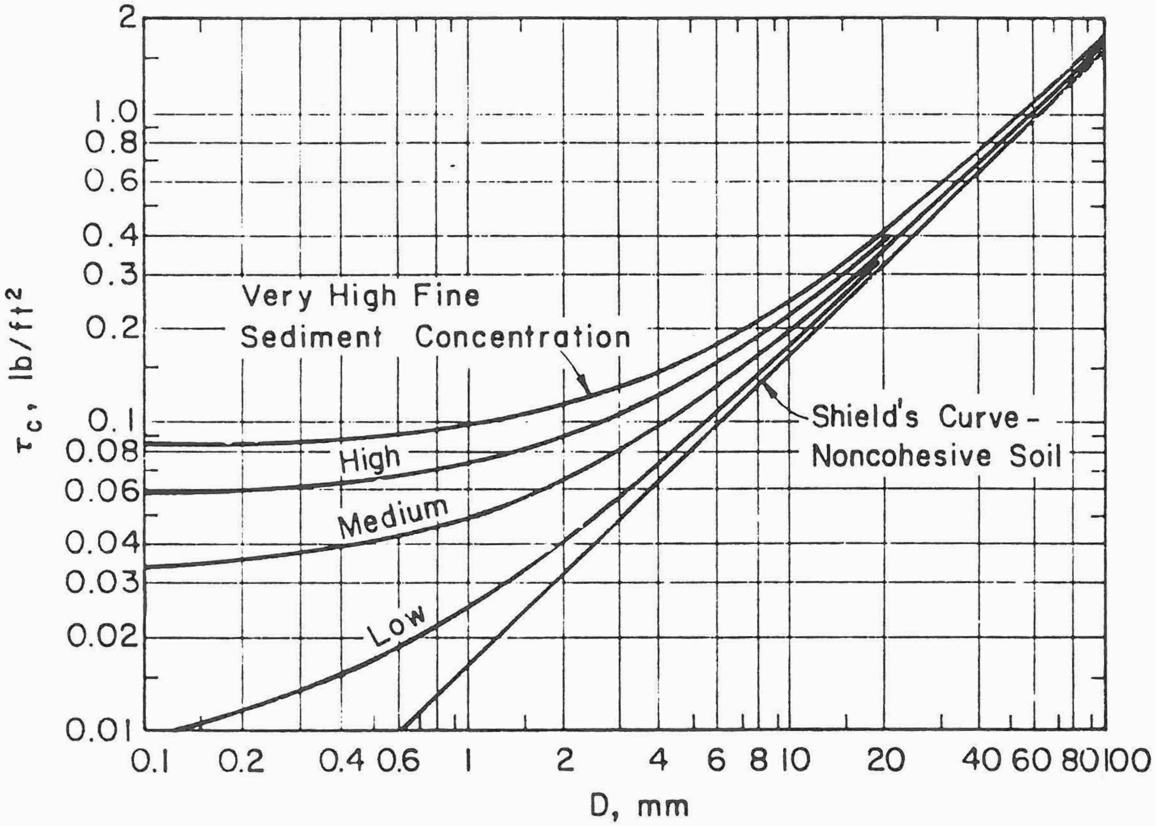


FIGURE B.5 CRITICAL SHEAR STRESS AS A FUNCTION OF BED MATERIAL SIZE AND SUSPENDED FINE SEDIMENT.

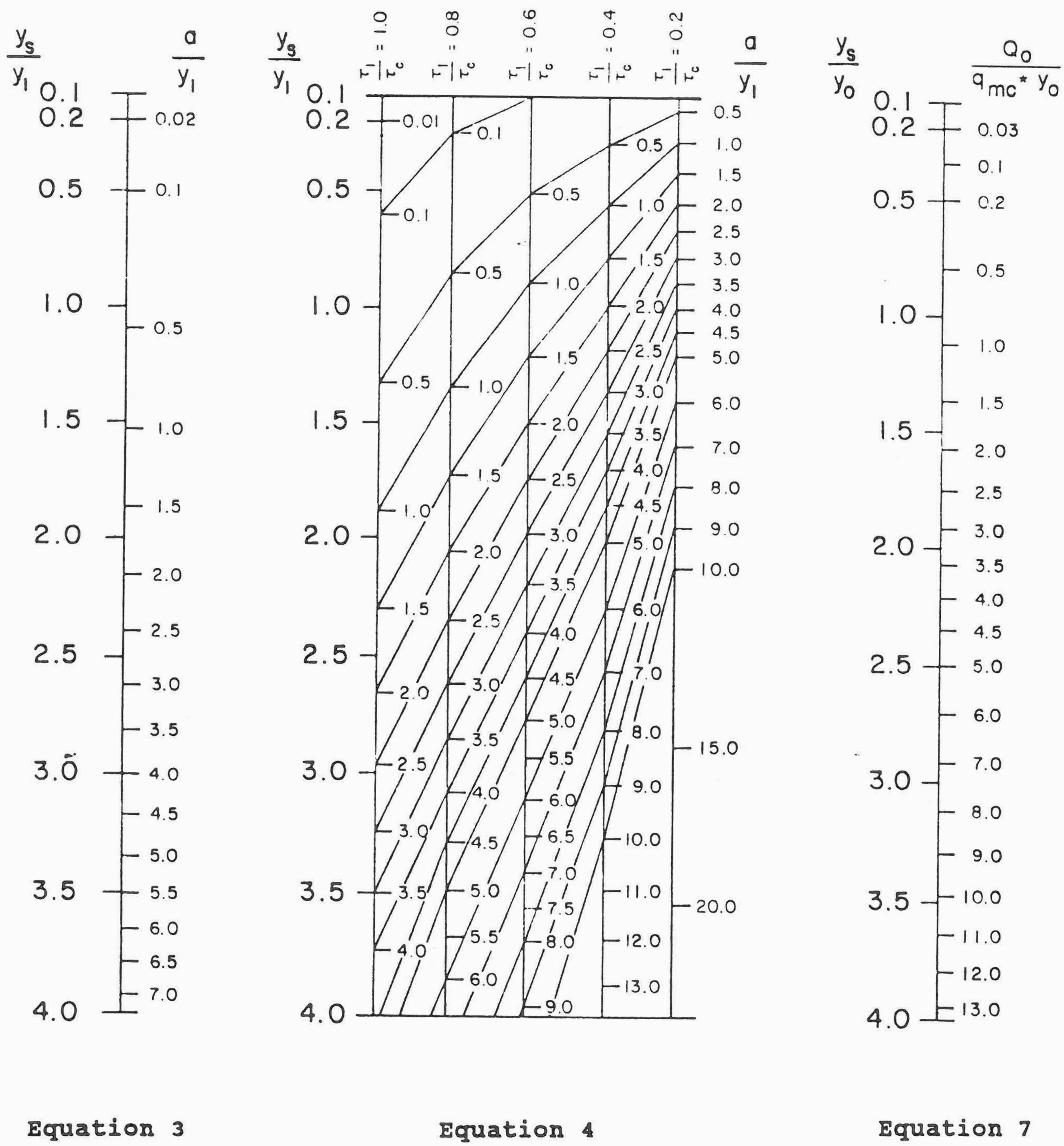


FIGURE B.6 NOMOGRAPHS FOR LAURSEN'S ABUTMENT SCOUR EQUATIONS

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.

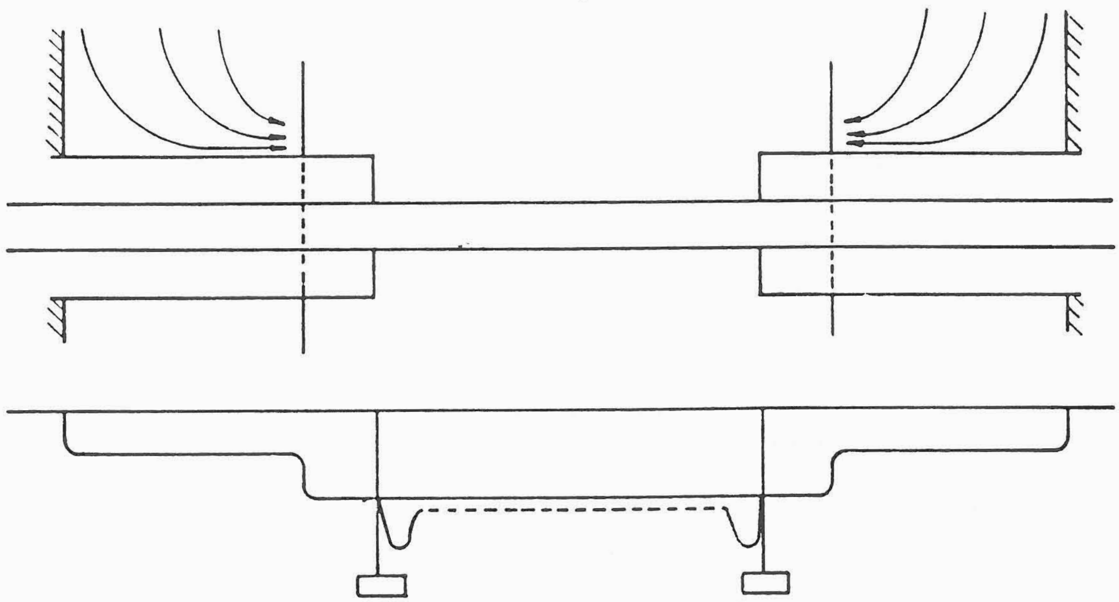


FIGURE B.7 BRIDGE ABUTMENT IN MAIN CHANNEL AND OVERBANK FLOW

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.

$$a = \frac{Q_o}{V_1 y_1} \qquad (6)$$

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.

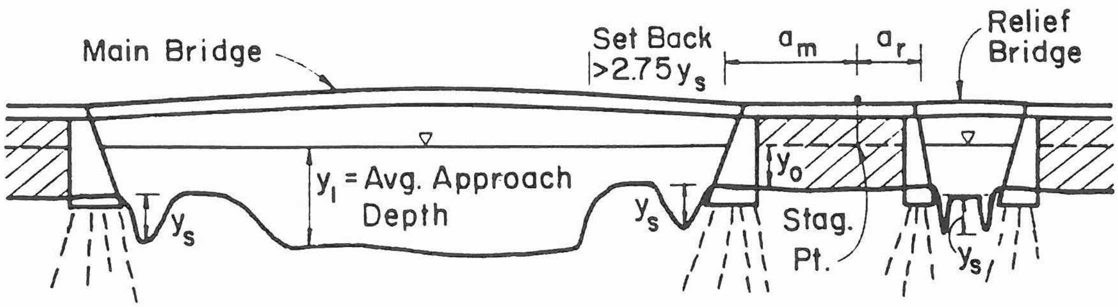


FIGURE B.8 BRIDGE ABUTMENT SET BACK FROM MAIN CHANNEL BANK AND RELIEF BRIDGE

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.

Notes.

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..

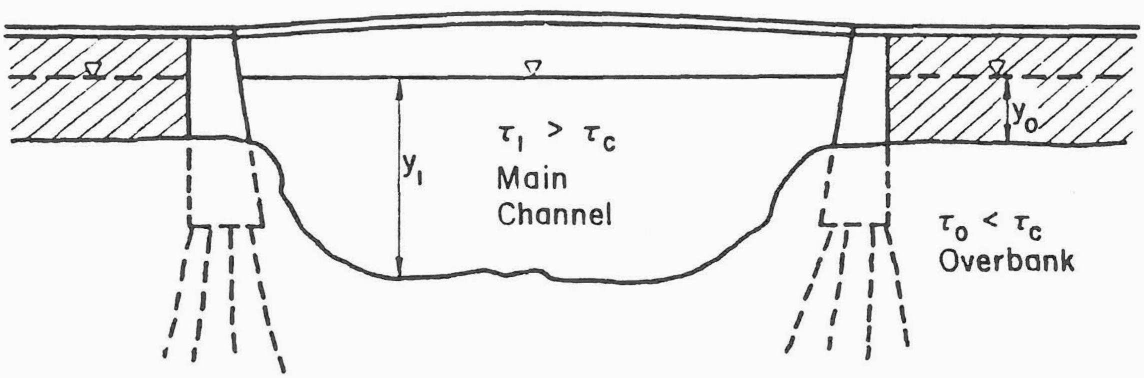


FIGURE B.9 ABUTMENT SET AT EDGE OF MAIN CHANNEL

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

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.

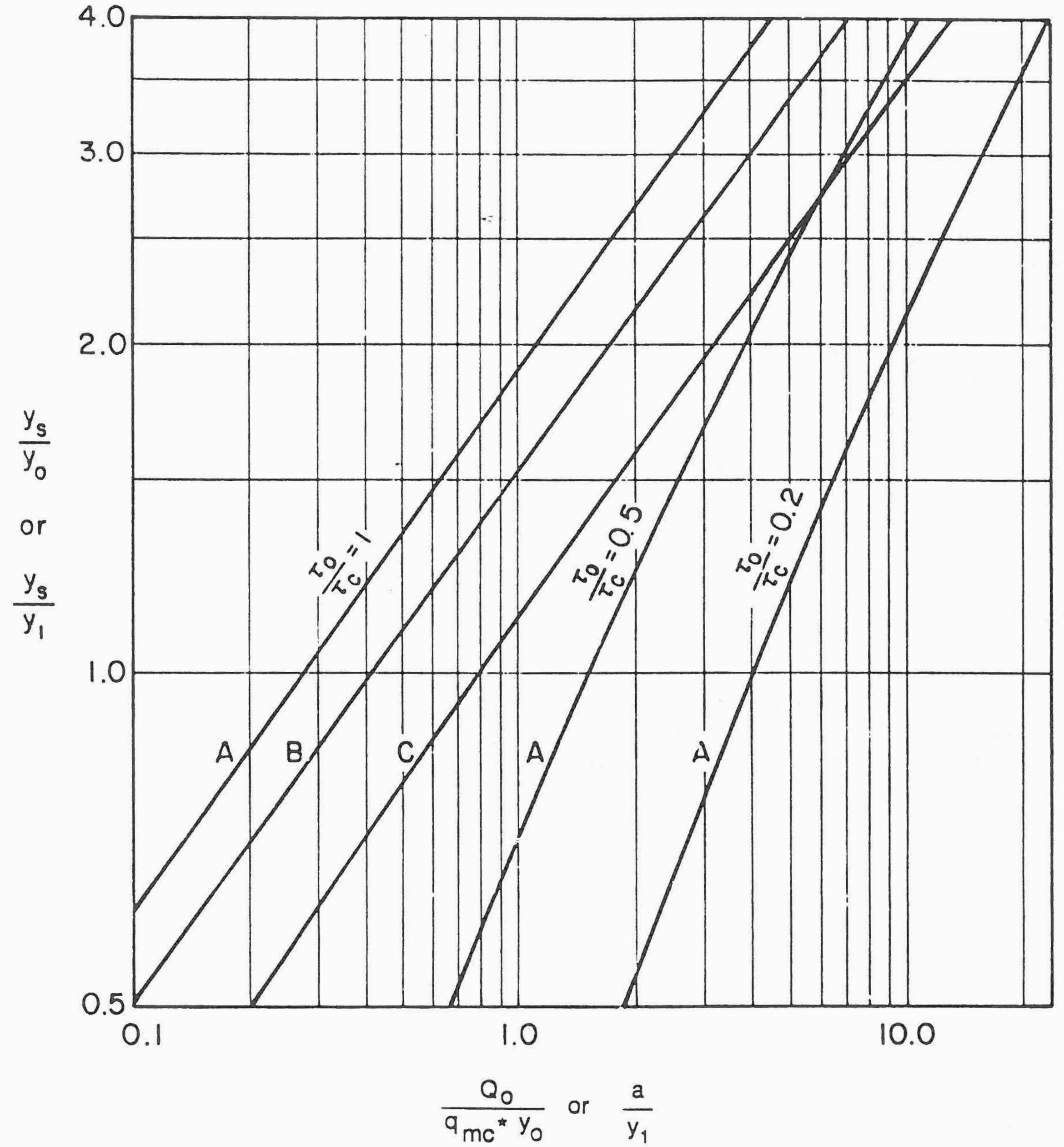


FIGURE B.10 VALUES OF CALCULATED SCOUR DEPTH FROM EQUATIONS 3, 4 and 7. (A is Eq. 4, B is Eq. 3 and C is Eq. 7)

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:

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

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).

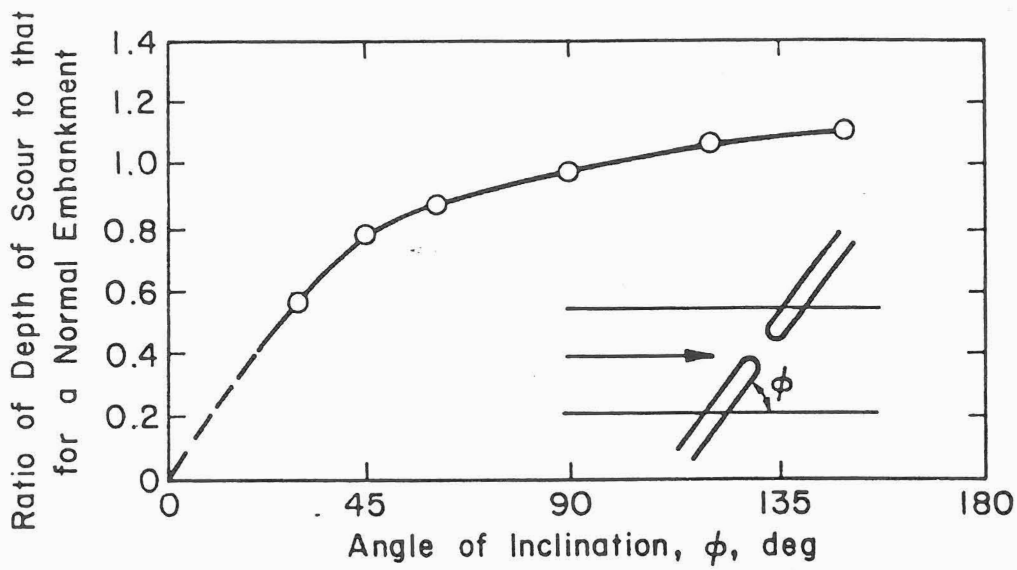


FIGURE B.11. SCOUR ESTIMATE ADJUSTMENT FOR SKEW.

LITERATURE CITED:

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2. Laursen, E. M., 1980, Predicting Scour at Bridge Piers and Abutments, Gen. Report No. 3, Eng. Exp. Sta.,College of Eng. Uni. of Arizona, Tucson, Az.

3. Froehlich, D. C., 1989, Abutment Scour Prediction, Paper presented at the 68 TRB Annual meeting, Washington D. C.

4. Gill, M. A., 1972, Erosion of Sand Beds Around Spur Dikes, Jour. Hyd. Div., ASCE, Vol. 98, No. Hy. 9, Sept., PP 1587-1602.

5. Chang, F. M., 1987, Personnel Communication.

6. E. V. Richardson, D. B. Simons, P. Julien, 1989, Highways in the River Environment," FHWA, U.S. Department of Transportation, (Revision of 1975 edition).

7. Ahmad, M., 1953, Experiments on Design and Behavior of Spur Dikes: Proc. IAHR, ASCE Joint Meeting, Univ of Minn., Aug.

APPENDIX C

WSPRO INPUT AND OUTPUT FOR EXAMPLE SCOUR PROBLEM

INPUT DATA FOR CHAPTER 4 EXAMPLE PROBLEM

```
T1          SCOUR EXAMPLE #2 - HYPOTHETICAL EXAMPLE
T2          CONTRACTION, PIER, AND ABUTMENT SCOUR CALCULATIONS
T3          HEC-18 - EVALUATING SCOUR AT BRIDGES
*
Q           30000
SK          0.002
*
XS    EXIT  750 * * * .002
GR          0,19   100,15   200,11   500,10.75   900,10   1100,9.0   1215,5.5
GR          1250,4.9   1300,3.05   1350,4.85   1385,5.1   1500,9.0   1700,10
GR          2100,10.75   2400,11   2500,15   2600,19
N              0.042     0.032     0.042
SA                 1100       1500
*
XS    FULLV 1400
*
BR    BRDG  1400
BL 1         650    1100    1500
BD           4  22
CD           3  50  2  22
AB           2
PW           5.65  30
N            0.042   0.032
SA              1100
*
AS    APPR  2100
*
HP 2 BRDG   13.82 * * 30000
*
HP 1 BRDG   13.54 1 13.54
*
HP 2 APPR   17.36 * * 30000
*
HP 1 APPR   17.36 1 17.36
*
EX
ER
```

OUTPUT

```
1
WSPRO          FEDERAL HIGHWAY ADMINISTRATION - U. S. GEOLOGICAL SURVEY
P060188             MODEL  FOR  WATER-SURFACE  PROFILE  COMPUTATIONS

           *** RUN DATE & TIME: 09-10-92  10:08

  T1         SCOUR EXAMPLE #2 - HYPOTHETICAL EXAMPLE
  T2         CONTRACTION, PIER, AND ABUTMENT SCOUR CALCULATIONS
  T3         HEC-18 - EVALUATING SCOUR AT BRIDGES
  *
  Q          30000
*** Q-DATA FOR SEC-ID, ISEQ =            1
  SK         0.002
  *
1
WSPRO          FEDERAL HIGHWAY ADMINISTRATION - U. S. GEOLOGICAL SURVEY
P060188             MODEL  FOR  WATER-SURFACE  PROFILE  COMPUTATIONS

        SCOUR EXAMPLE #2 - HYPOTHETICAL EXAMPLE
        CONTRACTION, PIER, AND ABUTMENT SCOUR CALCULATIONS
        HEC-18 - EVALUATING SCOUR AT BRIDGES
           *** RUN DATE & TIME: 09-10-92  10:08

*** START PROCESSING CROSS SECTION - "EXIT "
  XS   EXIT  750 * * * .002
  GR         0,19   100,15   200,11   500,10.75   900,10   1100,9.0    1215,5.5
  GR         1250,4.9   1300,3.05   1350,4.85   1385,5.1   1500,9.0    1700,10
  GR         2100,10.75   2400,11   2500,15   2600,19
  N             0.042    0.032    0.042
  SA                 1100       1500
  *

*** FINISH PROCESSING CROSS SECTION - "EXIT "
*** CROSS SECTION "EXIT " WRITTEN TO DISK, RECORD NO. =  1

--- DATA SUMMARY FOR SECID "EXIT " AT SRD =    750.  ERR-CODE =      0

      SKEW     IHFNO    VSLOPE        EK        CK
        .0        0.     .0020       .50       .00

X-Y COORDINATE PAIRS  (NGP =  17):
         X       Y          X       Y          X       Y          X       Y
        .0   19.00      100.0   15.00      200.0   11.00      500.0   10.75
     900.0   10.00     1100.0    9.00     1215.0    5.50     1250.0    4.90
    1300.0    3.05     1350.0    4.85     1385.0    5.10     1500.0    9.00
    1700.0   10.00     2100.0   10.75     2400.0   11.00     2500.0   15.00
    2600.0   19.00

 X-Y MAX-MIN POINTS:
      XMIN       Y          X    YMIN       XMAX       Y          X    YMAX
        .0   19.00     1300.0    3.05     2600.0   19.00         .0   19.00

SUBAREA BREAKPOINTS (NSA =  3):
        1100.   1500.

ROUGHNESS COEFFICIENTS (NSA =  3):
          .042    .032    .042
1
WSPRO          FEDERAL HIGHWAY ADMINISTRATION - U. S. GEOLOGICAL SURVEY
P060188             MODEL  FOR  WATER-SURFACE  PROFILE  COMPUTATIONS

        SCOUR EXAMPLE #2 - HYPOTHETICAL EXAMPLE
        CONTRACTION, PIER, AND ABUTMENT SCOUR CALCULATIONS
        HEC-18 - EVALUATING SCOUR AT BRIDGES
           *** RUN DATE & TIME: 09-10-92  10:08

*** START PROCESSING CROSS SECTION - "FULLV"
  XS   FULLV 1400
  *

*** FINISH PROCESSING CROSS SECTION - "FULLV"
```

```
*** NO ROUGHNESS DATA INPUT, WILL PROPAGATE FROM PREVIOUS CROSS SECTION.
*** CROSS SECTION "FULLV" WRITTEN TO DISK, RECORD NO. =  2

--- DATA SUMMARY FOR SECID "FULLV" AT SRD =    1400.  ERR-CODE =       0

     SKEW     IHFNO    VSLOPE        EK        CK
       .0        0.     .0020       .50       .00

X-Y COORDINATE PAIRS  (NGP =  17):
        X       Y           X       Y           X       Y           X       Y
       .0   20.30       100.0   16.30       200.0   12.30       500.0   12.05
    900.0   11.30      1100.0   10.30      1215.0    6.80      1250.0    6.20
   1300.0    4.35      1350.0    6.15      1385.0    6.40      1500.0   10.30
   1700.0   11.30      2100.0   12.05      2400.0   12.30      2500.0   16.30
   2600.0   20.30

 X-Y MAX-MIN POINTS:
     XMIN       Y           X    YMIN        XMAX       Y           X    YMAX
       .0   20.30      1300.0    4.35      2600.0   20.30          .0   20.30

SUBAREA BREAKPOINTS (NSA =  3):
        1100.   1500.

ROUGHNESS COEFFICIENTS (NSA =  3):
          .042    .032    .042
1
WSPRO          FEDERAL HIGHWAY ADMINISTRATION - U. S. GEOLOGICAL SURVEY
P060188              MODEL  FOR  WATER-SURFACE  PROFILE  COMPUTATIONS

         SCOUR EXAMPLE #2 - HYPOTHETICAL EXAMPLE
         CONTRACTION, PIER, AND ABUTMENT SCOUR CALCULATIONS
         HEC-18 - EVALUATING SCOUR AT BRIDGES
            *** RUN DATE & TIME: 09-10-92  10:08

*** START PROCESSING CROSS SECTION - "BRDG "
  BR   BRDG  1400
  BL 1        650    1100    1500
  BD          4  22
  CD          3  50  2  22
  AB          2
  PW          5.65  30
  N           0.042   0.032
  SA             1100
  *

*** FINISH PROCESSING CROSS SECTION - "BRDG "
*** CROSS SECTION "BRDG " WRITTEN TO DISK, RECORD NO. =  3

--- DATA SUMMARY FOR SECID "BRDG " AT SRD =    1400.  ERR-CODE =       0

     SKEW     IHFNO    VSLOPE        EK        CK
       .0        0.     .0020       .50       .00

X-Y COORDINATE PAIRS  (NGP =  13):
        X       Y           X       Y           X       Y           X       Y
    865.4   18.00       878.7   11.34       900.0   11.30      1100.0   10.30
   1215.0    6.80      1250.0    6.20      1300.0    4.35      1350.0    6.15
   1385.0    6.40      1500.0   10.30      1500.0   10.30      1515.4   18.00
    865.4   18.00

 X-Y MAX-MIN POINTS:
     XMIN       Y           X    YMIN        XMAX       Y           X    YMAX
    865.4   18.00      1300.0    4.35      1515.4   18.00       865.4   18.00

SUBAREA BREAKPOINTS (NSA =  2):
        1100.

ROUGHNESS COEFFICIENTS (NSA =  2):
          .042    .032

BRIDGE PARAMETERS:
 BRTYPE  BRWDTH     LSEL  USERCD  EMBSS   EMBELV  ABSLPL  ABSLPR
     3     50.0    18.00 *******   2.00    22.00    2.00 *******
```

```
DESIGN DATA:    BRLEN  LOCOPT  XCONLT  XCONRT
                650.0      1.   1100.   1500.

               GIRDEP  BDELEV   BDSLP   BDSTA
                 4.00   22.00 ******* *******

PIER DATA:  NPW =  1    PPCD = 0.
     PELV  PWDTH      PELV  PWDTH      PELV  PWDTH      PELV  PWDTH
      5.65  30.0
1
WSPRO         FEDERAL HIGHWAY ADMINISTRATION - U. S. GEOLOGICAL SURVEY
P060188             MODEL  FOR  WATER-SURFACE  PROFILE  COMPUTATIONS

        SCOUR EXAMPLE #2 - HYPOTHETICAL EXAMPLE
        CONTRACTION, PIER, AND ABUTMENT SCOUR CALCULATIONS
        HEC-18 - EVALUATING SCOUR AT BRIDGES
           *** RUN DATE & TIME: 09-10-92  10:08

*** START PROCESSING CROSS SECTION - "APPR "
  AS   APPR  2100
  *
  HP 2 BRDG  13.82 * * 30000

*** FINISH PROCESSING CROSS SECTION - "APPR "
*** NO ROUGHNESS DATA INPUT, WILL PROPAGATE FROM PREVIOUS CROSS SECTION.
*** CROSS SECTION "APPR " WRITTEN TO DISK, RECORD NO. =  4

--- DATA SUMMARY FOR SECID "APPR " AT SRD =   2100.  ERR-CODE =      0

     SKEW     IHFNO    VSLOPE        EK        CK
       .0        0.     .0020       .50       .00

X-Y COORDINATE PAIRS  (NGP =  17):
       X       Y          X       Y          X       Y          X       Y
      .0   21.70      100.0   17.70      200.0   13.70      500.0   13.45
   900.0   12.70     1100.0   11.70     1215.0    8.20     1250.0    7.60
  1300.0    5.75     1350.0    7.55     1385.0    7.80     1500.0   11.70
  1700.0   12.70     2100.0   13.45     2400.0   13.70     2500.0   17.70
  2600.0   21.70

 X-Y MAX-MIN POINTS:
     XMIN       Y          X    YMIN       XMAX       Y          X    YMAX
       .0   21.70     1300.0    5.75     2600.0   21.70         .0   21.70

SUBAREA BREAKPOINTS (NSA =  3):
       1100.   1500.

ROUGHNESS COEFFICIENTS (NSA =  3):
         .042     .032     .042

BRIDGE PROJECTION DATA:  XREFLT  XREFRT  FDSTLT  FDSTRT
                        ******* ******* ******* *******
1
WSPRO         FEDERAL HIGHWAY ADMINISTRATION - U. S. GEOLOGICAL SURVEY
P060188             MODEL  FOR  WATER-SURFACE  PROFILE  COMPUTATIONS

        SCOUR EXAMPLE #2 - HYPOTHETICAL EXAMPLE
        CONTRACTION, PIER, AND ABUTMENT SCOUR CALCULATIONS
        HEC-18 - EVALUATING SCOUR AT BRIDGES
           *** RUN DATE & TIME: 09-10-92  10:08



    VELOCITY DISTRIBUTION:  ISEQ =  3;  SECID = BRDG ;  SRD =    1400.

         WSEL      LEW     REW    AREA       K        Q     VEL
        13.82    873.8  1507.0  3286.9  470494.   30000.   9.13

X STA.       873.8      1003.3      1096.9      1150.0      1180.3      1203.9
  A(I)             346.5       305.9       225.0       166.6       149.6
  V(I)              4.33        4.90        6.67        9.00       10.03

```

```
X STA.        1203.9      1223.7      1241.9      1259.0      1274.4      1288.4
  A(I)              137.8       133.3       131.0       126.9       123.1
  V(I)              10.89       11.26       11.45       11.82       12.18

X STA.        1288.4      1301.6      1314.7      1329.0      1344.3      1361.3
  A(I)              122.0       120.7       123.8       124.5       131.2
  V(I)              12.29       12.43       12.11       12.05       11.43

X STA.        1361.3      1379.0      1397.3      1418.7      1447.3      1507.0
  A(I)              133.2       133.3       141.9       165.3       245.2
  V(I)              11.26       11.25       10.57        9.07        6.12
1
  *
  HP 1 BRDG  13.54 1 13.54
1
WSPRO          FEDERAL HIGHWAY ADMINISTRATION - U. S. GEOLOGICAL SURVEY
P060188              MODEL  FOR  WATER-SURFACE  PROFILE  COMPUTATIONS

        SCOUR EXAMPLE #2 - HYPOTHETICAL EXAMPLE
        CONTRACTION, PIER, AND ABUTMENT SCOUR CALCULATIONS
        HEC-18 - EVALUATING SCOUR AT BRIDGES
           *** RUN DATE & TIME: 09-10-92  10:08
    CROSS-SECTION PROPERTIES:  ISEQ =  3;  SECID = BRDG ;  SRD =     1400.

     WSEL  SA#    AREA        K    TOPW   WETP  ALPH    LEW    REW     QCR
             1    600.   40797.   226.   226.                          5553.
             2   2510.  392654.   406.   407.                         35385.
    13.54        3110.  433451.   632.   634.  1.16   874.  1506.  36279.
1
  *
  HP 2 APPR  17.36 * * 30000
1
WSPRO          FEDERAL HIGHWAY ADMINISTRATION - U. S. GEOLOGICAL SURVEY
P060188              MODEL  FOR  WATER-SURFACE  PROFILE  COMPUTATIONS

        SCOUR EXAMPLE #2 - HYPOTHETICAL EXAMPLE
        CONTRACTION, PIER, AND ABUTMENT SCOUR CALCULATIONS
        HEC-18 - EVALUATING SCOUR AT BRIDGES
           *** RUN DATE & TIME: 09-10-92  10:08

    VELOCITY DISTRIBUTION:  ISEQ =  4;  SECID = APPR ;  SRD =    2100.

         WSEL     LEW     REW    AREA        K        Q     VEL
        17.36   108.5  2491.5 11565.0 1414915.   30000.   2.59

X STA.         108.5       416.1       623.7       798.5       951.8      1077.6
  A(I)              978.0       823.0       752.7       711.6       658.1
  V(I)               1.53        1.82        1.99        2.11        2.28

X STA.        1077.6      1158.1      1204.1      1241.5      1274.0      1301.7
  A(I)              506.1       373.9       346.5       327.0       309.8
  V(I)               2.96        4.01        4.33        4.59        4.84

X STA.        1301.7      1330.6      1363.3      1399.1      1443.3      1522.7
  A(I)              318.4       327.1       340.0       368.6       502.7
  V(I)               4.71        4.59        4.41        4.07        2.98

X STA.        1522.7      1646.7      1803.5      1977.8      2184.8      2491.5
  A(I)              649.2       727.8       749.9       820.2       974.5
  V(I)               2.31        2.06        2.00        1.83        1.54
1
  *
  HP 1 APPR  17.36 1 17.36
1
WSPRO          FEDERAL HIGHWAY ADMINISTRATION - U. S. GEOLOGICAL SURVEY
P060188              MODEL  FOR  WATER-SURFACE  PROFILE  COMPUTATIONS

        SCOUR EXAMPLE #2 - HYPOTHETICAL EXAMPLE
        CONTRACTION, PIER, AND ABUTMENT SCOUR CALCULATIONS
        HEC-18 - EVALUATING SCOUR AT BRIDGES
           *** RUN DATE & TIME: 09-10-92  10:08
```

```
     CROSS-SECTION PROPERTIES:  ISEQ =  4;  SECID = APPR ;  SRD =    2100.

      WSEL  SA#     AREA        K   TOPW   WETP  ALPH     LEW     REW     QCR
              1     4049.  366963.   992.   992.                        46430.
              2     3467.  680989.   400.   400.                        57923.
              3     4049.  366963.   992.   992.                        46430.
     17.36         11565. 1414915.  2383.  2383.  1.53    108.  2492. 117067.
1
   *
   EX

+++ BEGINNING PROFILE CALCULATIONS --    1
1
WSPRO          FEDERAL HIGHWAY ADMINISTRATION - U. S. GEOLOGICAL SURVEY
P060188             MODEL  FOR  WATER-SURFACE  PROFILE  COMPUTATIONS

         SCOUR EXAMPLE #2 - HYPOTHETICAL EXAMPLE
         CONTRACTION, PIER, AND ABUTMENT SCOUR CALCULATIONS
         HEC-18 - EVALUATING SCOUR AT BRIDGES
            *** RUN DATE & TIME: 09-10-92  10:08

 XSID:CODE   SRDL    LEW     AREA    VHD    HF     EGL    CRWS      Q     WSEL
       SRD   FLEN    REW        K   ALPH    HO     ERR     FR#     VEL

EXIT :XS   ******   161.    6692.    .57 *****   13.14   11.86  30000.   12.57
      750. ******  2439.  670723.   1.83 ***** *******     .62    4.48

FULLV:FV     650.   161.    6706.    .57  1.30   14.44 *******  30000.   13.88
     1400.   650.  2439.  672489.   1.83   .00     .01     .62    4.47
          <<<<<THE ABOVE RESULTS REFLECT "NORMAL" (UNCONSTRICTED) FLOW>>>>>

APPR :AS     700.   161.    6700.    .57  1.39   15.84 *******  30000.   15.27
     2100.   700.  2439.  671817.   1.83   .00     .00     .62    4.48
          <<<<<THE ABOVE RESULTS REFLECT "NORMAL" (UNCONSTRICTED) FLOW>>>>>

             <<<<<RESULTS REFLECTING THE CONSTRICTED FLOW FOLLOW>>>>>

 XSID:CODE   SRDL    LEW     AREA    VHD    HF     EGL    CRWS      Q     WSEL
       SRD   FLEN    REW        K   ALPH    HO     ERR     FR#     VEL

BRDG :BR     650.   874.    3107.   2.69  2.01   16.23   13.27  30000.   13.54
     1400.   650.  1506.  432822.   1.86  1.07     .00    1.05    9.66

     TYPE PPCD FLOW      C    P/A    LSEL   BLEN   XLAB   XRAB
       3.   0.   1.   .734   .076   18.00   650.   879.  1500.

 XSID:CODE   SRDL    LEW     AREA    VHD    HF     EGL    CRWS      Q     WSEL
       SRD   FLEN    REW        K   ALPH    HO     ERR     FR#     VEL

APPR :AS     650.   108.   11574.    .16  1.02   17.52   14.56  30000.   17.36
     2100.   697.  2492. 1416461.   1.52   .28    -.02     .26    2.59

       M(G)   M(K)       KQ   XLKQ   XRKQ    OTEL
       .722   .430  811434.   891.  1521.   17.08

                      <<<<<END OF BRIDGE COMPUTATIONS>>>>>
  ER

1  NORMAL  END  OF  WSPRO  EXECUTION.
```

TABLE OF CONTENTS

APPENDIX D

NORTH CAROLINA

DEPARTMENT OF TRANSPORTATION

DIVISION OF HIGHWAYS

STRUCTURE SCOUR EVALUATION PLAN

FOR EXISTING STRUCTURES

JUNE 1990

INTERDISCIPLINARY SCOUR WORK GROUP

LIST OF TABLES

LIST OF FIGURES

LIST OF APPENDIXES

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.

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 Pla 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 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.

SCOUR EVALUATION PROCESS

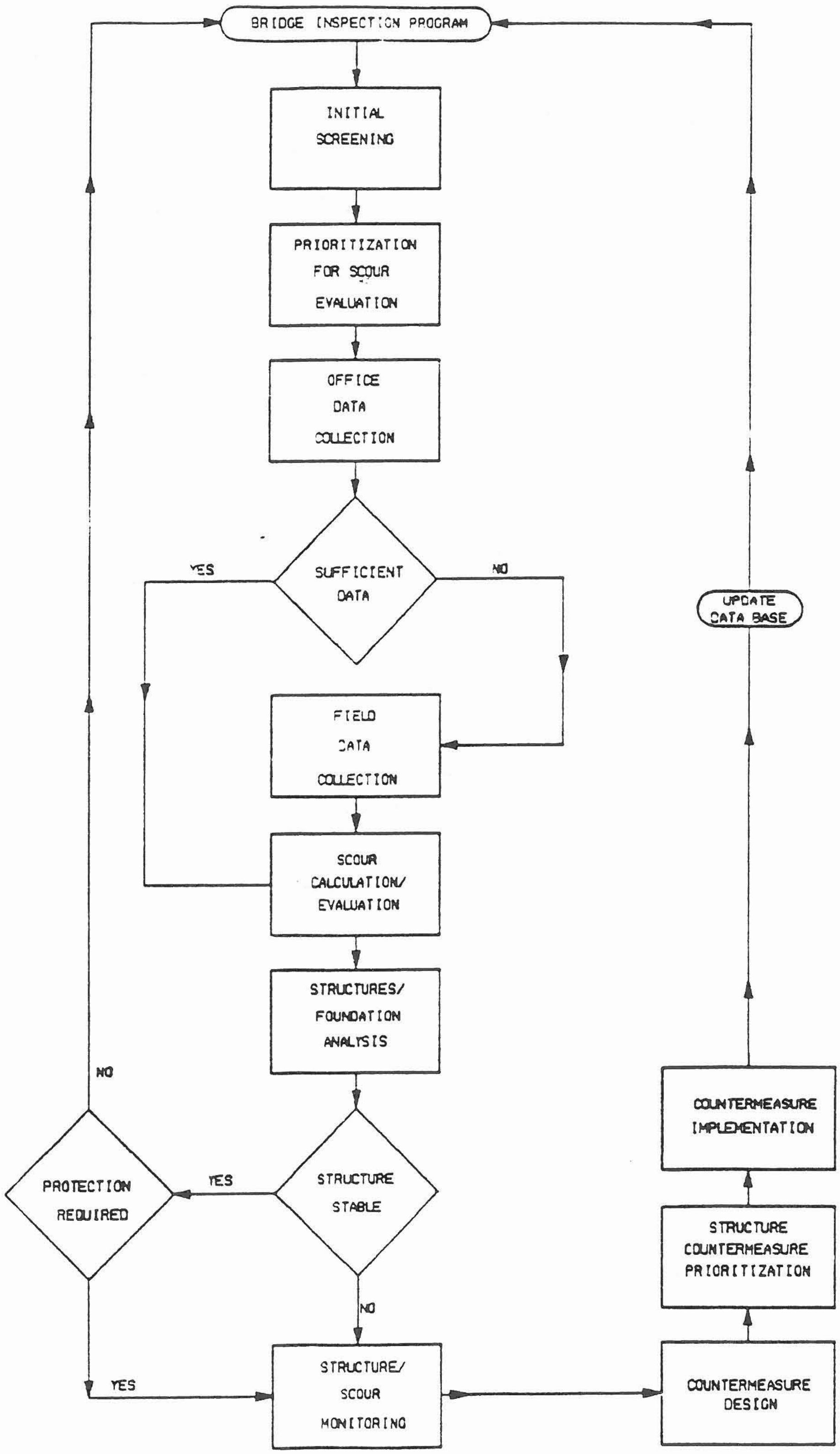


FIGURE 1

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	713
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

STATE_________

DATE_________

NUMBER OF BRIDGES

	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 FA	NFA	TOTAL	CLASSIFICATION	CATEGORY
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 FA	NFA	TOTAL	CLASSIFICATION	CATEGORY
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 correlated 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	3
	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	775

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.

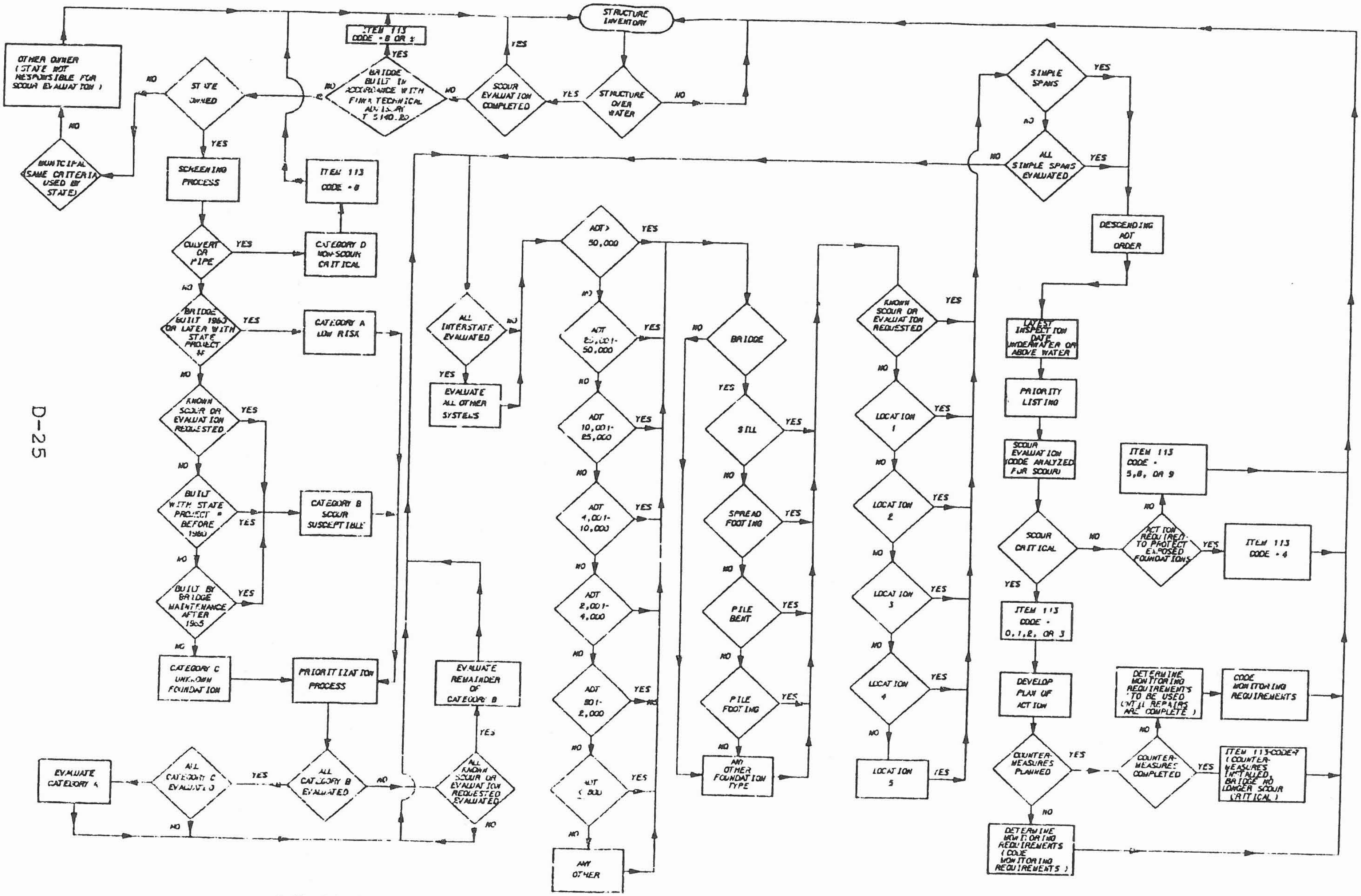
4.3 Prioritization For Scour Evaluation Data

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
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 31: 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	2,505	3,481	894	2,518	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	3,386	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

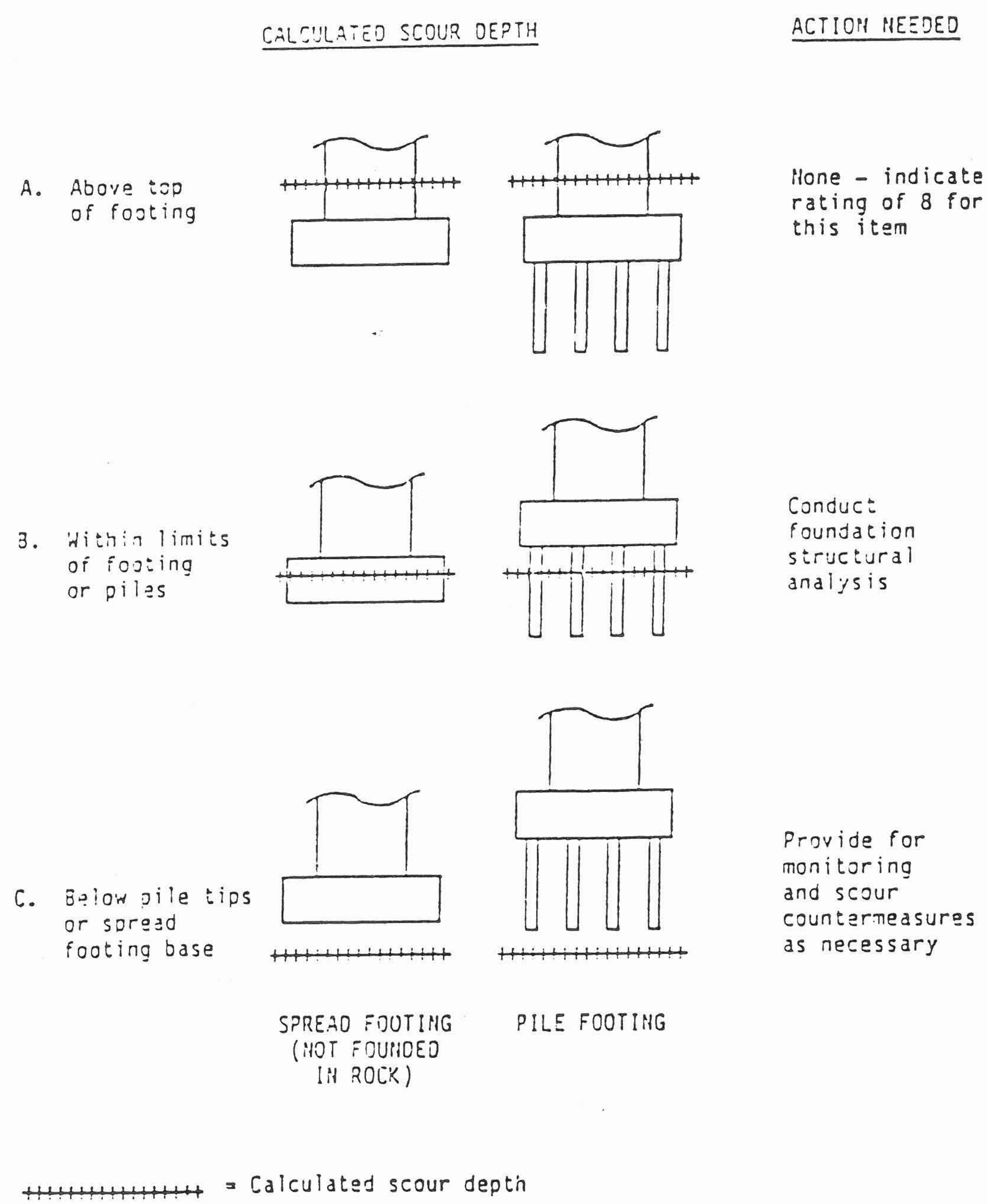
For each segment of this item, when applicable, code a 4-digit number to represent the month and year. The number of the month should be coded in the first 2 digits with leading zeros as required and the last 2 digits of the year coded as the third and fourth digits of the field. If the first digit of 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

LEVEL 1 AND LEVEL 2 STREAM STABILITY AND SCOUR ANALYSES

Replacement of Bridge on State Highway 144
South Platte River
Morgan County, Colorado

for

Colorado Department of Transportation

by

Resource Consultants & Engineers, Inc.
3665 John F. Kennedy Parkway
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APPENDIX F

CASE STUDY

Stream Stability and Scour Analysis for Colorado State Highway 144 Bridge on the South Platte River, Colorado

1. INTRODUCTION

This case study is based on a Preliminary Hydrology and Hydraulics Report for replacement of Colorado State Highway 144 bridge C-21-A over the South Platte River. This analysis was conducted by Resource Consultants & Engineers, Inc. (RCE) for BRW Engineering, Inc. in support of bridge design for District 4 of the Colorado Department of Transportation (CDOT). The project site is located in Morgan County southeast of the community of Weldona, Colorado.

The analysis is based on the procedures presented in HEC-20, "Stream Stability at Highway Structures" and HEC-18, "Scour at Highway Bridges." The Federal Highway Administration water surface profile computer model, WSPRO, was used to develop hydraulic variables necessary for the scour computations. The case study is intended to illustrate Level 1 and Level 2 analysis procedures and techniques available for hydraulic design of new or replacement bridges and scour vulnerability assessment at existing bridges.

2. PROJECT SITE DESCRIPTION AND LEVEL 1 ANALYSIS

2.1. General

The South Platte River originates in the Rocky Mountains of Colorado and is bounded to the west by the Continental Divide. Basin elevations along the western limits approach 14,200 feet. The river flows in a generally northerly direction through the Denver metropolitan area. After receiving flows from Bear Creek, Clear Creek and the St. Vrain, Poudre and Big Thompson Rivers, the South Platte turns eastward and crosses the eastern plains of Colorado to its confluence with the North Platte River near Ogallala, Nebraska. Elevations in the plains region vary from approximately 7,000 feet along the foothills to about 4,000 feet on the eastern plains.

Channel Morphology Changes

The South Platte Basin is located within the Southern Rocky Mountain Physiographic Province and the Colorado Piedmont Section of the Great Plain Physiographic Province. Total drainage area of the basin is 24,300 square miles and total river length equals 441 miles. River gradient exceeds 1,000 ft/mile near the headwaters, decreases to 21 ft/mile through the South Park Valley and then increases to about 69 ft/mile through the Lower South Platte Canyon. Downstream from Denver the slope decreases from approximately 16 ft/mile to about 5 ft/mile in the eastern plains of Colorado. Along the eastern plains in Morgan County, the South Platte River flows in a broad shallow valley ranging in width from as little as 1500 feet to about 21,000 feet.

Since the middle 1800s development of the South Platte basin has resulted in significant changes to the hydrologic characteristics of the basin and has led to significant morphologic changes of the river itself. Flows in the South Platte River are affected by transmountain diversions, dams which create on-stream reservoirs, diversion structures that route water to off-stream reservoirs and to irrigated croplands and power plants, evaporation from reservoirs, pumping of groundwater from alluvial aquifers, re-entry of flows from irrigation returns and hydropower releases and increased demands for urban areas as well as the water requirements of a much greater density of riparian vegetation.

The changed basin hydrology has resulted in a greatly changed river. Measurements of channel widths since about 1867 indicate that the width of the channel has reduced from about 1500-2000 feet in 1867 to about 100-300 feet in 1938 (Williams, 1978). The South Platte River experienced a very rapid reduction in width in comparison to the North Platte and Platte Rivers. By 1938 the present channel widths had been achieved on the South Platte, whereas channel narrowing continued into the 1960s on the other two rivers.

Figure 1 summarizes the geomorphic characteristics of the South Platte River at the Weldona site. Nadler (1978) reported that at the Weldona site the channel width reduced from about 1,500 feet in 1867 to about 180 feet in 1952; however, some channel widening occurred between 1952 and 1977, with a channel width of about 310 feet being reported in 1977. The recent (since 1952) changes in channel width can probably be attributed to the relatively high flows of the 1970s. Williams (1978) demonstrates that the 10-year averages of the mean annual flows at the Kersey gage since 1905 have been remarkably constant with the exception of the period from 1970 to 1977. The increased flows may well have led to increased channel widths.

At the Weldona site the planform characteristics of the river have also changed as a result of the changed basin hydrology. Sinuosity of the river has increased from about 1.02 in 1867 to 1.12 at the present. Concurrently, the slope of the river has been reduced from about 0.0015 to 0.0013.

Comparison of riparian vegetation descriptions from historical sources and old photographs indicates that in the mid and early 1800s there was little or no timber along the banks of the South Platte River and that the many islands were heavily vegetated with shrubs (willows and alders). In contrast the present day floodplain is heavily vegetated with cottonwoods that occupy the zone that used to be part of the channel in the 1800s. The increased density of the floodplain vegetation leads to heavier debris loading for the river and increases the potential for debris loading on bridges.

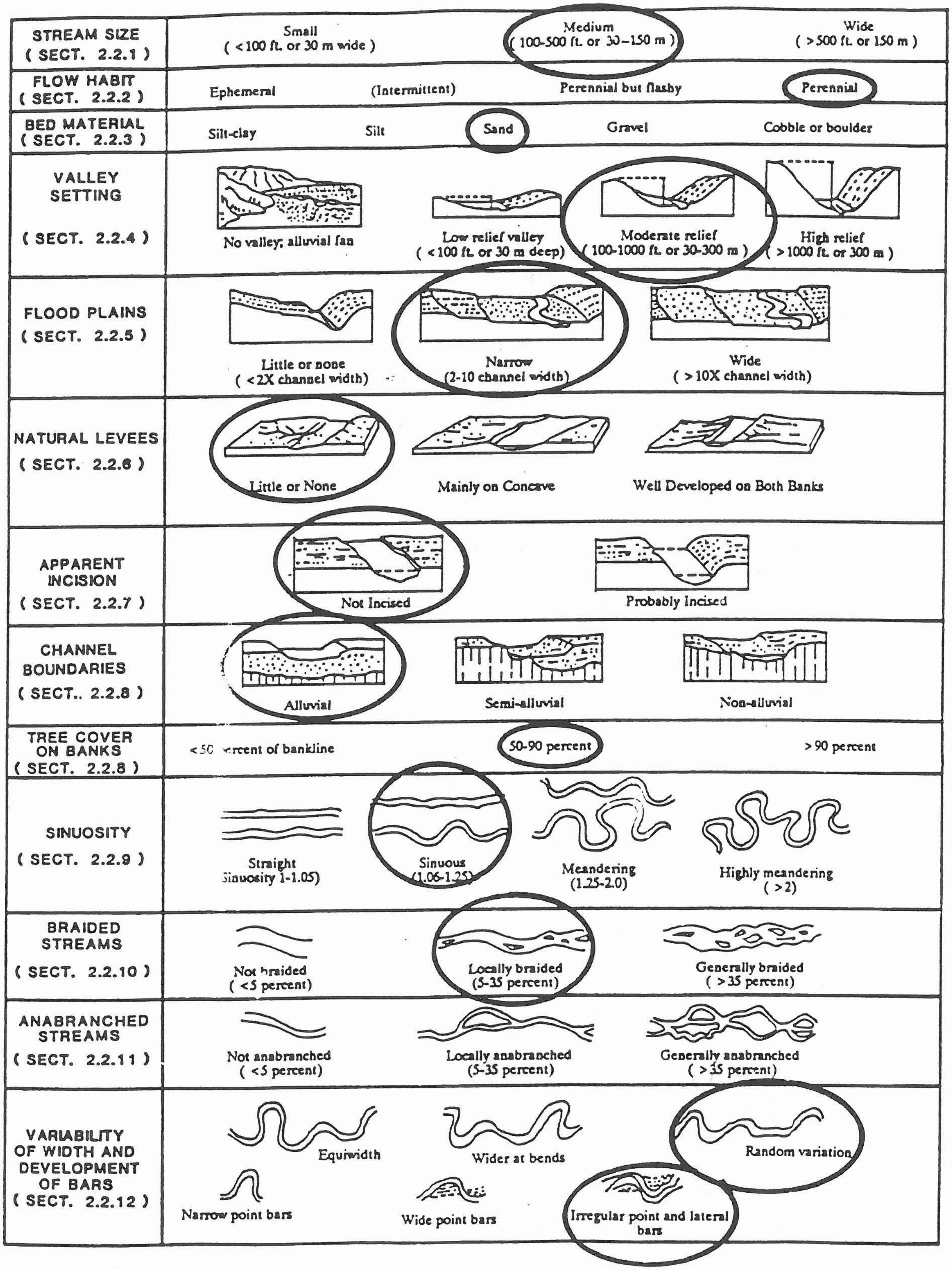

Figure F.1. Geomorphic Factors that affect stream stability, South Platte River at Weldona, Colorado.

2.2. Project Site

The project site is located in Morgan County near the town of Weldona, Colorado on a reach of the South Platte River called the Narrows. Within the project area, the South Platte River flows in a fairly well defined channel that exhibits a braided planform configuration. Numerous islands with various levels of vegetative establishment and stabilization are found within the main river channel. At the Narrows, in the vicinity of the project site, the river is generally constrained on the north and south by bluffs and the floodplain reduced from about 5,000 feet to about 2,000 feet. Materials found in the South Platte River valley consist primarily of alluvial sand, gravel and loam of Pleistocene and Holocene age. Adjacent plains are comprised principally of loess.

The current bridge crossing is located on the north side of the floodplain. Channel alignment in the reach centered on the existing bridge, is best described as a single high radius of curvature bend, and is controlled by the valley wall on the south bank at a distance of about 4,000 feet upstream of the bridge. Flows are deflected by the valley wall towards the north bank. Bank erosion on the north bank immediately downstream of the bridge is limited by riprap protection along the Union Pacific railroad whch parallels the South Platte River on the north. Because the Narrows reach is constrained by bedrock and historically has had limited sediment storage capacity as evidenced by the relatively narrow floodplain, it is unlikely that changes to the existing bridge configuration will have significant effects on channel stability. Therefore, the current channel alignment is likely to persist through the foreseeable future.

3. LEVEL 2 HYDROLOGIC ANALYSIS

3.1. Precipitation and Runoff

Mean annual rainfall in the South Platte River basin varies from west to east. Approximately 80 percent of the precipitation occurs between April and September. High-intensity, short-duration thunderstorms often occur during late spring and early summer. Historically, the river experienced large seasonal fluctuations in flowrate due to snowmelt runoff (Nadler & Schumm, 1981).

Runoff in the South Platte River is controlled by a number of factors including climatic, geologic and human activities. Human influences include water diversion and storage, irrigation and municipal water return flows, flood control and groundwater pumping from the South Platte alluvium. The net effect is that agricultural development and water development have caused major changes in the South Platte River system. Recharge to the river resulting from crop irrigation practices which began in 1885 altered the hydrologic character of the river. In general, the effect of irrigation was to raise the water tables above the river beds in late summer and to change stream flow from intermittent to perennial. Since the turn of the century, floodplain vegetation significantly increased along the South Platte River as a result of increased soil moisture. A change in the type of vegetation has also occurred with more woody vegetation now existing in the floodplain.

Proceeding east of the Weld-Morgan County line, most of the major tributaries drain plains areas to the south. These tributaries are intermittent and exhibit flashy response to thunderstorm events. Associated with the flashy response is the potential for these tributary streams to transport large quantities of sediment (USBR, 1950).

The nearest stream gage on the South Platte River is located downstream at the State Highway (SH) 144 crossing. This gage has been in operation since 1952. Stream gages are also located farther downstream near Fort Morgan and near Balzac, Colorado. **Table 1** shows the drainage areas and periods of record at these stream gaging stations. **Figure 2** shows the flow duration curve for the Weldona gage based on daily flow data for the period from 1953 through 1989. This figure indicates that a flow of 330 cfs is exceeded approximately 50 percent of the time at this location.

Table F.1. South Platte River Stream Gaging Stations in Morgan County, Colorado

Gage No.	Gage Location	Drainage Area (sq. mi.)	Period of Record
06758500	Weldona	13,245	October 1952 to present
06759500	Fort Morgan	14,810	1944 through 1958
06760000	Balzac	16,852	October 1916 to present

3.2. Flood History

The following flood accounts taken from the Corps of Engineers (1977), describe flood events that have occurred along the South Platte River in and adjacent to Morgan and Washington Counties. These descriptions are by no means a comprehensive history of flooding. They provide some insight regarding the source and magnitude of floods on the South Platte River in the general vicinity of the project site.

1921 - Heavy rainfall over much of the upper South Platte River basin caused flooding in Weld County in early June. Between Brighton and Orchard, the wooden bridges were impassable; two were destroyed and the approaches to the others were destroyed for a distance of several hundred yards. At Fort Morgan the water surface crest was comparable to that of the flood of 1894. The discharge of this flood at Kersey and at Balzac was the largest ever recorded at those locations except for the flood of May 1973 and the flood of June 1965, respectively.

1935 - Rains of cloudburst intensity over the basins of the plains tributaries to the South Platte River east of Denver occurred on 30-31 May following the wettest May in Colorado in 48 years. It was reported that a "veritable wall of water" appeared on Bijou Creek about 4 miles upstream from its confluence with the South Platte River. The flood crest reached Fort Morgan on 31 May and was reported to be 10 feet above flood stage having a discharge of 84,000 cubic feet per second. The flood crest attenuated rapidly as it flowed down the South Platte River from Fort Morgan.

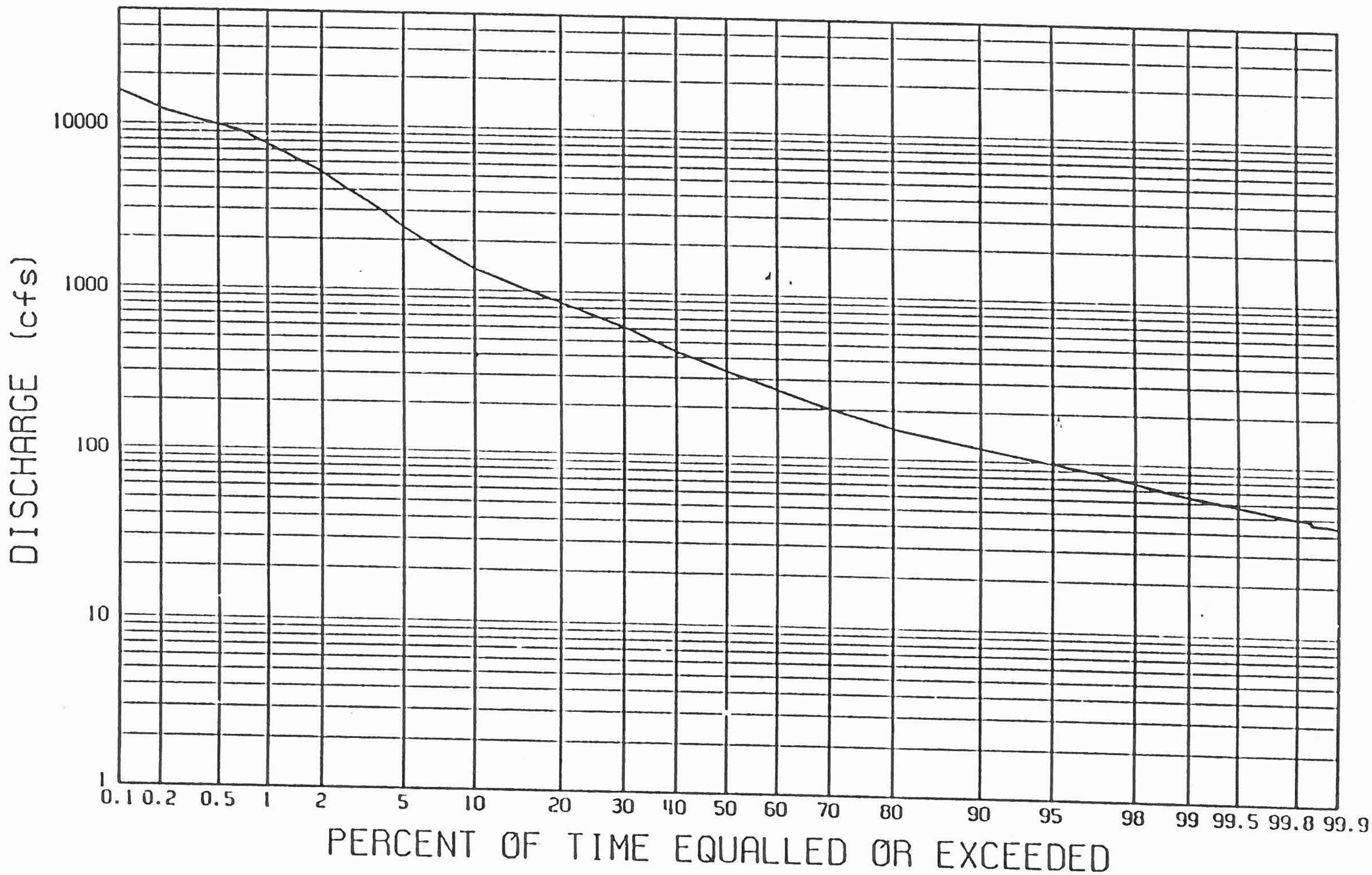

Figure F.2. Flow duration curve for Weldona gage - 1953 through 1989.

1938 - Heavy rainfall during the period from 30 August to 4 September over much of the upper South Platte River basin caused flooding on many of the mountain streams tributary to the South Platte River as well as the South Platte River itself. Relatively minor flooding was reported on the South Platte River.

1942 - The flood of April-May on the South Platte River was caused by excessive rainfall and snowmelt. The sustained high flows created considerable damage mainly due to erosion. Some levee failure occurred. Falling temperatures in late April turned the rainfall to snow as well as subsiding the melt of the existing snowpack thus preventing more serious flooding.

1949 - Heavy rainfall over a melting snowpack caused flooding on the South Platte River from mid-May to late June from Littleton, Colorado to North Platte, Nebraska. Considerable damage was incurred by homes, farm buildings, and crops along that reach of the river.

1965 - Heavy to torrential rainfall over large portions of the South Platte River basin created extensive flooding along the South Platte River. Heavy rainfall occurred over portions of the northern sections of the South Platte River basin on the 14th and 15th of June. As the storm system moved southward, torrential rainfall centered principally over the Plum Creek watershed on 16 June and on the Bijou Creek watershed on 17 June. Storm rainfall of the period extended over some 3,000 square miles of the South Platte River basin, including the Plum Creek, Cherry Creek, and Sand and Toll Gate Creek watersheds in the Denver region, and the Bijou Creek, Kiowa Creek, Comanche Creek, Badger Creek, and Beaver Creek watersheds to the east. Flooding occurred on the South Platte River from Plum Creek downstream to North Platte, Nebraska as a result of this rainfall.

1969 - Heavy rains during this period started on the afternoon of 4 May and continued with only intermittent breaks until 8 May. The storm covered an area along and near the eastern slopes of the mountains and extended into portions of the high plains. The heaviest amounts were centered 25 miles southwest of Denver and extended in a band along the foothills northward to near Estes Park. The weather station at Morrison reported a total storm rainfall of 11.27 inches and a maximum daily amount of 5.77 inches. General flooding resulted along the South Platte River.

1973 - Snowmelt runoff from the lower mountain area of the South Platte River basin began about the middle of April. Rainfall, amounting to as much as 6 inches, which was the major causative factor of the flooding in the South Platte River basin, began on 5 May. Sharp increases in flow as a result of the rainfall runoff were recorded at all gaging stations along the South Platte River from Littleton to the Colorado-Nebraska State line. The rainfall runoff was augmented by mountain snowmelt runoff which was also increasing during this period. The result was general flooding throughout the South Platte River basin; flooding was characterized by high, sharp hydrograph peaks from the rainfall runoff followed by a slow recession because of the continuing mountain snowmelt runoff. Bankfull discharges were experienced along portions of the main stem of the South Platte River for most of the month of May and on into June.

3.3. Previous Studies

The Omaha District of the U.S. Army Corps of Engineers (COE) conducted a "Flood Hazard Identification Study of the South Platte River in Weld, Morgan and Logan Counties in 1977. The reports for these three counties were published as Volumes 1, 2 and 3, respectively. At the project site, the peak discharges estimated by the COE for flood events having various recurrence intervals are as shown in **Table 2**. The COE report for Morgan County does not describe the methodology used to develop these peak discharges although it does reference the Weldona, Fort Morgan and Balzac stream gages on the South Platte River. The COE report also included computation of water surface elevations at various cross sections along the river. These cross sections were spaced large distances apart (as much as 10 miles) and bridge effects at S.H. 144 were not taken into account in any water surface profile analysis. Table 2 shows the water surface elevations computed at cross section number 35 as referenced in the COE study for various recurrence interval events. This cross section was located approximately 200 feet upstream of the SH 144 bridge.

In 1989, using the COE study as a basis, the Federal Emergency Management Agency (FEMA) developed floodplain maps for unincorporated areas of Morgan County along the South Platte River. These maps show floodplain limits, however no base flood elevations are provided.

3.4. Discharge-Frequency Analysis

An independent flood frequency analysis using annual peak flow records for the Weldona gage covering the period from 1953 through 1988 was conducted for this study. This evaluation was conducted using the U.S. Army Corps of Engineers program HECWRC (HEC, 1982). This program uses Water Resources Council (WRC) Method 17B to determine the discharge frequency relationship at this site. **Table 3** shows the results obtained from this procedure and compares them with the corresponding values developed by the COE (1977). Due to the rather minor differences between these computations and the fact that the current FEMA floodplain maps have been defined using the peak discharges from the COE study, the COE peak discharges were also adopted for this study.

Table F.2. Flood discharge frequency and corresponding water surface elevations near S.H. 144 (from COE, 1977).

Recurrence Interval (yrs)	Peak Discharge (cfs)	Water Surface Elevation[1] (ft)
10	13,500	4,316.2
25	21,500[2]	-------
50	30,500	4,318.6
100	42,500	4,319.8
500	82,500	4,323.0

1 Water surface elevations are for cross section No. 35 as defined in the COE (1977) study. This cross section was located approximately 200 feet upstream of S.H. 144.

2 Interpolated from log-probability plot.

Table F.3. Comparison of Flood Frequencies from WRC Method 17B and COE (1977).

Recurrence Interval (yrs)	Peak Discharge from WRC 17B (cfs)	Peak Discharge from COE (1977) (cfs)
10	15,200	13,500
50	34,000	30,500
100	45,000	42,500
500	79,800	82,500

3.5. Basis for Design Event

In the absence of economic risk analysis, CDOT criteria specify selection of the design event for a bridge crossing based on the magnitude of the 50-year peak discharge. At the Weldona site the 50-year flood event exceeds 4000 cfs; therefore, the 50-year flood becomes the basis for design (Colorado Division of Highways, 1984). The 50-, 100- and 500-year peak discharges used for design are shown in Table 3 (from COE).

CDOT criteria specify that the backwater effect created by a proposed bridge configuration be evaluated relative to an uncontracted or "natural" condition. As a guideline, an incremental increase in upstream water surface elevation of no more than 1 foot is referenced as desirable (Section 804.4 b.1). To represent this condition, all existing roadways and bridge effects were removed from the hydraulic model. This condition is hereafter referenced as the natural condition.

4. LEVEL 2 HYDRAULIC ANALYSES

4.1. Methodology

As mentioned in Section 3, floodplain maps have been developed for unincorporated areas of Morgan County along the South Platte River (FEMA, 1988). The FEMA floodplain report, however, does not provide any hydrologic or hydraulic information but rather references the COE (1977) report as the source of this information. Given the limited detail of the hydraulic analysis used to develop these maps and the fact that the S.H. 144 bridge crossing at this site was not considered in the COE analyses, existing hydraulic conditions as defined by detailed hydraulic analyses conducted for this project were used as a basis for comparison of the impacts of alternative bridge improvements.

Hydraulic analyses were conducted using the WSPRO bridge hydraulics model (FHWA, 1988). This model fully incorporates hydraulic procedures for analysis of bridge hydraulics and includes a design mode capability for evaluation of bridge alternatives.

Cross section data used in this study, were obtained from 1991 aerial photography and/or field surveys conducted during late 1991 and early 1992. Cross sectional data for the overbank areas and data describing the below-water portions of the cross sections were obtained by ground survey and coded into WSPRO formatted files.

The WSPRO model was developed using seven cross sections (including the bridge cross section) spaced at approximately 1000 foot intervals. Three cross sections were located upstream and downstream of the bridge cross section. Approach and exit cross sections then were located one bridge width upstream and downstream of the bridge. The locations of the approach and exit cross sections varied depending on the alternative under consideration.

Hydraulic roughness data were based on field observations of the sites conducted during late 1991 and early 1992. Due to the braided nature of the river and the existence of vegetated islands, a Manning n value of 0.04 was selected to represent the hydraulic roughness in the main channel. Overbank areas were modeled using an n value of 0.07. These n values reflect the presence of fairly dense vegetation and were judged to be appropriate for modeling flooding depths. To evaluate flow velocities through the bridges for use in evaluation of scour potential, the Manning n value for the main channel was reduced to 0.03. Default values for expansion and contraction losses (0.3 and 0.1) were used in the WSPRO model. The slope-area method with a slope of 0.0015 was used to define starting water surface conditions in the model.

4.2. No Bridge Condition

Hydraulic analysis of conditions without any bridges or approach roadways in place (unconstricted or natural flow conditions) results in the flood profiles shown in **Figure 3**. These profiles are used as a basis of comparison for all subsequent analyses.

4.3. Existing Conditions

The general configuration of the existing bridge crossing is shown in **Figure 4**. This figure shows an overall cross section depicting the location of the bridge, the existing topography and the existing road profile. The existing crossing is a 17 span timber stringer bridge founded on 12 inch diameter timber piling constructed in 1933. Bents are spaced approximately 20 to 26 feet on-center. Steel pipe piles have been added to replace or augment timber piles damaged by debris at various times and locations. The crossing is approximately 400 feet long and is located near the northernmost limits of the floodplain adjacent to the bluff line. The top of the bridge deck is approximately at elevation 4323 feet with the minimum low chord elevation equal to 4321 feet. The roadway profile will overtop at elevation 4321.0 feet.

Hydraulic analysis of existing conditions using the input data and hydraulic parameters described above produces the flow profiles shown in **Figure 5**. The existing bridge and embankment cause 1.5 feet of rise for a 50-year flood and 2.2 feet of rise for a 100-year flood, relative to natural (no bridge) conditions.

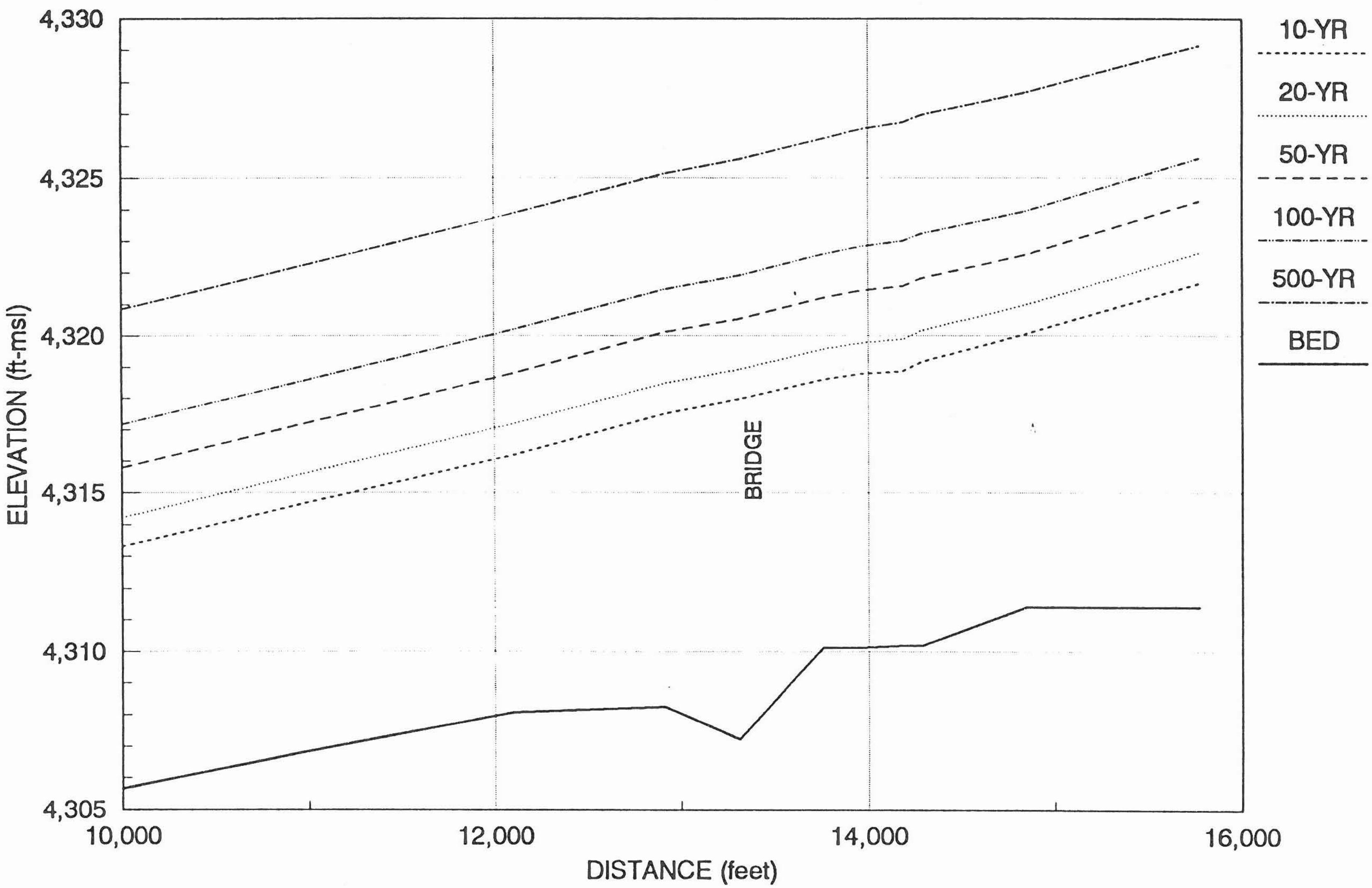

Figure F.3. Flood Profiles, no bridge condition.

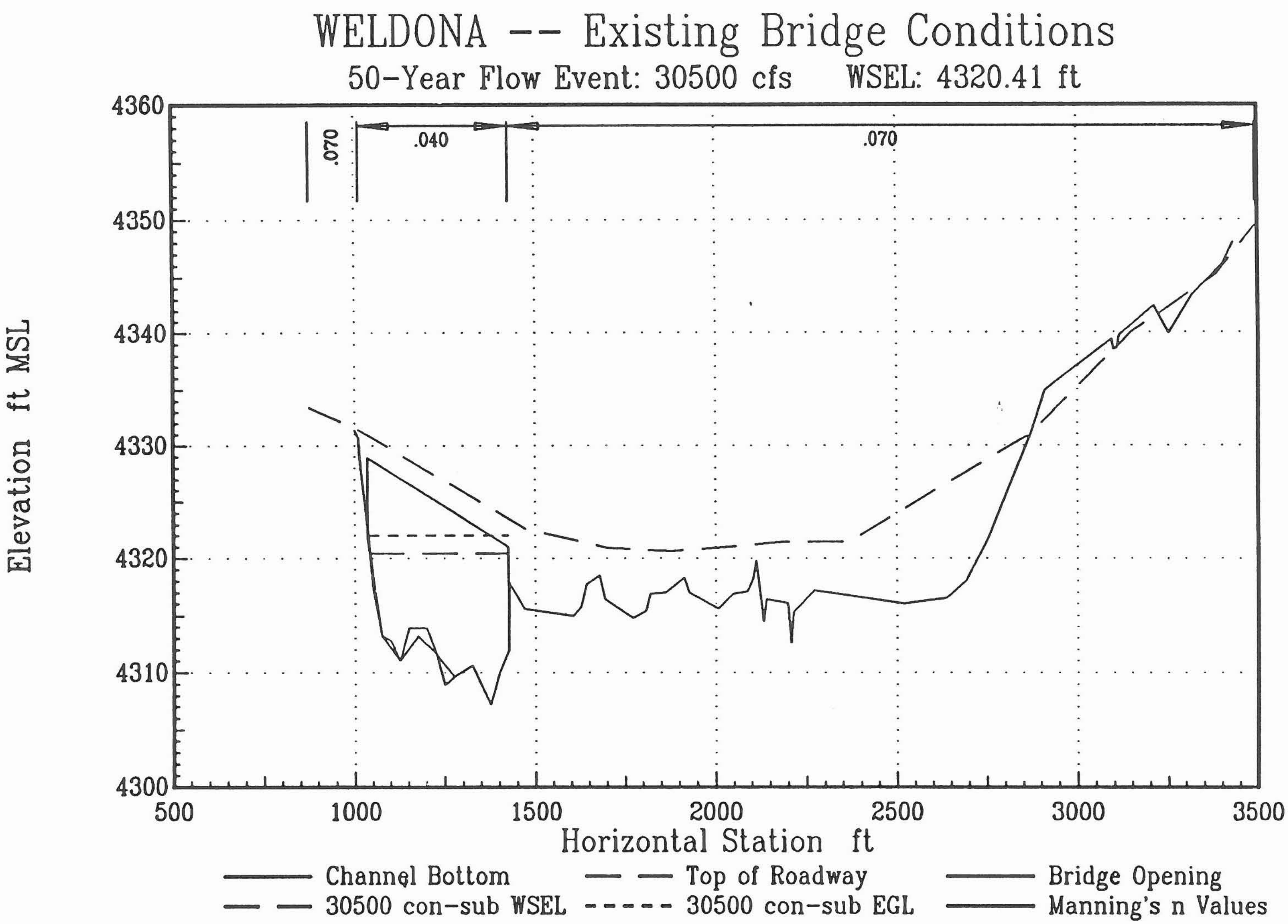

Figure F.4. Existing conditions bridge cross section.

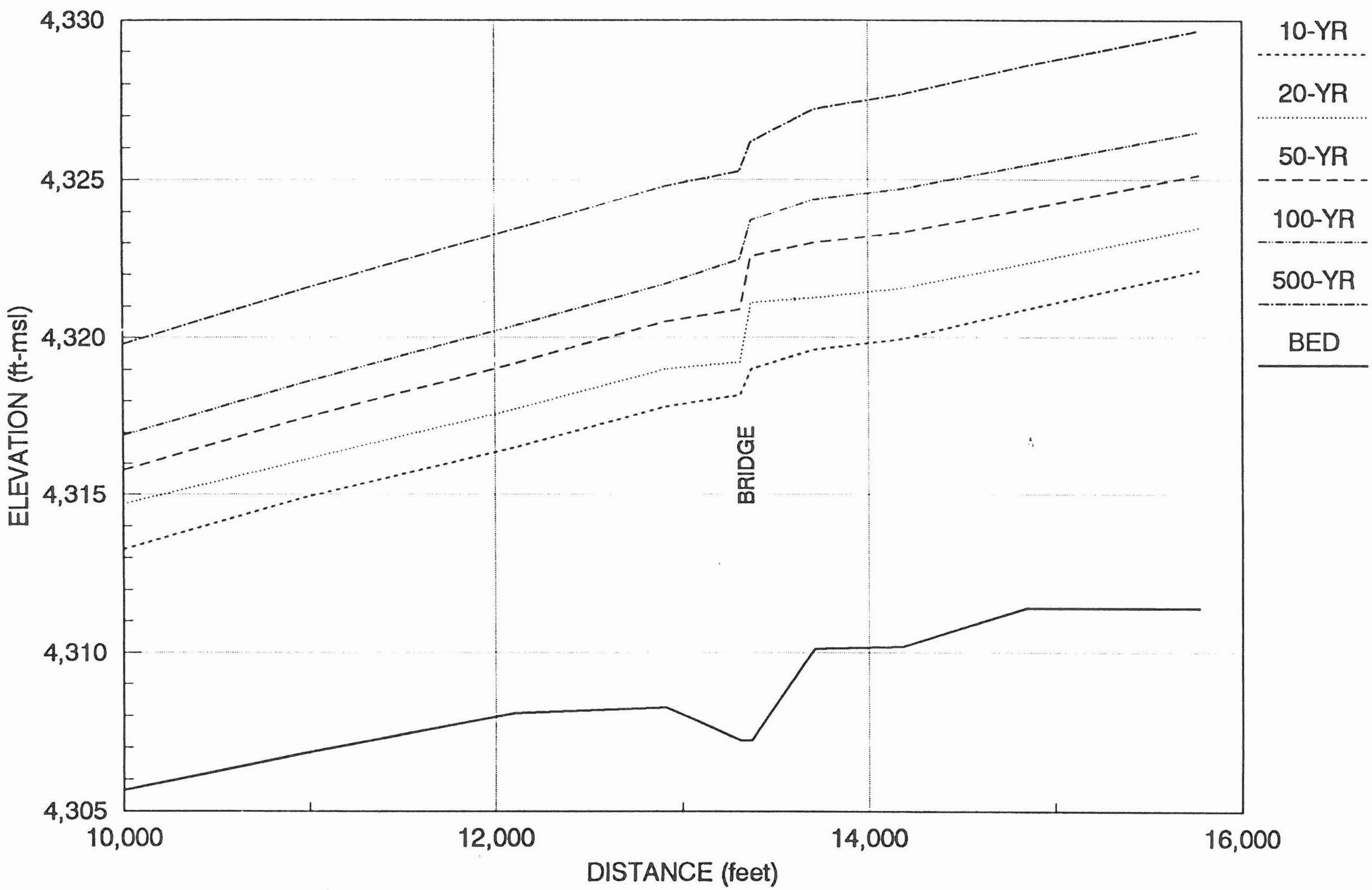

Figure F.5. Flood Profiles, existing conditions.

5. ALTERNATIVE ANALYSIS

5.1. General

Various alternative bridge configurations were evaluated at this site. For all of the alternatives piers were assumed to be spaced approximately 100 feet on-center, have a width of 2 feet and a pointed nose.

Initially, bridges lengths ranging from 600 feet to 1000 feet were analyzed. The maximum rise caused by a 800 foot bridge was slightly more than 1.0 foot. Very little reduction in water surface elevations occurred for 900 and 1,000 foot bridges. This is because the ground topography quickly approaches the road grade along the south approach where the channel widening would be located. Therefore, increasing the span length added little in bridge flow area. Inspection of the site topography and aerial photographs led to the conclusion that the area under the proposed bridges should be graded to enlarge the flow area. The low flow channel should remain unchanged and the remainder of the channel should be graded to match the upstream and downstream point bar surfaces.

5.2. Bridge Alternatives

Table 4 summarizes pertinent hydraulic results for existing conditions various alternatives. Each of the alternatives includes the channel grading described in the previous section. A 600 foot bridge causes 0.8 foot of rise for the design (50-year flood) event and 1.8 foot of rise for the 100-year event. This is the recommended alternative because the 500-foot bridge causes higher backwater for the 100-year event. All the options reduce backwater for the 50-year flood. Due to a higher road embankment, the 500-foot bridge increases backwater over existing conditions for a 100-year flood. **Figure 6** shows the proposed 600 foot bridge for the Weldona site. Also shown in this figure is the grading required to match upstream and downstream point bar surfaces.

Table F.4. Comparison of existing and various combinations of bridge sizes for a 50- and 100-year floods - Weldona.

Conditions[1]	WSEL Bridge (U/S face) (ft-msl)		Rise[2] (ft)		Bridge Velocity (fps)		Approach Velocity[3] (fps)	
	50-yr	100-yr	50-yr	100-yr	50-yr	100-yr	50-yr	100-yr
Existing	4322.2	4323.6	1.5	2.2	7.9	7.7	4.1	3.1
500'	4321.2	4324.0	1.3	2.5	7.5	8.9	2.7	3.0
600'	4321.0	4323.4	0.8	1.8	6.4	7.5	2.8	3.1
700'	4320.9	4322.9	0.5	1.4	5.3	6.2	2.9	3.3
800'	4320.9	4322.9	0.4	1.3	4.8	5.7	2.8	3.3

[1]Existing Condition is one 400' Bridge along the left bank.

[2]Maximum upstream rise over the unconstricted condition.

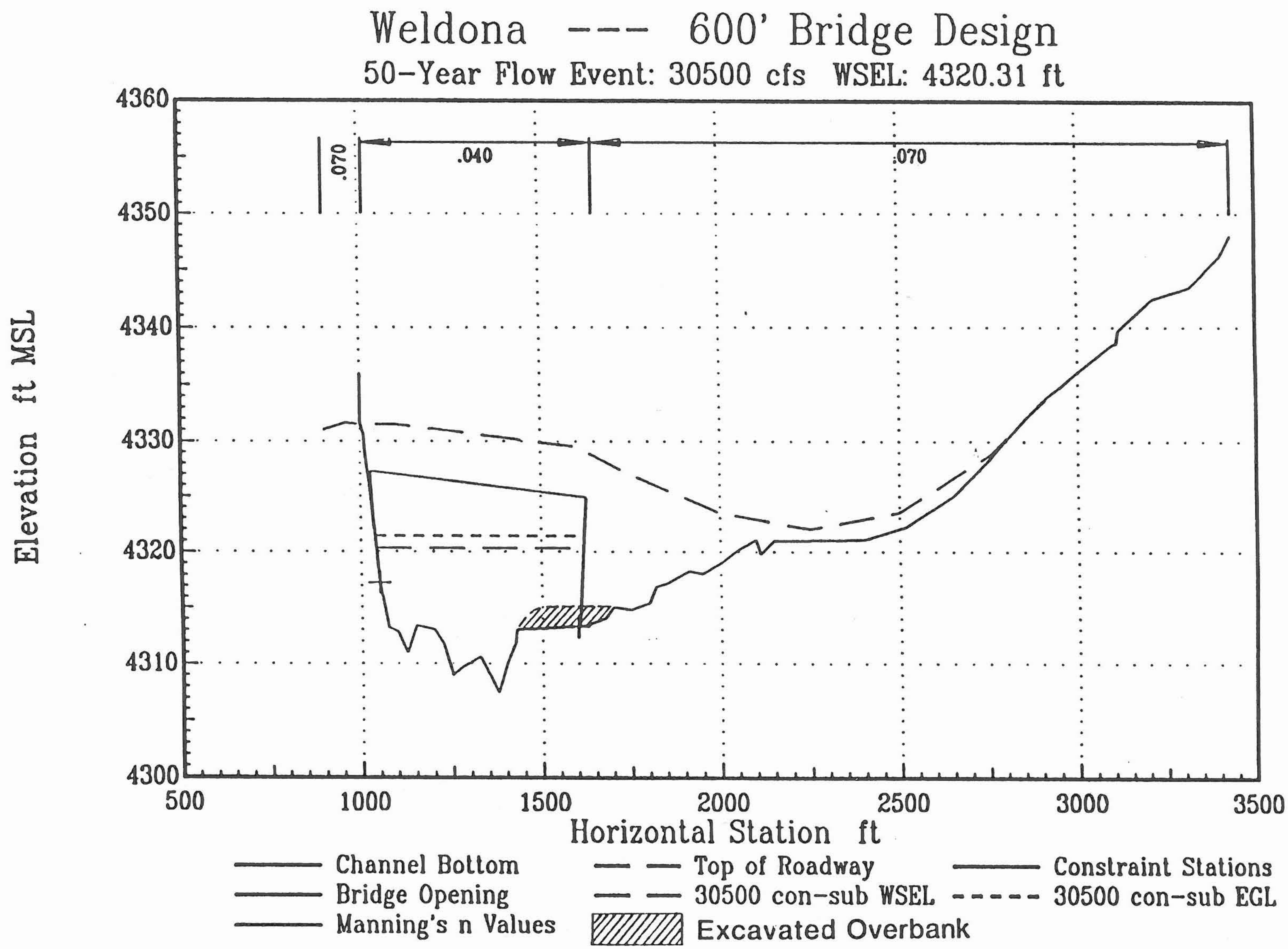

Figure F.6. Proposed bridge cross section.

6. LEVEL 2 SCOUR AND COUNTERMEASURE ANALYSIS

6.1. Introduction

The scour and countermeasures analyses were completed as outlined in FHWA, HEC-18, and HEC-20. A sumary of the results is presented in this section. Because of the significant potential for debris accumulation at bridges during flooding on this reach of the South Platte River (see historical summary Section 3.2) and resulting severe damages sustained by bridges on the South Platte River, the standard local scour computation procedures of HEC-18 were modified to consider debris.

6.2. Long Term Degradation

Long term aggradation/degradation trends for this reach of the South Platte River were evaluated by reviewing historical stage-discharge data at existing USGS gages. This evaluation indicated a condition of river bed stability with a possible slight tendency toward aggradation in some reaches. Therefore, a degradation component was not included in the total scour computation.

6.3. Scour

Scour computations were conducted using procedures outlined in FHWA Hydraulic Engineering Circular 18 (FHWA, 1991). Laursen's live bed scour equation was applied to evaluate contraction scour. Pier scour was determined for no debris, moderate debris (pier widths increased by 50 percent) and significant debris (pier widths doubled). Because of the history of debris problems at bridges in the South Platte, it was recommended that design be based on the potential for significant debris accumulation. In addition, upper regime bed forms are characteristic of the South Platte River during flood flows so local scour depths were increased by 10 percent. **Table 5** shows the results of the scour calculations. The bridge should be designed for 11 feet of pier scour during the 100-year flood and checked (for a factor of safety of 1) for 11.5 feet of pier scour during a 500-year flood.

Table F.5. Scour Analysis Results for the 600-foot Bridge Design - Weldona.

Recurrence Interval (yr)	Starting Bed El. (ft-msl)	Contraction Scour (ft)	Pier Scour (ft)			Total Scour (ft)	X1.1 Anti-dunes
			No Debris	Mod. Debris	Sis. Debris		
10	4307.5	1.8	2.8	3.6	4.4	6.6	4.8
20	4307.5	2.6	2.9	3.8	4.6	7.7	5.1
50	4307.5	4.0	3.1	4.0	4.8	9.3	5.3
100	4307.5	5.3	3.2	4.2	5.1	10.9	5.6
500	4307.5	4.7	3.9	5.1	6.2	11.5	6.8

6.4. Freeboard

Freeboard was computed for each bridge crossing as recommended in the CDOT "Roadway Design Manual." The 50-year flood discharge of 30,500 cfs and bridge flow velocity of 6.4 ft/sec results in a required freeboard of 2.6 feet. To provide some allowance for passage of debris, an additional two feet of clearance was added to the freeboard computed using the freeboard equation. A further check was also made to see that some freeboard was available during a 100-year event with the goal being to provide about two feet of clearance during this event.

6.5. Riprap Protection

A preliminary estimate of the required riprap protection was determined using procedures in the CDOT of the "Roadway Design Manual" (1984). Based on these criteria, the riprap protection would consist of a layer of stone 3.0 feet thick with a median diameter D_{50} of 18 inches. Riprap protection should be provided on the bridge abutments and extend upstream along the face of any guide banks (spur dikes) required.

6.6. Guide Banks

Due to the high degree of contraction and large volume of flow on the right overbank which would flow along the road embankment, a guide bank is recommended at the right bridge abutment to minimize abutment scour potential, improve conveyance in the bridge section, and help orient the flow perpendicular to the bridge crossing. Because of the low volume of flow on the left overbank, a guide bank will not be required at the left abutment.

7. REFERENCES

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APPENDIX G

SCOUR DETECTION EQUIPMENT

1. INTRODUCTION

As discussed in Chapter 7, scour monitoring is considered to be a suitable countermeasure for scour. Scour monitoring is differentiated from inspection in that monitoring implies the determination of the bed elevation at the time that scour is occurring. Although simple in concept, the ability to monitor scour during floods is inhibited by high flow depths and velocities, turbidity, floating debris, turbulence and ice. It is because of the adverse environment which exists in and around bridge piers and abutments during high flows (when scour occurs), that there are few instruments and techniques available to measure scour.

Past techniques to measure scour have focused on manual mechanical methods such as using a graduated rod to probe the scour hole, using a cable and lead weight, or similar techniques. Sonic fathometers have also been used with varying degrees of success. In a few notable cases divers have attempted to probe the scour holes around bridge piers at high water, but these few attempts have proven to be extremely dangerous given the nature of the turbulence around a bridge foundation.

More recently, newer techniques and adaptations of these past techniques have been, or are being developed to measure and monitor scour at bridge piers. These new techniques and instrumentation are the result of intensive research efforts which have been funded by the highway community. Some of these techniques can be also employed as post-flood inspection methods to determine maximum scour depths after floods.

The following text discusses some of the most promising techniques and instruments which are, or may be available in the future to monitor and measure scour at bridge piers and abutments. To begin this discussion, various geophysical tools which have been, or could be utilized for scour monitoring or post flood inspection, are described. Following this discussion deployment options of these and other techniques for either mobil or fixed installation scour monitoring devices are discussed.

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, the true extent of scour can be measured 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 its 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 in 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 transmits short, 80 to 800 MHz electromagnetic pulses into the subsurface and measures the two way travel time for the signal to return to 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, while some 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, whereas higher frequency 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 G-1 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 (20 KHz) which yield better penetration at the expense of

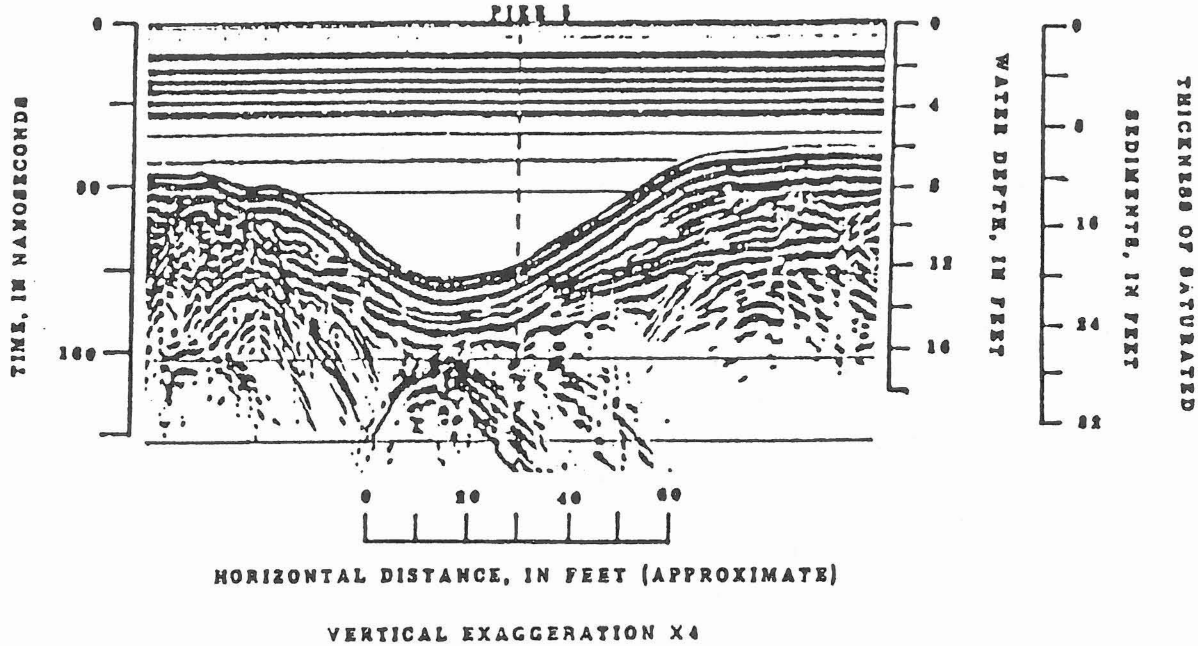


Figure G-1. Example of Ground Penetrating Radon.

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-G 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.

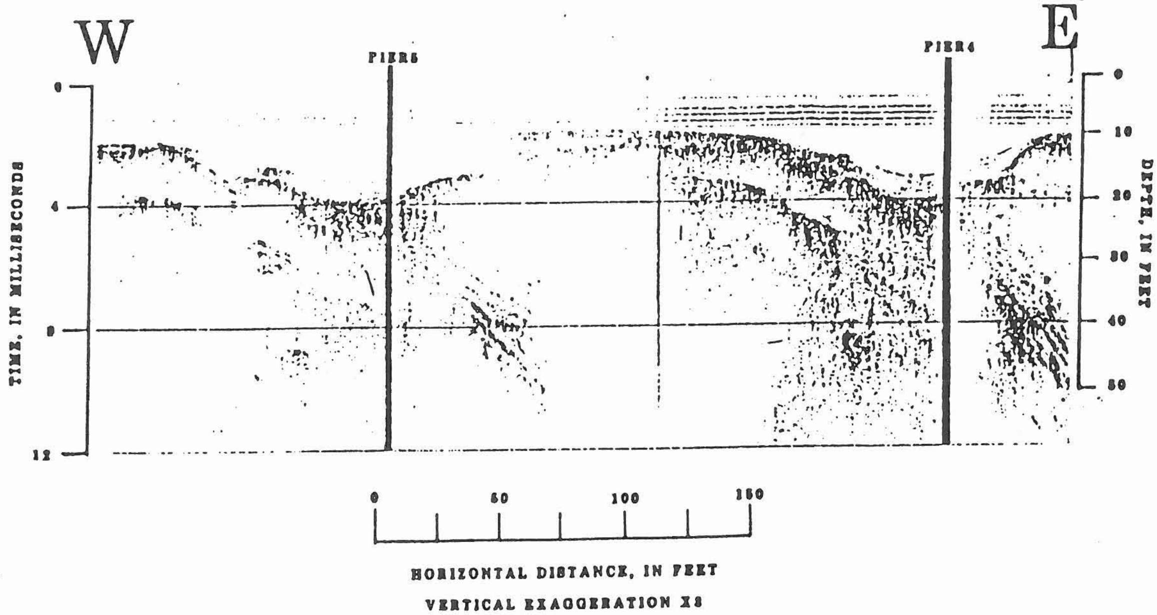


Figure G-2. Example of 14KHZ Tuned Transducer.

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 is easily 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 of 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 $1,000.

Figure G-3 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.

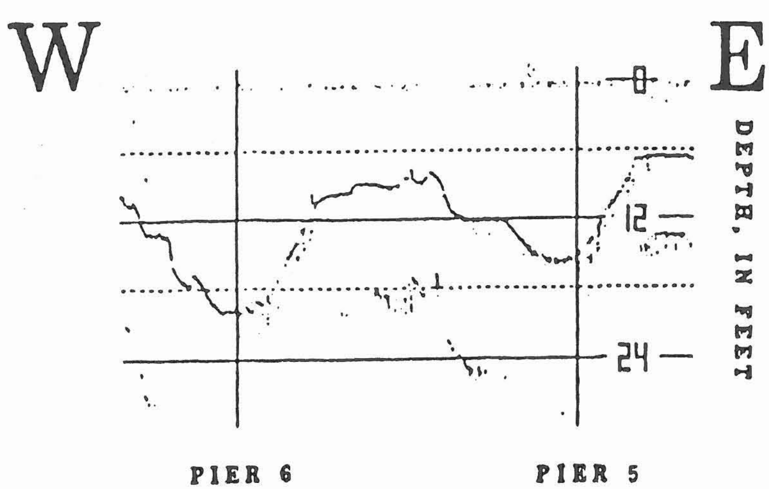


Figure G-3. Example of 200 KH_Z Fathometer.

Mobile Instrumentation

Mobile instrumentation comprises all instrumentation which can be brought to a bridge site to measure scour at flood flow conditions. Typically, these instruments are deployed on a boat, unmanned floating equipment platforms, from the bridge, or other means to sense the bed along and around the bridge piers and abutments. In some cases sonic transducers have been attached to sounding weights and suspended over the bridge rail using a portable crane and winch arrangement.

Mobile instrumentation can range from a simple black and white fathometer (typically used by sport fishermen) to ground penetrating radar, tuned transducers, color fathometers, or other geophysical techniques. Cable and a lead weight similar to that used for stream gaging are also used for scour measurement. More recently, two and three-dimensional sonic fathometers which can produce three dimensional images have become available for use by sport fishermen, however their use for monitoring scour has not been demonstrated.

An advantage of these techniques is that since the instrumentation is mobile, the equipment can be used to service several bridges within a highway department's region. Many state DOT's have been using black and white fathometers for developing cross sectional surveys of the bridge waterway area as well as for scour monitoring.

Disadvantages to mobile instrumentation relate to the inherent dangers and difficulties involved in collecting data during flood flows. In addition, some of the instrumentation requires technically qualified personnel to operate and maintain the device and interpret data.

Fixed Instrumentation

Scour monitoring equipment can be deployed in a fixed installation mode to provide a scour monitoring capability. In a typical installation an instrument, combined with a method to either manually or digitally record scour data, can be installed on or near a bridge pier or abutment to provide scour monitoring or measuring. These instruments include low-cost or more sophisticated sonic fathometers, sounding rods, buried rods, or other buried devices. Each of these classes of instrumentation is discussed separately.

Due to the wide variety of pier and abutment geometries, and because of the variability in river geometry, flow conditions, bed material and other characteristics of highway crossings, no single fixed instrumentation type will be applicable to meet the needs of all cases. Rather, there is a need to have a variety of fixed instrumentation to meet the needs for the many permutations of conditions found at bridges.

Sonic fathometers

Sonic fathometers can be attached to the bridge pier or abutment to monitor scour. Currently there are several research organizations which are experimenting and field testing these types of instrumentation. For example the USGS in Albany, New York has instrumented several bridges using both a simple "fish finder" and more sophisticated commercial sonic fathometers. The Virginia Transportation Research Council in Charlottesville, Virginia has installed multiple transducers on a bridge south of Richmond, VA. This installation is equipped with data logging and telemetering capability. Finally, Resource Consultants and Engineers (RCE, Fort Collins Colorado) have reported successful operation of a "fish finder" type sonic fathometer, linked to a data logger at a bridge over the Platte river near Orchard, Colorado, under an NCHRP project (Project 21-3) to develop scour instrumentation.

Although these research efforts have, for the most part, successfully demonstrated the applicability of these techniques to measure and monitor scour, the use of these instruments can be limited by factors such as ice, debris, or flows which have high concentrations of entrained air. In spite of these general limitations, the use of sonic fathometers to monitor and measure scour at bridge piers is considered to be both technically feasible and applicable to a wide range of bridges.

Sounding rods

In the context of fixed scour monitoring equipment, the use of sounding rods encompasses methods whereby a rod resting on the bed is allowed to slide vertically as scour develops. The rod is constrained to essentially vertical movement as scour develops by means of a sleeve or other method which will orient the sounding rod directly above the scour hole but will allow the rod to move vertically. Scour depths can be either determined manually or by using data logging techniques. One such instrument, known as the Brisco Monitor (use of trade names is for identification purposes only), is currently commercially available. This instrument measures scour by measuring the length of cable, which is attached to the top of the sounding rod, unwound from a spool in the data recording enclosure.

Sounding rods, such as the one described above, can be used as scour monitoring devices, however these instruments are limited by the expected ultimate depth of scour, and subsequently, the length of rod required to accurately track the development of scour. As the rod length increases, the weight of the rod bearing on the bed material also increases. The entire weight of the rod must be supported by the bed material of the scour hole. A footplate attached to the end of the sounding rod must be of sufficient size to prevent the rod from burying into the bed. In laboratory tests conducted at Colorado State University, and in field trials at an installation near Orchard, Colorado, it was found that for sand bed channels, the bearing stress of the rod and footplate needs to be below 400 to 600 psf to prevent the rod from burying.

Buried or Driven Rod Instrumentation

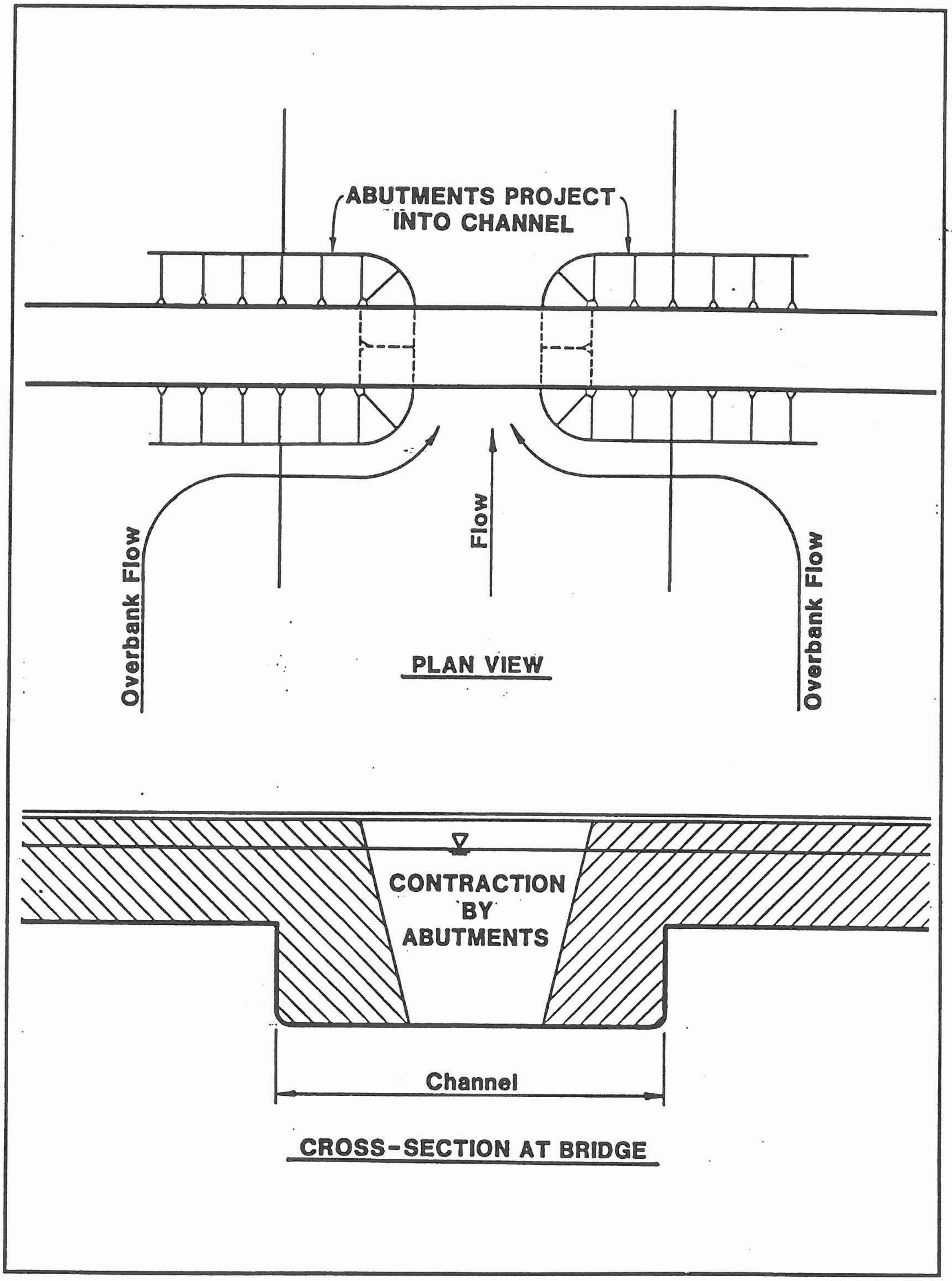
This class of devices encompasses all instrumentation which could be mounted in or attached to a vertical support which is either buried or driven into the channel bed at the location where scour is expected to occur. By sensing the channel bed/water interface, the progression of scour can be monitored or measured.

Various techniques can be utilized to measure scour with this class of instrumentation, however most are in experimental stages of development. These techniques include thermal and electrical conductance, sliding collars, or other sensors such as piezo-electrical strips or tip switches mounted externally on the support.

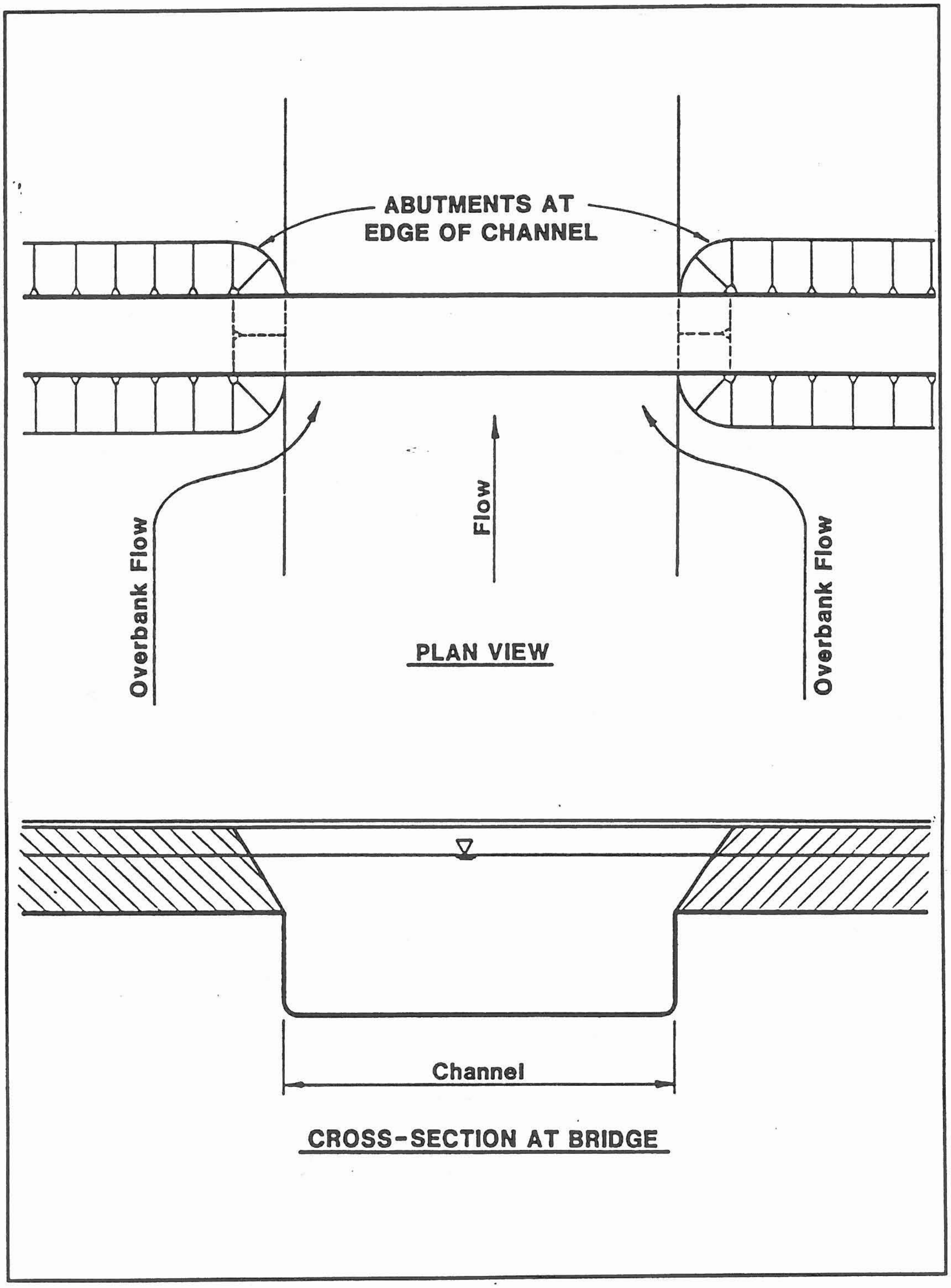
One of the most promising devices currently being developed (under NCHRP Project 21-3) consists of a buried rod with a sliding collar arrangement. Although still in development stage, this device shows promise as a simple, relatively inexpensive, easy to install and operate system for scour measuring and monitoring. Since testing is in progress, no further details of this instrument are available at this time, however it is believed that potentially this instrument could be utilized for a wide range bridge crossings.

APPENDIX H

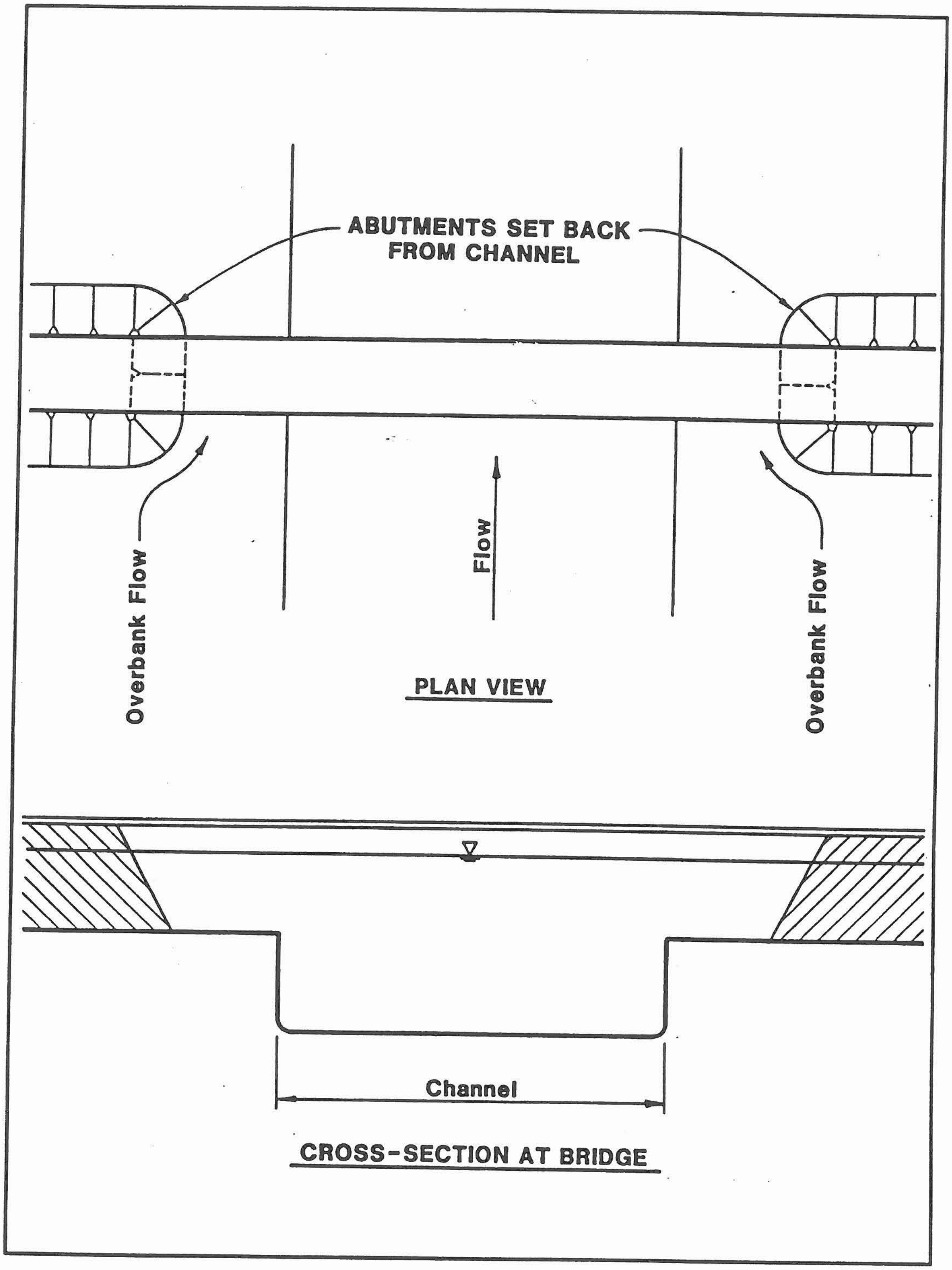
ILLUSTRATIONS OF THE FOUR MAIN CASES OF CONTRACTION SCOUR



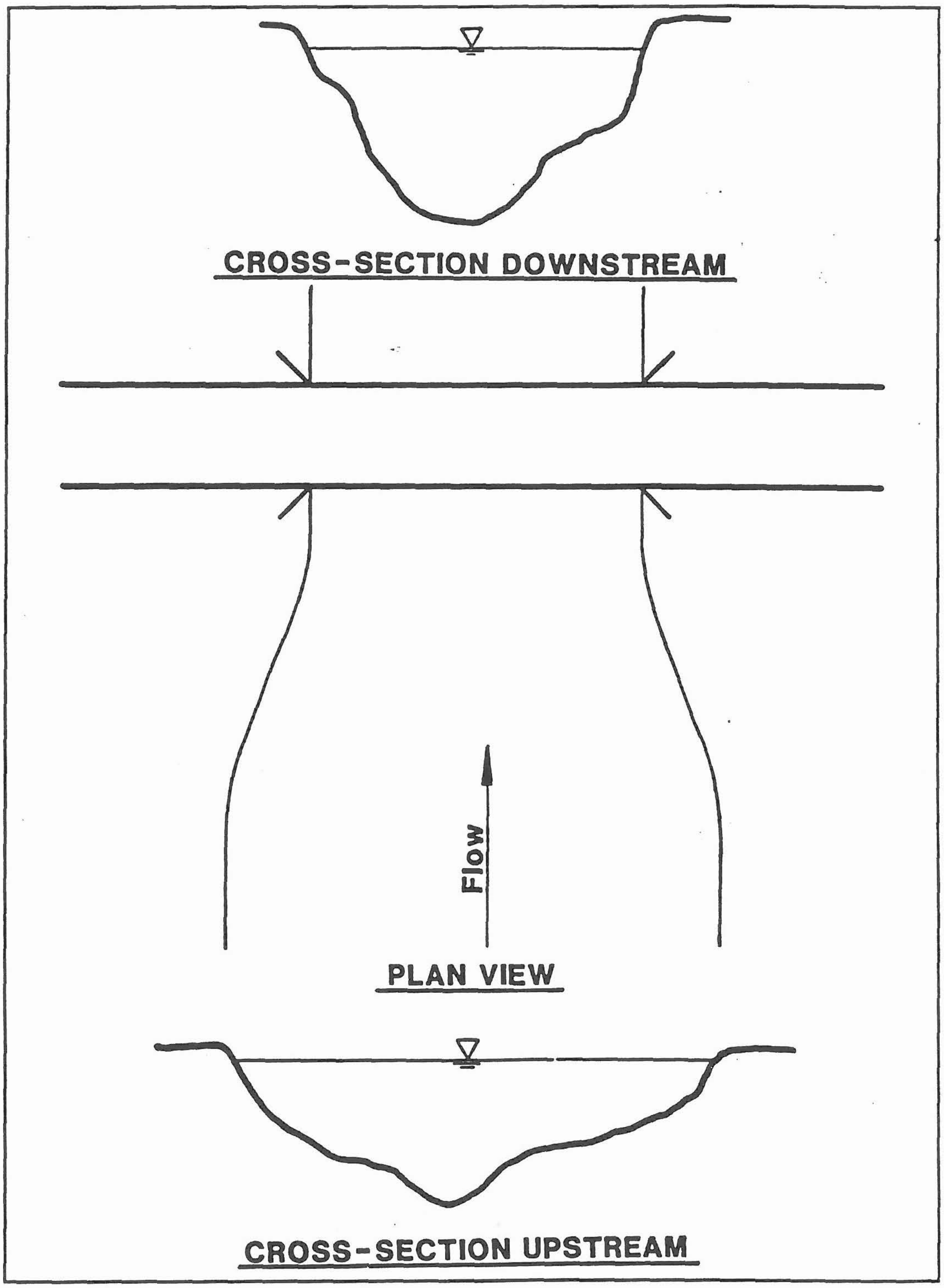
CASE 1A: ABUTMENTS PROJECT INTO CHANNEL



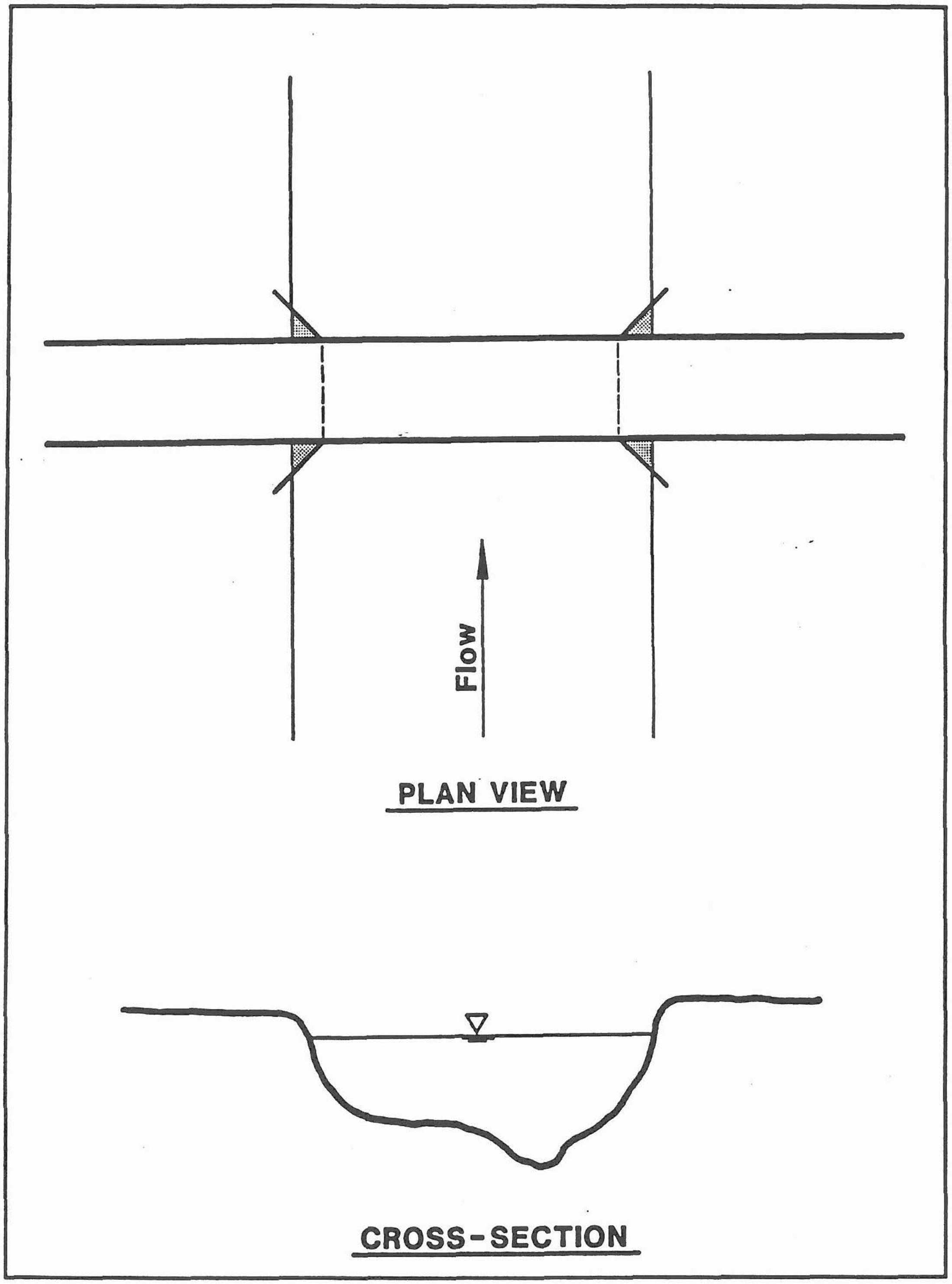
CASE 1B: ABUTMENTS AT EDGE OF CHANNEL



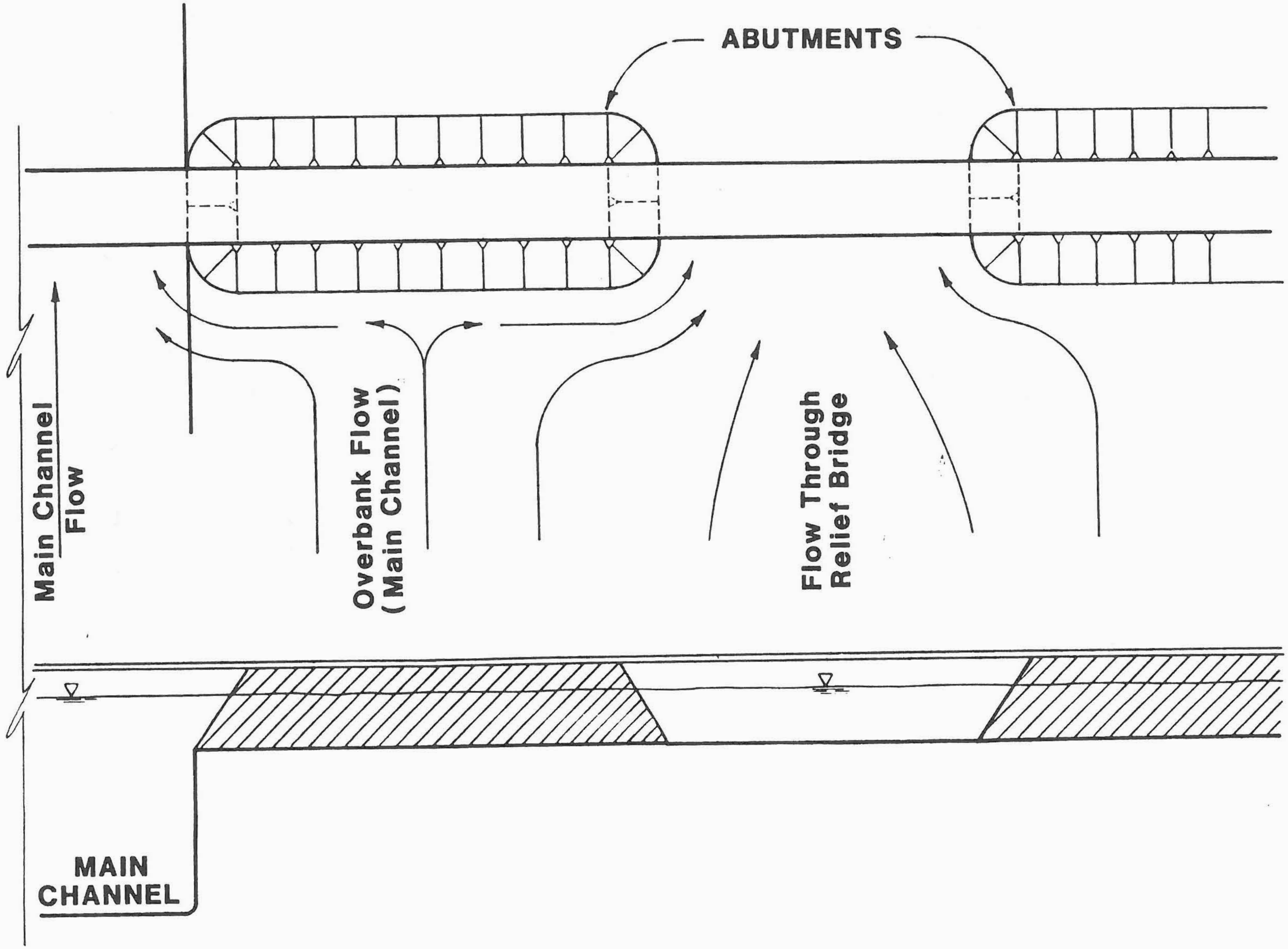
CASE 1C: ABUTMENTS SET BACK FROM CHANNEL



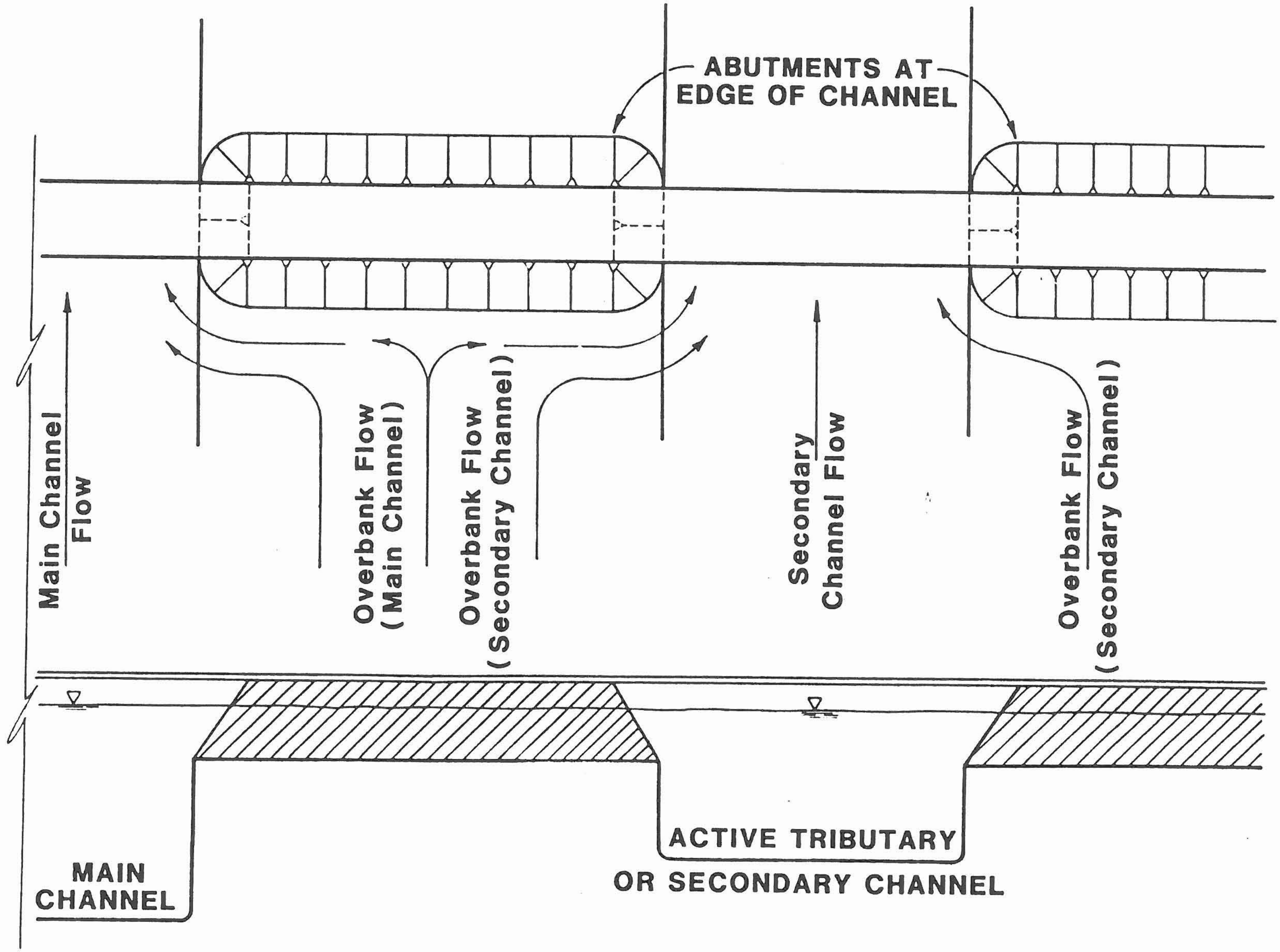
CASE 2A: RIVER NARROWS



CASE 2B: BRIDGE ABUTMENTS CONSTRICT FLOW



CASE 3: RELIEF BRIDGE OVER FLOODPLAIN



CASE 4: RELIEF BRIDGE OVER SECONDARY STREAM