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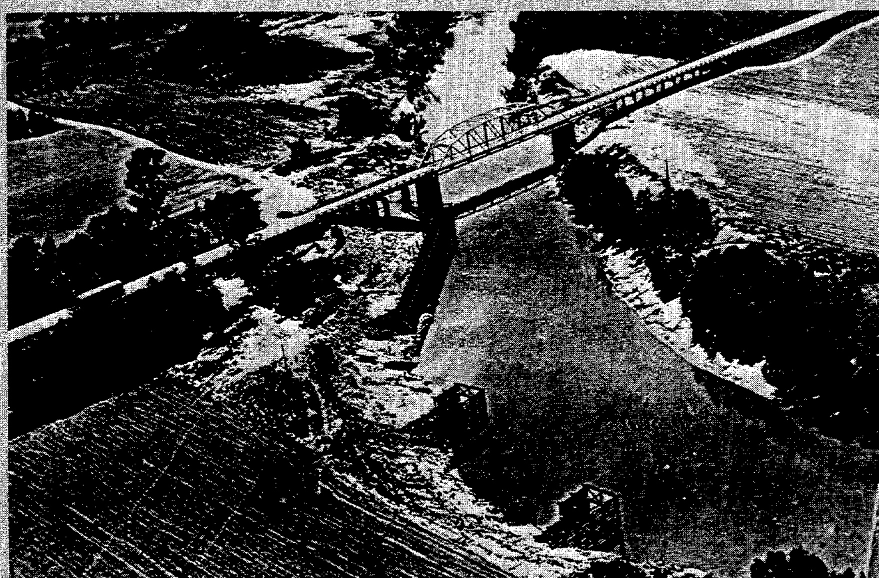
COUNTERMEASURES FOR HYDRAULIC PROBLEMS AT BRIDGES

Vol. I Analysis and Assessment



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Final Report

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Prepared for
FEDERAL HIGHWAY ADMINISTRATION
Offices of Research & Development
Washington, D. C. 20590

FOREWORD

This report is composed of two volumes: Volume I is an Analysis and Assessment; Volume II is a collection of 224 detailed case histories. The report provides guidelines to assist design, construction, and maintenance engineers in selecting measures that may be used to reduce bridge losses attributable to scour and bank erosion.

This research report is in the general subject area of hydraulics and hydrology. This research is in the Federally Coordinated Program (FCP) of Highway Research and Development. It is part of FCP Project 5H "Protection of the Highway System from Hazards Attributed to Flooding." The FCP project manager is Dr. Roy E. Trent and the task manager is Mr. Stephen A. Gilje.

Sufficient copies of Volume I and II will be distributed to provide a minimum of one copy to each FHWA regional office, division office and State highway agency. Direct distribution is being made to the division offices.

Charles F. Scheffey
Director, Office of Research
Federal Highway Administration

NOTICE

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16. Abstract

Guidelines have been developed to assist design, maintenance, and construction engineers in selecting measures that can be used to reduce bridge losses attributable to scour and bank erosion. These guidelines are based on case histories of 224 bridge sites in the U.S. and Canada, on interviews with bridge engineers in 34 states, and on a survey of published work on countermeasures. Each case history (in Vol. 2) includes data on bridge, geomorphic, and flow factors; a chronological account of relevant events at the site; and an evaluation of hydraulic problems and countermeasures. Problems at piers occurred at 100 sites and problems at abutments, at 80 sites. Problems are attributed to local scour at 50 sites, to general scour at 55 sites, and to lateral stream erosion at 105 sites. Performance ratings are given for rigid and flexible revetment, for flow-control measures (spurs, dikes, spur dikes, check dams, jack fields), and for measures incorporated into the bridge. Streams are classified for engineering purposes into five major types, each having characteristics of lateral stability and behavior that need to be taken into account in the design of bridges and countermeasures. Hydraulic analysis was carried out for flood conditions at 60 bridges, for which values of flow, bridge, and geomorphic factors are tabulated.

The report is in two volumes: Volume 1 - Analysis and Assessment
Volume 2 - Case Histories for Sites 1-283

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PREFACE

The first use of countermeasures to protect bridges from damage by scour and bank erosion seems not to be recorded, but an early use of riprap to protect piers evolved in France during the 1500's. Piers at some bridges were protected by a crib consisting of a ring of almost continuous piles driven around a pier and filled with stones. The piles tended to be uprooted by scour, leaving the pile of stones around the pier. This led to the practice of providing protection by riprap without a crib. The continued existence in France of bridges hundreds of years old, protected by riprap at piers, provides evidence for the effectiveness of riprap (Robinson, 1964, p. 54).

In the United States, protective measures seem to have been little used at bridges before 1920. Bridges tended to be located at favorable crossing sites and were rebuilt longer and higher if destroyed by flood. With the advent of roads built to higher standards--not subject to overtopping by minor floods--an increased proportion of overbank flow was diverted through the bridge waterway, and the need for protective measures became more apparent. In addition, the choice of crossing sites became increasingly dictated by factors other than hydraulic suitability.

Countermeasures are now used by all states and Canadian provinces, but the extent of use and the practices differ greatly from one state to another, partly because of differences in stream type, in climate, and in geology. However, experience with countermeasures in one region is transferable to another, with some allowance for regional differences. Experience with countermeasures seems best conveyed and documented by means of case histories.

This report on hydraulic problems and countermeasures at bridges has been prepared by the U.S. Geological Survey under an inter-government agreement with the Federal Highway Administration. Volume I is a general treatment of hydraulic problems and countermeasures, and it is based mainly on a set of case histories which are reproduced in Volume II. Relevant published information was also used in the preparation of Volume I, as was information from interviews with state bridge engineers. Volume I serves as a guide to the case histories of Volume II. It contains, under the appropriate major headings, a listing of the case-history site numbers to which any particular bridge factor, geomorphic factor, kind of hydraulic problem, or kind of countermeasure is relevant. The case histories are for 224 bridge sites in 27 states and three Canadian provinces. Each case history includes information on the engineering and hydrologic features of the sites. Sites most suitable for hydraulic analysis, and for which adequate data could be obtained, have been written by Blodgett who wrote case histories 1-148.

Acknowledgements--The project was initiated by Stephen A. Gilje of the Office of Research, Federal Highway Administration, who served as contract manager and provided much help in carrying out the project. Comments and guidance were provided by J. Sterling Jones, Roy E. Trent, and Frank Johnson of the Federal Highway Administration.

U. S. DEPARTMENT OF TRANSPORTATION

FEDERAL HIGHWAY ADMINISTRATION

SUBJECT FHWA-RD-78-162 and FHWA-RD-78-163, "Countermeasures for Hydraulic Problems at Bridges," Vol. 1 and Vol. 11	FHWA BULLETIN June 15, 1979

This Bulletin covers distribution of the subject two-volume report which describes measures which have been used to protect highway bridges from scour and bank erosion. Volume I is an analysis and assessment of stream hazards, countermeasures and success or failure of measures as applied in the river environment. Volume II is a collection of 224 detailed case histories which were used in developing the first volume. The report will be of interest to highway design, construction and maintenance engineers working in the river environment.

Volume I documents a survey of stream hazards to bridges, provides a statistical analysis of frequency of problems, and rates the performance of rigid and flexible revetments, and flow-control measures (spurs, dikes, spur dikes, check dams, jack fields). Measures incorporated into the design of the bridge are also evaluated. Streams have been classified for engineering purposes so that designs can take into account differing characteristics of lateral stability and stream behavior. Volume I includes detailed hydraulic analysis for 60 bridge sites. Volume II contains detailed case histories for use as a guide to measures which have been used in the field and tested. A slide presentation outlining salient portions of Volume I is also being prepared for distribution in the near future.

Sufficient copies of the report are being distributed to provide a minimum of one copy to each regional office, division office, and State highway agency. Direct distribution is being made to division offices. Additional copies for official use may be requested from Mr. David Solomon, Chief, Environmental Design and Control Division, FHWA, HRS-42, Washington, D.C. 20590. These requests will be filled while the limited supply lasts. Additional copies for the public are available from the National Technical Information Service, U.S. Department of Commerce, 5285 Port Royal Road, Springfield, Virginia 22161. A small charge will be imposed for each copy ordered from NTIS.

G. D. Love
Associate Administrator for
Research and Development

Attachments

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The work was done under the general direction of Harry H. Barnes, Jr., Chief, Surface Water Branch, U.S. Geological Survey. The following members of the Geological Survey are regarded as co-authors because they assembled data and wrote preliminary drafts for many of the case histories: P. J. Carpenter (Washington); M. F. Cook (Louisiana); G. S. Craig, Jr., (Wyoming); D. A. Eckhardt (Pennsylvania) M. S. Hines (Arkansas); K. L. Lindskov (Minnesota); D. O. Moore (Nevada); R. S. Parker (Tennessee); A. G. Scott (New Mexico); and K. V. Wilson (Mississippi).

The assistance and cooperation of many individuals and agencies in the preparation of this report is gratefully acknowledged. Particular thanks are due to the state and district engineers who not only identified sites and supplied general information, but also furnished bridge and countermeasure plans, and photographs, for specific sites. The help of Canadian engineers and the Project Committee on Bridge Hydraulics of the Roads and Transportation Association of Canada is also gratefully acknowledged. G. M. Mazurek of Alberta Transportation took Brice on a field trip to several Alberta bridge sites and furnished information for several bridge sites, as did D. L. Sullivan of the New Brunswick Department of Transportation.

Lester A. Herr of the Federal Highway Administration gave permission for the writing of letters of inquiry to Division Bridge Engineers of his organization, many of whom responded with useful information. L. D. Bruesch, Chief Highway Structures Engineer for the U.S. Forest Service, gave the addresses of Forest Service Regional Bridge Engineers; and several Forest Service bridge sites were obtained by correspondence, particularly through the assistance of Philip Keasey and Stephen Bunnell. Data for several sites were furnished by the U.S. Army Corps of Engineers. Gay D. Jones kindly lent the file of responses to a questionnaire on bridge scour that had been sent in connection with the preparation of National Cooperative Highway Research Program Synthesis 5 (1970).

The many line drawings of bridges and countermeasures in Volume II are based on engineering drawings obtained from the state or province in which the bridge is located. In order to be presented in the format of this report, the drawings had to be much reduced and generalized. We tried to represent the essential features as accurately as possible, but we have not attributed the specific sources because of possible inaccuracies or misinterpretations. All photographs obtained from persons or agencies outside the Geological Survey are acknowledged in the report.

Acknowledgment is made to the U.S. Army Research Office for a grant (No. DA-ARD-D-31-124-70-G89; Airphoto interpretation of the form and behavior of alluvial rivers) to Brice for the period 1970-74, during which sequential aerial photographs and other data on 200 stream reaches were collected. Interpretations of stream morphology presented here are based mainly on this work.

CONTENTS

CONTENTS

LIST OF ABBREVIATIONS AND SYMBOLS

Symbol	Definition	Unit
A	Cross-section area of stream or waterway	ft^2
A_j	Area of piers or piles in waterway	ft^2
b	Width of main channel	ft
β	Skewness of piers to flow	degrees
b_n	Net width of bridge normal to flow	ft
D_n	Size of material based on sieve analysis. Median size D_{50} (also referred to as mean size) is size of material for which 50 percent of material is finer or coarser.	mm
e	Eccentricity of bridge flow	--
F	Froude number	--
FM	Farm-to-market road	--
FS	Forest Service road	--
γ	Specific weight of water	lb/ft^3
g	Gravitational constant (acceleration)	ft/sec^2
I	Interstate highway	--
j	Ratio of pier area to gross area of waterway	--
L	Length of bridge	ft
LR	Local road	--
M	Bridge opening (contraction) ratio	--
n	Mannings roughness coefficient	$lb^{1/6}$
P	Wetted perimeter of channel	ft
p	Effective width of pier normal to flow	ft
PH	Provincial highway	--
P_L	Length of pier	ft
P_w	Width of pier	ft

LIST OF ABBREVIATIONS AND SYMBOLS

Symbol	Definition	Unit
R.I.	Recurrence interval of flood	--
ϕ	Skewness of bridge to flow	degrees
Q_a, Q_c	Constricted overbank discharge	ft^3/s
Q_b	Unconstricted discharge	ft^3/s
Q	Discharge	ft^3/s
Q_t	Total discharge	ft^3/s
R	Hydraulic radius of channel bed (A/p)	ft
τ_c	Critical shear stress at the boundary	lb/ft^2
τ_o	Shear stress at the boundary ($\gamma \overline{Y} S_o$)	lb/ft^2
S_f	Slope of energy grade line	--
S_o	Slope of water surface or channel bed	--
SR	State route	--
t	Bridge submergence	ft
US	U.S. highway	--
V	Velocity of flow	ft/s
$\overline{V}$	Mean velocity of flow	ft/s
Y	Depth of flow	ft
$\overline{Y}$	Average depth of flow	ft

LIST OF CASE HISTORY SITES

LIST OF CASE HISTORY SITES (CONTINUED)

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LIST OF CASE HISTORY SITES (CONTINUED)

Chapter 1

INTRODUCTION

Damage to bridges and highways from floods amounted to an estimated $100 million in 1964 and also in 1972, years in which exceptionally large floods occured (Chang, 1973). The average loss for recent years is about $50 million a year. Stream-related damage and maintenance problems also occur without floods, but the expense of such damage and problems is more difficult to assess. Experience has shown that losses, damage, and maintenance problems from stream-related causes can be reduced by the use of suitable countermeasures.

Scope and objective--The objective of this study is to develop guidelines to assist design, construction, and maintenance engineers in selecting measures that can be used to reduce bridge losses and damage attributable to scour and bank erosion. Emphasis is directed toward measures that can be applied to existing bridges to overcome deficiences such as (1) underdesign for scour, (2) excessive scour due to reoriented flow which may effectively change a streamlined pier shape into a blunt shape, (3) excessive scour at a pier or an abutment due to flow concentration, and (4) excessive bank erosion due to shifting meanders and flow concentrations. The problem of accurately predicting scour depths is considered only to the extent necessary to evaluate scour countermeasures. With regard to methods of controlling bank erosion, the study is limited to the application of existing technology to the problem of protecting bridges.

Method of investigation--An initial survey was made through U.S. Geological Survey district (state) offices in order to inventory the range of hydraulic variables that should be considered and to survey countermeasure practices and the incidence of bridge losses in different states. Preliminary reports from the districts served to identify states from which adequate information could be obtained.

A preliminary stream classification was adapted (from Culbertson, Young, and Brice, 1967) for the ultimate purpose of relating hydraulic problems and countermeasure practices to stream properties and stream type. Sites were selected in Mississippi, Tennessee, Missouri, and Iowa, at which channel instability problems were known to exist at bridges. These sites were inspected and the preliminary classification was revised.

A plan was developed to survey current countermeasure practices and to determine how effectively specific countermeasures have been used to help safeguard bridges against failures associated with flooding. A further objective was to identify sites for more detailed investigations. A set of questions was written, for which responses were obtained by interview with bridge and maintenance engineers. Originally, the study was to be restricted to 10 states, which were chosen partly to provide geographical diversity and partly because of the availability in a particular state of U.S. Geological Survey personnel to work on the project.

It was found that the 10 states originally chosen did not provide a sufficient diversity of hydraulic problems and countermeasure practices. With the advice of the contract manager, the scope of the study was extended by visits to highway agencies in other states. In all, the highway agencies of 34 states were visited. Not all of these reported a site suitable for detailed investigation, but all were interviewed and their responses to general questions are summarized herein.

In addition to visits at state offices, letters of inquiry regarding possible sites were sent to 17 district offices of the U.S. Army Corps of Engineers; to 22 division bridge engineers of the Federal Highway Administration; and to nine Forest Service regional bridge engineers. About 20 of the sites reported as case histories in Volume II came from these letters of inquiry. Correspondence with members of the Project Committee on Bridge Hydraulics of the Roads and Transportation Association of Canada, and a visit to Alberta and Saskatchewan by Brice, resulted in eight case histories.

Identification of sites suitable for study depended on the memory and experience of the persons contacted. No state or agency was found to have a file of sites where hydraulic problems (or bridge losses) had occurred and countermeasures had been applied. If there is a bias in the preliminary identification of sites, it is probably toward sites where problems have occurred most recently, because these are most easily recalled.

Evaluation of Hydraulic Problems and Countermeasures

Site selection--As a result of the survey described above, about 325 bridge sites potentially suitable for further investigation were identified. File folders on these sites were examined by the contract manager. For about 285 sites that best met the objectives of the project, efforts were made to collect necessary data, but this was not always possible. In some cases, the needed data were not avaialble; in others, no qualified persons were available to search for additional data and to visit the site.

Because the usefulness and validity of this study depends in large degree on the detailed evaluation of specific sites (reported as case histories in Volume II, the criteria for site selection require further discussion. The kind of site to be considered for a case history was mainly determined by the requirement for countermeasure evaluation . We concluded that the kind of site best suited for countermeasure evaluation was one where a hydraulic problem had occurred, a countermeasure had been applied, and the countermeasure had then been tested by a flood. We searched for sites of this kind, and most of the case histories involve this sequence of events. At another kind of site, a countermeasure had been applied before the occurrence of a problem, and a flood resulted in no damage to the bridge. Some cases of this kind have been used, but the countermeasure may or may not have presented damage. A few of the sites included serve only to document hydraulic problems, countermeasure practices, or stream type.

Although no random sampling procedure was used in arriving at the particular set of sites reported as case histories in Volume II, there is no apparent reason for bias toward selection of sites representing any particular kind of hydraulic problem and countermeasure. Also, all major physiographic regions of the United States are represented, except New England. The different backgrounds of the authors (Blodgett in Civil Engineering, and Brice in Geology) has introduced some differences in presentation and emphasis, as may be discerned from comparison of case histories 1-148, written by Blodgett, with 150-283, written by Brice. Sites most suitable for hydraulic analysis, and for which adequate data could be obtained, have been written by Blodgett.

Standardization and collection of data--The objectives of this project, particularly with regard to countermeasure performance, involve a consideration of many factors. To standardize data collection and analysis, the factors considered to be most relevant were divided into three sets, called bridge factors, geomorphic factors, and flow factors. The factors in each set are listed and discussed more fully in separate chapters of this report, and definitions of the factors are provided in the Glossary. In general, bridge factors include general information about a bridge, such as its length, as well as factors specifically related to a problem, such as location at a meander. Geomorphic factors include general information about a stream, such as its degree of sinuousity, as well as factors specifically contributing to a problem, such as mining of gravel from the channel. Flow factors include such items as recurrence interval of a particular flood and water velocity in the channel. A few factors, such as valley or channel slope, belong to two sets.

Hydraulic problems and countermeasures are defined and classified (see appropriate chapter and Glossary), and the relation between these and the three sets of factors is conceived as shown schematically in fig. 1. According to this scheme, a bridge-crossing situation is represented, at low flow, by a set of bridge factors, a set of geomorphic factors, and a countermeasure, as shown in the top row of boxes, fig. 1. (If no countermeasure exists, the countermeasure box is disregarded.) Both bridge factors and geomorphic factors interact, or potentially interact, with the countermeasure. If a flow event occurs, either a flood or a high flow within bankful stage, the performance of the countermeasure is tested by its interaction with bridge factors, geomorphic factors, and flow factors. If no countermeasure is present, the hydraulic performance of the bridge is tested. As a result of the flow event, a hydraulic problem may or may not result. If a problem results, a new or additional countermeasure may be installed, and the sequence of testing will be repeated at the next flow event.

Information on the bridge, countermeasures, and chronological sequence of events was obtained from the agency responsible for the bridge and recorded on a standardized data-collection form. Depending on the filing system and other circumstances, information on a bridge site can be readily obtained in the central files of some states, but only with difficulty in others. Additional information from district highway offices is usually helpful, but time did not permit this for many sites. Hydrologic and geomorphic information was obtained from publications or files of the Geological Survey, or from maps and aerial photographs. Nearly all of the sites were observed in the field, either by the authors and the contract manager or by a designated member of the Geological Survey, but some (about 5 percent) were not.

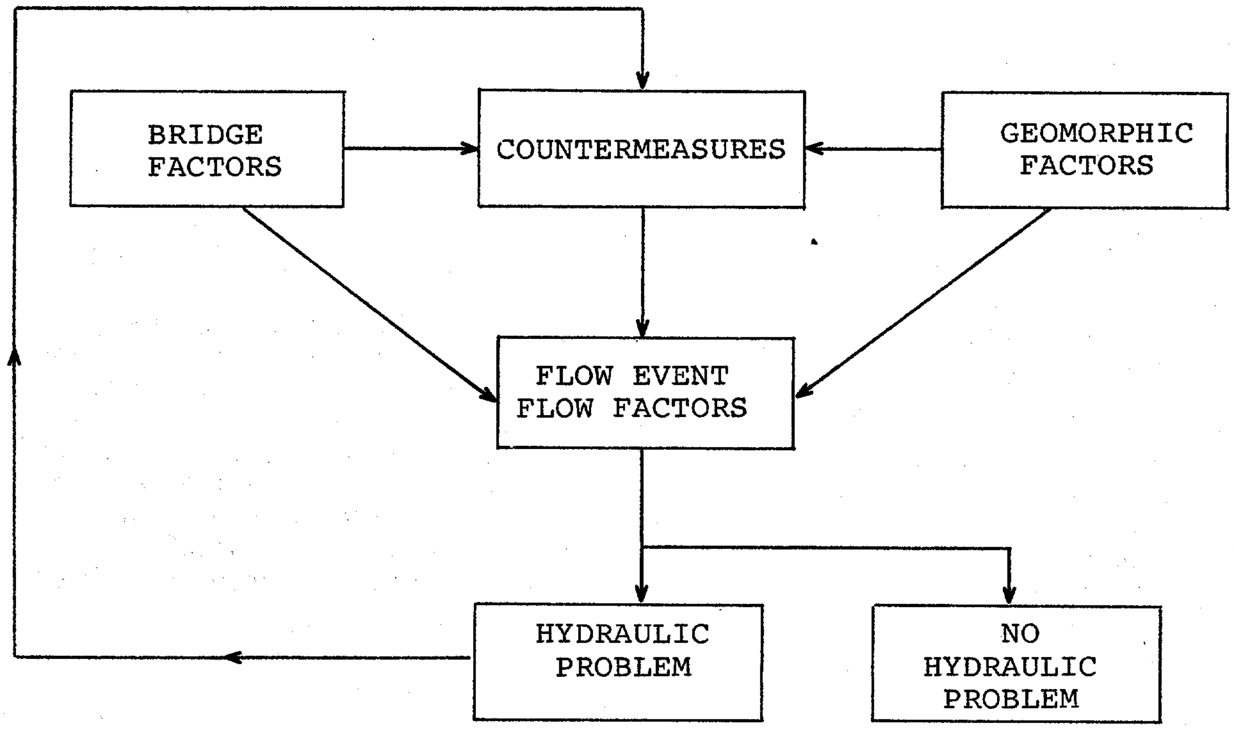

Figure 1. Schematic relation of countermeasures, hydraulic problems, and related factors.

Previous Work

Although there has been no previous summary of national scope on hydraulic problems and countermeasures at bridges, a great deal of work has been published on fluvial hydraulics and on countermeasures, particularly in relation to river training and bank stabilization. Reference will be made here only to those works that have been found most generally useful in the preparation of this report. These include the work of the Project Committee on Bridge Hydraulics of the Roads and Transportation Association of Canada (Neill, 1973), which contains a brief but very useful treatment of bridge hydraulics and experience with countermeasures, particularly in Canada. Problems, practices, and countermeasure performance in California are fully treated in California Division of Highways (1970), which contains a great deal of information that is both useful and of general application. However, information on any particular type of countermeasure tends to be distributed under different chapter heading (design, construction, planning, maintenance, prior reports), such that the California report requires careful reading. Keeley (1971) has published a valuable study, supported by 20 detailed case histories, of problems and practices in Oklahoma. Chang (1973) has reported on the causes of bridge failures in the United States during certain major floods of the period 1969-72. Experience of the U.S. Army Corps of Engineers with measures for channel stabilization in major alluvial rivers has been summarized by Lindner (1969). A national study on evaluation of control methods for streambank erosion has been begun by the U.S. Army Corps of Engineers (Pickett and Brown, 1977).

Chapter 2

HYDRAULIC PROBLEMS

DEFINITION AND ANALYSIS

A hydraulic problem is defined here as an effect of streamflow, tidal flow, or wave action on a crossing, such that traffic is immediately or potentially disrupted. As so defined, a hydraulic problem may range in degree from rather trivial, requiring only routine maintenance for correction, to partial or complete failure of the structure; and it is difficult to set definite limits within these extremes. However, most of the problems documented in this report lie in a middle range between the extremes, in that they present a definite hazard to the bridge and are subject to corrective action less drastic than partial or complete replacement of the structure. For bridges that have been swept away in catastrophic floods, it is usually difficult to decide which part failed first or what practical countermeasures might have been taken to prevent the loss.

Hydraulic problems are described here according to the location directly affected (such as pier, abutment), the hydraulic process involved in the problem (such as scour, lateral erosion, wave action), and the extent of the problem (such as undermining of spread footing, or subsidence of pier or abutment). Terms used are defined in the Glossary, and most of the definitions are conventional. However, an attempt has been made to restrict and sharpen the meaning of some terms applying to hydraulic process, in order to analyze hydraulic problems and countermeasure performance more usefully. A countermeasure that is effective for one hydraulic process may not be effective for another.

In common engineering usage, and as defined in Highway Research Board (1970, p. 3), the term "scour" includes all erosive action of running water in streams, including erosion of both bed and banks. In geologic and hydrologic usage, the term usually indicates a downward erosive action, as implied by the opposing meaning of terms in the phrase "scour and fill". For purposes of this report, it is useful to distinguish an erosive process that acts mainly downward, or vertically, such as bed erosion at a midstream obstruction, from one that acts mainly in a lateral direction, such as bank erosion at the outside of a meander bend. The causes and countermeasures for scour at a pier are not the same as those for lateral channel migration at a pier or abutment. The typical effects of scour and lateral erosion, as defined here, are illustrated in figs. 2, 3, and 4.

As used here, and as generally used, the term "local scour" refers to scour that is localized at (and, by implication, caused by) a pier, abutment, or other obstruction to flow. The meaning of the term "general scour" is less clear, but for present purposes it refers to scour that is not localized at a pier or other obstruction (fig. 3). General scour commonly, but not necessarily, affects all or most of the cross-sectional width of a channel. Any scour, whether general or local, is discontinuous along the length of a channel (Colby, 1964).

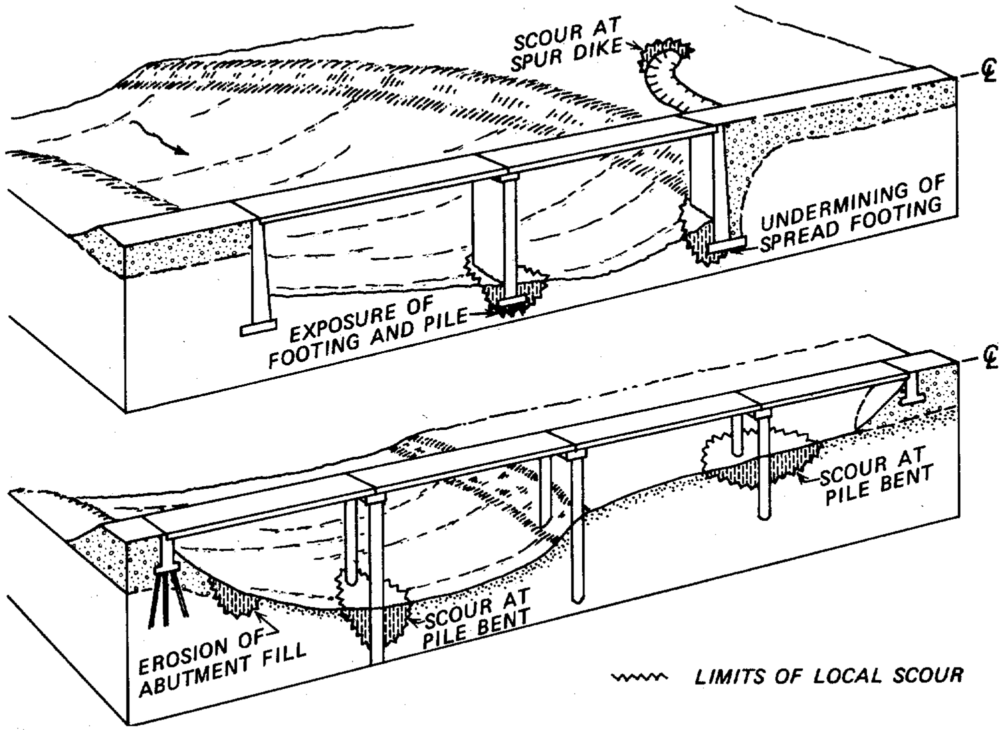

Figure 2. Local scour and related hydraulic problems at bridges, attributed to the effects of obstructions to flow.

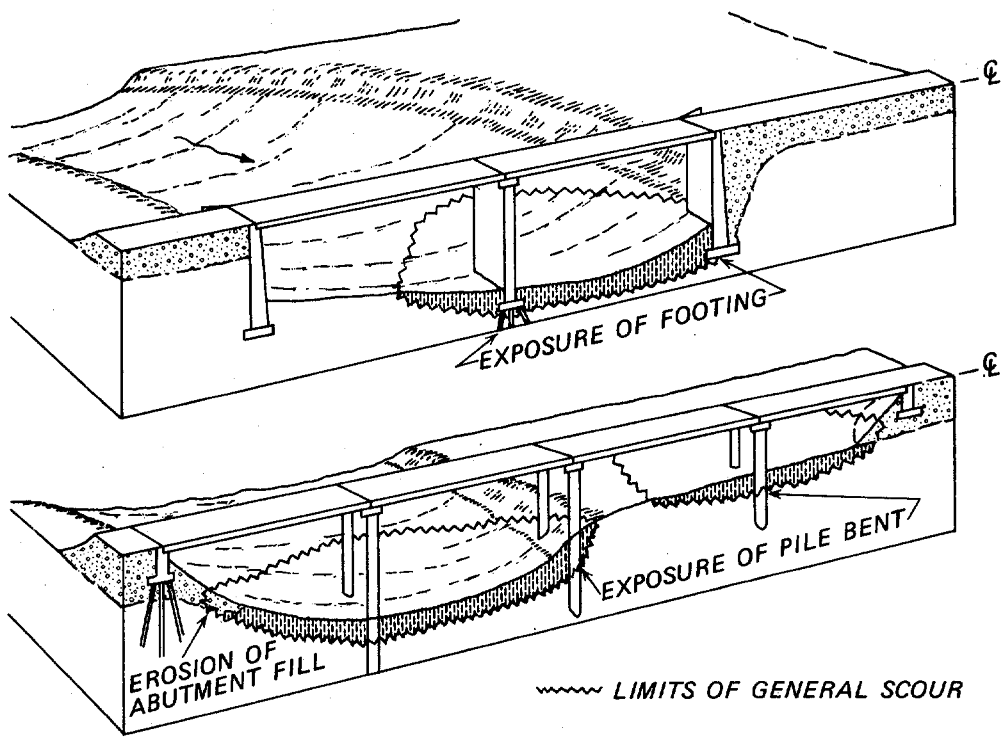

Figure 3. General scour and related hydraulic problems at bridges, attributed to contraction of flow or to channel deepening at the outside of a bend.

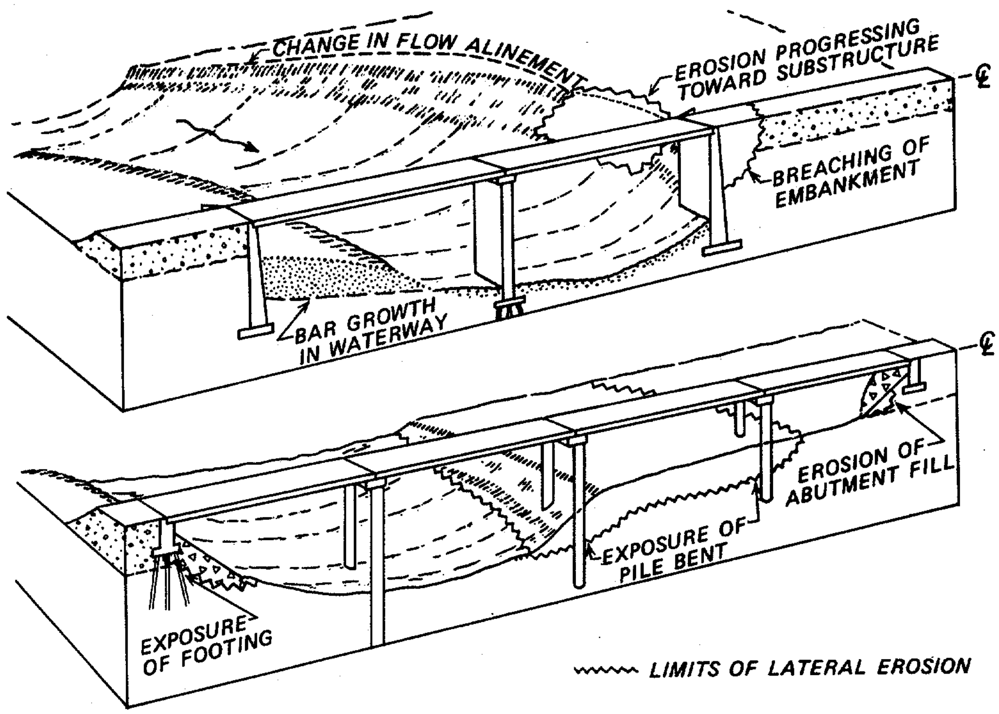

Figure 4. Lateral stream erosion and related hydraulic problems at bridges, attributed to erosion at a bend or to lateral migration of the channel.

The term "degradation" applies to a continuous, long-term, and usually progressive lowering of the channel bed through a channel reach of significant length, for example, a length greater than about 10 channel widths. Scour is more restricted in time and space than degradation, and more likely to be followed by fill (aggradation).

CHARACTERISTICS OF HYDRAULIC PROBLEMS AT STUDY SITES

In table 1, sites to which each of the problem characteristics--of location, process, or extent--apply are listed by site number. This table is intended to serve as an index to the case histories and also as a general summary of the occurrence of problems as documented in the case histories. For interpretation of the table as a summary, the following points are relevant: (1) Some sites have no problems, and hence are not listed in this table; other sites have several problems or the same problem at several locations, and hence are listed under several headings. (2) At some sites, the case history involves two or more bridges, built at different times, and having different structural features.

Location of Problem

Problems at piers (or pile bents) are somewhat more common at the study sites than problems at abutments (table 1). Contrasting results were reported by Chang (1973), who found damage to the pier in 24.5 percent of 383 cases of bridge failure that he studied and damage to the abutment in 71.8 percent of the cases. However, the cases studied by Chang include many smaller and older bridges which probably had no piers and which, at least in Pennsylvania, had masonry abutments on shallow spread footings. Also, Chang's study is on the effects of very large or catastrophic floods. Most of the bridges in the present study are more than 100 ft (30.5 m) in length, have spillthrough abutments, and piers in the channel. Piers, particularly those in a channel, are more subject to certain problems such as local scour, catchment of drift, and abrasion by sediment. Problems at piers outside a channel usually involve lateral bank erosion or flood plain scour during overbank flow. As for abutments, the greater number of listings for the spillthrough type reflects the fact that most bridges in the case histories have spillthrough abutments. Erosion of abutment fill-slopes, with or without the exposure of pile-supported footings, is due in most cases either to lateral erosion or to general scour.

Channel problems, as interpreted here, are mostly those that affect the channel independently of the bridge, such as degradation of the channel bed and lateral erosion of the channel banks. Also, at most of the sites listed under "banks", the bank erosion has not yet reached the bridge.

Although the bridge superstructure failed at about 19 of the sites, at only three sites was damage to the superstructure attributed directly to hydraulic forces. In most of the 19 sites, failure of the superstructure is attributed to failure of piers or abutments. Chang (1973) reported damage to the bridge superstructure in 14.9 percent of 383 cases of bridge failure that he analyzed. However, most of the bridges in the present study have not failed, and the problems reported do not, for the most part, reflect the effects of catastrophic flooding.

Table 1. Characteristics of hydraulic problems at study sites

LOCATION OF PROBLEM

1. Pier or pile bent,

   in channel (bed or banks):
   Sites 1, 2, 3, 4, 7, 8, 9, 10, 11, 16, 17, 23, 24, 26, 31, 33, 41, 42, 44, 46, 47,49, 50, 51, 52, 53, 54, 61, 68, 69, 70, 85, 89, 90, 103, 122, 123, 124, 125, 127, 130, 131, 132, 133, 139, 141, 146, 148, 154, 160, 164, 168, 174, 185, 193, 197, 198, 201, 204, 205, 207, 208, 215, 218, 223, 227, 242, 246, 247, 257, 261, 262, 264, 265, 267, 268, 269, 272, 276, 277, 282.

   outside of channel:
   Sites 17, 35, 38, 52, 69, 145, 162, 170, 174, 177, 209, 210, 211, 213, 219, 228, 238, 258, 263, 281.

2. Abutment,

   spillthrough, at channel:
   Sites 8, 12, 14, 26, 28, 30, 37, 39, 41, 42, 45, 48, 50, 54, 55, 56, 60, 95, 131, 143, 148, 153, 165, 166, 167, 169, 172, 173, 175, 176, 178, 180, 181, 183, 186, 187, 196, 199, 200, 206, 220, 225, 226, 233, 235, 242, 243, 245, 246, 248, 250, 260, 266, 267, 270, 272, 273.

   spillthrough, set back from channel:
   Sites 162, 174, 212, 214, 221.

   vertical:
   Sites 20, 21, 23, 26, 27, 31, 32, 36, 42, 182, 194, 201, 202, 224, 236, 237, 239, 252, 254.

3. Channel,

   bed: Sites 1, 4, 20, 24, 49, 51, 53, 55, 71, 89, 90, 134, 143, 146, 148, 159, 161, 163, 172, 173, 174, 184, 203, 212, 222, 224.

   banks: Sites 4, 7, 10, 28, 37, 39, 46, 55, 60, 73, 85, 91, 99, 115, 120, 125, 130, 131, 133, 148, 153, 159, 161, 169, 170, 171, 173, 174, 175, 177, 178, 185, 188, 192, 212, 213, 216, 220, 222, 223, 224, 226, 236, 240, 241, 248, 252, 253, 254, 257, 258, 260, 263, 273, 282.

4. Superstructure:
   Sites 11, 138, 250.

5. Approach roadway embankment,

   adjacent to abutment:
   Sites 8, 14, 25, 27, 29, 36, 41, 42, 50, 54, 56, 60, 95, 99, 155, 173, 181, 193, 194, 211, 234.

   not adjacent to abutment:
   Sites 14, 18, 19, 22, 55, 56, 95, 154, 157, 195, 201, 204, 217, 234, 279.

6. Spur dike:
   Sites 30, 34, 47, 68, 144, 156, 204, 221.

Table 1. Characteristics of hydraulic problems at study sites (continued)

HYDRAULIC PROCESS INVOLVED IN PROBLEM

1. Scour,

    local (effects known to be localized at pier, abutment, or embankment):
    Sites 1, 2, 10, 12, 14, 16, 17, 21, 23, 30, 32, 33, 35, 38, 40, 42, 44, 46, 51, 68, 88, 122, 139, 144, 156, 162, 165, 166, 168, 177, 183, 185, 204, 205, 206, 211, 221, 238, 239, 257, 262, 264, 265, 268, 269, 270, 282.

    general (effects not known to be localized at pier, abutment, or embankment):
    Sites 4, 7, 8, 9, 10, 11, 17, 19, 20, 24, 25, 26, 27, 29, 31, 37, 41, 42, 45, 47, 48, 49, 50, 52, 53, 54, 56, 61, 69, 70, 103, 123, 124, 125, 132, 141, 143, 146, 154, 155, 163, 164, 172, 184, 201, 203, 215, 218, 222, 237, 245, 250, 262, 267, 276.

2. Lateral erosion,

    by stream action:
    Sites 1, 4, 7, 8, 10, 12, 28, 36, 37, 41, 42, 46, 48, 50, 54, 55, 56, 60, 73, 85, 91, 95, 97, 99, 103, 115, 120, 125, 130, 131, 133, 144, 145, 148, 153, 154, 159, 161, 169, 170, 171, 173, 174, 175, 176, 178, 179, 180, 181, 182, 183, 185, 186, 188, 192, 193, 194, 195, 196, 204, 207, 208, 209, 210, 211, 212, 213, 214, 215, 216, 219, 220, 222, 223, 224, 225, 226, 227, 228, 233, 234, 235, 236, 238, 239, 240, 241, 242, 243, 245, 246, 248, 252, 253, 254, 257, 258, 260, 261, 263, 266, 273, 279, 280, 281, 282.

    by wave action:
    Sites 167, 187, 201, 202, 267.

3. Slumping of bank or embankment:
    Sites 131, 148, 163, 167, 173, 199, 200, 217, 263.

4. Channel degradation:
    Sites 1, 7, 24, 39, 45, 52, 85, 89, 90, 124, 148, 159, 160, 161, 168, 169, 170, 173, 174, 193, 197, 198, 205, 212, 220, 227, 238, 239, 242, 246, 247, 261, 277, 282.

5. Channel aggradation:
    Sites 25, 55, 57, 123, 266.

6. Accumulation at bridge,

    of floating debris:
    Sites 1, 8, 17, 25, 27, 32, 42, 51, 52, 58, 70, 88, 155, 168, 170, 174, 194, 226, 257, 261, 262, 264, 266, 268, 270, 276.

Table 1. Characteristics of hydraulic problems at study sites (continued)

HYDRAULIC PROCESS INVOLVED IN PROBLEM (continued)

6. Accumulation at bridge,

    <u>of ice</u>
    Sites 14, 239.

    <u>of sediment</u> (<u>bar growth</u>)
    Sites 8, 42, 68, 163, 176.

7. Impact,

    <u>by floating debris</u>
    Sites 88, 132, 250.

    <u>by water</u>
    Sites 11, 138.

    <u>by ice</u>
    Sites 248, 250.

    <u>by boulders</u>
    Site 8.

8. Abrasion,

    <u>by sediment</u>
    Sites 3, 8, 17, 54, 95, 127.

9. Buoyant forces
    Site 138.

EXTENT OF PROBLEM

1. Land loss--erosion of banks or flood plain
    Sites 30, 46, 91, 115, 248, 253.

2. Change in flow alinement
    Sites 68, 216.

3. Erosion progressing toward substructure or roadway embankment
    Sites 46, 120, 131, 134, 144, 159, 161, 163, 169, 171, 180, 181, 184, 187, 188, 195, 203, 207, 208, 212, 216, 219, 220, 221, 222, 225, 236, 240, 241, 254, 258, 281.

4. Clearance of superstructure reduced by aggradation of channel
    Site 71.

5. Abrasion or impact damage to pier
    Sites 3, 8, 17, 54, 95, 127, 132.

Table 1. Characteristics of hydraulic problems at study sites (continued)

EXTENT OF PROBLEM (continued)

6. Erosion of abutment fill
   Sites 12, 26, 28, 32, 39, 48, 54, 60, 133, 143, 153, 162, 165, 166, 172, 175, 179, 182, 194, 196, 198, 199, 200, 202, 206, 211, 214, 224, 235, 248, 280.

7. Erosion of roadway embankment fill
   Sites 8, 19, 22, 25, 27, 29, 41, 54, 56, 95, 99, 165, 183, 193, 194, 201, 204, 217, 234, 243.

8. Breaching of roadway embankment
   Sites 14, 42, 95, 155, 182, 233.

9. Outflanking of bridge by stream
   Site 233.

10. Pile bent exposed to hazardous depth
    Sites 41, 47, 53, 131, 164, 197, 198, 209, 211, 213, 218, 223, 227. 228, 263.

11. Pile-supported footing (of pier or abutment) exposed to hazardous depth
    Sites 1, 4, 7, 16, 17, 21, 24, 42, 49, 85, 89, 130, 141, 145, 154, 163, 173, 178, 183, 185, 186, 204, 205, 226, 227, 242, 245, 246, 247, 252, 265 266, 267, 268, 269.

12. Undermining of spread footing
    Sites 2, 8, 23, 26, 27, 44, 90, 122, 124, 125, 139, 177, 265, 270, 272, 273.

13. Subsidence of pier or abutment
    Sites 9, 26, 27, 51, 52, 70, 88, 139, 148, 164, 168, 201, 215, 233, 261, 264, 272, 273, 276, 277.

14. Collapse of span
    Sites 11, 31, 32, 36, 37, 50, 52, 55, 61, 88, 103, 123, 138, 146, 174, 239, 262, 270, 282.

EFFECT ON BRIDGE USE

1. Major effect--bridge closed
   Sites 8, 9, 11, 14, 26, 31, 36, 37, 50, 51, 52, 55, 61, 70, 88, 99, 103, 123, 138, 139, 146, 155, 174, 182, 233, 234, 239, 261, 262, 270, 272, 273, 276, 277, 282.

2. Minor effect--traffic disrupted
   Sites 2, 17, 21, 25, 27, 29, 38, 60, 148, 210, 250, 264.

3. Potential effect
   (All other sites)

Problems at the approach-roadway embankment include both breaching and erosion of embankment fill, and the sites listed include only those in which there were problems at the bridge as well as the embankment. Cases in which only the approach embankment is eroded are probably rather common, but such cases were not included in this study.

Problems at spur dikes include the development of scour holes of usual size at the spur dike, and partial or complete destruction of the spur dike. Although the probability of these occurrences is anticipated in spur-dike design, they are reported here as part of the information on spur-dike performance.

Hydraulic Process Involved in Problem

Because the effects of local scour are commonly superimposed on the effects of general scour, a clear separation of the two effects is difficult if not impossible. This difficulty is circumvented in the tabulation by listing under local scour only those cases in which scour is <u>known</u> to have been localized at a pier, abutment or embankment, whether or <u>not</u> the localized scour is accompanied by general scour. Local scour may be of much more common occurrence than indicated by the tabulation, but its incidence is not known either because no soundings were made at all, or no soundings were made at the peak of a flood. However, the results do indicate that local scour is not a dominant cause of pier or abutment <u>failure</u>, and this is corroborated by the interviews with state bridge engineers. Most cases of general scour reported herein resulted from constriction of overbank flow by the approach embankments, but some resulted from constriction of flow in the channel by abutments or piers.

Lateral erosion by stream action is a very common problem, and it is the dominant problem in states like Mississippi where the rivers are meandering and have relatively high rates of lateral erosion. Also included here under lateral erosion are the effects of flow expansion and eddy action downstream from a bridge constriction, bank erosion that may occur downstream from a check dam, and the channel widening that commonly follows large-scale channelization. At a bridge, lateral channel migration may cause problems at both piers and abutments, and bar formation may occur in the bridge waterway on the side opposite the eroding bank. Several cases of lateral erosion by wave action are reported, some of which are at reservoirs and some at estuaries.

Although most lateral bank erosion involves small-scale bank slumping, the cases reported under bank slumping are those in which the slumping is of a larger scale, extending more than about 10 ft (3 m) landward from the bankline and directly affecting the bridge substructure or the approach embankment. The term "slumping" is applied here to any mass gravitational movement of bank or embankment material, without regard to rate of movement or viscosity of slumped material. However, most of the cases tabulated involve slumping in the strict sense, in which the slumped mass is solid and a scarp, curved in plan view, is left upslope from the mass of slumped material.

Channel degradation, as defined here, refers to a lowering of the channel bed through a reach of substantial length (more than 10 channel widths). At a bridge waterway the effects of degradation may be like those of general scour, but degradation is not restricted to the immediate area of a bridge, and it is likely to be more difficult to control because it is progressive and is caused by factors beyond the control of the bridge engineer. Degradation is among the more common causes of hydraulic problems at bridges, and its causes are discussed more fully in Chapter 4. Channel aggradation, for which only a few cases were found, is apparently a much less common problem. Upstream degradation in a channel is not necessarily accompanied by downstream aggradation, because, particularly in humid regions, the material removed by degradation is transported to large rivers and thence to the sea. Also, aggradation in a bridge waterway does not reduce the stability of foundations, and does not become a problem until the waterway area or bridge clearance is reduced below the minimum value needed to convey floods.

Accumulation of debris is, according to the tabulation, a less important cause of hydraulic problems than might be anticipated. Among the reasons for this are the following: (1) Accumulations of debris are usually removed by maintenance personnel before the debris causes a serious problem (2) the contribution of debris to bridge failures is probably greatest during very large or catastrophic floods, the effects of which are not fully represented in this study, and (3) the contribution of debris to a bridge failure or to a hydraulic problem is commonly difficult to assess. With regard to accumulation of ice--undoubtedly a serious potential hazard--the finding of only one problem site is attributed to the bridge design countermeasures that are taken where ice is likely to accumulate. Local accumulation of sediment, or bar growth in the bridge waterway, is apparently not a widespread problem.

Each of the remaining hydraulic processes--impact, abrasion, and buoyant forces--is represented at one or a few sites. Abrasion damage to bridge piers by sediment in transport may be a more common problem than is generally recognized.

Extent of Problem

An attempt was made to categorize the extent of problems reported in the case histories, according to the items listed in table 1. In a general way, the items are listed according to increasing severity of the problem, with "land loss" being the least, and "collapse of span" the most, severe. For sites in which a lesser problem, such as subsidence of a pier, led to a greater problem, the site is listed under both categories of problem. Within many of the categories, there are different degrees of severity, but an attempt to distinguish between such degrees would make the table too complex. For example, "erosion of abutment fill" may range from slight to serious. In general, no site is listed under a category unless the problem at the site is judged sufficient to represent a definite hazard to the bridge. For items whose meaning is not apparent, a definition sketch is provided by figs. 2, 3, and 4.

Effect on Bridge Use

Sites are listed under three categories with regard to effect on bridge use, although for many sites we had difficulty in obtaining specific information on this effect. For most of the sites, however, traffic was not significantly affected and the effect of the problem is regarded as potential. Both hydraulic problems and the performance of countermeasures can be, in most cases, more accurately determined at bridges that are not destroyed or seriously damaged.

SURVEY OF HYDRAULIC PROBLEMS

During interviews with state bridge engineers, an effort was made to determine the incidence of damage to bridges or highway approaches by streams and the occurrence of different kinds of hydraulic problems. Two categories of stream-related damage are distinguished, one of which will be characterized here by the term "loss" and the other by "problem". "Loss" refers to cases in which the bridge needs to be wholly or partly replaced. "Problem" refers to a wide gamut of hydraulic problems, ranging from subsidence of a pier or abutment because of scour, or breaching of an approach embankment, to erosion of an abutment fill-slope. A problem either disrupts traffic or represents a potential hazard to the bridge unless it is corrected.

In interpreting the results reported here, the following points should be kept in mind. (1) No systematic record of bridge losses and hydraulic problems, kept separately from the files on individual bridges, was found in any state or agency; therefore, the estimates given depend on the memory and experience of the persons interviewed. Problems or losses that occurred within the past year are more likely to be recalled than those that occurred ten years ago. (2) The estimates given apply only to roads in the state system; county roads are excluded and these, being designed for floods of lesser recurrence interval, are more subject to damage during a given flood than are state bridges. (3) The estimates apply to the period 1960-76, approximately. The damage reported from a particular state depends heavily on the occurrence of major floods during this period. (4) Most of the interviews were held with either design or maintenance engineers (or both) at the state level. Many maintenance problems may be handled at the local level, perhaps without the specific knowledge of engineers at the state level.

Probably the most reliable information on bridge losses and hydraulic problems was obtained in Mississippi, where interviews were held at both the state level and at all six highway districts by a Geological Survey engineer, K. V. Wilson, who has been associated with bridge-site investigations in that state for about 25 years. For Mississippi, the estimate obtained is: loss, 16 of 4,500 bridges; problems, 750 of 4,500 bridges. Many floods occurred in Mississippi during the period 1960-76, many of the streams have degraded and eroded laterally because of channelization projects, and the streams tend to have wide flood plains that are frequently inundated. The incidence of losses and hydraulic problems is probably above the national average, but is exceeded in states where catastrophic floods occurred, as in Pennsylvania.

Estimates of bridge losses and hydraulic problems, in relation to the total number of bridges in the state road system, are given below by state. Also given are responses received to the question, "What are the most troublesome hydraulic problems in your state (or district)?". For a few states, unpublished information is given from a 1969 survey on scour at bridge waterways, carried out for the Highway Research Board and described in Highway Research Board (1970); this information is identified by the phrase "HRB Survey".

Alaska, Fairbanks District only--No bridges lost in 1967 flood, when damage was mainly to embankments. Problems at small percent of bridges, including those on braided streams, where the channel tends to build up on one side and shift to the other side. Crossing sites on alluvial fans are potentially hazardous; general solution is to move crossing as close to mountain front as possible, where channel does not shift as readily. On small streams, there are problems with icing. The U.S. Forest Service reports problems at 8 native log-stringer bridges in southeastern Alaska, all involving lateral erosion of abutment fill.

Arizona--750 bridges: loss, 8; problems 20. Problems with instability of channel alinement, especially on braided streams, leading to erosion of streambanks and abutment fill.

Arkansas--6,400 bridges; loss, 3; problems, 1,000. All districts interviewed. Major problems are undermining of abutment fill-slopes, scour around piers, and channel degradation.

California, District 3 only--500 bridges; loss, 2; problems, 19. For the state as a whole, bridge losses and damage in the catastrophic flood of 1964 are described in California Div. of Highways (1965). A total of 104 county bridges were lost, and 18 state bridges were lost in District 1 alone. In the northwestern part of the state, very high flood elevations were accompanied by vast quantities of debris, some of which was derived from the flooding of lumber mill storage yards. Debris accumulation was an important factor in the destruction of bridges. Near the town of Scotia, US-101 crossed the Eel River on a pair of bridges, one older than the other. The older bridge, a through-truss type that stood upstream, survived the flood; but the newer bridge, which was opened to traffic in 1961, was lost. Although the newer bridge was higher in deck elevation, it was of the deck truss type, and its truss extended 18 ft (5.5 m) below the deck. Because of debris accumulation in the steel truss, the newer bridge was pushed from its piers.

Florida--2,500 bridges; a few small bridges in northern part of state have been lost in major floods. Bridge inspection program, which involves 6 teams of 2 divers each, has found 200-300 cases of pier subsidence since program began, and has probably prevented bridge losses. Few problems with bridges in general because limestone bedrock is near surface at most crossings, and stream velocities tend to be low.

Idaho, District 3 only--Major problem is reduction in channel capacity due to aggradation and vegetal growth downstream from dams.

Illinois--6,000 bridges; loss, 10; problems, 30-40. No particular kind of problem more prevalent than others. Problems with overbank flow from ice jams in northern part of state; additional freeboard sometimes recommended, also additional spans.

Indiana--3,500 bridges; loss, 2. Some bedrock in southeastern part of state is erodible interbedded limestone and shale, and footings may be undermined.

Iowa--4,000 bridges; loss, 1; problems, 15. Does not include effects of 1958 flood, in which 6 state bridges and many county bridges were lost. Most problems are with lateral erosion on channelized streams in western part of state; some with scour due to contraction of sandbed streams. The Illinois Central Railroad had severe problems with channel degradation and lateral erosion on the channelized Boyer River in eastern Iowa, and several millions of dollars were spent in repairing and rebuilding bridges (HRB Survey).

Kansas--Most problems have been on Cimarron River and tributaries in southwestern part of state; these involved lateral erosion and, to a lesser degree, scour. Similar problems occur on the Republican River. There have been no major floods since 1952, and the newer bridge designs and the countermeasures have consequently not been tested. Problems at abutments have been much more common than problems at piers (HRB Survey).

Louisiana--Problems with exposure of foundation piling by channel degradation and scour on lower Pearl River; guidelines needed regarding percent of original penetration that can be lost without endangering bridge. Other problems involve lateral migration of channels and slumping of high approach embankments.

Maryland--800 bridges; loss, 55; problems, 100. (Catastrophic flood in 1972). Problems are erosion at abutments, also undermining of cantilever piers and abutments on spread footings, during extreme floods. Environmental regulations regarding channel changes greatly increase the difficulty of designing and constructing stream crossings.

Massachusetts--HRB Survey: 207 state and town bridges were lost and required replacement during the 1955 hurricanes, and another 35 required extensive repairs. The contribution of scour at piers and abutments to this damage could not be assessed. In the opinion of maintenance officials, the accumulation of debris, and consequent scour, contributed significantly to the losses. Because of flood control works subsequently completed by the Army Corps of Engineers, the extensive losses of 1955 would probably not recur with a storm of similar intensity. Also reported from Massachusetts was scour by tidal currents at a bridge on Cohasset Narrows, which was finally solved by extensive pressure grouting around abutments.

Minnesota--No loss of bridges on primary roads; problems at about 100 bridges on secondary roads, mainly involving approach embankments. Most problems on streams with wide flood plains, especially the Red River. No ice problems on major streams, problems with snow and ice accumulation at waterways of bridges on small streams.

Mississippi--4,500 bridges; loss, 16; problems 750. The U.S. Geological Survey representative in Mississippi (K. V. Wilson) categorized the problems at 39 sites he investigated as follows: shifting of stream channel, 15 sites; scour at abutment, 10 sites; streambank erosion, 5 sites; general scour, 3 sites; channel degradation, 3 sites; scour at piers, 1 site; scour of approach channel, 1 site; and slumping of approach embankment, 1 site.

Missouri--4,000 bridges; loss, 0; problems, 25. Most problems are with lateral stream migration resulting from non-highway channel alterations, in northeastern part of state. On alluvial plain of Mississippi River, problems with aggradation of drainage ditches that have been widened. Few problems in Ozark region, where bedrock is at or near surface.

Montana--1,800 bridges; loss, 20; problems, 25. Most losses occurred in 1964 flood. Damage by lateral stream migration, particularly of braided streams, is a common hydraulic problem.

Nebraska--2,500 bridges; loss, 2; problems, 30. Previous substantial losses and damage occurred during floods in 1944 and 1947. Among the hydraulic problems are lateral migration of stream, and channel instability associated with headcutting. Few bridge losses have resulted from channel degradation, because degradation is considered in the placement of foundations.

Nevada--165 bridges; loss, 0; problems, 1. Potential problems are mainly associated with flash floods and flood debris.

New Mexico--700 bridges; loss, 1; problems, 20. Major problems are channel degradation in upper reaches of streams, and aggradation in lower reaches.

North Carolina--Of problems reported on 6 bridge sites, one is bottom scour at a tidal inlet, four are lateral bank erosion, and one is general scour of bridge waterway.

North Dakota--800 bridges; loss, 2; no estimate of problems, which are mostly handled by district maintenance personnel. More troublesome problems result from bank slumping along the Red River, lateral bank erosion in southwestern part of state, and headcutting of channels.

Ohio--Few problems reported, most now prevented by bridge design. Of seven problem sites reported, four involved lateral erosion at abutment or banks, two involved undermining of pier footings, and one involved the undermining of piers downstream from a reservoir spillway.

Oklahoma--3,000 bridges; loss, 5; problems, 45. In the eastern part of the state, there are few problems because the stream channels are well defined and underlain by bedrock, into which foundations are set. In the western part of the state, the streams are wide and meandering, and stream control is frequently a problem. Piers are set on bedrock or on piles, and the bottom of pile-supported footings are usually placed about 10 feet below the channel. Laterial migration of channels is a much more common source of trouble than is scour at piers.

Oregon--1,350 bridges; loss, 10; problems, 130. Scour at bridge piers is a significant problem and is commonly associated with the accumulation of logs and debris at the bridge. A few problems relate to the effects on flow of an additional crossing upstream from a bridge, or to borrow excavations in the streambed.

Pennsylvania, District 8 only--3,500 bridges; loss, 150; problems, 300. Most of damage occurred during the "Agnes flood" of 1972, but other damage occurred during one flood in 1973 and two floods in 1975.

Excerpts from an unpublished report by a District Eight bridge engineer (M. Hegarty) who inspected many of the damaged bridges in nine counties of south central Pennsylvania, are as follows: Some losses occurred by the impact of water against the bridge superstructure, but most cases of total or partial collapse resulted from scouring and undercutting of foundations. No bridge built after 1940 collapsed as a result of scour. All foundations that failed from scour were set on erodible materials, generally at a depth of four feet or less below the streambed. At some locations, foundation failure from scour is attributed to accumulation of debris. At many locations, poor stream alinement or formation of new channels during flood was an important factor in damage; another important factor was inadequate area of waterway opening. The type of structure most susceptible to collapse from scour was found to be stone-masonry piers and abutments without concrete footings. All factors that contributed to the failure of older bridges by scour are considered in current design criteria, and the existing inspection program should lead to the remedy of any problems before they become serious; therefore, the failure from scour of bridges as presently built and maintained is improbable. A recurrence of a flood equal in magnitude to the 1972 flood would probably cause less than 10 percent as much bridge collapse-damage, because most of the more vulnerable bridges have been replaced or underpinned since 1972.

South Dakota--Approximately 1,000 bridges; loss, 12; problems, 60. Most losses occurred in Black Hills flood of 1972. Most problems are with lateral channel migration, some problems with degradation on tributaries to the Missouri River, such as the James, Vermillion, and Sioux Rivers.

Tennessee--About 6 problem sites reported, mostly on alluvial plain of Mississippi River in western part of state, and most related to channel degradation and lateral erosion resulting from channelization projects. Clearing of flood plains for agricultural purposes has also contributed to channel instability and to increased problems with floating debris. Few problems reported from central and eastern part of state, where bedrock is usually present at shallow depth in stream channels.

Texas--21,000 bridges; loss, 300; problems, 1,000. The most common problems are lateral migration of channels, accumulation of drift at piers, erosion of embankment at abutments, slumping of embankment at high water stages, channel changes downstream from reservoirs, and channel instability resulting from channelization projects. One case of aggradation (Sulphur River at SR-37) was reported.

Utah--600 bridges; loss, 0; problems, 11. Channelization projects are a common cause of hydraulic problems.

Wisconsin--Of 5 problem sites reported, one involved scour at piers resulting from ice and drift; two (I-90 at Wisconsin River and at Turtle Creek) involved local scour at piers; one, inadequate clearance of superstructure; and one, loss of a county bridge from scour at abutment.

Wyoming--1,400 bridges; loss, 1; problems, 12. Most common of the problems is erosion of abutment fill slopes, in connection with channel degradation or contraction scour.

Missouri Pacific Railroad--(operates in 13 states, including, Mo., Ark., La., Texas, Okla., Nebr., Colo.) 200 miles of bridges; loss, 0 bridges; problems, 12 bridges. Scour at piers more common than scour at abutments, but flood plains are crossed mostly by trestle rather than by embankments. Problems include degradation and lateral erosion of channelized streams, and accumulation of drift at timber trestle structures, for which spacing of bents is only 14 feet (4 m). More problems in Louisiana and Oklahoma than in other states.

CHAPTER 3

COUNTERMEASURES

CLASSIFICATION, DEFINITION, AND EVALUATION

A countermeasure is here defined as a measure, either incorporated into the design of a bridge or installed separately at or near the bridge, that serves to prevent or control hydraulic problems. Countermeasures are classified here according to the following major categories, for each of which examples are given:

- Flexible revetment or bed armor
  - Dumped rock riprap, rock-and-wire mattress, gabion, car body, planted vegetation, precast-concrete block, willow mattress
- Rigid revetment or bed armor
  - Concrete pavement, sacked concrete, concrete-grouted riprap, concrete-filled fabric mat, bulkhead
- Flow-control structures
  - Spur, retard, dike, spur dike, check dam, jack field
- Special devices for protection of piers
  - Drift deflector, abrasion armor at pier nose
- Modifications of bridge, approach roadway, or channel
  - Underpinning or jacketing of pier, construction of overflow section on roadway, realinement of approach channel
- Measures incorporated into design of a replacement bridge
  - Increased bridge length, fewer or no piers in channel

Definitions for terms in the above list are provided in the Glossary, and further discussion of terminology is given below. Countermeasure types that were not encountered in the course of this study, and for which no information regarding performance was found, are not included in this report.

The performance of a countermeasure is rated according to the following scheme:

1. Function
   - A. prevented or controlled hydraulic problem
   - B. reduced or partially controlled hydraulic problem
   - C. no apparent effect on hydraulic problem
2. Damage to countermeasure
   - A. sustained no significant damage
   - B. sustained significant damage
3. Unwanted effects of countermeasure
   - A. caused no problem
   - B. caused some problem

For example, the rating of AAA for a countermeasure indicates that it prevented or controlled a hydraulic problem, was not significantly damaged, and caused no problem. In this chapter, ratings are given for countermeasures at study sites where sufficient information was available for evaluation. Because unwanted effects of countermeasures are not common, the third letter of the rating scheme is omitted except for those sites where such effects were discerned. In the case histories, a more detailed explanation of countermeasure performance is given, rather than a rating by letters.

No definite criterion of magnitude or recurrence interval for the flow that adequately tests countermeasure performance could be established. Instead, the recurrence interval of the flood is given, where it could be determined. A flood of frequent recurrence (2-5 yr) is regarded as a dubious test, whereas an event of infrequent recurrence (50 yr) is regarded as a severe test. If a small flood is preceded by a sequence of small floods, its effect on a bridge or countermeasure may be affected by the antecedent events. On many streams, the banks may be eroded rapidly--and bank protection may therefore be tested--at stages below bankfull.

Damage to a countermeasure is regarded as a relatively unimportant aspect of its performance. According to the "principle of expendability" (as paraphrased after California Division of Highways, 1970, p. 15), a countermeasure that serves its purpose most economically is likely to be damaged. Where cost is an important consideration, highway agencies should consider the construction of inexpensive structures (especially spurs and spur dikes) that provide adequate protection but may require maintenance after floods.

A few examples of problems caused, either directly or indirectly, by a countermeasure are given in the case histories. Among these are scour and lateral erosion downstream from a check dam (Site 159); erosion by overfall at riprap placed in the bridge waterway (Site 222); erosion of a streambank opposite a spur (Site 215); and decreased waterway capacity attributed to riprap at a pier (Site 38). Scour holes at spur dikes are common, but no hydraulic problems at bridges have been attributed to these.

REVETMENT AND BED ARMOR

The revetment terminology in tables 2 and 3 is adapted from California Division of Highways (1970, p. 89-92), from Searcy (1967), and from Normann (1975). The purpose of revetment is to provide an erosion-resistant surface for a bank or embankment; and bed armor, which consists of the same kinds of materials, serves the same function for the channel bed. Flexible revetment has the property of conforming to minor changes (brought about by subsidence or erosion) of the underlying surface, usually without being seriously damaged. Rigid revetment does not conform to such changes, and thus may fail because of lack of support.

Except for the term "riprap", the terms applying to revetment are consistent in usage and generally understood. As defined for this report, riprap refers to a layer or facing of broken rock or concrete, dumped or placed to protect a structure or embankment from erosion (fig. 5); it also refers to the broken rock or concrete suitable for such use. According to Searcy (1967, p. 11-2), highway engineers have broadened this usage to include sacked-concrete revetment and concrete slope pavement, thus the

Table 2. Use and performance of flexible revetment and bed armor at study sites.

Type; and part of crossing protected	Study sites where used; and performance rating
Dumped riprap	
abutment	1, 8, 9AA, 10BB, 13AA, 15, 20BB, 30AA, 33AAB, 36, 37AA, 38AA, 42AA, 43, 48, 52CB, 53BB, 68, 74, 101AA, 115, 151AA, 153AAB, 163CB, 167BA, 169, 171, 173CB, 174CB, 176CBB, 180BA, 181, 182, 187, 196, 198BB, 199CB, 217, 226, 235BB, 136BB, 243, 248AB, 250AA, 258, 260AA, 266AA, 273CB.
pier	4BA, 16AA, 17BB, 24AA, 38AAB, 42, 49, 50, 122, 125, 126, 127, 130, 139AA, 162AAB, 177AA, 184AA, 185, 201AA, 205, 207, 210, 211AA, 213, 218, 219AA, 222CBB, 228, 258, 263, 264, 265AA, 267CB, 273, 281, 282.
channel . . . . . (bed or banks)	4BA, 13, 14, 15, 30AA, 33, 36CB, 37, 43AA, 52, 92AA, 99BB, 101AA, 115AA, 116, 123, 131, 153AAB, 157, 159CB, 161, 169, 171, 174, 180, 182, 184, 188, 192, 213AA, 219, 222, 223CB, 244, 248, 260, 281, 282AA.
embankment	8, 9, 14AA, 30, 42AA, 120AA, 150AA, 154AB, 156, 181, 192, 195BA, 201CC, 204, 211, 234, 243.
Rock-and-wire mattress	
abutment	12BB, 54AB, 55AA, 56AB, 58AA, 60BB, 61AA, 103AA, 275, 276AB, 277BA, 278BB, 279.
pier	263, 277BA.
channel . . . . . (bed or banks)	54, 58, 60, 85, 103AA, 276.
embankment	55, 56.
Gabion	
abutment	5, 6, 74, 138, 252, 257.
pier	138
channel	252
embankment	5, 6, 100AA, 267.
Car body	
abutment	1AB, 174CBB, 244.
channel . . . . . (bed or banks)	41, 174, 216AA, 223CB, 224AA, 240CB.
Planted vegetation	
channel	221AA
embankment	30
Precast-concrete blocks	
abutment	21BB, 202BB.

Table 3. Causes of riprap failure at study sites.

Cause of failure	Location of countermeasure; and site number			
	abutment	pier	channel	embankment
Inadequate size	10, 20, 53, 166, 273	4	4, 36, 159	201
Direct impingement of current	176, 180, 235	. . .	. . .	154
Channel degradation . . .	52, 174	222	. . .	. . .
Internal slope failure	167, 173, 199	. . .	. . .	. . .
Steep slope of revetment	198	. . .	159, 223	. . .
Use of broken concrete	198, 199	. . .	223	195
High percentage of fines	. . .	267	. . .	. . .
Use of rounded stones	20, 236	. . .	. . .	. . .
Lack of filter blanket	. . .	. . .	. . .	201
Wave action	. . .	267	. . .	. . .
Overtopping of revetment	. . .	. . .	99	. . .
Underscour in sandbed stream . .	. . .	17	. . .	. . .
Damage by ice	248	. . .	. . .	. . .

Figure 5. Installation of dumped rock riprap, with trench at toe and cutoff trench on slope. (Cache Creek, Calif., Site 1.)

term riprap would apply to all the commonly used kinds of revetment. On the other hand, it was found during the course of this study that engineers in some states do not apply the term riprap to randomly dumped rock, but reserve it for hand-placed rock or concrete pavement. Some precision of usage is gained by restricting the term riprap to revetment consisting of broken rock or broken concrete, and this usage is followed consistently here.

Specifications and guidelines on rock riprap and other commonly used kinds of revetment are given in Normann (1975), Searcy (1967), and California Division of Highways (1970), and are not repeated here.

Dumped Riprap

Although the design information on dumped riprap in Searcy (1967) has been superceded by that in Normann (1975), the size gradation classes remain the same and are used as a standard of reference in the case histories. Most of the state engineers we interviewed have used Searcy (1967) as a reference, but many states have established their own size-gradation classes which do not necessarily conform to the three classes established in Searcy and Normann. For some descriptions of riprap in the case histories, we have quoted which of the three classes the size gradation in question was most nearly equivalent. For convenience of reference, the essential features of the size gradation of each class are given below:

Class I: Weight range, 100 lb (45 kg) to 2 lb (1 kg).
D_{50}, 25 lb (11 kg).
Class II: Weight range, 700 lb (318 kg) to 20 lb (9 kg).
D_{50}, 200 lb (91 kg).
Class III: Weight range, 2000 lb (909 kg) to 40 lb (18 kg).
D_{50}, 700 lb (318 kg).

For many of the case histories in which riprap is a countermeasure, we were able to obtain only rough approximations of the size gradation.

Dumped riprap is the most widely used kind of revetment in the United States. Its effectiveness has been well established where it is of adequate size, of suitable size gradation, and properly installed. In table 2, use of dumped riprap as a countermeasure is documented at a total of 110 sites, and performance was rated at 58 of these. Of the 58 sites evaluated, the riprap served its protective function (A rating) at 34 sites, partially served its function (B rating) at 12 sites, and failed to serve its function (C rating) at 12 sites. But most of the sites selected are problem sites that present difficult conditions for function of a countermeasure; and the performance of riprap at bridges in general is no doubt better than indicated by these ratings. At some of the sites (for example, Site 184), riprap probably saved the bridge from destruction.

Keeley (1971, p. 67) examined many installations of dumped rock riprap (bank revetment) in Oklahoma, ranging in construction date from 1926 to 1969, and found no evidence of significant failure. He concludes that the riprap design in use there provides a basic and efficient bank control on the meandering streams of Oklahoma.

Where riprap is supported by erodible sandy or silty ground, failure may occur when the supporting ground is eroded through the interstices of the riprap. Under such circumstances, the need for a filter blanket beneath the riprap is well established, and a striking example of failure from lack of a filter is given (Site 201). On the other hand, the performance of filters is difficult to assess, because it is difficult to know what would have happened in the absence of the filter, or in the presence of a filter of another kind. Plastic filter cloth, instead of a granular filter, has been used at several bridge sites in Florida and in Louisiana, and an example of the use of filter cloth for scour countermeasures at a pier (in Minnesota) is given in Site 205.

Dumped riprap keyed (or plated) by tamping (fig. 6) is reported from one site (Site 126), where it proved to be effective. Guidelines for placement of keyed riprap have been developed by the Oregon Department of Transportation and distributed by the Federal Highway Administration, Region 15. In the keying of a riprap, a 4,000-lb (1,818 kg) or larger piece of steel plate is used to compact the rock into a tight mass and to smooth the revetment surface. Keyed riprap is more stable than loose riprap revetment because of reduced drag on individual stones, its angle of repose is higher, and its cost is less because a lesser volume of rock per unit area is required.

On the method of placement of riprap at piers, some relevant information was obtained in this study and additional information has been published by Nece (1974), in a study of the effectiveness of riprap practice at bridge piers in Washington state. In the present study, evidence was found at two sites (Sites 16 and 17) to show that riprap placed level with the streambed is effective. From his study of six sites in Washington state, Nece concluded that the Washington state method placement, by which riprap is placed from the top of the pier footing to the streambed, has worked

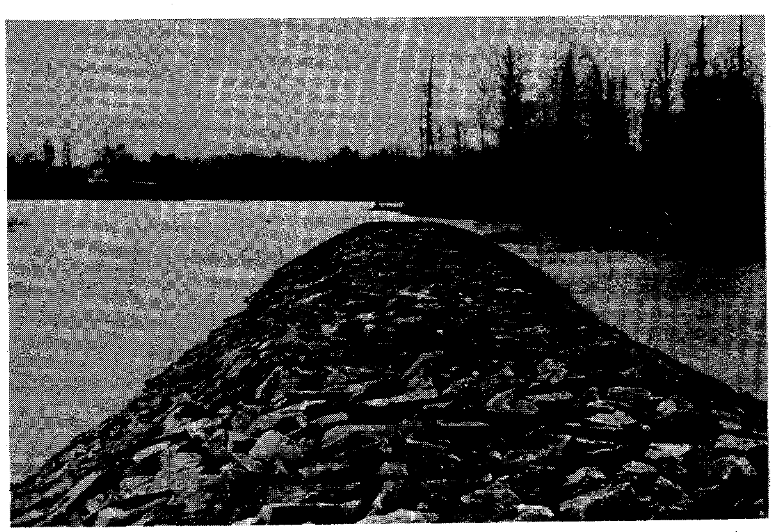

Figure 6. Dike revetted with dumped rock riprap, keyed in place by tamping. (South Santiam River, Oreg., Site 126; photograph from Oregon Dept. of Highways.)

generally well. However, most of the six sites had not been tested by severe floods. At two of the six sites (Carbon River and Baker River bridges) scour problems have occurred or seem potentially troublesome. At the Carbon River site, channel shift leading to bad pier orientation was a factor. At the Baker River site, progressive channel degradation is occurring because of the effects of an upstream reservoir. Both Nece (1974, p. 74-79) and Klingeman (1971) recommend placement of the riprap so that the top is below, rather than level with, the streambed. Exposed riprap may create a flow disturbance that will undermine the edges of the riprap blanket, or cause scour elsewhere in the bridge waterway.

Because of the general effectiveness of dumped riprap, a more detailed analysis of the relatively small number of cases in which it failed, to some degree, is appropriate here. Assigned causes for failure at specified sites, and at parts of the crossing it was intended to protect, are given in table 3. In this table, "pier" refers to piers and pile bents on channel banks and flood plain, as well as in the channel. "Embankment" refers not only to the approach roadway embankment, but also to dikes and spur dikes.

Inadequate size was assigned as a cause of unsatisfactory riprap performance for sites where the size seemed clearly too small. As with other countermeasures, the size of riprap used depends on considerations of cost, availability, and degree of protection required. A detailed discussion of the theoretical and practical factors in determination of riprap size (and other design considerations) is given in Simons and Senturk (1977, p. 399-447).

"Direct impingement of current" refers to situations in which the current strikes the revetment at a sharp angle, rather than flowing parallel with it. In a summary of design factors for riprap, California Division of Highways (1970, p. 111) recommends heavier stone, thicker section, flatter slope, and deeper toe for riprap subject to direct impingement. This guideline is supported by the findings herein.

Channel degradation, which accounted for riprap failure at three sites, is probably a common cause of failure. In spite of the flexibility of riprap, and even where the toe is well protected, channel degradation is likely to cause failure. Linder (1976, p. 2-168 to 2-179) has shown that the effects of degradation can be offset by providing a volume of reserve riprap in the revetment toe. Most other countermeasures are similarly susceptible to failure by channel degradation, except for some types of spurs or retards that are supported by deeply driven piles.

Internal slope failure, which riprap failed to halt at three sites, is also difficult to control with other countermeasures. Riprap would not be expected to be effective against it unless the slope angle was reduced to a stable value. Before a countermeasure is designed for a bank or embankment, the site should be examined and a search made for evidence of internal slope failure, particularly fissures or small scarps at a distance of 10 ft (3 m) or more from the edge of the embankment. High approach embankments, built to keep the approach roadway above the level of extreme floods, are susceptible to internal failure.

Most criteria for riprap design specify that the slope be no steeper than 1.5:1. This criterion is widely followed for abutment fill-slopes, but it is sometimes disregarded for streambanks. Use of riprap on steep ungraded streambanks, even in emergency situations, is probably not worth the cost.

Broken concrete is used for riprap in many states where rock riprap is unavailable or unusually expensive, but we found no specifications for the use. Broken concrete riprap proved to be more or less unsatisfactory at three sites.

Each of the other causes for riprap failure is represented by only one site, and no generalizations about the causes are warranted from the case histories. Riprap that contains a high percentage of fine material, even if it also contains large stones, is liable to failure because of the washing out of the fines (fig. 7). Rounded stones, either from streambeds or from glacial drift, are widely used for riprap in regions where they are readily available (North Dakota, Saskatchewan, Montana). Although these are less desirable than angular stone, they are nevertheless effective (see, for example, Sites 250 and 260). Rounded stones less than 5 in (127 mm) in diameter have a significantly lower angle of repose than angular stones, but the angle of repose is not significantly different from larger diameters (see Simons and Senturk, 1977, fig. 7.16, p. 442).

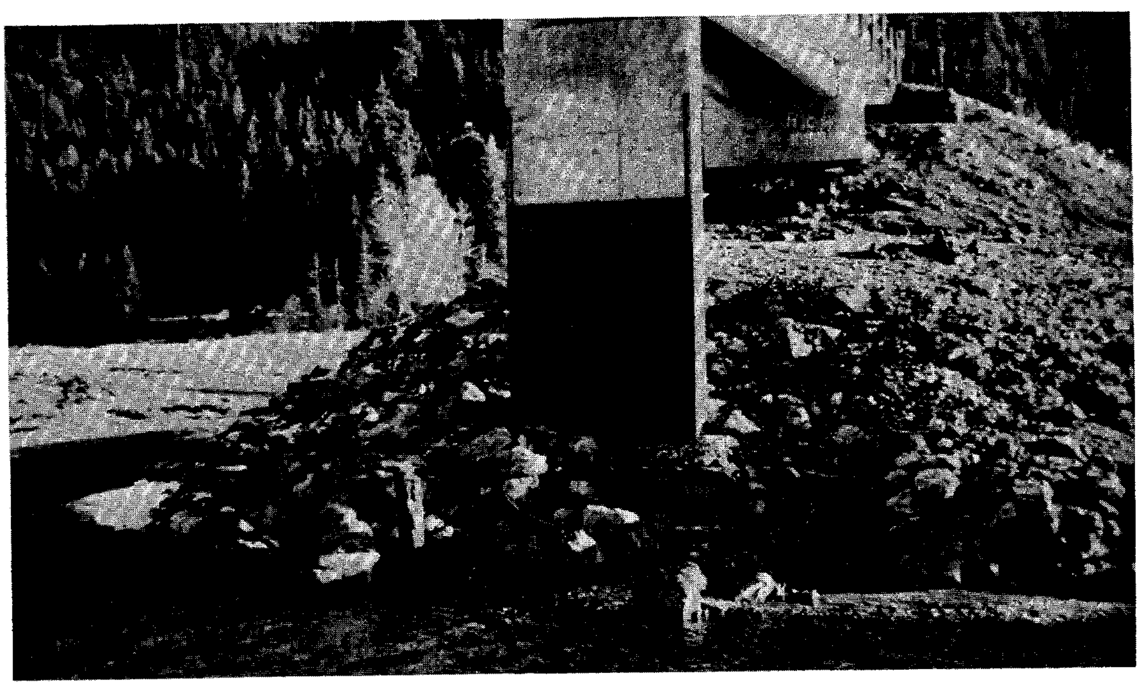

Figure 7. Erosion of dumped rock riprap having a high percentage of fine-grained material. (North Fork Skokomish River, Wash., Site 267.)

Rock-and-Wire Mattress and Gabion

A distinction is made here between rock-and-wire mattress, which is 1 ft (30.5 cm) or less in thickness, and a gabion, which is thicker and more nearly equidimensional. Both types are called by Normann (1975) "wire-enclosed riprap", which consists of mats or baskets fabricated from wire mesh, filled with stone, connected together, and anchored to the slope. Details of construction differ with degree of exposure and purpose, whether for revetment or for toe protection at other types of revetment. For wire mesh, galvanized woven fencing is specified in Normann, and experience in Wyoming has shown that welded-wire mesh tends to break at the welds.

As indicated in table 2, rock-and-wire mattress has performed generally well at the case history sites, although local failure of the wire mesh, and spilling out of the riprap, is not uncommon. Good performance is documented by several case histories from New Mexico and Arizona (Sites 54,55,56,58,60, and 61), where the mattress is held in place against the bank by steel rails driven into the streambed, and called "railbank protection". Rock-and-wire mattress, held in place by stakes, is also considered to be effective in Wyoming. In California, use of rock-and-wire mattress has diminished because of high cost of labor, questionable service life of the wire mesh, and the efficiency of modern methods of excavation for dumped riprap toe protection. Good performance in southern California of the mattress had previously been reported by the Los Angeles Flood Control District (California Division of Highways, 1970, p. 317), and wire mesh in service from 13 to 15 yr showed no surface evidence of corrosion. On the other hand, abrasion damage to wire is reported in Montana and Maryland.

The economic use and good performance of rock-and-wire mattress is evidently favored by the following factors:

(1) Arid climate. Corrosion in arid regions is slow, and many of the streams are ephemeral, so that the wire mesh is not subject to continuous abrasion by sediment. In addition, many ephemeral channels in the southwestern United States have unstable vertical banks at which the staked rock-and-wire stays in place better than dumped riprap (fig. 8).

(2) Availability of stones of cobble size. In localities where such stones are readily available, but large riprap is not, the use of rock-and-wire mattress may be advantageous in spite of a probability of eventual corrosion or abrasion of the wire.

The use of gabions, or rock-filled wire baskets, is reported at several sites, but at only one of these--where it performed satisfactorily--was it adequately tested. Gabions have a long history of use, but they have not been widely used by bridge engineers in the United States.

Other Flexible Revetment or Bed Armor

Hand-placed riprap is evidently not in current use, and it has a low degree of flexibility. Its poor performance record as cited in Searcy

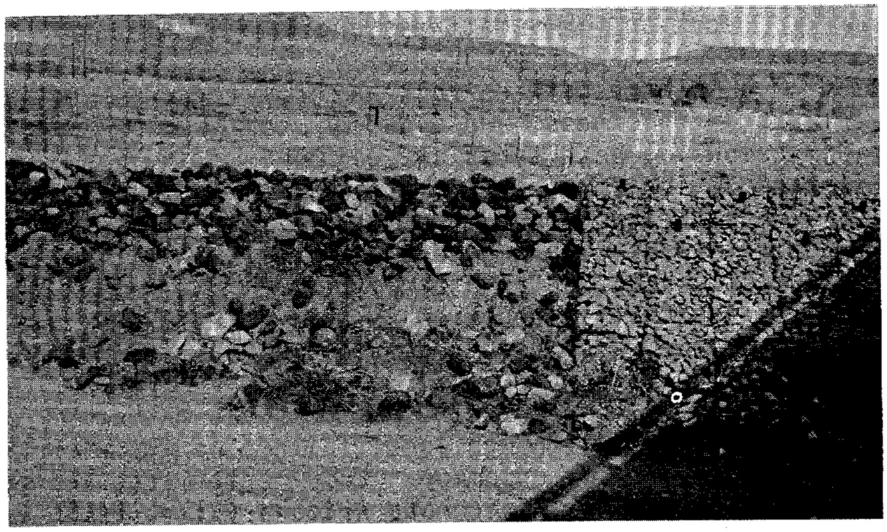

Figure 8. Dumped rock riprap, left, failed because of inadequate toe protection. Rock-and-wire mattress, right, remained intact despite inadequate toe protection. (Whalen Draw east of Guernsey, Wyo.; photograph from Wyoming Dept. of Transportation.)

(1967, p. 11-10) is supported by the findings of this study. Car-body revetment is rarely used by highway agencies, but it is capable of satisfactory performance where properly emplaced (Sites 216, 224).

Planting of vegetation is evidently seldom relied on as a countermeasure by bridge engineers. Vegetation near a bridge may reduce channel capacity, and its effects in erosion control are not very immediate or positive. In Iowa, however, sod is relied on for protection of spur dikes. Failure of sod as bank protection is reported from Arkansas. Although planting of vegetation seems to be seldom practiced, the establishment of natural vegetation at, or around, other countermeasures used for bank protection is known to be important and, in many cases, critical. Planting of stakes or poles of willow and poplar has been widely used for bank protection in New Zealand, and several methods of planting stakes are described in Atcheson (1968, p. 72-83). Such planting is described as "the simplest and sometimes the cheapest way of establishing protection, but damage and mortality are often high and considerable patience and judgment are required". Stake planting is usually more effective on small to moderate sized rivers, whose load includes fine sediment. Severely eroding bends at critical points are protected by other means.

Examples of the use of precast-concrete blocks, both articulated and not articulated, are given in California Division of Highways (1970, p. 90, 93,145), but few examples were found in this study. Articulated concrete blocks cemented to plastic filter cloth are available in a patented design ("Gabimat"). These have been mostly used for shore protection, but favorable indication of their performance at bridges is reported from Louisiana, where riprap may be locally unavailable. Vegetation grows through interstices between the blocks and contributes to appearance and stability.

Concrete Pavement

Information on the design and construction of concrete pavement as a countermeasure is given in Normann (1975, p. 118-212) and California Division of Highways (1970, p. 126-132). Thickness may range from 3 to 6 in (7.5 to 15 cm) and the slabs may be plain or reinforced. In Illinois, where concrete pavement is in general use at bridge abutments, a typical design specifies a 6-in thickness, reinforced with welded-wire mesh, placed on a 2:1 slope; an anchor wall is placed at the center, and a concrete wall extending to a depth of 3 ft (1 m) is placed at the toe. The paving is carried a distance of about 10 ft (3 m) behind the abutment. Any failure of concrete slope pavement tends to be progressive, and special precautions (as compared with dumped riprap) must therefore be taken to prevent undermining at the toe or ends, or failure because of an unstable foundation. California Division of Highways favors concrete slope pavement only for situations where flow is controlled or maximum flow is limited.

As indicated in table 4, concrete paving has failed, to some degree, at a rather high percentage of the sites where it was used. Causes of failure are categorized as follows:

Undermining of toe: Sites 48, 91, 165, 166, 172, 183
Erosion at end of paving: Sites 23, 28, 45, 152, 172
Eddy action downstream from bridge constriction: Sites 28, 45
Channel degradation: Sites 45, 55
High water velocities: Sites 26, 45
Overtopping: Sites 27, 28
Hydrostatic pressure: Site 134

Although a substantial number of failures are documented here, concrete slope pavement is widely used in Illinois, Florida, and Texas, and the number of failures reported from those states is small. In Alberta, concrete slope-paving is preferred as revetment because of its effectiveness and neat appearance, although it is well protected at the toe by a heavy riprap launching apron or by a concrete cutoff-wall extending to bedrock. The following guideline seems warranted: Well-designed concrete paving is satisfactory as revetment on streams having low gradients, or in other circumstances where it is thoroughly protected from undermining at the toe and ends. It should seldom be used as streambank protection, except in situations where the whole channel is lined. Weep holes for relief of hydrostatic pressure are required for many situations, although only one failure from hydrostatic pressure is documented here.

Sacked Concrete

Use of sacked-concrete revetment (fig. 9) is documented in 20 case history sites but no highway agency reported a general use of sacked concrete as revetment. In California, sacked concrete is regarded as expensive revetment, and is almost never used unless stream gravel (for aggregate) is available at a location and satisfactory riprap is not (California Division of Highways, 1970, p. 133). For urban areas, sacked concrete has a more natural appearance than does concrete slope pavement, and this is enhanced

Table 4. Use and performance of rigid revetment and bed armor at study sites.

Type; and part of crossing protected	Study sites where used; and performance rating
Concrete pavement	
abutment	23, 25AA, 26CB, 27AA CB, 28BB, 39CB, 45BB, 47AA, 48CB, 75, 132AA, 143AB, 151AA, 165CB, 166CB, 170, 172BB, 178AA, 183CB, 184, 206AB, 212AA.
pier	55BB, 141.
channel	23BB, 27, 91BB, 134AB, 141AA.
embankment	19AAB, 22, 144AB, 150AA, 152AB, 184.
Sacked concrete	
abutment	1AB, 20, 26AA, 28BB, 29AA, 41AA, 45BB, 187BB, 193, 217, 261.
embankment	1, 18.
Concrete-grouted riprap	
abutment	7AA, 75, 140.
pier	124AA.
channel	13AA, 140.
embankment	7AA, 95AA.
Concrete-filled fabric mat	
abutment	183.
pier	44BB.
Bulkhead	
abutment	17AA, 63, 131BB, 145, 173AB, 194BA, 199AA, 200BA, 224BB, 243AA, 244AA, 245AA, 247AA, 260.
pier	145.
channel	55BB, 131, 252CB.
embankment	194, 217AA.

by the establishment of vegetation in interstices between the bags. We have insufficient evidence for judging the relative effectiveness of sacked concrete and concrete pavement, but they are subject to failures from the same causes. Failures of sacked-concrete revetment at the case history sites are as follows:

Undermining of toe: Sites 1, 45.
Erosion at end: Site 28.
Channel degradation: Sites 1, 45.
Wave action: Site 187.

Concrete-Grouted Riprap

No failures of concrete-grouted riprap (fig. 10) are documented in the case histories, but it is subject to the same hazards as other types of rigid revetment. As compared with dumped riprap, it allows the use of smaller stone and a lesser thickness of revetment. Several cases were reported from Georgia (but not documented as case histories) in which scour at piers was successfully controlled by paving the streambed with concrete grouted riprap. Early installations of concrete-grouted riprap in California (California Division of Highways, 1970, p. 243) failed because of improper penetration of grout, but this difficulty has been corrected.

Figure 9. Sacked-concrete revetment, slightly undercut at upstream (right) end. (Stevens Creek, Calif., at SR-85.)

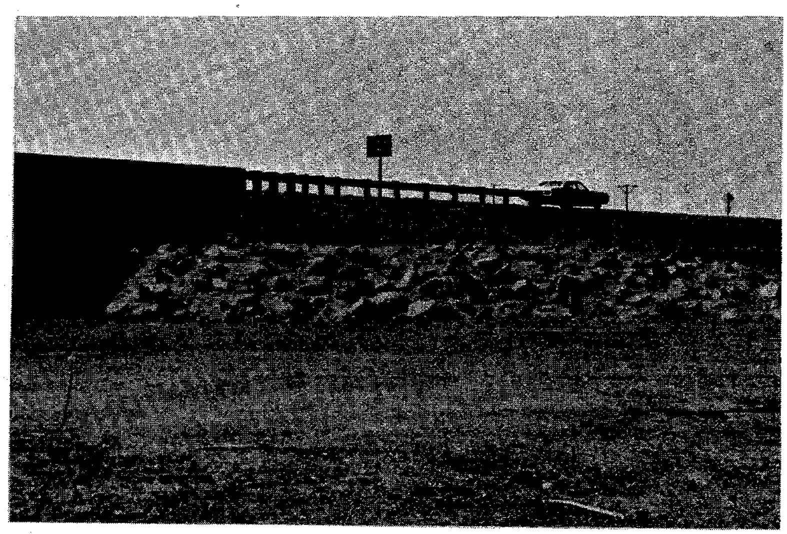

Figure 10. Use of concrete-grouted riprap for protection of abutment. (Stony Creek, Calif.; Site 75.)

Concrete-filled Fabric Mat

Concrete-filled fabric mat (fig. 11) is a patented product ("Fabriform") consisting of porous, pre-assembled nylon fabric forms or envelopes which are placed on the surface to be protected and then filled with high-strength mortar by injection. Variations ("Fabricast" and "Enkamat" consist of nylon bags that are similarly filled. An application of this countermeasure, not yet tested by flood, is provided by Site 183. In a case history provided by the manufacturer, but not included among the case histories of Vols. II and III, "Fabriform" revetment was applied as bank protection along the East Fork of 102 Mile River, at Bedford, Iowa. Shortly after completion, the small but high-banked stream flowed at near bankfull stage and water velocities reached 20 ft/s (6 m/s). The revetment performed satisfactorily except at one isolated locality, where shale bedrock on the creek bottom was scoured from beneath the mat.

Other applications of concrete-filled fabric mat at bridges have been reported from South Dakota (Karim, 1975). Two bridges on SR-79, south of Rapid City, S. Dak., one over Spring Creek, and the other over Battle Creek, were damaged during the catastropic flood of 1972. The abutment fill-slopes were eroded, and hydraulic analysis showed that the waterway openings of both bridges were inadequate for a flood of 50-yr recurrence interval. A countermeasure was needed to protect the abutments and provide maximum flow capacity. Concrete-filled fabric mat was chosen in preference to riprap or rock-and-wire mattress, partly because of cost advantage. In 1974, concrete-filled fabric mats were installed around the abutment fill slopes (2:1 slope) and on the bottom of the channel. In order to prevent undermining, both upstream and downstream ends of the mat were trenched in about 4 ft (1.2 m) below the channel bed. By 1978, the countermeasure had been tested by two floods of approximately 10-yr R. I., and its performance is judged to be

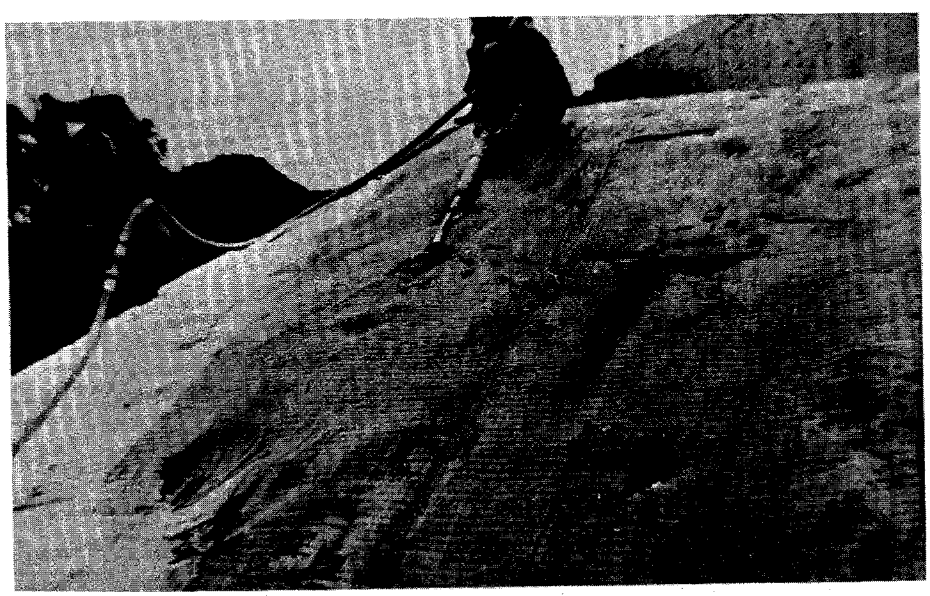

Figure 11. Installation of concrete-filled fabric mat. (Spring Creek, S. Dak.; photograph from South Dakota Dept. of Transportation.)

good (M. Karim, oral communication, 1978). Some details of the original installation were not completely satisfactory, because of the contractor's inexperience with the specialized techniques of construction.

Concrete-filled fabric bags were installed in 1974 as a countermeasure for scour at piers on the Susquehanna River in Pennsylvania (Site 44). During a subsequent flood, some of the bags were displaced and previously undermined areas were partially re-exposed. Heavier bags, extending farther from the pier footing, are apparently needed.

Bulkheads

A bulkhead is defined as a steep or vertical wall against a natural or artificial slope, for the purpose of supporting the slope or protecting it from erosion. Bulkheads are constructed of concrete, timber pile, sheet piling (fig. 12), or cribbing. A bulkhead usually projects above the ground; whereas a cutoff wall, which is similar in construction and function, is usually below ground. Some of the structures described herein as bulkheads are mostly below the ground surface and are therefore transitional to cutoff walls. Also, the distinction between a retard, as defined here, and a bulkhead is not sharp in some cases, although a retard is not intended as a supporting structure. By far the most common bulkhead at stream crossings is the wingwall (or endwall) at bridges or culverts, but wingwalls were not specifically investigated in this study.

As indicated in table 4, most bulkheads reported at study sites are at abutments. They are most useful at abutments in the following situations: (1) on braided sandbed streams having erodible sandy banks, as in Sites 17, 63, 243, 244, and 247, (2) where banks or abutment fill-slopes have failed by slumping, as in Sites 173, 199, and 200; other countermeasures are not likely to be effective against slumping, unless the embankment is graded

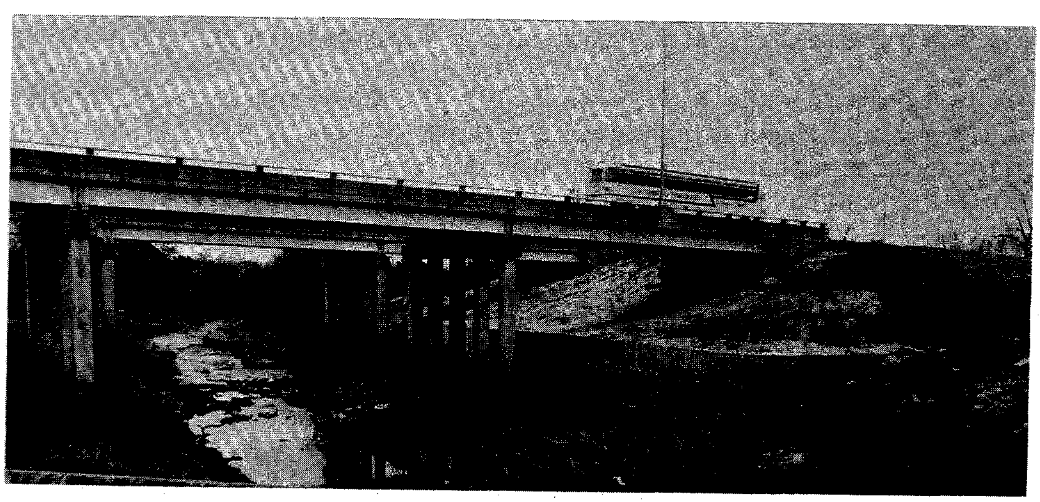

Figure 12. Use of steel sheet-pile bulkhead as a countermeasure for slumping of abutment fill-slope. (North Branch Ward Creek, La., Site 199.)

to a low angle, and, (3) in cases where the stream alinement is poor, to provide a transition between the streambanks and the bridge opening, as in Sites 224 and 245.

At Sites 194 and 217, bulkheads have satisfactorily protected the roadway embankment in situations where the stream flows directly alongside the embankment. Few examples of the use of bulkheads as channel bank protection were found; at one of these (Site 55) steel sheet-pile bulkheads proved to be less effective than rock-and-wire mattress.

At the five sites (55,194,200,224, and 252) where bulkheads failed, to a greater or lesser extent, the causes of failure were not entirely clear, but were attributed to undermining. In California Division of Highways (1970, p. 271-272), failure of bulkheads is attributed to loss of foundation support, erosion of natural banks adjoining the structure, and loss of fill material.

FLOW-CONTROL STRUCTURES

A flow-control structure is defined here as a structure, either within or outside a channel that acts as a countermeasure by controlling the direction, velocity, or depth of flowing water. Structures within this category are sometimes called "river training works" (see, for example, Neill, 1973, p. 114), but it is doubtful whether protection of a bank from erosion--a common function of a flow-control structure--should be regarded as river training, particularly if the erosion is induced by a bridge or other artificial structure. Among the most important properties of a flow-control structure is its degree of permeability. An impermeable structure may deflect a current entirely, whereas a permeable structure may serve mainly to reduce water velocity. As used here, the term "permeable" means that a structure has definite openings through which water is intended to pass, such as openings between adjacent boards or pilings, or the meshes of wire. Structures made of riprap, or filled with riprap, have some degree of permeability, but these are classed as impermeable because they act essentially as impermeable barriers to a rapidly moving current of water.

Usage of terms for the different types of flow-control structures is not consistent from one highway agency to another, although nearly all agencies in the United States apparently agree on use of the term "spur dike". Types of flow-control structures are distinguished herein mainly on the basis of their position relative to channel banks, crossing, and flood plain, as shown in fig. 13. We have tried to maintain consistent internal usage according to this scheme, and, in addition, to give locally used terms where they seemed relevant.

Spurs

A spur is defined as a linear structure, permeable or impermeable, projecting into a channel from the bank for the purpose of altering flow direction, inducing deposition, or reducing flow velocity along the bank. Alternative terms used by various highway agencies are: jetty, groin, dike,

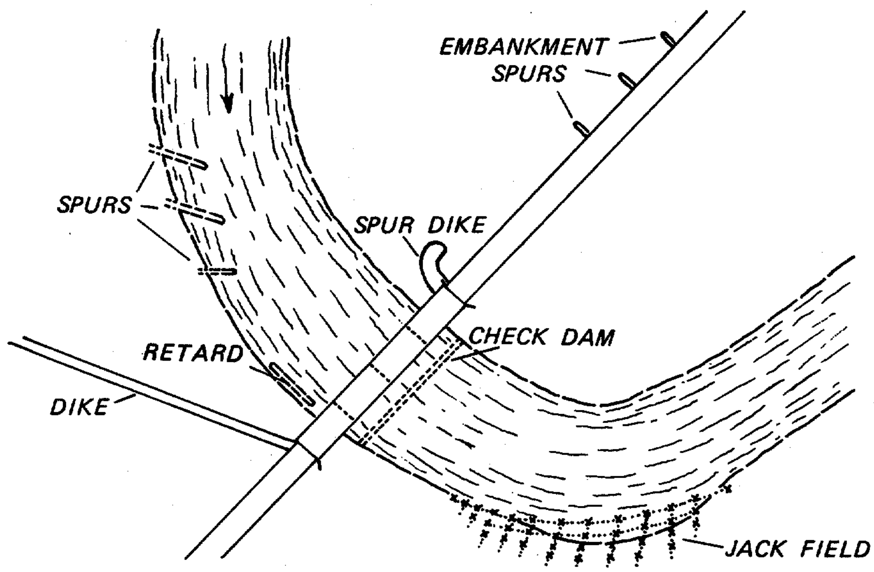

Figure 13. Placement of flow-control structures relative to channel banks, crossing, and flood plain. Spurs, retards, dikes, and jack fields may be either upstream or downstream from the bridge.

deflector, and wing dam. The term "spur" was chosen here because no other meaning has been attached to it (as a countermeasure) and because it conforms with usage by the Roads and Transportation Association of Canada. Linear structures projecting from an embankment, to decrease flow along the side and thus protect it from lateral erosion, are called embankment spurs. Such spurs may project into the flood plain rather than into a channel and thus function as spurs only during overbank flow.

A useful summary of information on the application, and design, of spurs, including a survey of available literature, has been prepared by Richardson and Simons (1974). According to these authors, spurs, by deflecting the current from the bank and causing deposition, may protect the streambank more effectively and at less cost than riprap revetment. Among other uses they give for spurs are the constriction of long reaches of a wide braided river to establish a stable channel, constriction of short stream reaches, control of flow at a bend or through a crossing, and to direct flow through a bridge opening. They find that spurs have been used mostly on sandbed streams with relatively large sediment concentrations, with lesser use on gravel bed streams and on streams characterized by icing. In general, the use of spurs is to establish a pre-determined flow alinement. Factors that enter into the design and performance of spurs are discussed below, and the performance of different structural types is given in table 5.

Table 5. Use and performance of spurs at study sites.

Structural type	Study sites where used; and performance rating
Impermeable, earth or rock embankment	36, 43AA, 46, 92AA, 116, 125, 151AB, 181CB, 243AA, 253AB, 257AB, 258AB, 279.
Impermeable, sheet pile	175AA, 186.
Impermeable, timber pile	226CB.
Permeable, timber pile	130BA, 174BB, 185, 209AA, 210AA, 215AAB, 216, 220AA, 225AA, 239BB, 244AA, 261.
Permeable, steel pile, pipe, or rail	131, 178AA, 179AA, 186, 207AA, 209AA, 241BA, 242BB, 283AB.

Structural types of spurs--Different structural types are named here according to the dominant material used in construction, or to the composition of the main supporting members. The types we found in use (table 5) are: earth or rock embankment (figs. 14 and 16), timber pile, sheet pile (fig. 15), and steel pile (fig. 17). Most of the rock embankment spurs are constructed of heavy riprap, and the earth embankment spurs are revetted with riprap.

Timber-pile structures in a wide range of designs are described in the case histories. Variations include number of rows of pile, use of 3-pile clusters, type of bracing, type of sheathing, and use of riprap with the pile. Among the more successful timber-pile structures are those used in Mississippi, which typically have timber sheathing and willow poles on the upstream face. The main disadvantages of timber-pile structures are expense, susceptibility to damage by fire and vandalism, and susceptibility to damage by drift and ice. According to California Division of Highways (1970, p. 153), use of preservative-treated, rather than untreated, pile and timber approximately doubles the cost. For streams where deposition at the spurs (and consequent establishment of vegetation) is probable, the use of expendable structures of untreated wood should be considered.

From his detailed observations of timber-pile spurs (diversion structures) in Oklahoma, Keeley (1971) concluded that the use of wire fencing on the lower part of the structure has merit because it serves to prevent underscour of the face planking. The face planking should be extended to or below river bed elevation; otherwise, the stream will flow beneath the planking at low stage and no sediment will accumulate.

Sheet pile spurs, called "wing dams" (fig. 15), are described at two sites, (175 and 186), both in South Dakota. These consist of two walls of sheet pile, filled with earth. They are admittedly expensive, but have been successful where risk of damage from ice and floating debris is high. One application has been to place a single sheet-pile spur upstream from steel H-pile spurs, for the purpose of deflecting drift and ice.

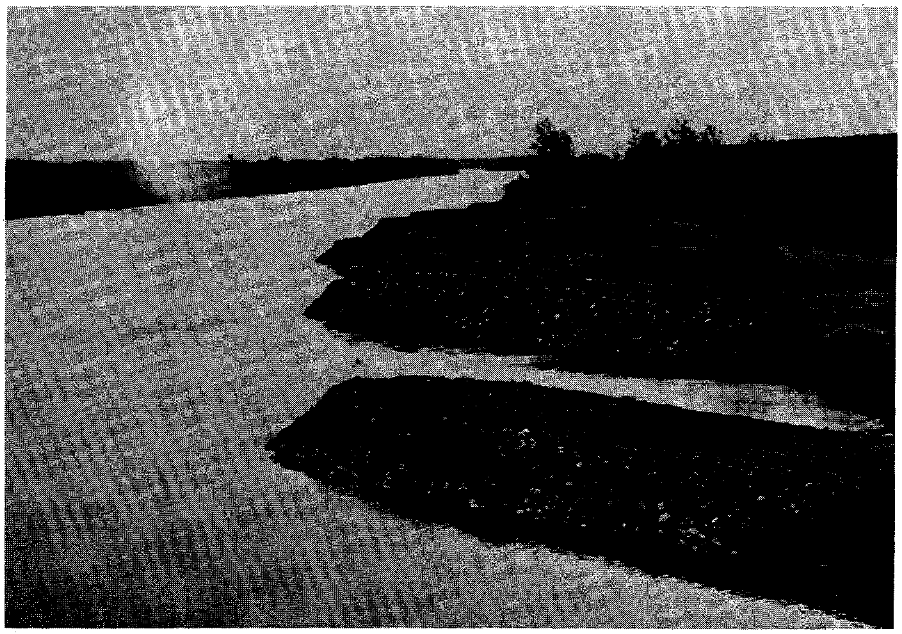

Figure 14. Revetted earth embankment spurs, inclined upstream. (North Saskatchewan River, Site 258; photographs from Saskatchewan Dept. of Highways and Transportation.)

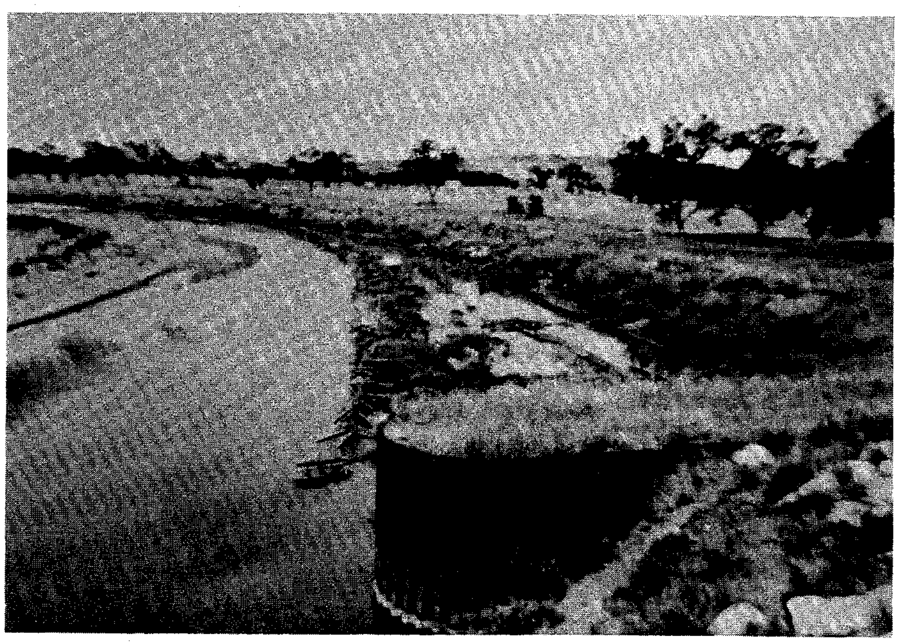

Figure 15. Steel sheet-pile spur (wing dam) upstream from bridge. Kellner jack field along bankline beyond spur. (Grand River, S. Dak., Site 175; photograph from South Dakota Dept. of Transportation.)

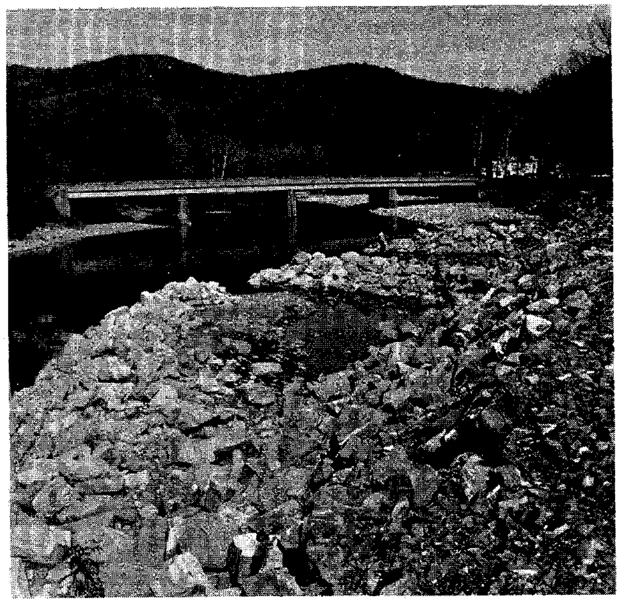

Figure 16. Rock spurs, inclined downstream. (Lycoming Creek, Pa.; Site 43.)

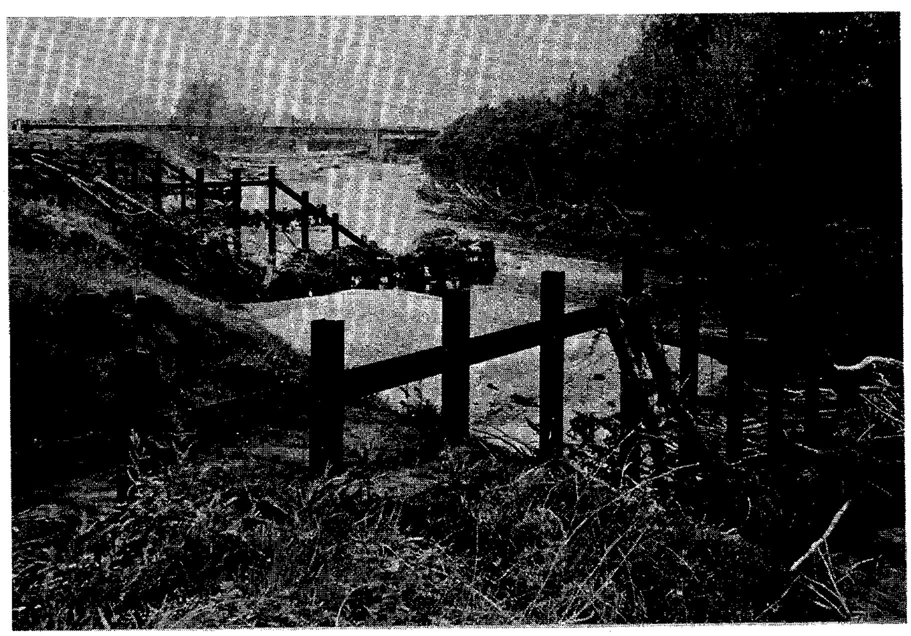

Figure 17. Steel H-pile spurs, with attached tree trunks. (Grand River, Mo., at SR-169 crossing, photograph by Mark Looschen, 1962.)

The structural type described as steel pile, pipe, or rail includes a wide variety of designs. Among these are fence-like structures which differ from an ordinary wire fence in that the posts must be driven to sufficient depth to prevent loss by scour. The posts may be in single, double, or triple row; the facing is usually of wire mesh, but may be of timber; and double or triple fences may be filled with material such as rock or brush. Permeability can be varied according to the mesh of the wire and the type of filling. In California, fence-like spurs, usually impermeable, have been used for deflection and training of shallow streams. However, the term "fence" seems inappropriate for the spurs of steel H-pile, faced with heavy wire mesh, that have been successfully used for control of bank erosion in Iowa (Sites 178 and 179), Missouri, and Nebraska. Heavy welded-wire mesh (surplus from concrete highway construction) is well regarded in Iowa, but not considered an acceptable substitute for woven wire in California (California Division of Highways, 1970, p. 223). Steel spurs, as opposed to timber pile, have the advantage of greater resistance to damage by fire, vandalism, ice or floating debris; and at one site in Mississippi (Site 207) steel piles were used instead of timber because a stratum of clay prevented sufficient penetration of timber pile. Spurs consisting of lines of steel jacks or tetrahedrons have been used (see, for example, California Division of Highways, 1970, p. 247), but according to our findings, highway agencies nearly always deploy jacks in fields, which are discussed separately under "jack fields".

Except for the lighter fence-like structures, most of the spurs we encountered were built as strongly and rigidly as possible to resist impacting forces and to require minimum maintenance. Such structures are expensive. Also, they lack flexibility, in that the bottom of the sheathing (or wire) is fixed in position and cannot shift downward to accommodate erosion of the channel bed. A more flexible and expendable spur has been designed and patented by K. W. Henson, and two applications of the Henson spur jetty are described in Sites 131 and 283. In brief, the spur is constructed of permeable wood-and-steel panels, about 25 ft (7 m) in length, that slide vertically on steel-pipe pile (fig. 18). These spur jetties have been in use since 1956; most applications have been at pipeline crossings, but one planned highway application is described in Site 131. Reports on performance are given in Henson (1967, 1971); Mistrot (1966); and O'Donnell (1973). Maintenance of the spur jetty during the first few years after installation--during the time that sediment is accumulating and vegetation is becoming established--is likely to be needed.

<u>Permeability of spurs</u>--Structural types such as embankments and sheet-pile spurs are inherently impermeable, but the other types are either permeable or impermeable, depending on how they are constructed. Only limited success with impermeable spurs is reported by California Division of Highways (1970, p. 234), because of eddy action and bank erosion between the spurs. This can be avoided, at greater expense, by closer spacing or by bank protection between spurs. For river control work in New Zealand, the use of impermeable spurs (groins) has diminished (Acheson, 1968, p. 96). Among the reasons for this are the severe disturbance to flood flow induced by the impermeable spurs, the uncertainty of effects, and the high cost. Permeable spurs (groins) have been most effective on alluvial rivers with substantial bed load and high sediment concentration. Where spurs are used for the purpose

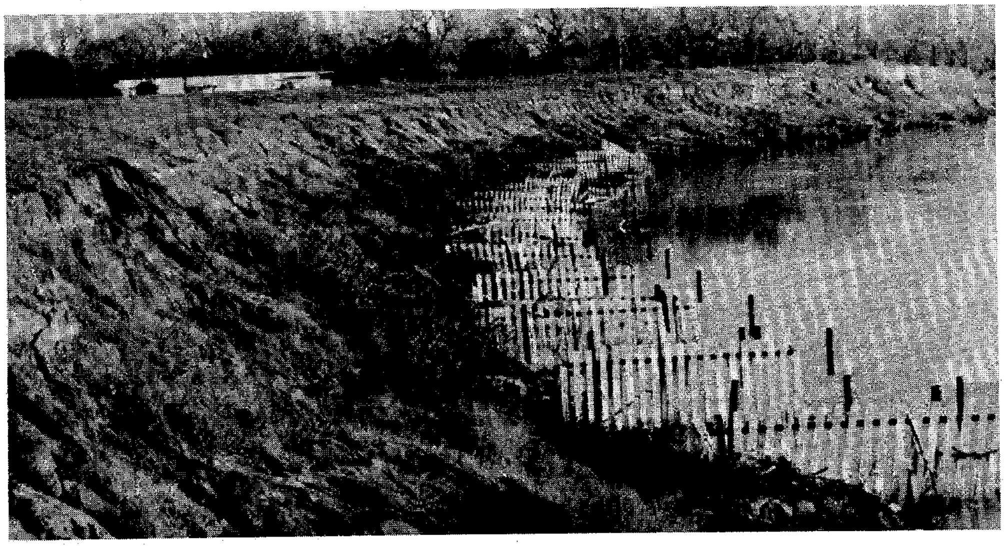

Figure 18. Henson patented "Spur Jetties". (Brazos River, Tex.; Site 283.)

of constricting a wide braided river, as on the South Island of New Zealand and in parts of Canada (Site 151), permeability may have little if any advantage.

Permeable spurs have the inherent potential of reducing water velocity with a minimum of eddy action. Atcheson (1968, p. 97) recommends that the spurs should be sufficiently permeable that they will not get blocked up easily with floating debris. Judging from the study sites, most of the spurs in use in the United States are permeable.

Shape of spurs in plan view--Richardson and Simons (1974) describe spurs according to shape as straight, T-head, L-head, and hockey, but recommend the straight spur as most cost-effective. The T-head and L-head shapes are apparently used mainly for large-scale river training projects; and the hockey form, which has no special advantages, may induce greater scour of the streambed. Only straight spurs were encountered in this study, except at Sites 209 and 210, where a downstream-curving spur was used.

Angle of spur to bank--As shown in fig. 19, spurs may be perpendicular to the bank inclined downstream, or inclined upstream. The angle of inclination is measured counterclockwise from the downstream bank. In a review of the literature, Richardson and Simons (1974) found descriptions of spurs for which the angle of inclination ranged from 30° to 120°, and a similar range was found in this study. In Neill (1973, p. 129), an upstream inclination is recommended for embankment-type spurs, on the grounds that an area of low water-velocity is thus formed upstream from the spur, which prevents erosion of the upstream face and permits placement of revetment on the spur nose only. Franco (1967), who studied inclination angles of 60°, 90°, and 120°, reported that the downstream-inclined spur performed best for channelization to improve navigation, but produced a greater tendency for scouring at their

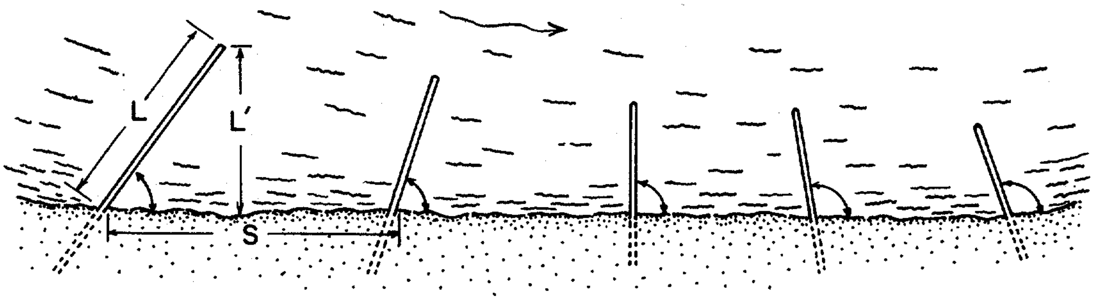

Figure 19. Definition sketch for length, spacing, and inclination of spurs.

bank end than spurs inclined upstream. On the other hand, Richardson and Simons (1974, p. 18) find that spurs angled upstream will produce deeper scour holes in the streambed, and recommend a 90° inclination as most economical for bank protection. Only one example of upstream-inclined spurs, in the main channel of a river, was found for this study; these spurs performed satisfactorily, but the river is wide and braided.

In New Zealand, where spurs (groins) have been widely used for river control, permeable spurs are invariably constructed with a downstream inclination of 30° to 60° with the thalweg, depending on the purpose of the spur and the sharpness of curvature (Acheson, 1968, p. 101). The spurs are varied in length to insure that the noses line up to a smooth curvature. Where the spurs are to have a diversionary effect, the upstream spur should be at a low angle to the bank and subsequent spurs at increasing angles. As to the impermeable spurs, the generally accepted practice in New Zealand is to use angles ranging from 90° to 120°, depending on conditions. The sharper the curvature of a bend and the steeper the river gradient, the closer together the impermeable spurs should be, and the more acute the angle with the upstream bank.

From the available evidence, it is reasonable to conclude that permeable spurs should be placed at right angles to the bank or inclined downstream. For impermeable spurs, an upstream inclination may have advantages on wide shallow channels, but for other situations the advantage of an upstream inclination has not been established.

Length, spacing, and number of spurs--The length of a spur (L in fig. 19) is measured along the crest from bankline to tip to spur. The projected length (L') is a more useful indication of channel constriction induced by the spur, but actual lengths are given for spurs described here. The "root" of a spur, by which it is keyed into the bank (dashed lines in fig. 19), is not measured in the determination of L, but it is an important measure against outflanking of the spur by the stream. So many variables enter into the determination of spur length that no firm guidelines can be given. These variables include stream-channel width, stream type, degree of channel contraction that can be tolerated, permeability of the spur, and cost considerations. According to Richardson and Simons (1974), the minimum length is 50 ft (15 m) for bank protection on straight reaches, bends of long radius, and braided channels, and the maximum length should be less than 10-15 percent of bankfull channel width. The 50-ft (15 m) minimum is based on cost considerations, as shorter spurs may cost more than riprap revetment along the bank.

The spacing (S in fig. 19) is measured along the bankline, between the centerline of adjacent spurs. For bank protection, the spacing and number of spurs are related to the length of bank protected by each spur, which, according to Neill (1973, p. 129), appears to be at least twice its projected length (L' in fig. 19). Other variables in spur spacing, besides spur length, are water velocity, angle of flow with the spurs, and curvature of the bank. Richardson and Simons recommend, in general, a spacing between adjacent spurs of 1.5 to 6 L', where L' is the projected length of the upstream spur.

For sites in this study where spurs performed well, the spacing ratio (S/L') was in the range of 0.8 to 3.5. Except at short impermeable spurs (Sites 181, 257), we found no evidence of bank erosion between spurs. In most cases, bank protection is likely to be required between short impermeable spurs, although it apparently was not required at Site 253, on a small gravel-bed stream.

According to Neill (1973, p. 129), spurs should normally be used in groups, and a single spur should in general be avoided where the main current would impinge against it. Richardson and Simons (1974) recommend that spurs to protect streambanks or to contract the stream should number no less than three. At each of three sites of the present study (Sites 175, 225, and 207), two spurs have performed satisfactorily, and Neill (1973), p. 130) gives an example of the successful use of one spur.

<u>Height of spur</u>--The height of a spur is measured relative to low-water elevation, to design-flood elevation, or to bankfull stage. Heights relative to bankfull state, or to the height of the bank from which the spur projects, are given in the case histories. The crest of an individual spur may be level or it may slope streamward. The heights of successive spurs may be uniform, increase in a downstream direction, or decrease in a downstream direction. Richardson and Simons (1974) recommend that the elevation of the crest of the spur should be 1 ft (0.3 m) above the elevation of bottom-fast ice, or at bankfull elevation, whichever is higher. For many installations, catchment of debris on the spur is a major consideration, and the top of a pile structure should be low enough to pass flood debris (California Division of Highways, 1970, p. 154). Most of the spurs described herein are near bank height, but permeable spurs well below bank height may perform satisfactorily (Sites 210 and 382).

Retards

A retard is a permeable or impermeable linear structure in a channel, parallel with the bank and usually at the toe of the bank (fig. 20), intended to reduce flow velocity, induce deposition, or to maintain an existing alinement of flow. All of the structural types previously described for spurs--earth or rock embankment, timber pile, sheet pile, or steel pile--apply also to retards. In addition, jacks or tetrahedrons, which are rarely used for spurs, are sometimes used for retards. A retard differs from a spur in orientation relative to bankline (parallel rather than inclined) and from a dike in crest height relative to bank height (at or below bank height, rather than well above bank height). Unlike armor or revetment, a retard is not supported by the bank.

As shown in table 6, most of the retards at study sites are permeable, and most have a good record of performance. The retards have proved to be useful in the following kinds of situations: (1) For alinement problems that occur very near a bridge or roadway embankment, particularly those involving rather sharp channel bends and direct impingement of flow against a bank (Sites 115, 226, 169, 171, 208, 211, 216, 223, 281, 180). (2) For other bank erosion problems that occur very near a bridge, particularly on streams that have a wandering thalweg or very unstable banks (Sites 1, 7, 21, 227, 178, 261, 219).

As compared with spurs, a retard can have a more positive action in maintaining an existing alinement, and it can be more effective in preventing lateral erosion at a sharp bend. As compared with revetment along a bankline, the retard can be oriented to provide better flow alinement, and

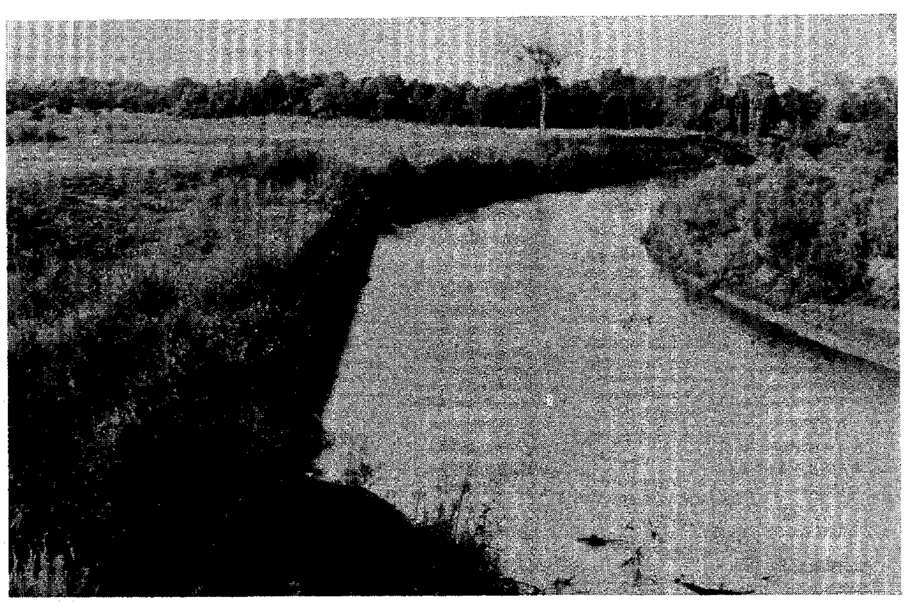

Figure 20. Timber-pile retard, at bend in channel upstream from bridge. (Washita River, Okla., Site 115.)

Table 6. *Use and performance of retards at study sites.*

Structural type	Study sites where used; and performance rating
Impermeable, earth or rock embankment . .	226AA, 227BA, 254.
Impermeable, timber pile	85, 192, 211, 245BB.
Impermeable, steel pile, pipe, or rail. .	1, 7, 75, 275.
Permeable, timber pile	21, 63, 115, 169AA, 171BA, 178CA, 187, 208AA, 211AA, 216AA, 223CB, 281BA.
Permeable, steel pile, pipe, or rail . .	180BA, 219AA.
Permeable, jack or tetrahedron	73, 222.

it can provide an outlying line of defense against bank erosion. In addition, a retard can withstand direct impingement of flow better than bank revetment unless the bank revetment is heavily reinforced.

Although the retards at study sites are mostly strong and rigid structures designed to cope with hazardous situations close to a bridge, other retard types have proved to be useful for control of bank erosion along a stream channel. The Soil Conservation Service in Mississippi has developed guidelines, based on more than 10 years of experience, for design and installation of streambank stabilization measures, including two types of retards. One of the retard types is a line of concrete jacks (fig. 21), each jack consisting

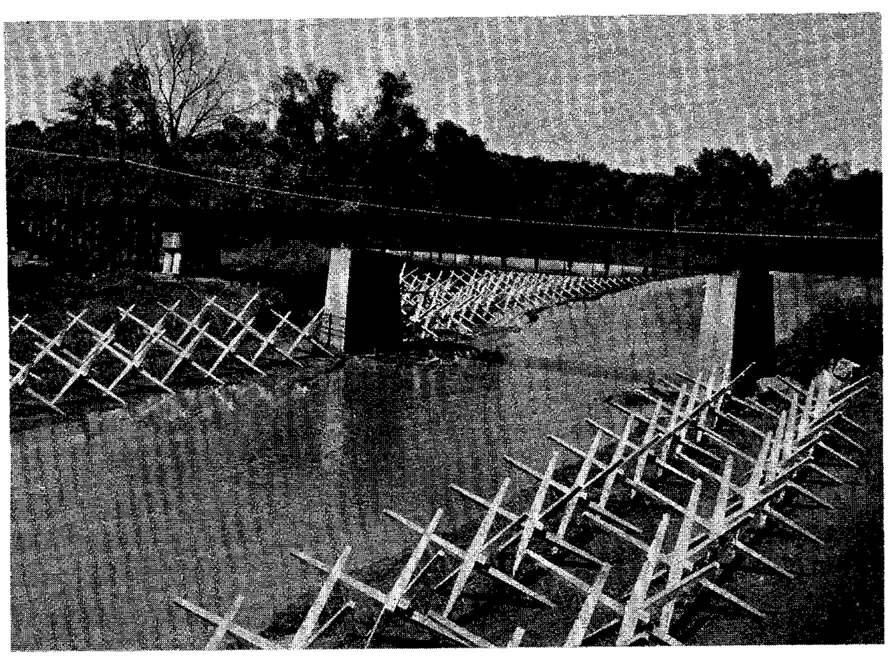

Figure 21. Concrete jacks used as retards, photographed soon after installation in 1970. (Tallahatchie River at New Albany, Miss.; U.S. Dept. of Agriculture, Soil Conservation Service.)

of three 16-ft (5.3 m) reinforced-concrete beams that are bolted together at right angles at the midpoints. The other consists of slotted-board fencing, which is supported by 10-in (25-cm) wood pile treated with a preservative.

According to interviews by K. V. Wilson with Soil Conservation Service engineers, concrete jacks are used because the steel jacks were too light in weight to withstand the water velocities. Initial failures of both concrete and steel jacks resulted from catchment of drift on anchor posts and consequent failure of the posts from scour. Other anchor posts failed when the jacks subsided. Only two failures of slotted-board fence were reported, both attributed to channel degradation. About 17 different types of bank protection measures were tried on Topashaw Canal in the vicinity of Calhoun City, Miss., including several variations of the slotted board fences and also planting of willows. (Velocities of flood flow in Topashaw Canal reach about 7 ft/s (2 m/s), according to U.S. Geological Survey measurements). The slotted-board fences, in different variations, performed satisfactorily. The concrete jacks are satisfactory, except in bends of more than 5 degrees of curvature, and are less expensive than the fences, which cost about $80-90 per linear foot (in 1976). Difficulty was experienced with the planting of willows, whose establishment is affected by the occurrence of floods.

Fence-type retards are used in California (California Division of Highways, 1970, p. 232) for protection (1) at the toe of highway embankments in direct contact with the stream, (2) training and control to inhibit bank erosion upstream and downstream from river crossings, and (3) control of erosion and redeposition of material where progressive embayments are creating a hazard. Examples of use are given in Sites 1 and 7.

Dikes

A dike is an impermeable linear structure for the control or containment of overbank flow. Most dikes are on flood plains (fig. 13) but in some situations (as on wide braided rivers or on alluvial fans) they may be within channels. All of the flow-control structures of this report, except jack fields, are termed "dikes" by Lindner (1969, p. VIII-23), and this usage seems to be generally followed by the Army Corps of Engineers. However, the term "dike" seems inappropriate for many flow-control structures, particularly the more permeable ones.

Dikes at the study sites (table 7) are used to prevent flood water

Table 7. Use and performance of dikes at study sites.

Structural type	Study sites where used; and performance rating
Earth or rock embankment	7, 9AAB, 14AA, 16AA, 27 CB AA, 74, 92, 95, 126, 140, 151AA, 157, 184, 243AA.
Steel pile, pipe, or rail . . .	275.

from bypassing a bridge (Sites 9, 14, 16, 27), or to confine channel width and maintain channel alinement (Sites 151, 243). Performance of the dikes was judged generally satisfactory. However, at Site 9, diversion of overbank flow resulted in scour at the bridge waterway.

Some dikes extend upstream from one or both sides of the bridge opening, as illustrated in Site 275; these are similar in function to spur dikes, but are usually much longer and may extend to the valley side (fig. 22). Such dikes are commonly called "training dikes" and examples of use are given in California Division of Highways (1970, p. 58, 100, 238, and 242). The use of fence-type structures for training dikes seems mostly restricted to arid and semi-arid regions.

Spur Dikes

A spur dike is a straight or outward-curving structure that extends upstream from the approach embankment at either or both sides of the bridge opening, for the general purpose of directing flow through the opening (fig. 23). Some spur dikes extend downstream from the bridge, as well as upstream. A spur dike is the same as a guide bank, as defined in Neill (1973, p. 114, 121). The major use of spur dikes in the United States, as described in Bradley (1973, p. 51) and in Colson and Wilson (1974), is to prevent erosion by eddy action at bridge abutments or piers, where concentrated flood flow travelling along the upstream side of an approach embankment enters the main flow at the bridge constriction. According to

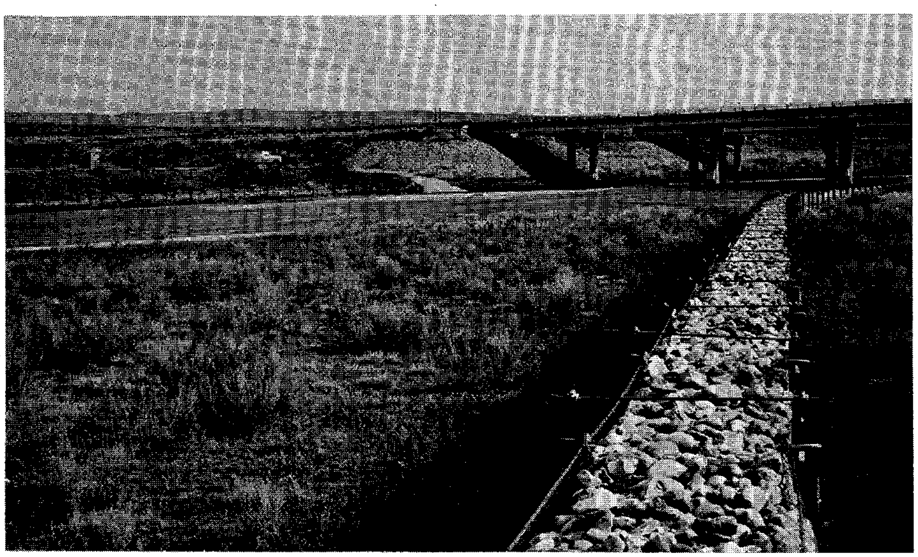

Figure 22. Training dike made of a double row of steel posts, faced with wire mesh and filled with riprap. (South Fork Powder River, Wyo., Site 275.)

Figure 23. Revetted spur dikes of elliptical shape. (Snow River, Alaska, Site 156; from Norman, 1975, p. 127.)

Karaki (1960) a properly proportioned spur dike is effective in reducing the gradient and velocity along the embankment by moving turbulence to the upstream end of the dike. Scour, if it occurs, is near the upstream end of the dike, away from the bridge. By establishing smooth parallel streamlines in the approaching flow, the spur dike improves flow conditions in the bridge waterway. As summarized in Neill (1973, p. 121), guide banks (spur dikes) "may be used to confine the flow to a single channel, to improve the distribution of discharge across the waterway opening, to control the angle of attack on piers, to break up meander patterns, and to prevent erosion of approach roads". The major purpose of a spur dike may differ with stream type and other crossing factors, and it seems improbable that any particular design criteria for shape or length will best serve all purposes.

Structural type and revetment--Nearly all of the spur dikes encountered in this study are revetted earth embankments (table 8). A rock embankment is

Table 8. Use and performance of spur dikes at study sites.

Structural type	Study sites where used; and performance rating
Revetted earth or rock embankment	30, 34, 35, 41, 46, 47, 49, 50, 57, 58, 68, 101, 150AA, 151AA, 152AB, 156AA, 158, 192, 193BB, 204BB, 206AA, 212AA, 214AA, 233, 257AA, 259, 260AA, 276, 280AA.
Earth embankment, not revetted	180AA.

described in Site 35, and a dike of concrete rubble masonry, in Site 280. Typical placement of revetment is shown in fig. 24. Rock riprap is most generally used for revetment, but also used are concrete pavement (Site 144), concrete pavement with riprap toe-protection (Sites 150, 151, and 152), rock-and-wire mattress (Site 58), gabions (Site 5), and grass sod which is considered satisfactory for many sites in Iowa (see, for example, Site 180). Inasmuch as the "principle of expendability" applies to spur dikes as well as to other countermeasures, careful consideration should be given to the more general use of vegetation as revetment. Timber-pile spur dikes were formerly used in Louisiana and in Georgia, but no recent construction of these was found.

Shape and orientation to flow--Spur dikes shaped in the form of a quarter of an ellipse, with a ratio of major axis (L_S in fig. 24) to minor axis (B in fig. 24) of about 2.5:1, are in general use. Of 23 study sites at which the performance of spur dikes was evaluated (table 8) the dikes were of elliptical shape at 16 sites, straight at three sites (Sites 47, 50, and 150), and straight with curved ends at five sites (Sites 35, 46, 150, 151, and 152). The straight dikes performed satisfactorily, although there is evidence at Site 35 that the flow does not follow the nose as well as might be desired (see also Bradley, 1973, p. 55). In Iowa, the elliptical shape has been modified for greater simplicity of construction. On the basis of observations and measurements of flood flow at elliptical spur dikes in Mississippi, Colson and Wilson (1974, p. 32) concluded that the existence of eddies between the dikes indicates deficiencies in the

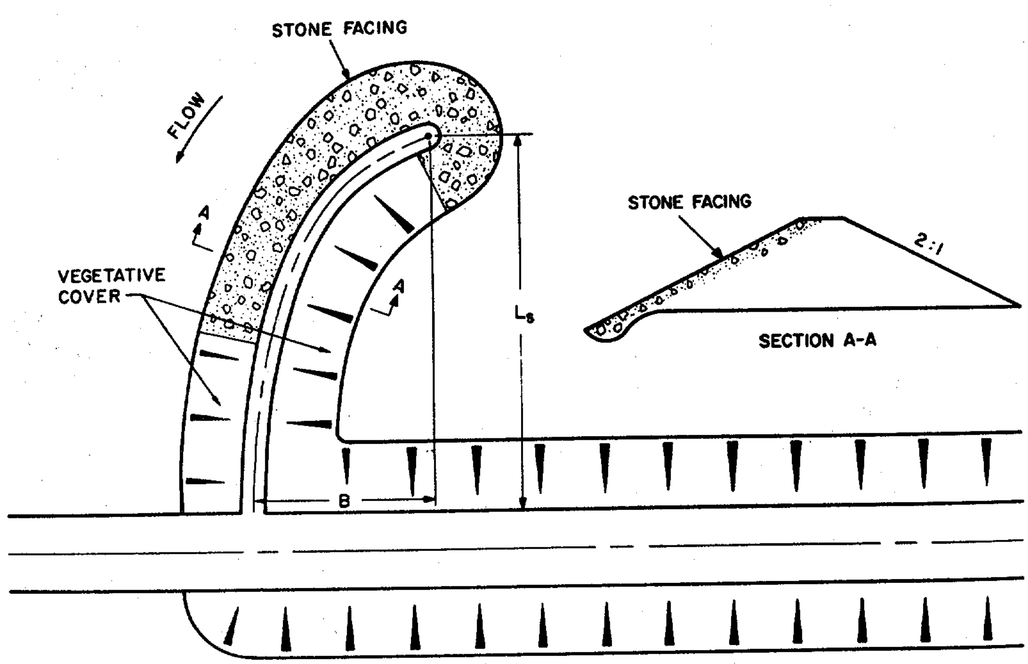

Figure 24. Plan and cross section of an elliptical spur dike. (From Bradley, 1973.)

design shape. However, no evidence of unsatisfactory performance of the elliptical shape was found in the present study.

The performance of spur dikes is affected by their orientation with respect to the main channel approaching the bridge. There seems to be a tendency to construct spur dikes perpendicular to the bridge center line, regardless of orientation of flow, although Bradley (1973 fig. 30, p. 54) implies that spur dikes at skewed crossings should be parallel to the direction of approach flow. In extreme cases, where channel flow (either a main channel or a subsidiary channel) approaches the bridge almost parallel to the approach embankment, spur dikes are likely to be severely damaged and only partially effective (see Sites 68 and 193). Such a situation is clearly a severe test for any countermeasure, and flow-control measures in addition to the spur dike are needed. Where the crossing is both skewed and eccentric, as in Site 204, spurs along the roadway embankment (in addition to the spur dike at the bridge abutment) should be considered to prevent erosion of the embankment. In some early applications (as at Site 280), two or more dikes were used on one side of the bridge opening, and there is no reason to expect one dike to be sufficient in all situations.

Spur dikes have been recommended for control of meanders that are migrating toward a bridge or approach embankment (see case described in Neill, 1973, p. 125). No examples of such use were found in the present study, but the general effectiveness of spur dikes alone for this purpose is open to serious question. On most rivers, erosion of the spur dike by an encroaching meander is probable. Spurs, retards, or revetment are more suitable for the control of meander migration.

For some situations, orientation of the spur dike with respect to the general trend of the valley, or direction of valley flood flow, should be considered. Colson and Wilson (1974, p. 31) describe the crossing of Souinlovey Creek at I-59 near Pachuta, Miss., where the spur dikes are alined with the creek but skewed to the valley flow. They conclude that "alinement of spur dikes and abutments with the valley flood flow for skewed crossings of heavily wooded flood plains with shallow depths appears to increase the effectiveness of the dikes".

Downstream extensions of spur dikes, which are commonly used in other countries, were found in the United States only at Sites 46, 156, and 214. At Site 46, the extension was added to control downstream bank erosion; at Site 214, extensions were added to prevent erosion of abutment fill. At Site 214, a major purpose of the spur dike is to confine a wide braided channel (in Alaska), and the spur dikes with downstream extensions in Canada (Sites 150, 151, and 152) serve a similar purpose.

<u>Length and number</u>--No criteria for the determination of spur-dike length have been well established. Most of the published criteria relate dike length to length of bridge (width of waterway opening). Recommendations for the upstream length (of straight spur dikes) are in the range of 0.75 times bridge length to 1.5 times bridge length (Richardson and Simons, 1974, p. 24). According to Bradley (1973, p. 55), however, there is no direct relation

between length of spur dike and length of bridge; and he prefers to use as a criterion the discharge in the first 100 ft (30.5 m) of waterway adjacent to the abutment, in relation to the discharge on one side of the flood plain. A chart of these parameters, incorporating also the average velocity through the bridge opening, is tentatively given for design purposes by Bradley.

At the 23 study sites where spur-dike performance was rated, spur dikes tend to be rather long by comparison with those listed by Bradley (1973, table C-1, p. 111). All are longer than 150 ft (46 m) except at Sites 41, 47, 212, 260, and 280, where lengths range from 15 ft (4.6 m) to 75 ft (23 m). The mean of the lengths at 23 sites is 172 ft (52 m). Ratios of spur-dike length to bridge length range from 0.014 (at Site 47) to 0.81 at (Site 151). The mean of this ratio for 23 sites is 0.42. Inasmuch as nearly all of the spur dikes performed well, it seems apparent that spur dikes in the range of 0.75 to 1 bridge length are too long. On the other hand, the length of a spur dike depends to some degree on its major purpose, which may be, in general terms, either (1) control of flood-plain flow or (2) control of channel. Most of the longer spur dikes in the study sites have channel control as their major purpose (Sites 150, 151, 152, 156). Bradley's design chart for spur-dike length evidently applies to spur dikes that serve for control of flood-plain flow, that is, as a countermeasure for lateral flow concentrations that enter the bridge constriction from the upstream side of approach embankments.

Information obtained in this study is insufficient for proposal of guidelines regarding spur-dike length; the performance of short spur dikes is inadequately represented in the study sites. For spur dikes whose major purpose is control of flood-plain flow, the tentative design chart of Bradley is probably the best available guide. For spur dikes whose major purpose is control of channel, the guideline in Neill (1973, p. 124) is probably the best available.

As to number of spur dikes at a bridge opening, there was only one dike at 15 of 23 study sites, two dikes of the same length at six sites (Sites 41, 58, 68, 150, and 124) and two dikes of different length at one site (Site 152). In view of the generally good performance of the dikes, it is apparent that a single dike is satisfactory in many situations.

Check Dams

A check dam is a low dam or weir across a channel, for the control of water stage or velocity, or to prevent channel degradation. Check dams at the study sites (table 9) were built of either rock riprap, concrete, sheet pile, rock-and-wire mattress, gabions, or concrete-filled fabric mat. No structural failures of check dams were documented, and the number of sites is insufficient to establish the superiority of any particular structural type. Most of the check dams were installed at smaller streams as a countermeasure against moderate channel degradation and they have proved effective for this purpose. In alluvial streams, however, an unacceptable amount of scour may be induced downstream from a check dam, as at Sites 159, 161, and 177.

Table 9. Use and performance of check dams at study sites.

Structural type	Study sites where used; and performance rating
Rock riprap	10BA, 90AA.
Gabion	85, 89AA, 274.
Concrete	177BBB.
Rock-and-wire mattress	60BB.
Sheet pile	85, 159AAB, 161AAB.
Concrete-filled fabric mat	183.

Jack or Tetrahedron Fields

A jack is a device for flow control and protection of banks against lateral erosion. Jacks function as flow-control measures by reducing the water velocity along a bank, which in turn results in accumulation of sediment and establishment of vegetation. The most common type has six mutually perpendicular arms rigidly fixed at the center and strung with wire. Kellner jacks are made of three steel struts; concrete jacks are made of three reinforced-concrete beams bolted together at the midpoints. All of the jacks described in the case histories are of the Kellner type.

Steel tetrahedrons are similar in function to jacks; they are apparently not in current use by highway agencies, but use at one site (Site 73) is described.

Jacks are usually deployed in fields, consisting of rows of jacks tied together with cables, with some rows generally parallel with the bank and some at an angle (fig. 25). Such fields are sometimes called "jetty fields".

Of the four sites where steel jack fields were used, the performance was rated satisfactory at two (Sites 120 and 133), partly satisfactory at one (Site 175), and unsatisfactory at one (Site 281). As a basis for discussing the major factors that determine the performance of jack fields, the observations of Keeley (1971, p. 54-61; 274), from his study of many jack fields in Oklahoma, are freely paraphrased below.

1. Jack fields are not likely to perform well on streams that have erodible, non-coherent banks and carry little floating debris. If debris does not accumulate at the jacks, they offer little or no protection to the bank, which continues to erode.
2. Jack fields will serve for bank protection but not for altering the course of a stream. When placed in the river bed, they become ineffective through burial.
3. Jack fields, like other flow-control measures, are vulnerable to outflanking, and the upstream terminal of the field should therefore be at a place resistant to erosion.
4. Individual jacks that have been severely damaged by floating debris may still function as part of the jack field.

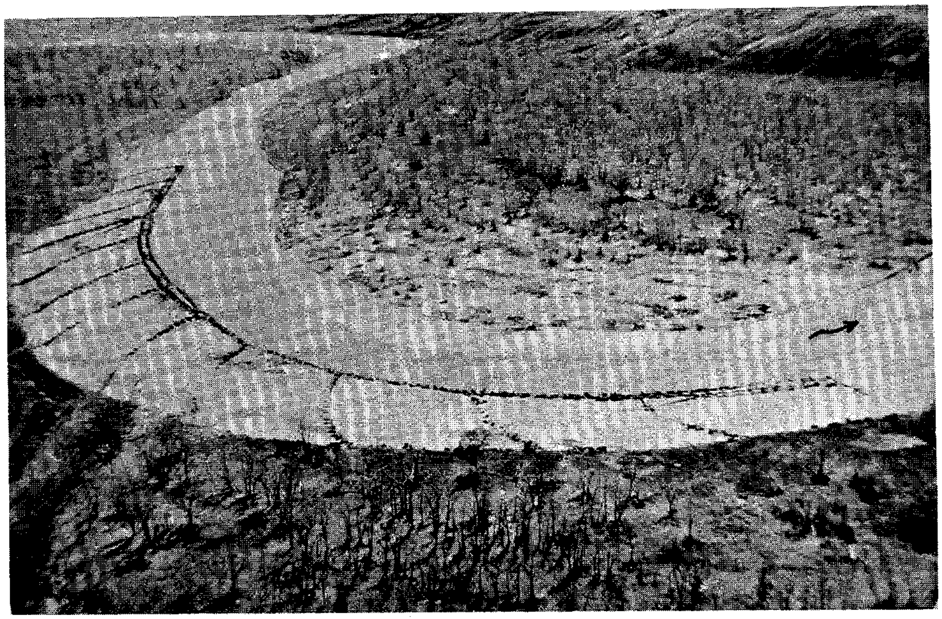

Figure 25. Steel-jack field at outside of meander. Sediment has accumulated at jacks, and performance of jacks in controlling bank erosion is good. (Cheyenne River near Creston, S. Dak.; photograph from South Dakota Dept. of Transportation.)

5. On streams that have banks moderately resistant to erosion and that transport adequate quantities of floating debris, jack fields may provide good and economical protection to an unstable bank. The debris must accumulate at the jacks before they are undermined by scour and thus lowered below streambed elevation.

Thus, according to Keeley, the major factors that enter into the performance of jack fields are availability of floating debris, erodibility of the bank, and placement, whether at bankline or in the main channel of a stream.

Judging from Keeley's observations in Oklahoma, a key factor in poor performance of steel-jack fields is burial, as brought about by failure to accumulate drift or by placement in the main channel of a stream. In some cases, complete burial may be prevented by use of larger jacks (or tetrahedrons). The steel jacks discussed by Keeley were evidently made of 16-ft (5-m) members; tetrahedrons made of 30-ft (9-m) members were found effective in the deep silty sands of the Colorado River by California Division of Highways (1970, p. 246). On the sandy bed of the braided Salinas River near King City, California, the Army Corps of Engineers (Pickett and Brown, 1977, p. 34-40) has also demonstrated the effectiveness of 30-ft (9-m) steel tetrahedrons; and a further example of the use of large tetrahedrons on the Salinas River is given in Site 173 of this report. In Mississippi, the Soil Conservation Service found that steel jacks were too light to resist lateral movement, and heavier concrete jacks were found to be effective.

As to the ineffectiveness of jack fields for altering the course of a stream (as compared with their use for protection of an existing bank), the findings of this study do not contradict Keeley's observation. At Site 120, which is also reported on by Keeley (1971, p. 244-251), a jack field has been employed in shifting the river course away from the outside of a meander bend, by a distance of about 250 ft (76 m). However, few of the jacks were actually placed in the river bed, as the river course was realined and the former channel filled in before the jacks were placed. Also, the river transports abundant floating debris. At Site 175, the jack field extended into the main channel of the meandering Grand River in South Dakota, but the downstream end was anchored to a massive sheet-pile spur. Failure is attributed to the fact that ice severely damaged the steel jacks before sediment could accumulate. At the Chenenne River near Creston, South Dakota, steel jacks were used successfully to build out the eroding concave bank at a bend. Although jacks were placed in the river channel, the stream is wide and shallow and has a wandering thalweg.

On the basis of Keeley's observations and the findings of this study, the following guidelines for the use of jack fields as countermeasures are proposed:

1. On any river, the probability of good performance of a jack field is greatly increased if small floating debris and sediment load are in sufficient amount to form accumulations within the first few years after construction.
2. Jack fields may serve for protection of an existing bank, but are not usually reliable for altering the course of a stream by shifting the bankline channelward. On wide shallow channels, which are commonly braided, jack fields may serve to shift the bankline channelward, particularly if jacks (or tetrahedrons) of large dimensions are used.

SPECIAL DEVICES FOR PROTECTION OF PIERS

Special devices for protection of piers and pile bents are listed in table 10 and described at the sites referenced in the table. In addition to these, Chang and Karim (1972) have done an experimental study on the installation of protective piles upstream from piers where hazardous scour holes have developed. An experimental field installation was made at the I-29 crossing of the Big Sioux River south of Brookings, South Dakota, but the results are as yet inconclusive (Karim, oral communication, 1978).

Table 10. Special devices used at study sites for protection of piers.

Device	Study sites where used; and performance rating
Flow deflector plate attached to pier	164BB
Debris deflector	21AA, 153
Abrasion armor at pier nose	8AA, 60, 140
Sheet pile driven around pier	174
Grout curtain around pier foundation	24AA
Concrete curtain wall between steel pile . . .	60

Only a few examples of debris deflectors were found. A concrete-fender debris deflector has functioned satisfactorily at Site 21; and steel-rail deflectors worked for a time at Site 174, where severe channel degradation eventually rendered all countermeasures ineffective. At Site 153, large sheet-pile cassions create a bow wave that serves to deflect drift.

Several examples of damage to concrete piers by boulders or other sediment in transport were found, and an effective abrasion armor is described in Site 8.

MODIFICATIONS OF BRIDGE, APPROACH ROADWAY, OR CHANNEL

Sites at which modifications of bridge, roadway, or channel were used as countermeasures are referenced in table 11. The most commonly used modification involves the underpinning or jacketing of a pier or abutment. Underpinning was widely used, and with evident success, in Pennsylvania at old bridges with masonry abutments that were damaged during the catastrophic floods of 1972. With regard to underwater pressure grouting as a scour countermeasure, a cautionary note is given by the Massachusetts State Highway Department, from their experience with a bridge on Cohasset Narrows, where the abutment was endangered by tidal scour. Large amounts of grout can be lost by underground diffusion.

MEASURES INCORPORATED IN DESIGN OF A REPLACEMENT BRIDGE

Countermeasures incorporated into the design of a bridge are undoubtedly the most widely used and most effective for the prevention of hydraulic problems such as scour at piers. However, the emphasis of the present study is on countermeasures at existing bridges. Sites at which countermeasures

Table 11. Bridge, approach roadway, or channel modifications used as countermeasures.

	Study sites where used; and performance rating
Underpinning or jacketing of pier or abutment	2AA, 3AA, 7, 23AB, 70, 122, 148, 174CB, 247AA, 264, 268, 269AA, 282.
Addition of spans to bridge	8, 148, 174CB, 194, 196, 198.
Raising of bridge superstructure	71AA
Realinement of approach channel	13, 16, 192CB, 213, 226CB.
Construction of overflow section on roadway	18, 19, 22, 100.
Driving pile bents to greater depth	160, 197.
Addition of webbing or enclosure to pier	8, 60, 276.
Deeper placement of footings	16

have been incorporated into a replacement bridge, mostly as a result of problems encountered at a previous bridge, are referenced in table 12.

SURVEY OF COUNTERMEASURE PRACTICES

In interviews with state bridge engineers, questions were asked about bridge design practices that serve to prevent hydraulic problems, and about practices regarding countermeasures installed separately at or near the bridge, either before or after hydraulic problems occur. With regard to design practices, the structural function of a particular feature cannot always be separated from its function as a countermeasure. For example, a pile foundation set deep enough to provide the required bearing may also be deep enough to provide protection against scour. A pier made solid or webbed for structural reasons, may also inhibit the accumulation of drift. Generalizations on some aspects of design and countermeasure practices are summarized below, and additional information about the practices of particular states follows.

Design Flood Frequency

Flood frequencies used by states for bridge design purposes depend on many factors, including the class of road and land use on the flood plain. Most states tend to design bridges on primary roads for the flood of 50-yr recurrence interval, with consideration being given to conveyance of the 100-yr flood without significant increase in flood hazard. Two states (California, Texas) may use a 50-yr design flood for the bridge superstructure and a 100-yr flood for the substructure. In Idaho and Georgia, the 50-yr flood is used for both primary and secondary bridges, but in Georgia the approach embankment may be designed for overtopping by the 25-yr flood. With few exceptions, bridges on secondary roads are designed for the 25-yr flood. The advantages of designing a crossing for overtopping of the roadway during floods have been emphasized by Bradley (1973, p. 45), but we found little evidence of this practice for primary roads. Instead, some approach embankments are so high that they have failed by slumping.

Table 12. Countermeasures incorporated into a replacement bridge.

Countermeasures	Study sites where used; and performance rating
Increased bridge length	5, 6, 8, 27, 57, 61, 97, 99, 120, 130, 150.
Increased clearance (freeboard)	8, 57.
Overflow section on roadway	100, 158.
Webbed or wall-type piers	1, 8.
Fewer or no piers in channel	31, 32, 52, 97, 99, 123.
Piers designed to deflect boulders and debris . .	8AA, 52.
Guard rails that minimize flow resistance	11, 132AA.
Realinement of approach channel	5, 6, 8, 15, 101.

Measures to Protect Piers From Scour

With regard to design practices, a question was asked about the preferred type of pier, but this preference depends so much on circumstances that no generalizations can be made. As would be expected, streamlined or round piers are in general use. Spread footings are rarely used unless they can be set on bedrock. Footings with pile foundations are usually set a minimum of 6 ft (2 m) below streambed, or 10 ft (3 m) where there is reason to believe that the streambed is easily erodible, as on sandbed streams. As to estimation of depth of scour and depth of pile foundations, no information was gained beyond that previously published (for example, Highway Research Board, 1970; Breusers, Nicollet, and Shen, 1977).

At bridges on very large rivers, such as the Mississippi, or on large rivers in which scour to great depths is known to have occurred, such as the Atchalafaya, protection is placed around piers at the time of bridge construction. This protection usually consists of large mats, of willow or filter cloth, weighted with rock riprap. At bridges on other rivers, countermeasures are rarely used at piers until after a scour problem has been detected.

Measures to Protect Abutments From Scour or Lateral Erosion

Spillthrough abutments are in very general use, although some design engineers expressed a preference for vertical full-height abutments. In some states, abutment fill-slopes are revetted on all or most bridges at the time of construction; in others, revetment is applied only in cases where it is thought to be needed, or after a problem occurs. Dumped rock riprap is by far the most common revetment, but in two states (Illinois, Florida) concrete slope pavement is in general use.

Measures to Prevent Accumulation of Debris

Except for well-known design features relating to bridge clearance, pier spacing, and in particular to webbing or enclosure of multiple column piers or pile bents, no successful devices for the prevention of debris accumulation were reported in the interviews. In Wisconsin (Government Bridge over Bad River in Ashland County), an ice and debris deflection structure was built upstream from the bridge, for the purpose of directing floating debris between the piers. Accumulated debris at the structure has deflected flow toward the approach embankments, causing erosion of the embankment toe.

Measure for Prevention of Ice-related Problems

For prevention or control of ice accumulation at bridges, no agency reported any measures other than those involved in bridge design, such as increased bridge length, wide pier spacing, battered piers, and avoidance of structural features beneath the bridge deck that might cause lodgment of ice. In northern regions, piers are designed to withstand dynamic ice forces (Neill, 1976), but the necessity for armoring pier noses for prevention of ice damage by impact or abrasion is unclear. Pier noses are commonly armored in Alaska and Alberta, armored on some bridges in Wyoming, North Dakota, and South Dakota, but rarely armored in Saskatchewan. In one notable case from Saskatchewan (Site 258, Borden Bridge on North Saskatchewan River), pointed concrete pier noses having no armor are undamaged after exposure to ice since 1936. Some damage resulting from icing on small streams was reported from Alaska, but such damage was not mentioned elsewhere. Water running over the surface of ice, which has built up by icing to stream-bank level, may erode abutment fill-slopes. One countermeasure tried in Alaska (on Crooked Creek, Steese Highway) was to confine the channel with gabion bank revetment, in order to prevent icing by increasing the channel depth. This measure has reduced, but not entirely prevented, maintenance from icing problems.

Use of Flow Control Measures (Including Spur Dikes)

Practice in the use of spur dikes and other flow-control measures, (such as spurs, jetties, retards) differs greatly from one state to another. There are no spur dikes in some states and several hundred in others. The spur dikes in use are, with few exceptions, earth embankment of elliptical shape, as recommended in Bradley (1973). Spur dikes are, in many cases, built at new bridges; the other flow-control measures are rarely included as part of bridge design, but are built only after a hydraulic problem has occurred. In some states, no flow-control measures have been constructed at bridges in the state road system. Use of flow-control measures may require easements or the acquisition of property beyond the right-of-way, both for construction and maintenance, and many bridge engineers are reluctant to undertake this procedure. There is also the possibility that flow-control measures may be interpreted as interference with the natural channel and thus pose a liability threat, although no examples of this were found in the present study.

Channel Alterations for Bridge

The practice of altering or relocating a stream for better flow alinement at the bridge has been curtailed in most states and discontinued entirely in some, because of state and federal environmental regulations as enforced by various agencies. Also, the potential of lawsuits, relating to damage attributed to the alteration, has tended to curtail the practice. Since the bridge engineer can, in most cases, neither select the crossing site nor relocate the channel where approach alinement is bad, he may be forced to design a longer and more expensive bridge. In addition, poor channel alinement may result in continuous and expensive maintenance problems at a bridge and may be a hazard to the public safety. Extensive channelization projects, usually done for purposes of improving flood-plain drainage, have resulted in serious channel degradation and lateral erosion, as documented in the case histories of this report. However, this should not be confused with the alteration of a short reach in the vicinity of a bridge, for which there is little documentation of upstream or downstream environmental damage.

Practices as Reported by Individual States

Responses to questions regarding specific countermeasures used for the protection of piers and abutments, and the use of flow-control measures, are summarized below by state. Also summarized are responses to the question, "What countermeasures have you found unsatisfactory, either in general or in particular situations?" For Maine, unpublished information is given from a 1969 survey on scour at bridge waterways, carried out for the Highway Research Board and described in Highway Research Board (1970); this information is identified by the phrase "HRB Survey".

Alaska--Rock riprap at abutments, a few spur dikes in use. Failure of impermeable spurs on Tanana River, Richardson Highway near Donnelly in 1963. Spurs caused deposition, and the river cut behind them; the agency concluded that spurs were not suitable for braided streams of high bedload, and used riprap for bank protection. On Bear Creek, grouted riprap at an abutment failed because of undermining.

Arizona--At abutments, wire-enclosed rock, dumped rock riprap, or grouted rock riprap. Spur dikes in use 10 years. Timber-pile spurs found effective on Little Colorado River. Failures of steel jacks and grouted rock riprap.

Arkansas--At piers, rock riprap used after problem occurs. At abutments, rock riprap. Spur dikes used but rarely. Timber-pile spurs and retards effective on Red River. Failures of sod as bank protection, and of undersized riprap.

Florida--At abutments, concrete slope pavement or sacked concrete. Only one or two spur dikes in state, no other flow-control measures.

Georgia--For pier protection after problem, may pave streambed with concrete or grouted riprap. At abutments, granite riprap at most bridges, sacked concrete at some. Granular filter or plastic filter cloth used in some situations, but no cases know where lack of filter has caused problem. About 150-200 spur dikes in state, in use since 1960, now of elliptical design but formerly of timber pile. No other flow-control measures used.

Idaho--At abutments, rock riprap, grouted riprap, or gabions. No spur dikes or other flow control measures. Solid or webbed piers used to prevent accumulation of debris. Ice is a problem, at some bridges the bottoms of the bridge stringers have been enclosed to prevent lodgment of ice.

Illinois--At piers, concrete-fabric mat, rock riprap, and sheet piling have been used after problem occurs. At abutments, concrete slope pavement in general use, 6-in (15-cm) thickness. Spur dikes rarely used, other flow-control measures not used. Failure of concrete slope pavement at a few bridges.

Indiana--At abutments, rock riprap. Spur dikes rarely used, other flow-control measures not used.

Iowa--Abutment slopes not usually protected; where done, rock riprap is used or, less commonly, concrete pavement or sheet piling. Spur dikes in fairly common use, earth embankment type of modified elliptical shape. Revetment not generally used on spur dikes, reliance placed on grass sod. There are several installations of steel H-pile spurs and retards. Failure in one case of short impermeable spurs or "hard points"; other cases in which stream has cut behind an upstream steel H-pile spur.

Louisiana--As a substitute for dumped-rock riprap (which is expensive in state), has in some cases found articulated concrete blocks (on plastic filter cloth) more satisfactory than concrete-filled fabric mat, which failed because weep holes were not adequate for relief of hydrostatic pressure. Also, vegetation becomes established on cellular concrete blocks.

Maine--Experience with relocating sections of streams has been generally unsuccessful; streams tend to revert to former channels during flood. Abrasion of wooden piling by ice has been of some concern (HRB Survey).

Maryland--At piers, rock riprap after problem, less commonly sheet piling. At abutments, concrete slope-pavement with concrete cutoff walls, less commonly timber bulkheads. Among unsatisfactory countermeasures, underpinning of pier regarded as expensive and of uncertain effectiveness. Wire-enclosed riprap is subject to damage because of abrasion of wire, even PVC-coated wire.

Minnesota--At piers, rock riprap after scour problem, filter cloth used in a few cases. At abutments, rock riprap is used more often than concrete slope pavement, but one or the other is used at nearly all bridges. Riprap is used with granular filter. A few spur dikes are in use.

Mississippi--At piers, rock riprap after scour problem. At abutments, mainly rock riprap. Many spur dikes in use. Permeable spurs or retards, of either timber or steel pile, in use at many sites; these measures are regarded as effective but expensive.

Missouri--At abutments, riprap with granular filter is used where overbank velocity exceeds 5 ft/s (1.5 m/s). Spur dikes are used mainly on major rivers. Spurs or retards in use at about 15 sites; most of these consist of a single row of timber pile, but there are at least two installations of steel-pile spurs.

Montana--No revetment at most abutments, rock riprap where present. Quarried rock is preferred, but rounded boulders are used for emergencies and on low-velocity streams. About 10 spur dikes in state, built according to guidelines in Bradley (1973). Little or no use of spurs and retards. Overspill sections are used on some roadway embankments. Wire-enclosed riprap is not favored because of its expense and the probability of abrasion damage to the wire.

Nebraska--At abutments, riprap of rock or broken concrete is used where required but abutments are not generally revetted. No filter is used, specifications include gradation of sizes. Minor use of spur dikes and other flow-control measures. Steel spurs successful at some sites, but ineffective at one site where channel is degrading.

Nevada--At abutments, rock riprap, or gabions where large riprap is not available. Little or no use of spur dikes and other flow-control measures. Some failures of concrete slope pavement.

New Mexico--At piers, rock-and-wire mattress has been used for scour problems. At abutments, rock-and-wire mattress, 12-in (30-m) in thickness is specified for all bridges; no filter is used with the mattress. Spur dikes are used on channels with large overflow sections. Impermeable spurs are in use at some sites, steel jacks are rarely used. Dumped rock riprap (without wire) does not stay in place as well as wire-enclosed riprap.

North Dakota--Riprap, usually of "fieldstone" (rounded glacial boulders) is in general use at abutments, and revetment is usually extended upstream from bridge. Spur dikes of elliptical shape in use at about 200 bridges. Little or no use of other flow control measures. For check dams, reinforced concrete has been found more satisfactory than sheet piling. Many bridge waterways are of trapezoidal shape, and steep side slopes are avoided; formerly used 2:1 slopes, now uses 3:1 or 4:1 slopes. Where ice is a problem, steel noses are used on piers, and piers at some sites are battered.

Ohio--Rock riprap is used on most abutment slopes. There are about 10 spur dikes in the state, and no other flow-control measures are used. Some failures of concrete slope pavement are reported.

Oklahoma--At abutments, rock riprap, concrete pavement, Bermuda grass sod, and, in a few cases, soil cement, are used. There are only a few spur dikes in use. Other flow-control measures in use are timber-pile spurs, steel-jack fields, and short impermeable rock spurs. Although effective in many cases, timber-pile structures are subject to destruction by fires and vandals. Concrete slope pavement requires maintenance.

Pennsylvania, District 8 only--Piers and abutments damaged by undermining are commonly repaired by underpinning with concrete and jacketing with concrete. At abutments, rock riprap is the most common revetment. Failures of rock riprap are attributed to improper installation or to inadequate size of rock.

South Dakota--At piers having scour problems, rock riprap, gabions, and concrete-fabric mat have been used. Abutments usually are revetted with rock riprap, but concrete slope pavement is used at some bridges on interstate system. Spur dikes are used at many sites, and all of these are not revetted. For flow control, spurs of sheet pile or steel pile are preferred to riprap on banks. Of two steel jack installations, one has performed well. Concrete slope pavement has failed in severe floods.

Texas--At abutments, concrete slope pavement with rock riprap at toe is used, also riprap of broken concrete, and sheet pile. Spur dikes in use since 1965, regarded as effective. Other flow-control measures in use are steel-jack fields, impermeable rock spurs, and timber-pile spurs. Car body revetment proved satisfactory on Red River. Some failure of sacked-concrete riprap owing to weathering.

Virginia--At abutments, rock riprap. Spur dikes and other flow-control measures rarely used. Failures of concrete slope-pavement, where toe was not on bedrock.

Wyoming--At abutments, rock-and-wire mattress. About 6 spur dikes (or training dikes) in state, some of fence type. Welded-wire mesh not satisfactory for rock-and-wire mattress, because the welds tend to break; woven-wire mesh is preferred.

Missouri Pacific Railroad--As remedial measures for scour at piers, sheet pile or riprap are used. Riprap may cause scour hole downstream from pier. A major expense in remedying hydraulic problems is that of underpinning piers. Spur dikes not required, because trestles are used to cross wide flood plains.

CHAPTER 4

GEOMORPHIC FACTORS

DEFINITIONS

The term "geomorphic" is applied here to properties of the landscape in which a bridge is located, properties of the stream that is crossed, and certain effects of man on the stream and the landscape. Only those properties (and effects) that are considered most important for the analysis of hydraulic problems and the performance of countermeasures are included here. Use of technical geomorphic terms has been kept to a minimum. The terms used are defined in the Glossary, but the usage of two basic terms needs to be discussed here. (1) A "stream" is defined as a body of water, which may range in size from a large river to a small rill, flowing in a channel. A river is a large stream, but no definite limits of size for a river have been established. (2) The term "channel" refers to the bed and banks that confine the surface flow of a natural or artificial stream. A channel is usually discernible whether it contains water or not. Many of the properties used in classification apply more specifically to the channel than to the stream. The term "stream" is often used when "channel" is actually meant, but there seems no good way to avoid this ambiguity. In this report, as in much of the technical literature on streams, the terms are sometimes used interchangeably.

USE OF AERIAL PHOTOGRAPHS AND MAPS

Many stream properties can be observed more accurately (and much more quickly) on vertical aerial photographs than on maps in the field. In the field, views of the stream are commonly obscurred by vegetation, and a vantage point from which to observe the plan of the channel is rarely available. Although modern topographic maps are prepared from aerial photographs, the rendition of channel details on maps is necessarily a simplified and subjective interpretation by the cartographer, and much detail is lost. Both photographs and maps represent the stream as it existed at some time in the past, and a field investigation is required to ascertain the present condition of the stream. In addition, field investigation is needed to verify observations made on photographs or maps, to observe features that may have been missing or obscurred, and to record changes.

Most highway agencies routinely acquire aerial photographs, by contract or with their own aircraft, of crossing sites for design purposes, and many also acquire the photographs for sites where hydraulic problems have occurred. Such photographs are as essential for engineering purposes as radiography is for medical purposes. The usefulness of vertical aerial photography is greatly enhanced by stereoscopic viewing. Because of the vertical exaggeration provided by stereoviewing, channel and flood-plain features marked by scarps appear more sharply defined on the photographs than in the field.

Comparison of aerial photographs taken at intervals in the past with recent photographs is a valuable technique for evaluation of stream behavior, and it should be applied to all new crossings and to the planning of countermeasures for an existing bridge. Much of the United States, particularly in agricultural regions, has been photographed periodically since about 1935 by the Department of Agriculture, and much has also been photographed by other governmental agencies. A computerized record of aerial photography for all parts of the U.S. is maintained by the National Cartographic Information Center of the U.S. Geological Survey in Reston, Virginia. If given the geographical coordinates of an area of interest, the Center will furnish information on available photographs from which the photographs can be ordered. Comparison of photographs made on different dates can be made by enlarging them to the same scale; by photocopying them on ordinary 35-mm color film to produce slides and projecting them to the scale (Brice, 1974, p. 187); or by the use of a zoom transfer scope.

In addition to aerial photographs, on which details of the channel are shown, a topographic map is needed for broader representation of the stream, its valley, and the surrounding terrain. Most of the channel (or valley) slopes in this report are from topographic maps. The 7.5' quadrangle maps (scale 1:24,000) of the Geological Survey are most useful, but the 15' maps (scale 1:62,500) can be used for areas not mapped at the larger scale.

The properties of a stream may change from place to place along its course, sometimes rather abruptly, and a description (or classification) applies only to a segment or reach that is reasonably consistent in its properties. Such a reach may be only a few hundred yards (meters) in length for a small stream, or tens of miles (kilometers) in length for a large stream. Properties upstream from a crossing are most important, but downstream properties should also be noted.

Stream descriptions for the study sites of this report apply to the reach at the crossing site and are based on aerial photographs, topographic maps, and observations in the field. Most of the aerial photographs were obtained from state highway agencies, but photographs from the Department of Agriculture, the Geological Survey, the Forest Service, or other governmental agencies were also used. At sites for which aerial photographs were not obtained, topographic maps were used and the stream descriptions are incomplete.

WHY ONE STREAM IS DIFFERENT FROM ANOTHER

Before a stream classification is presented, the genetic factors that cause one stream to be different from another will be discussed. According to Lane (1957, p.8), the most important of these genetic factors are (1) stream discharge, (2) longitudinal slope, (3) sediment load, (4) resistance of banks and bed to movement by flowing water, (5) vegetation, (6) temperature, (7) geology, and (8) works of man.

Stream discharge depends on precipitation and on the characteristics of the drainage basin, and it is among the few variables that cannot be modified by the activities of the stream. An alluvial channel becomes adjusted in size to transmit the discharge that it receives, although investigation has not yet established just which measure of discharge is most effective in the establishment of channel size. In selecting a particular statistical measure of flow for plotting channel size against discharge, Leopold and Maddock (1953) and Lane (1957) chose average discharge because, for most streams, this is the only readily available measure. However, Leopold and others (1964, p. 83) reason that the work of perennial streams in scour and fill and in transport of sediment is accomplished principally by flows near or above bankfull stage--flows that occur less than 0.4 percent of the time or roughly once a year.

Leopold and Maddock (1953) showed that, at a given cross section, width, depth, and velocity increase systematically with discharge. Also, as the discharge of a stream increases in a downstream direction, width, depth, and velocity of the flow (at a given flow frequency) increase systematically with discharge. For rivers studied, average width increases downstream at approximately the 0.5 power of average discharge, average depth as the 0.4 power, and average velocity as the 0.1 power. This does not mean, of course, that all rivers have the same width, depth, and velocity at a given discharge; the absolute values of these depend mainly on the resistance of the banks, and the systematic downstream increase applied only if bank resistance remains constant in a downstream direction. Also, each value of width, depth, and velocity used by Leopold and Maddock applies only to the measured section at a gaging station, which may not be representative of the reach in which the gaging station is located. For some channels, especially wide shallow channels, the standard deviation of width, depth, and velocity in a given reach is likely to be large.

The longitudinal slope of a channel is initially established by the geologic character of the region through which the channel runs: channel slopes in mountain belts are likely to be steep, whereas channel slopes in plains regions are likely to be gentle. The initial slope is modified to a greater or lesser extent by a stream, or by its ancestral streams. With the passage of time, falls and rapids in the channel tend to be obliterated and the longitudinal profile commonly assumes a shape that is concave upward. For practical purposes, however, the longitudinal slope of a channel is largely predetermined by regional geologic slope, and this predetermined slope has an effect on channel morphology and behavior.

The cause-and-effect relations between predetermined slope on the one hand, and channel morphology and behavior on the other, are complex. The relation between slope and channel pattern is affected by discharge, bank resistance, and size of bed material, as well as other variables. On a given regional slope, a small stream may be sinuous because its banks have the requisite resistance relative to discharge; on the same slope, a larger stream may be braided because its banks have a lesser resistance relative to discharge. In general, the braided pattern is established when the banks have a low resistance relative to slope and discharge and the sinuous pattern, when the banks have a moderate resistance. But braiding can arise from causes other than low bank resistance; among these are a large supply of coarse bed material, as from a melting glacier.

The effects of sediment load depend both on the quantity and the particle size of the load. The quantity of load depends mainly on erosion rate in the drainage basin, and particle size depends on the geology and soils in the basin.

The effects of sediment load on channel morphology and behavior are mainly related to (1) the cohesiveness of the load when deposited as bank material and (2) attainment of the slope required for transport of the bedload. If a stream lacks sufficient slope to transport the material being supplied it from the drainage basin, the channel will fill until sufficient slope is attained. Attainment of the requisite slope may be accompanied by a change from a sinuous to a braided pattern. Leopold (1964, p. 293) has shown that braided channels (mainly in Wyoming and Montana) typically have steeper slopes than meandering channels, but the cause-and-effect relations are not resolved.

Resistance of banks to erosion, an important factor in stream morphology, depends on properties of the bank materials and on the vegetal cover. Meandering streams of nearly uniform width and deep, narrow cross section usually have banks that are resistant because of high clay content or dense vegetal cover. Streams having non-resistant banks, composed mainly of sand and lacking a dense vegetal cover, are usually braided and have wide, shallow cross sections.

Leopold and Wolman (1957) concluded tentatively that resistance to flow is related to the granular roughness of the bed material and to large-scale irregularities on the bed surface. They believe also that a reach of stream will adjust in velocity, depth, and slope simultaneously to accommodate a change in roughness due to bed form for a given flow and sediment discharge. If the flow and sediment discharge remain steady for a period of time, it is reasonable to assume that the roughness will attain a steady value. However, during periods of rising and falling stages in natural streams the changes in flow conditions may occur too rapidly for the bed to adjust its configuration to the new flow and sediment discharge. In such cases, the relationship between bed configuration and flow can be somewhat indeterminate. Therefore fluctuations in flow and sediment discharge may be significant factors in the development of channel pattern.

Because temperature has an effect on other variables that are known to influence channel form, it is listed here as an effective variable; but the precise effects of temperature have not been isolated. In northern regions, the formation of ice jams in the spring causes the water level to rise, promotes meander cutoffs, and increases scour. Freezing of banks in the winter may increase their cohesiveness, but thaw in the spring leads to rapid bank sloughing (Leopold and others, 1964, p. 87). Lane and others (1949) attribute a major increase in sediment transport of the Colorado River in the winter to low-water temperatures. Changes in viscosity influence not only the fall velocity and vertical distribution of sediment but also the thickness of the laminar sublayer and the bed configuration and resistance to flow; large sediment discharges and small flow resistances are associated with low temperatures. Obviously, the temperature of a region has an important influence on the kind and density of vegetal cover along the streambanks, and hence on the resistance of the banks.

As a variable in connection with stream morphology and behavior, geology refers to the relief of the drainage basin, the character of the bedrock and soil, and the erosion rate; the effects of these have been discussed. In addition, geology refers to the geologic history of the channel. Most stream valleys have had a long geologic history and may have been occupied in the past by streams that differed substantially in discharge and morphology from the modern stream. The deposits of these ancestral streams, as well as their valley slopes, can have an important influence on the modern stream.

For purposes of this report, the "works of man" are divided roughly into two categories. Structures at a crossing are considered separately (in the chapters on bridge factors and on countermeasures) from works that are not part of the crossing. Of the many external works that have an impact on the crossing, only those that have a rather immediate and apparent impact could be considered. These include general channel alteration projects, sand or gravel mining, and the building of reservoirs.

In discussing the effects of different variables on channel morphology and behavior, it is implied that morphology and slope are adjusted to the integrated effects of all these variables. This idea of adjustment is called "quasi-equilibrium" by Leopold and others, (1964, p. 266) and "regime theory" by Blench (1969). In the literature of geology and geomorphology, streams that exhibit evidence of such adjustability and stability have been called "graded". Mackin (1948, p. 471) defined a graded stream as "one in which, over a period of years, slope is delicately adjusted to provide with available discharge and with prevailing channel characteristics, just the velocity required for the transportation of the load supplied from the drainage basin." However, Mackin has placed too much emphasis on slope; adjustment of width and channel pattern is equally or more important. If the water velocity in an alluvial channel becomes, for any reason, too great for containment by the bank materials, it can be reduced effectively by an increase in width (and the accompanying decrease in depth) than by a decrease in slope. An increase in width may be accompanied by a change in channel pattern from sinuous to braided.

No practical criteria are available for the immediate identification of a graded stream in nature. For practical purposes, it must be assumed that all alluvial channels have attained some degree of equilibrium, and that a pronounced change in any of the controlling variables will lead to scour or fill. The scour or fill may represent only temporary fluctuations about some mean value, or may represent a long-term trend. If the trend is over a period of years or decades, the tendency to scour is usually referred to as degradation, and the tendency to fill as aggradation. Obviously, deep temporary scour can be just as disastrous to a bridge as long-term degradation.

STREAM CLASSIFICATION FOR ENGINEERING PURPOSES

The stream classification presented here is based on stream properties that are observable on aerial photographs and in the field. Its major purpose is to facilitate the assessment of streams for engineering purposes, with particular regard to lateral stability. It is based on two premises: (1) lateral stability, as well as several other aspects of stream behavior, is reflected in the physical appearance of the stream and its channel and (2) the best guide to the future behavior of a stream, during the life span of an engineering structure, is its behavior during the immediate past. The behavior of a stream is defined as changes in its position or its form properties with time. Form properties include channel width, sinuosity, meander form, bars, and islands. Aggradation and degradation, although important aspects of river behavior for engineering purposes, are difficult to assess from the physical appearance of a stream except in rather obvious cases, such as migrating scarps ("headcuts") in the channel (Site 277). They are best assessed from historical changes of bed elevation in relation to fixed structures or datum planes.

Each of the 14 properties listed in the left column of figure 26 could be used as the basis of a valid stream classification. For example, classification of streams as small, medium, or large is useful for some limited purposes, and classification as alluvial or non-alluvial is useful for others. The most widely used classification of streams is based on two properties (sinuosity and braiding) and streams are classified according to it as straight, meandering, or braided. This classification has limited usefulness, and it neglects the fact that many braided streams are straight, and many meandering streams are braided. If all 14 properties of figure 26 (and the categories shown for each) were incorporated into a single classification, several thousand stream types would be generated, even with the deletion of improbable types. Two ways of solving this dilemma will be used here: (1) The engineering significance of each property will be discussed, insofar as it applies when considered alone. (2) The most common stream types represent a characteristic association of properties; these common types will be described, and their engineering significance will be discussed.

Stream Size

Definition and measurement--The size of a stream can be indicated by its discharge, by its drainage area, or by some measure of its channel dimensions, such as width or cross-sectional area. No single measure of size is satisfactory because of the diversity of stream types. In the case histories, the measures of size given are bankfull discharge (where available), drainage area, and width. For purposes of stream classification (fig. 26), bank-to-bank channel width is chosen as the most generally useful measure of size, and streams are arbitrarily divided into three size categories on the basis of width.

Along some rivers, bank-to-bank width is difficult to define for purposes of measurement because one of the banks is indefinite. This is particularly true at bends, where the outside bank is likely to be vertical and sharply defined by the inside bank slopes gradually up to flood-plain level. The position of the line of permanent vegetation on the inside bank is the best available indicator of the bankline, and it tends to be rather sharply defined along many rivers in humid regions. The width of a stream is measured along

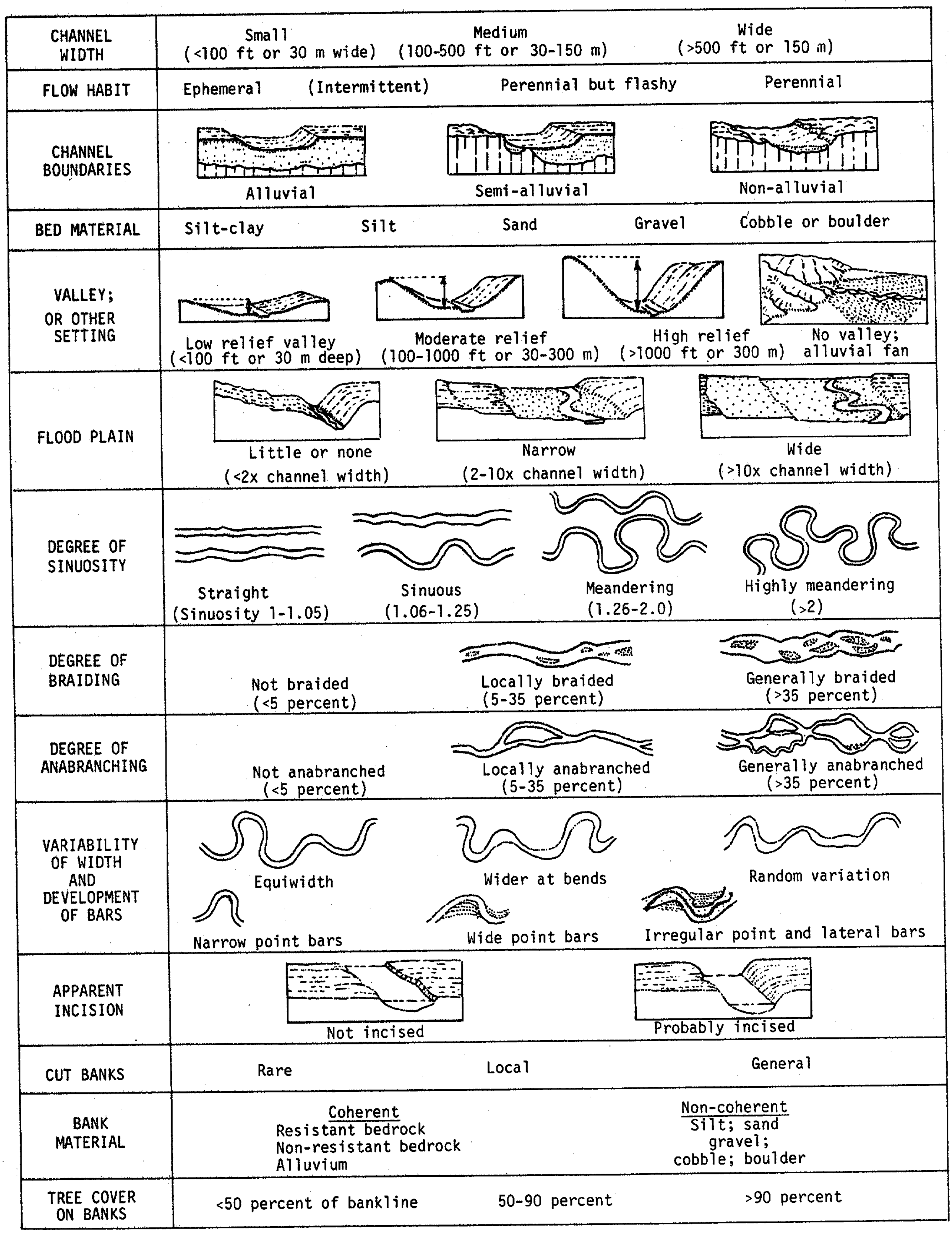

Figure 26. Stream properties for classification and stability assessment.

a perpendicular drawn between its opposing banks, which are defined either by their form or as the riverward edge of a line of permanent vegetation. For sinuous or meandering streams, width is measured at straight reaches or at the inflections between bends, where it tends to be most consistent. For multiple channel streams, width is the sum of the widths of individual unvegetated channels.

Because the width of a stream (as well as other aspects of its appearance) changes with river stage, all streams would ideally be measured and described at a corresponding stage, that is, at a stage corresponding to a particular point on their flow duration curves. For representing a stream on a topographic map, the Topographic Division of the Geological Survey uses, in so far as possible, the so-called "normal" stage, or the stage prevailing during the greater part of the year. They find that the "normal" stage for a perennial river usually corresponds to the water level filling the channel to the line of permanent vegetation along its banks. For this report also, stream descriptions apply to the "normal" stage.

In table 13, streams at the study sites are divided into six size categories according to drainage area, and three categories according to width. The study includes few streams of small size, of the sort that are commonly called "creeks" and also few major rivers, such as the Mississippi, Columbia, and Missouri Rivers. Most of the streams are rivers of moderate size.

Engineering significance--The depth of a stream tends to increase with size, and potential for scour tends to increase with depth of flow. On major rivers, the potential for deep scour requires the most thorough attention to scour depths and pier design, and extensive scour protection is commonly provided for piers, as at Sites 203 and 228. Each increment of increased footing depth, provided as a safeguard against scour, adds greatly to the cost. On streams of small or moderate size, added increments of foundation depth may represent a substantially smaller percentage of total bridge cost.

Although accurate determination of scour depths is not yet possible, the potential for deep scour on large rivers seems to be fully appreciated by bridge engineers. On the other hand, the potential for lateral erosion by large rivers seems to be less fully appreciated. At several of the study sites, piers originally placed well landward from the main channel are now within it (see, for example, Sites 170, 228, and 238). Not much information has been assembled on lateral migration rates for streams, but the potential for lateral erosion increases with stream size. For example, a major river like the Mississippi in its lower course, where the width is about 5,000 ft (1,525 m) may shift laterally by 100 ft (30.5 m) or more during a single major flood. A river of moderate size like the Sacramento in its middle course, where the width is about 1,000 ft (305 m), is unlikely to shift more than 25 ft (8 m) during a single flood, even though its migration rates are relatively high. A still smaller stream, whose width is about 100 ft (30.5 m) is unlikely to shift more than 10 ft (3 m) or so.

Table 13. Drainage area and channel width of streams at study sites.

DRAINAGE AREA

Less than 10 mi^2 (26 km^2)

Sites 2, 45, 166, 176, 199, 254, 266, 270, 277.

10-100 mi^2 (26-259 km^2)

Sites 5, 6, 8, 10, 12, 13, 20, 31, 32, 33, 36, 48, 53, 54, 56, 60, 61, 70, 74, 100, 125, 132, 155, 168, 183, 193, 194, 198, 206, 208, 216, 220, 224, 227, 233, 237, 245, 246, 248, 252, 253, 267, 272, 273, 274, 278.

101-1000 mi^2 (259-2590 km^2)

Sites 3, 7, 9, 16, 23, 25, 26, 27, 29, 34, 37, 38, 42, 43, 46, 47, 52, 55, 57, 58, 60, 68, 71, 75, 92, 127, 130, 156, 157, 159, 163, 165, 174, 177, 180, 181, 182, 192, 195, 196, 197, 200, 209, 211, 212, 213, 214, 217, 222, 236, 242, 263, 264, 265, 276, 280, 281.

1001-10,000 mi^2 (2591-25,900 km^2)

Sites 1, 11, 17, 24, 30, 41, 49, 50, 73, 88, 90, 91, 101, 115, 116, 122, 138, 141, 150, 152, 153, 158, 172, 185, 207, 210, 215, 218, 219, 223, 225, 226, 234, 235, 239, 241, 243, 261, 262, 268, 275, 279, 282.

10,001 - 100,000 mi^2 (25,901 - 259,000 km^2)

Sites 4, 21, 28, 35, 44, 51, 63, 103, 124, 131, 133, 139, 169, 170, 171, 184, 188, 204, 205, 238, 244, 247.

Greater than 100,000 mi^2 (2,590,000 km^2)

Site 228.

CHANNEL WIDTH

Less than 100 ft (30.5 m)

Sites 2, 3, 8, 13, 14, 20, 29, 31, 32, 34, 36, 47, 53, 70, 100, 125, 132, 134, 165, 166, 176, 182, 183, 194, 199, 206, 208, 211, 212, 213, 214, 216, 217, 222, 224, 235 245, 254, 260, 261, 263, 274.

100-500 ft (30.5 - 152 m)

Sites 4, 11, 12, 15, 16, 26, 27, 30, 33, 37, 41, 42, 43, 49, 52, 54, 55, 61, 68, 69, 74, 85, 90, 91, 92, 101, 115, 116, 120, 123, 126, 133, 138 141, 143, 148, 153, 157, 159, 160, 162, 163, 173, 175, 177, 178, 179, 180, 181, 185, 186, 193, 197, 200, 204, 207, 209, 210, 215, 219, 220, 223, 225, 226, 227, 234, 236, 239, 241, 242, 246, 250, 252, 253, 257, 262, 264, 265, 266, 270, 275, 280, 281, 282.

Greater than 500 ft (152 m)

Sites 1, 7, 17, 21, 24, 35, 44, 51, 57, 58, 63, 71, 73, 75, 103, 122, 124, 131, 140, 150, 151, 152, 154, 156, 169, 170, 171, 174, 188, 205 228, 238, 243, 244, 247, 259, 267, 279.

Except for the observation that the potential for lateral migration increases with stream size, no useful generalizations about lateral migration rates are now feasible. Rates are likely to be highest at the bends of meandering rivers, but all bends do not migrate at the same rate. Also, a particular bend may remain stable for a period of time, and then begin to migrate. The best method for estimation of lateral migration rate at a crossing site is to compare aerial photographs taken at different times in the past with the most recent photographs.

A correlation between stream size, as expressed by average discharge, and stream slope has been shown by Lane (1957) for several types of streams. In general, slope decreases as the fourth root of average discharge, such that large streams have lower slopes than small streams. Because of its correlation with stream size, slope was not included in fig. 4.1 as a geomorphic property, although values for valley slope are given in the case histories. For geomorphic purposes, valley slope is considered a more fundamental measure than channel slope, because channel slope varies with sinuosity and appears to be dependent on valley slope.

Flow Habit

A perennial stream is defined here as one that flows continuously for all or most of the year, and an ephemeral stream as one that does not flow continuously for most of the year. In the strict sense, an ephemeral stream flows only briefly and in direct response to precipitation, but the term as used here includes intermittent streams. A perennial but flashy stream is characterized by rapid changes in stage response to precipitation. As indicated in table 13, most of the streams at study sites are perennial.

In humid regions, the channels of ephemeral streams are likely to be small and to pose no particular problems in bridge and countermeasure design. In arid regions, however, ephemeral channels may be several hundred yards (meters) in width and they may pose difficult problems. Among these problems are determination of stage and discharge of the design flood; estimation of depth of scour; indeterminate position of the thalweg; and the probability of channel degradation by headcutting. Countermeasures for a variety of hydraulic problems at ephemeral channels are described at the study sites listed in table 14.

Table 14. Flow habit of streams at study sites.

Ephemeral
Sites 1, 5, 6, 7, 10, 12, 13, 54, 55, 56, 57, 58, 60, 71, 75, 89, 132, 134, 140, 141, 144, 277.

Perennial but flashy
Sites 11, 36, 45, 61, 63, 85, 100, 158, 194, 199, 242, 245, 246, 254.

Perennial
(all other sites)

Channel Boundaries

Although no precise definitions can be given for alluvial, semi-alluvial, or non-alluvial streams, some distinction among streams with regard to the erosional resistance of the earth material in their channel boundaries is needed. In geology, bedrock is distinguished from alluvium and other surficial materials mainly on the basis of its age, rather than on its resistance to erosion. A compact alluvial clay is likely to be more resistant than a weakly cemented sandstone that is much older. Nevertheless, the term "bedrock" does carry a connotation of greater resistance to erosion and it is used here in that sense. An alluvial channel is in alluvium, a non-alluvial channel is in bedrock, and a semi-alluvial channel has both bedrock and alluvium in its boundaries. The bedrock of non-alluvial channels may be wholly or partly covered with sediment at low stages, but likely to be exposed by scour during floods.

Table 15. Channel boundaries of streams at study sites.

Non-alluvial
Sites 3, 23, 138, 177, 264.

Semi-alluvial
Sites 2, 8, 11, 24, 34, 46, 54, 68, 69, 89, 97, 154, 156, 158, 182, 205, 217, 219, 265, 267, 272, 278.

Alluvial
(all other sites)

As shown in table 15, most of the streams at study sites are alluvial. This is partly a reflection of the fact that most streams large enough to be crossed by bridges are alluvial, but it is also an indication that bridges on non-alluvial streams are less subject to hydraulic problems. Few hydraulic problems are reported from areas where bedrock is at or near the surface. In Missouri, for example, nearly all the problems reported are from the parts of the state that are mantled with glacial drift, loess, or alluvium; and few problems are reported from the rocky Ozark Region.

Nevertheless, the security of foundations in bedrock depends on the quality of the rock and the care with which the foundations are set. At the US-14 crossing of Lance Creek, west of Pierre, South Dakota, the channel degraded into shale bedrock and caused exposure of pier footings. At White Oak Creek, in a bedrock area of southern Ohio, a pier was undermined by erosion of interbedded limestone and shale. At Site 44, on the Susquehanna River in Pennsylvania, steeply dipping strata of shale have proved to be a poor foundation material for piers and have been eroded by high water velocities. At Site 11, on the Canadian River in New Mexico, the failure of piers set 0.5 ft (0.15 m) into bedrock is attributed to scour and lateral forces on the bridge during partial submergence. The bedrock is probably sandstone. Steel-pile bents for the replacement bridge were set in holes drilled 6 ft (1.8 m) into bedrock. According to Highway Research Board (1970, p. 18) serious problems and failures have been encountered with piers founded on erodible shale, sandstone, and other erodible rocks. In the present study, however, only the problems mentioned above were found.

Table 16. Bed material size of streams at study sites.

Silt-clay
Sites 20, 53, 173, 188, 199, 200, 245, 246, 260, 263, 269, 274, 277.

Sand
Sites 4, 10, 15, 17, 25, 29, 32, 41, 45, 47, 48, 50, 51, 52, 55, 60, 70, 71, 73, 85, 88, 101, 103, 115, 116, 120, 130, 131, 133, 141, 143, 145, 156, 164, 169, 170, 171, 174, 175, 178, 179, 180, 181, 183, 185, 186, 192, 193, 194, 195, 196, 201, 205, 207, 209, 210, 212, 213, 216, 218, 222, 223, 225, 226, 227, 228, 239, 241, 242, 243, 244, 247, 257, 259, 261, 262, 268, 275, 280, 281, 282.

Gravel
Sites 1, 5, 6, 7, 11, 12, 16, 21, 26, 28, 31, 37, 38, 42, 44, 46, 54, 56, 57, 61, 68, 69, 90, 91, 92, 95, 99, 122, 123, 124, 125, 126, 127, 132, 134, 138, 144, 148, 150, 151, 152, 155, 157, 158, 163, 165, 168, 176, 177, 182, 197, 198, 204, 220, 224, 233, 235, 248, 252, 253, 254, 265, 267, 270, 272, 273, 276, 279.

Cobble or boulder
Sites 2, 3, 8, 9, 13, 14, 23, 24, 27, 30, 33, 34, 35, 36, 43, 74, 89, 97, 140, 153, 236, 264, 266.

Bed Material

Streams are classified, according to the dominant size of the sediment on their beds, as silt-clay bed, sand bed, gravel bed, and cobble or boulder bed. Accurate determination of the particle size distribution of bed material requires careful sampling and analysis, particularly for coarse bed material (Kellerhals and Bray, 1971). Most of the bed material designations for streams given here are only rough approximations, based on visual observation. Although not suitable for quantitative purposes, such as calculation of scour depths from formulas, they are useful for qualitative description. For some sites, we were unable to obtain information on bed material. As shown in table 16, most of the streams at study sites have been designated as either sand bed or gravel bed, although silt-clay and cobble-boulder streams are adequately represented. Many of the stream beds designated as gravel also have abundant sand, but gravel is the dominant component.

No relation was found between bed material size and the incidence of scour problems at piers, abutments, or embankments at study sites. Of 76 sites where scour problems were identified (table 1), 5 occurred at streams with silt-clay beds, 27 at streams with sand beds, 33 at streams with gravel beds, and 11 at streams with cobble-boulder beds. For each kind of stream bed, the number of problem sites is roughly half of the total number of sites in the category. Neill (1964, p. 14), among others, has shown that particle size has only a small effect on the depth of scour produced by vortex and wake action around piers.

The greatest <u>depths</u> of scour at study sites, however, were reported for streams having sand or sand-silt beds. For example, general scour to a depth of 28 ft (8.5 m) is documented at Site 70 on Big Running Water Creek, Arkansas; scour to about 40 ft (12 m) at Site 103, Rio Grande, New Mexico; and scour to about 100 ft (30.5 m) at Site 203 downstream from the I-10 bridge on the Atchafalaya River (Whiskey Bay Pilot Channel) in Louisiana. Anderson (1966) has described a scour hole 40 ft (12 m) in depth downstream from the failed I-29 bridge on the Big Sioux River, where the bed material is silt-clay. On the other hand, Holmes (1974) reports scour depths up to 41 ft (12.5 m) in "shingle", from his useful study of 36 railroad bridges in New Zealand where scour had occurred. ("Shingle" is usually defined as rounded stones ranging in diameter from 20 to 200 mm, or 1 to 8 in). Holmes found no obvious relation between depth of scour and size of bed material. Most of the streams in Holmes' study had steep channel slopes, in the range of 0.0035 to 0.0080. But scour to a probable depth of 15 ft (4.6 m) is reported at a crossing on the cobble-bed Chemung River in Pennsylvania, where the slope is 0.0007. (Site 30).

The general conclusion from the present study is that scour problems are as common on streams having coarse bed material as on streams having fine bed material. However, very deep scour is more probable in fine bed material.

Table 17. Valley relief, or other setting, of streams at study sites.

<u>Low relief valley (relief less than 100 ft, or 30.5 m)</u>
Sites 15, 25, 29, 31, 34, 41, 45, 47, 48, 49, 52, 53, 70, 85, 88, 116, 131, 133, 134, 143, 145, 148, 151, 152, 159, 160, 162, 165, 169, 170, 171, 179, 185, 192, 193, 195, 196, 197, 198, 199, 200, 212, 213, 216, 217, 218, 222, 227, 228, 236, 246, 248, 257, 263, 268, 277.

<u>High relief valley (relief greater than 1000 ft or 305 m)</u>
Sites 3, 9, 13, 23, 26, 28, 36, 74, 103, 125, 127, 138, 139, 154, 156, 158, 182, 264, 266, 267, 272, 273, 279.

<u>No valley, stream on alluvial fan or piedment slope</u>
Sites 1, 5, 6, 7, 12, 16, 27, 75, 123, 233.

<u>Moderate relief valley (relief 100-1000 ft, or 30.5 - 305 m)</u>
(all other sites)

Valley or Other Setting

For streams in valleys, valley relief is used as a means of indicating whether the surrounding terrain is generally flat, hilly, or mountainous. For a particular site, relief is measured (usually on a topographic map) from the valley bottom to the top of the highest adjacent divide. Relief greater than 1000 ft (or about 300 m) is regarded as mountainous, and relief in the range of 100-1000 ft (30-300 m) as hilly. Streams in mountainous regions are likely to have steep slopes and coarse bed material. In many regions, channel slope increases with increase in steepness of valley side slopes.

As shown in table 17, 23 of the study sites are in mountainous terrain. All of these have beds of coarse material (gravel or cobble-boulder), and no specific hydraulic problems are associated with bridges on them, beyond those discussed in connection with bed material. Streams in regions of lower relief are more likely to be troublesome, because of more rapid lateral erosion.

Streams on alluvial fans or on piedmont slopes in arid regions pose special problems. A piedmont slope is a broad slope along a mountain front, and streams issuing from the mountain front may have shifting courses and poorly defined channels, as on an alluvial fan. Countermeasures for these problems are described at the study sites indicated in table 17.

Flood Plain

A flood plain is described in the Glossary as a nearly flat alluvial lowland bordering a stream, formed by stream processes, that is subject to inundation by floods. Many geomorphologists prefer to define a flood plain as the surface presently under construction by the stream, which is flooded with a frequency of about 1.5 years. According to their definition, surfaces flooded less frequently are terraces, abandoned flood plains, or "flood-prone areas". However, surfaces flooded with a frequency of 50 to 100 years are considered herein as part of the flood plain. Most engineers do not have the time or the inclination to study stream terraces, which may be difficult to distinguish from modern or active flood plains.

Streams having little or no flood plain may pose difficult problems with roadway or crossing location, particularly in mountainous regions. These problems are discussed in Calif. Div. of Highways (1970, p. 55-70), but are not considered here because no unique countermeasures are indicated. The use of spur dikes on flood-plain streams is discussed in the chapter on countermeasures.

In figure 26, flood plains are arbitrarily divided into three categories a according to their width, relative to channel width. Most of the study sites are on flood plains classed as wide according to this scheme (table 18).

Table 18. Width of flood plain at study sites.

Little or none (width less than 2 times channel width)
Sites 2, 3, 8, 11, 13, 23, 24, 29, 44, 54, 74, 89, 95, 134, 138, 139, 150, 153, 154, 156, 158, 182, 236, 254, 258, 259, 264, 266, 267, 273, 277, 278.

Narrow (width 2-10 times channel width)
Sites 9, 10, 14, 26, 30, 35, 42, 43, 51, 56, 58, 61, 69, 71, 73, 85, 92, 99, 103, 124, 125, 132, 143, 148, 151, 152, 155, 157, 175, 194, 205, 220, 234, 238, 243, 250, 252, 253, 257, 265, 270, 272, 275, 276.

Wide (width greater than 10 times channel width)
Sites 4, 15, 17, 20, 21, 25, 28, 31, 32, 33, 34, 37, 38, 41, 47, 48, 49, 50, 52, 53, 63, 68, 70, 88, 90, 91, 101, 115, 116, 120, 122, 126, 127, 131, 133, 145, 159, 160, 162, 163, 165, 166, 168, 169, 170, 171, 173, 174, 177, 178, 179, 180, 181, 183, 185, 186, 188, 192, 193, 195, 196, 197, 200, 204, 206, 207, 208, 209, 210, 211, 212, 213, 214, 215, 216, 217, 218, 219, 221, 222, 223, 225, 226, 227, 228, 235, 239, 241, 242, 244, 245, 246, 247, 248, 260, 261, 262, 279, 281, 282.

Sinuosity

Sinuosity is a dimensionless measure of the "crookedness" of a stream. It is the ratio of a stream reach as measured along its centerline, to length as measured along the valley centerline, or along a straight line connecting the ends of the reach. The valley centerline is preferable when the valley itself is curved. Straight stream reaches have a sinuosity of one, and the maximum value of sinuosity for natural streams is about four. For this study, dividers were used for measurement of stream length and valley length on maps or aerial photographs, as recommended by Water Resources Council (1968). Inasmuch as the sinuosity of a stream is rarely constant from one reach to the next, no very refined measurement of sinuosity is warranted. The four classes of sinuosity in figure 26 are arbitrary, for sinuosity is a continuous property.

Sinuosity can also be defined as the ratio of stream slope to valley slope. A straight stream, or one that directly follows the valley centerline, has the same slope as the valley; as the sinuosity of the stream increases, its slope decreases in direct proportion. Similarly, if a sinuous channel is straightened, the slope increases in direct proportion to the change in length.

Table 19. Sinuosity of streams at study sites.

Straight, altered
Sites 45, 52, 55, 134, 148, 176, 179, 200, 203, 212, 222, 242, 262.
Straight, not altered
Sites 12, 14, 51, 60, 89, 100, 132, 143, 150, 154, 158, 166, 243, 244, 246.
Sinuous
Sites 1, 2, 4, 8, 10, 13, 15, 16, 17, 21, 26, 27, 29, 30, 31, 32, 33, 34, 35, 36, 37, 38, 42, 43, 44, 53, 56, 57, 58, 61, 63, 68, 69, 70, 71, 73, 74, 75, 85, 90, 95, 97, 99, 103, 123, 124, 125, 126, 127, 138, 151, 152, 155, 159, 160, 162, 163, 165, 168, 178, 180, 182, 183, 185, 192, 193, 199, 205, 208, 219, 224, 227, 233, 236, 238, 241, 246, 247, 252, 253, 254, 258, 259, 263, 264, 265, 268, 270, 272, 273, 275, 279, 282.
Meandering
Sites 20, 24, 28, 41, 47, 48, 49, 50, 88, 91, 101, 115, ,20, 130, 131, 133, 153, 157, 169, 170, 171, 173, 174, 175, 181, 188, 194, 195, 196, 197, 198, 204, 207, 209, 210, 213, 214, 215, 216, 217, 220, 223, 225, 226, 228, 234, 235, 239, 245, 248, 257, 261, 274, 276, 277, 278, 280, 281.
Highly meandering
Sites 186, 206, 211, 218, 250, 260.

The size, form, and regularity of meander loops is an aspect of sinuosity. Symmetrical meander loops are not very common, and a sequence of two or three identical symmetrical loops is still less common. In addition, meander loops are rarely of uniform size. The largest is commonly about twice the diameter of the smallest, and the size-frequency distribution of loop radii tends to be normal. Some geomorphologists have classified meander patterns according to form, applying such terms as regular, irregular, and tortuous, but no relation of meander form alone to lateral stability has been demonstrated. Small kinks in the meander pattern of an alluvial stream may indicate local instability, as noted by Kellerhals, Neill, and Bray (1972).

As indicated in table 19, most of the stream reaches at study sites are either sinuous or meandering. Reaches that are straight for distances greater than about 10 channel widths are not common in nature, and many of the straight reaches at study sites have been artificially straightened. Most highly meandering streams are of small or medium size, and they tend to occur where banks have a rather high degree of resistance, due either to clay content or vegetal cover. Most streams of high sinuosity have sand or silt-clay beds, but gravel-bed streams of high sinuosity are common, particularly in Alaska.

There is little relation between degree of sinuosity, as considered apart from other properties, and lateral stream stability. A highly meandering stream may have a lower rate of lateral migration than a sinuous stream of similar size. Assessment of stability is based mainly on additional properties, especially on bar development and the variability of channel width. However, many hydraulic problems are associated with the location of crossings at a meander or bend. These include the shift of flow direction at flood stage, shift of thalweg toward piers or abutments, and lateral channel erosion at piers or abutments.

For the planning of new crossings and of countermeasures at existing crossings, it is very useful to have some insight into not only the probable rate of meander migration, but also the probable way that the meander loop will migrate or develop. No two meanders will behave in exactly the same way, but the meanders on a particular stream reach tend to conform to one or another of several modes of behavior. Both the probable rate and mode of behavior are best assessed by study of a particular meander loop, or pair of loops, as shown on aerial photographs taken over a period of at least 10 years, and preferably 20 or 30 years.

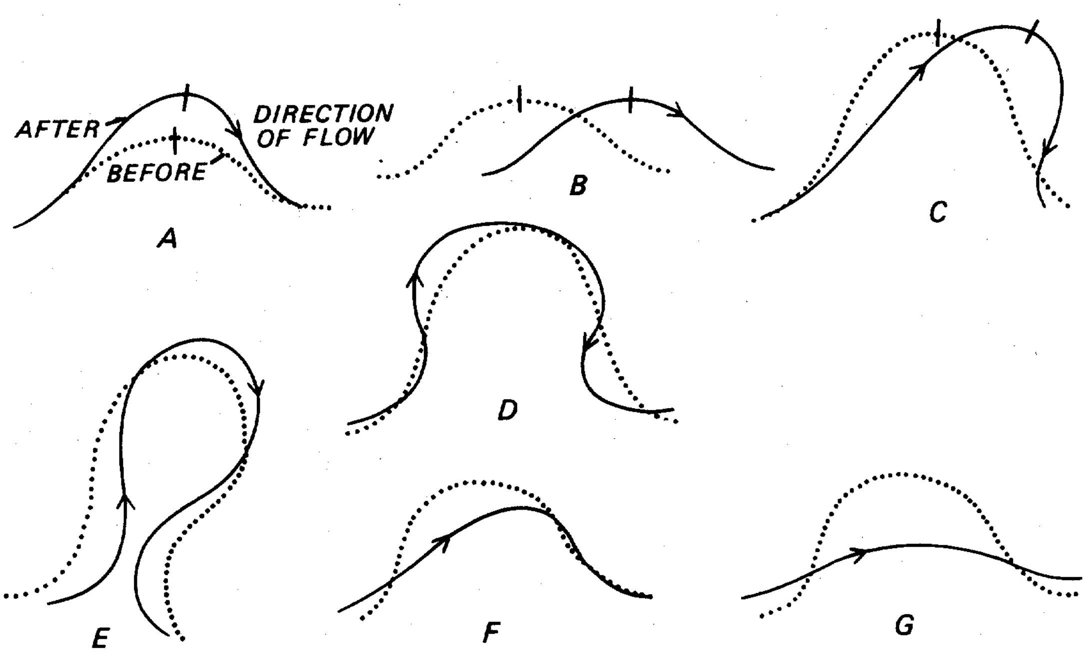

Figure 27. Modes of meander loop behavior. A, Extension. B, Translation. C. Rotation. D, Conversion to a compound loop. E, Neck cutoff by closure. F, Diagonal cutoff by chute. G, Neck cutoff by chute. (From Brice, 1977, p. 38.)

A meander is likely to follow one or another of the modes of behavior illustrated in figure 27, which is based on a study of about 200 sinuous or meandering stream reaches. Mode A represents the typical development, on many streams, of a loop of low amplitude, which decreases in radius as it extends slightly in a downstream direction. Mode B rarely occurs unless meanders are confined by valley sides on a narrow flood plain, or are confined by artificial levees. It may also occur on steep, gravel-bed streams. Well-developed meanders on streams that have moderately unstable banks are likely to follow Mode C. Mode D applies mainly to large loops on meandering or highly meandering streams. The meander has become too large in relation to stream size and regimen, and secondary meanders develop along it, converting it to a compound loop. Mode E also applies to meandering or highly meandering streams. The banks have been sufficiently stable for an elongated loop to form (without being cut off), but the neck of the loop is gradually being closed and cutoff will eventually occur at the neck. Modes F and G apply mainly to locally braided sinuous or meandering streams having unstable banks. Loops are cut off by chutes that break diagonally or directly across the neck.

The most rapid bank erosion is predictably at the outside of meanders, downstream from the apex of the loop. Parsons (1960) has developed a method for predicting the point of maximum erosion. This method has been found useful by Soil Conservation Service engineers in Mississippi, but it has not been tested for the present study. Some random factors seem to be involved in the migration of meanders, and the exact rate or place of erosion is probably not predictable.

The cutoff of a meander, whether done artificially or naturally, causes a local increase in channel slope and a more rapid growth rate of adjoining meanders. On the Marias River in Montana (Site 234), an increase in lateral erosion rate at the crossing is attributed to the natural cutoff of a large upstream meander. Comparison of sequential aerial photographs (Brice, 1974, p. 191) indicates that the effects of the cutoff do not extend very far upstream or downstream, for example, no farther than the next two or three loops. Adjustment of the channel to increase in slope seems to be largely accomplished by increase in channel width (wetted perimeter) at and near the point of cutoff.

Some generalizations can be made, from general knowledge of stream behavior, about the probable consequences of controlling or halting the development of a meander loop by the use of countermeasures. None of the study sites were analyzed for this purpose, and we received no reports of litigation. From his study of streams in Oklahoma, Keeley (1971, p. 275) suggested that control of erosion at a particular concave bank would increase the rate of erosion at the next (unprotected) concave bank downstream. This idea is apparently supported by the hypothesis that, if the river is deprived of sediment load by an erosion countermeasure at the first bank, it will make up for this by increased erosion at the second. However, it seems unlikely that any significant part of the sediment load of a stream is derived from erosion at one concave bank. A more probable consequence relates to change in flow alinement (or lack of change, if the position of a naturally eroding bank is held constant). The development of a meander is affected by the alinement of the flow that enters it. Any artificial influence on flow alinement is likely to affect meander form.

Downstream bank erosion rates are not likely to be increased, but the points at which bank erosion occurs are likely to be changed. In the case where flow is deflected directly at a bank, an increase in erosion rates would be expected.

Braiding

A braided stream is one whose flow is divided at normal stage by mid-channel bars or small islands. An island differs from a bar in that it is covered with permanent vegetation. A typical braided stream has the aspect of a single large channel, usually with fairly well-defined banks, within which there are subordinate channels, called "braids". The aspect of a braided stream changes with water stage, as bars are exposed or covered with water; therefore, a particular stage, the normal stage, is specified for consistency in description. As with other stream properties, a description of braiding can apply only to a single homogeneous reach.

Streams have all degrees of braiding. Some have only an occasional bar or island from place to place along their course, and others have very many within a short reach. Degree of braiding is indicated, rather crudely, by the percent of reach length that is divided by bars or islands. A reach is described as locally braided if 5 to 35 percent of reach length is divided by bars or islands, and as generally braided if more than 35 percent is divided. A generally braided stream that has a complex braided pattern, such that more than one bar or island is likely to be traversed by any line across the stream, can be described as fully braided. However, this designation was not used in table 20.

Table 20. Degree of braiding of streams at study sites.

Generally braided

Sites 5, 6, 12, 27, 51, 54, 56, 58, 60, 63, 71, 73, 75, 85, 103, 140, 150, 154, 158, 243, 244, 247, 258, 259, 266, 267, 275, 279.

Locally braided

Sites 1, 4, 7, 9, 10, 11, 16, 17, 21, 24, 26, 30, 35, 36, 37, 42, 43, 44, 57, 61, 68, 74, 88, 90, 92, 99, 115, 120, 122, 125, 126, 132, 133, 151, 152, 153, 155, 163, 169, 170, 171, 175, 177, 205, 228, 233, 234, 235, 236, 238, 239, 241, 253.

Braiding of streams at study sites in indicated in table 20. About 12 percent of the streams are generally braided; about 23 percent, locally braided; and about 65 percent, not braided. These percentages probably approximate the incidence of braiding among streams in general. However, braided streams tend to be common in the arid and semi-arid parts of western U.S. and in regions having active glaciers, such as parts of Alaska and the Canadian Rockies. They are uncommon east of the Mississippi River. The upper Mississippi River and the Illinois River have an exceptional kind of braiding, characterized by narrow wooded islands, which are apparently related to post-glacial infilling of their valleys. Generalizations given here about braiding do not apply to these rivers.

In comparison with an unbraided stream of the same discharge, a braided stream tends to have a wider and more shallow cross section, a steeper slope, and a higher ratio of bed load to suspended load. Braiding occurs in sand-bed streams--the Platte and Loup Rivers of Nebraska are well-known examples--but it is more common in gravel-bed streams.

Braided streams require long bridges if the full channel width is crossed, but effective flow-control measures for constriction of the channel have been developed in Alberta, Saskatchewan, Alaska, Nebraska, and California, as described in the case histories. The banks are likely to be easily erodible, and unusual care must be taken to prevent lateral erosion at or near abutments. In Canada, guide banks (spur dikes) are used at abutments (Sites 150 and 259), and in Nebraska, abutments may be protected with steel sheet piling (Sites 244, 245, and 247). The position of braids is likely to shift during floods, resulting in unexpected velocities and depths of flow at individual piers. However, measurements of scour at bridge piers in Alaskan braided streams do not indicate any unexpected depths. Lateral migration of braided streams takes place by lateral shift of a braid against the bank, but available information indicates that lateral migration rates are generally less than for meandering streams. Along braided streams, however, migration is not confined to the outside of bends but can take place at any point by the lateral shift of individual braids. The banks of braided streams are typically scalloped by this process. Of the 11 study sites where shift of the thalweg was particularly noted as a factor in hydraulic problems, 4 were on generally braided streams (Sites 56, 243, 266, and 267) and 3 were on locally braided streams (Sites 17, 153, and 233).

Locally braided streams share the characteristics described for braided streams, but to a lesser degree. Development of an isolated bar near the bridge may change flow alinement and cause scour at the bridge, as at Site 153. Also, local braiding carries an implication of erodible, and hence unstable, banks. Of 76 study sites where scour problems were identified, the streams were locally braided at 21 sites. On the other hand, local braiding, like general braiding, suggests a relatively high transport of bedload, which is important for the success of many flow-control countermeasures.

Some braided streams are aggrading, but most are evidently not aggrading. The possibility of aggradation on a braided stream is best assessed from historical changes in bed elevation.

Anabranching

An anabranched stream differs from a braided stream in that the flow is divided by islands rather than bars, and the islands are large in relation to channel width. This usage of the term "anabranch" is after Dury (1969). The anabranches, or individual channels, are more widely and distinctly separated and more fixed in position than the braids of a braided stream. An anabranch does not necessarily transmit flow at normal stage, but it is an active and well-defined channel, not blocked by vegetation. Although the distinction between braiding and anabranching may seem academic, it has real significance for engineering purposes. Neill (1973, p. 16) and Mollard (1973, p. 18) distinguish anabranching streams for engineering purposes, but Neill calls them "wandering" and Mollard, "anastomosing". The term "split channel" is applied by some Canadian engineers. In the U.S., individual anabranches are sometimes called "bypass channels" or "overflow channels". Streams exhibit all degrees of anabranching, and arbitrary categories are delineated here (fig. 26) in the same way as previously described for degrees of braiding. As indicated in table 21, anabranching was discerned at only a small percentage of the streams at study sites.

Anabranching is less common than braiding, but it occurs in a wide range of geographic situations, each of which produces a rather different type of anabranching. The types will be mentioned only briefly here, since the engineering problems associated with them are similar. The most distinctive type, which is illustrated in figure 26, is best developed in the valleys of mountainous or cold regions. Examples are the Tanana River downstream from Fairbanks, the Yellowstone River along much of its course in Montana, the Klickitat River in Washington at Site 138, and the Red Deer River in Alberta at Site 151. A second type occurs on alluvial fans or piedmont slopes in dry climates, where distributaries divide and rejoin. Examples are Salmon Creek, California (Site 5) and Brawley Wash, Arizona, (Site 57). A third type occurs on wide flood plains in warm rainy climates. Here anabranching is evidently associated with tributaries that cross the flood plain and flow nearly parallel with the main channel before entering it. During sustained floods, flow breaks across the divide, and the lower end of the tributary becomes an anabranch of the main channel. Examples are the Cossatot River in Arkansas (Site 162) and the Tickfaw River in Louisiana (Site 196). Yet a fourth type, represented in the study sites only by the Pearl River in Louisiana (Site 49) occurs on deltas or deltaic plains where distributaries divide and rejoin, in a manner analogous to that on alluvial fans.

Table 21. Degree of anabranching of streams at study sites.

<u>Generally anabranched</u>
Sites 5, 6, 7, 90, 151, 157, 162, 197, 279.

<u>Locally anabranched</u>
Sites 9, 15, 34, 38, 46, 47, 49, 57, 68, 88, 91, 126, 138, 152, 155, 163, 196, 236, 265, 270.

Problems associated with crossings on anabranched streams can be avoided if a site where the channel is not anabranched can be chosen. If not, the designer may be faced with a choice of either building more than one bridge, building a long bridge, or diverting anabranches into a single channel. Inasmuch as anabranches are permanent channels that may transmit a substantial amount of flow, diversion and confinement of an anabranched stream is likely to be more difficult than for a braided stream. Measures that have been used for confinement are spur dikes, dikes, and spurs. See, for example, Sites 5, 6,7,57, and 151. Such measures are subject to lateral erosion by an active anabranch (Site 151) and must be designed to avoid outflanking of the bridge by an anabranch. Problems with flow alinement may occur if a bridge is built at or near the junction of anabranches, as at Sites 163 and 196. Where anabranches are crossed by separate bridges, as at Site 138, the design discharge for the bridges may be difficult to estimate. If one anabranch should become partly blocked, as by floating debris or ice, an unexpected amount of flow may be diverted to the other.

Variability of Width and Development of Bars

For single channels (excluding multiple-channel anabranched or braided reaches), variability of unvegetated channel width is a useful indication of lateral stability or instability of the channel. The visual impression of unvegetated channel width on aerial photographs depends on the relatively dark tones of vegetation bordering the channel, as contrasted with the lighter tones of exposed sediment or of water. Under some lighting conditions, water appears dark but because of the smoothness of the surface there is usually no problem in distinguishing it from vegetation. A channel is considered to be of uniform width (equiwidth) if the average width of the unvegetated channel as observed at the widest places (which are usually at bends) is not greater than about 1.5 times the average width as observed at the narrowest places. Natural channels that have no bordering vegetation are rare, and this means of assessing lateral stability cannot be applied to them.

The relation between width variability and lateral stability depends on the development of point bars and lateral bars. A point bar forms on the inside or convex bank of a bend, usually somewhat downstream from the apex of the bend, and it grows channelward as the opposite or concave bank is eroded. A lateral bar forms along the side of a channel at some place other than a bend. If the concave bank at a bend is eroding slowly, the point bar will grow slowly, and permanent vegetation will become established on it. The unvegetated part of the point bar will appear as a narrow crescent. If, on the other hand, the bank opposite the point bar is eroding rapidly, the bar will grow more rapidly than permanent vegetation can become established on it, and it will be wide and conspicuous. A point bar whose unvegetated width exceeds the width of flowing water at the bend is considered to be wider than average.

If the bare point bars in a reach tend to exceed average width, lateral erosion rates are probably high enough to cause hydraulic problems at the crossing. Because the exposed width of a point bar changes with river stage, aerial photographs taken at either high or low river stages are not suitable for this assessment. Also, the establishment of vegetation on a point bar may depend on factors besides rate of bar growth, for example, on climate and the timing and intensity of floods. For streams in rainy southern climates, in particular, vegetation may become established rapidly, and cut banks at the outside of bends may be more reliable than point bars as an indication of instability.

In reaches of low sinuosity, or in very large meander loops, a lateral bar may form at the inside of a curve in the thalweg. Such a bar may be the precursor of a point bar and a developing bend. Careful attention should be paid to such lateral bars in the design of crossings and countermeasures. At Site 228 on the Red River, for example, a lateral bar caused erosion at the inside bank of a large bend.

Three categories of width variability are distinguished, but the relative lateral stability of these must be assessed in connection with bar development and other properties. In general, equiwidth streams having narrow point bars are the most stable laterally, and random-width streams having wide, irregular point bars are the least stable. Vertical stability, or the tendency to scour, cannot be assessed from these properties. Scour may occur in any alluvial channel. In fact, the greatest potential for deep scour might be expected in laterally stable equiwidth channels, which tend to have relatively deep and narrow cross sections and bed material in the size range of silt and sand.

Equiwidth streams--The stability of equiwidth streams is due either to a continuous cover of permanent vegetation along the banks, to coherent bank materials, or to both factors. Good examples are the Qu'Appelle River in Saskatchewan (Site 260), the Noxubee in Mississippi (Site 217), and the Obion River in Tennessee (prior to channelization; see Site 262). The Red River of the North, which forms the border between Minnesota and North Dakota, is particularly stable because it is incised into coherent material, although slumping of the banks from internal slope failure is a problem at some bridges. Stability will likely decrease if the stream is channelized or if the forested flood plain is cleared; both factors have caused instability and problems at bridges on the Obion and the Forked Deer River in Tennessee (Sites 261 and 262) and on many rivers in Mississippi.

Equiwidth streams with narrow point bars probably have a relatively low ratio of bedload to suspended load, by comparison with streams that have wide point bars or a braided pattern. However, there is too little information on the bedload of rivers for this to be well established.

Streams having well-developed natural levees tend to be equiwidth and to have low rates of lateral migration. A natural levee typically slopes gently away from the stream and is best observed during a flood, when its crest appears as an emergent strip along the stream. Well-developed natural levees usually occur along the lower courses of streams, or where the flood plain is submerged for several weeks or months a year. Once formed, the levees create basins (flood basins) between the river and the valley sides, from which water may drain but slowly. If the natural levee is deeply breached during a flood, the stream course may change in a new direction through the breach. Natural levees were detected at only two study sites (223 and 260), and no problems specifically connected with them were reported.

Streams wider at bends--As shown in table 22, streams wider at bends are more common at the study sites than other types. They tend to be laterally unstable, and the degree of instability is assessed from the development of point and lateral bars, and the presence of cut banks.

The Red River along the Arkansas-Texas state line, and in Louisiana, is wider at bends and has a rapid rate of lateral erosion, which has caused problems at many bridges. Examples are Sites 169, 170, 171, and 188. On recent aerial photographs of the Red River, the point bars are mostly vegetated because regulation by dams has reduced flood peaks. Flows near average discharge are sustained, and the flood of 50-yr recurrence interval is confined to the channel. The effect of regulation on the rate of lateral erosion is not yet clear, but erosion is continuing.

Of the sites where rapid lateral erosion was particularly noted as a factor in hydraulic problems, almost all were on streams classed as wider at bends with point bars. These include, besides the Red River sites mentioned above, Deer Creek in California (Site 16); Washita River in Oklahoma (Site 116); the Trinity and Brazos Rivers in Texas (Sites 130, 131, 145, 283); the Marias and Teton Rivers in Montana (Sites 234 and 235); the Buffalo River in Mississippi (Site 281); the Elkhorn River in Nebraska (Site 240); and the Grand River in South Dakota (Site 175).

Table 22. Width variability of streams at study sites.

Equiwidth
Sites 29, 30, 35, 49, 53, 69, 131, 173, 185, 206, 211, 214, 217, 218, 219, 223, 257, 260, 261, 262, 282.

Wider at bends
Sites 4, 16, 21, 26, 28, 33, 34, 101, 116, 125, 130, 145, 153, 157, 169, 170, 171, 174, 175, 180, 186, 188, 195, 204, 207, 208, 209, 210, 213, 215, 216, 226, 234, 235, 239, 240, 245, 248, 252, 280, 281, 283.

Random variation
Sites 1, 7, 38, 44, 63, 68, 73, 92, 115, 120, 133, 163, 177, 225, 228, 236, 265.

A high degree of lateral instability is represented by the Nodaway River in Missouri (Site 226) and the Nishnabotna in Iowa (Site 226). Both of these streams have been straightened in the past and have since become sinuous or meandering. The thalweg tends to wander somewhat within a wider channel, particularly on the Nodaway, and bars (point or lateral) tend to be almost continuous along the low-water channel. Aerial photographs of several unaltered streams, also unstable, having bars of this type are in Keeley (1971, p. 148 and 166); for example, Salt Fork of the Red River west of Altus, Oklahoma, and North Fork of the Red River west of Tipton, Oklahoma.

Random-width streams---Random width variation is related to a tendency toward either braiding or anabranching. On some streams, as on the Mississippi River along the Arkansas-Tennessee state line (Site 228), random width variation is evidently related to lack of uniformity in resistance of banks to erosion. On others, as on the Washita River in Oklahoma (Site 120), random width variation is evidently related to a wandering thalweg, which may widen the channel by lateral erosion at random places. A wandering thalweg not only contributes to lateral erosion but also to alinement problems at a bridge.

Random-width streams tend to be laterally unstable, and the degree of instability is assessed from the development of point and lateral bars. Where these bars are narrow and the channel boundaries are well vegetated, a low degree of instability is indicated. Where the lateral bars are prominent, point bars are irregular, and the thalweg is meandering, a high degree of instability is indicated as, for example, on the Washita River at Site 120. Other examples of unstable random-width streams among the study sites are Cache Creek in California (Site 1); the Salinas River in California (Site 73); and the Little Colorado River in Arizona (Site 63). The Cimarron River along much of its course in Oklahoma, as represented by several sites in Keeley (1971), is a striking example.

Apparent Incision

The apparent incision of a stream (or channel) is judged from the height of its banks at normal stage, relative to its width. For a stream whose width is about 100 ft (30.5 m), bank heights in the range of 6-10 ft (2-3 m) are about average, and higher banks indicate probable incision. For a stream whose width is about 1,000 ft (305 m), bank heights in the range of 10-15 ft (3-5 m) are about average, and higher banks indicate probable incision. Incised streams tend to be fixed in position and are not likely to bypass a bridge or to shift in alinement at a bridge. The lateral erosion rates are likely to be slow, except in the case of western arroyos with high, vertical, and clearly unstable banks (Sites 245, 246, and 277). Also, the incised Brazos River in Texas (Sites 131 and 283) tends to have moderately high lateral erosion rates where the banks are vertical and slumping.

Cut or Slumped Banks

The presence of raw, vertical cut banks along a stream is clearly an indication of lateral instability, and it should always be noted in connection with other criteria that have been described here. Cut banks can be observed in the field, and they are best observed on aerial photographs by stereoviewing. Slumping, which occurs mainly at vertical banks, is an additional indication of lateral instability. Cut or slumped banks are difficult to observe where the bankline is densely forested, but they will usually be evident from trees that have toppled into the channel.

Bank Materials

The resistance of streambanks to erosion is undoubtedly an important factor in both channel stability and stream morphology, but it is very difficult to quantify. Resistance depends on the nature of the bank materials and on the nature and extent of vegetal cover. If only the bank materials are considered, and the lack of homogeneity typical of banks is disregarded, erosional resistance is determined by several variables. For non-cohesive materials, resistance tends to increase with increase in particle diameter, from fine sand to cobbles and boulders. For cohesive materials, resistance depends on, among other variables, the type and quantity of clay and on the orientation of the clay particles. According to Partheniades (1971, p. 30), "very small changes in the amount of cohesive binder are enough to change drastically the erosional characteristics of a soil of given mechanical composition". In addition, dessication and aeriation may be important factors in increasing the stability of cohesive soils against erosion.

In view of these difficulties, no attempt was made to relate the composition of cohesive bank materials to erosional resistance and lateral stability. As shown in figure 26, bank materials are described qualitatively as coherent or non-coherent. A coherent material is described as one that forms hard lumps or clods, when dry, that are not easily crushed with the hands. Non-coherent materials are described according to dominant size. For streams at many sites, no satisfactory description of bank materials was obtained.

In a general way, the erosional resistance of streambanks increases with increase in clay content. Also, non-coherent sand banks are more easily eroded than non-coherent gravel or cobble banks. However, problems with lateral erosion at study sites are almost as common on streams with gravel or cobble-boulder banks as at streams with sandy banks. This is probably due to the fact that streams having coarse bank materials also have steeper gradients and greater competence for erosion of coarse material.

Tree Cover on Banks

Vegetal cover along a streambank tends to increase the resistance of banks to erosion and thus has an important effect on both channel stability and stream morphology. The effects of vegetal cover would logically depend on the kind of vegetation, its density, its continuity along the banks, and its stage of growth. Quantification of these variables would obviously be complicated. A simplified scheme for expressing the extent of tree growth along the bankline is given in figure 26. For a given reach, the percent of bankline (along both banks) occupied by trees is estimated from an aerial photograph, and the reach is placed in one of three categories according to this percentage.

An historical example showing the drastic effects of destruction of vegetation along a stream in a semi-arid region is given by Schumm (1971, p. 4-6). Before 1914, the Cimarron River in southwestern Kansas was highly sinuous and its channel was narrow and deep. A major flood occurred in 1914. Between 1914 and 1939, the channel width increased from an average value of 50 ft (15 m) to 1,200 ft (366 m), braiding occurred, and a wandering thalweg developed. These changes are related to destruction of natural vegetation by the flood, by drought, and by man. Between 1942 and 1951, rainfall was above average and no major floods occurred. With the restoration of vegetation, the average width of the channel decreased from 1,200 ft (366 m) to 500 ft (152 m).

ALLUVIAL STREAM TYPES

The large number of stream properties are combined in various ways in individual streams, no two of which are identical. However, certain associations of properties tend to recur, and those associations that recur most often represent a stream type. For engineering purposes, five alluvial stream types are distinguished. Most rivers and large creeks can be considered as belonging to one or another of these types. Very small streams tend to be erratic in their properties and may be difficult to classify, probably because of the pronounced effect of vegetation on their development. Bedrock plays a similar role in the development of non-alluvial streams. Each alluvial stream type is named below according to the properties most critical for its recognition, and its typical association of properties is given.

Type A: Equiwidth, Point-bar Stream

Bars--Point bars only, lateral bars rare. Not braided. Point bars mostly covered with permanent vegetation, but narrow crescents of bare sediment may be visible at normal stage. If markings are visible on point bars, these tend to be concentric scrolls.

Sinuosity--A given reach may be straight, sinuous, meandering, or highly meandering.

Other properties--Cut banks rare, banks tend to be well vegetated. Natural meander cutoffs are at neck, leaving crescentic oxbow lakes on flood plain.

Engineering significance--Most laterally stable of all stream types, but meanders gradually migrate. With much clearing of vegetation along channel, stability may quickly deteriorate; cut banks are an early indication of this. Rate of bedload transport, probably small in relation to suspended load.

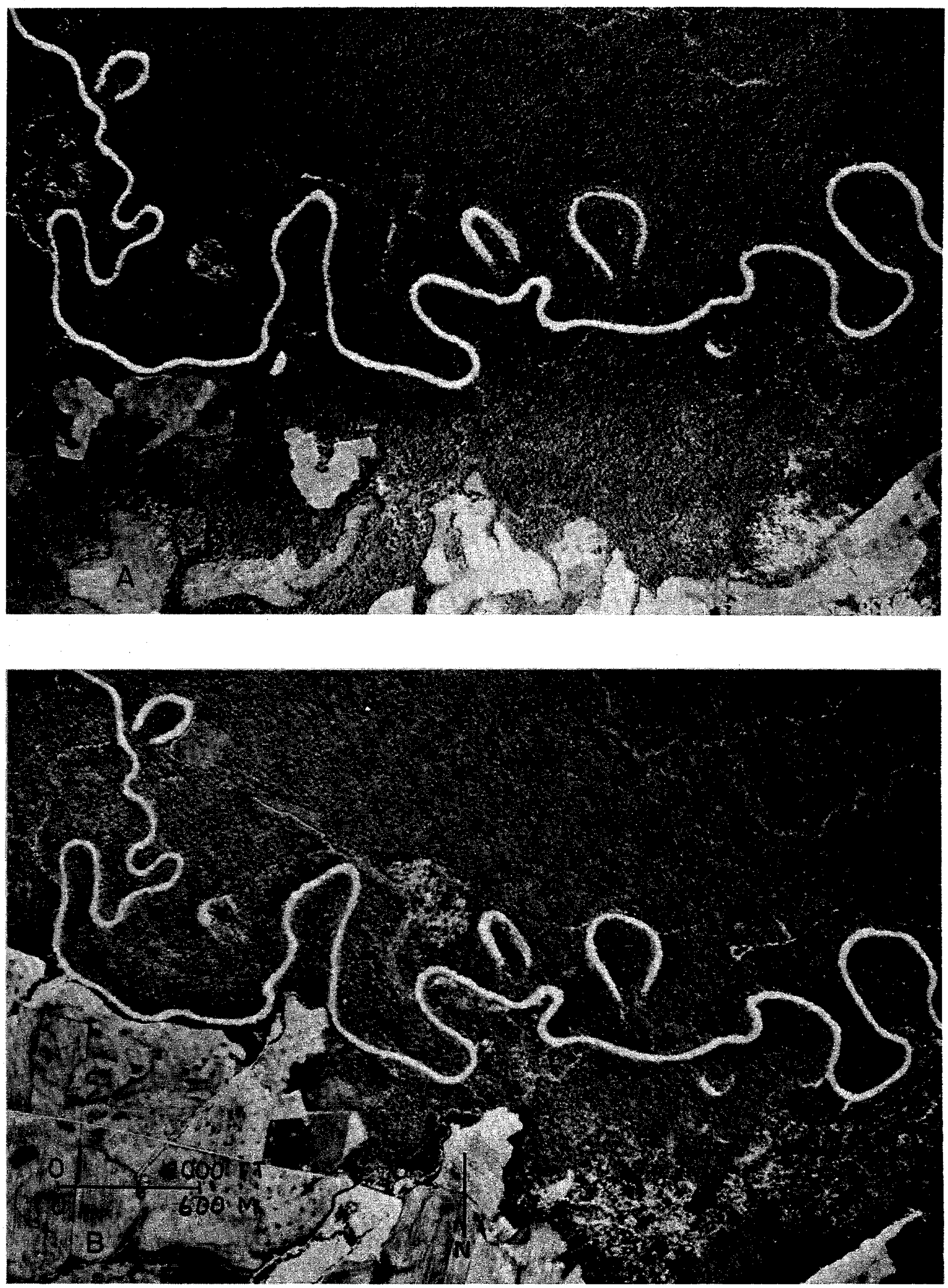

Figure 28. Sequential aerial photographs of an equiwidth point-bar stream, Hatchee River near Stanton, Tenn. A, Photograph on April 26, 1938; B, Photograph on May 20, 1966. (From U.S. Dept. of Agriculture.)

Examples--Equiwidth streams adequately illustrated by an aerial photograph or map in Vols. II and III are as follows: North Branch Susquehanna River, Pa. (Site 35); Bowie River, Miss. (Site 53); Leaf River, Miss. (Site 209); Noxubee River, Miss. (Site 217); Yazoo and Tallahatchie Rivers, Miss. (Site 218); Beaver River, Saskatchewan (Site 257); Qu'Appelle River, Saskatchewan (Site 260).

The Hatchee River in Tennessee has characteristics typical of an equiwidth stream on a forested flood plain, and evidence of lateral stability is provided by sequential aerial photographs (fig. 28). The reach shown is meandering as a whole, but shorter reaches that may be considered for a crossing site are straight, sinuous, or highly meandering. Most point bars are permanently vegetated with dense hardwood forest that covers the flood plain, but a few unvegetated crescents can be seen. The curved oxbow lakes on the flood plain indicate, by their shape, that natural meander cutoffs are of the neck type. Little change in the position of the channel took place during the 28-yr period between the photographs. Note that clearing of the forest during this period has encroached on the channel, and further clearing (which has subsequently occurred) will lead to a deterioration of lateral stability.

Type B: Wide-bend, Point-bar Stream

Bars--Point bars mainly, but may have a few lateral bars and be locally braided. Unvegetated point bars are of average width or wider. Markings on point bars, if visible, tend to be concentric.

Sinuosity--A given reach may be straight, sinuous, or meandering.

Other properties--Cut banks local or general, depending on degree of stability. Natural meander cutoffs are of neck or chute type.

Engineering significance--Potentially high rate of lateral migration at bends. Straight reaches may remain stable for decades. Substantial transport of bed material, either sand or gravel. Most rivers in the U.S. are of this type.

Examples--Wide-bend, point-bar streams adequately illustrated by an aerial photograph or map in Vols. II and III are as follows: Musselshell River, Mont. (Site 101); Trinity River, Tex. (Site 130); Brazos River, Tex. (Sites 131 and 283); Red River, Ark. (Site 170); Grand River, S. Dak. (Site 175); Marias River, Mont. (Site 234); Elkhorn River, Nebr. (Site 240); Leaf River, Miss. (Site 207); Buffalo River, Miss. (Site 281).

The Pearl River in Mississippi has characteristics typical of a wide-bend point-bar stream on a forested flood plain, and lateral migration can be discerned by comparison of sequential aerial photographs (fig. 29). The meander loops of the Pearl are unusual in their regularity and uniformity of size, but there are some short straight reaches between loops. Exposed point bars, which appear lighter in tone, are wider on the 1969 photograph because the river

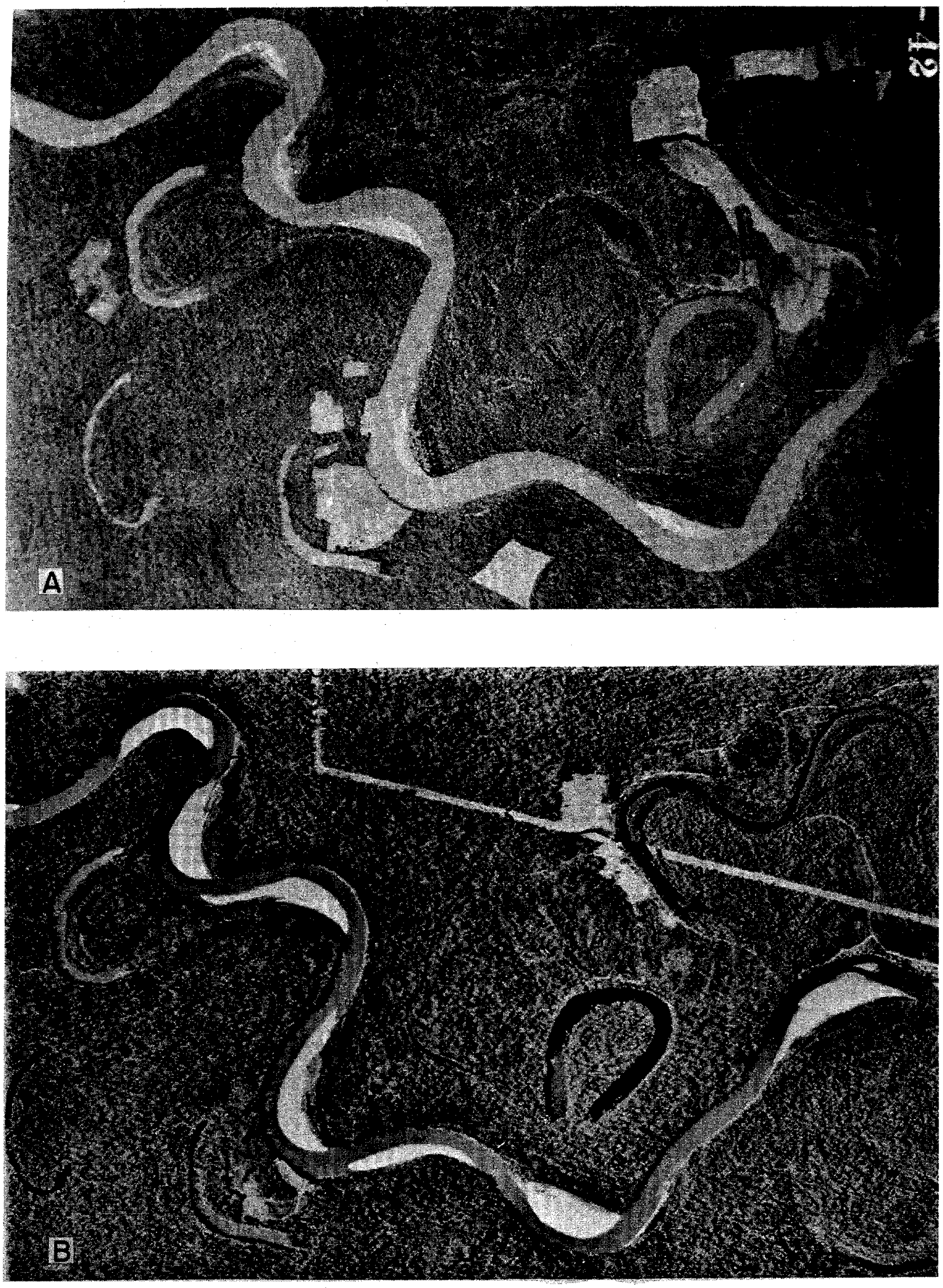

Figure 29. Sequential aerial photographs of a wide-bend point-bar stream, Pearl River near Bogalusa, Miss. A, Photograph on April 2, 1942; B, Photograph on Dec. 17, 1969. (From U.S. Dept. of Agriculture.)

stage is lower. Cut banks opposite the point bars are discernible. The maximum amount of bank recession, at the outside of bends, was about 170 ft (52 m) during the 27-yr period between photographs. Such a rate is considered low to moderate for a stream of this size. Average discharge is about 8,900 ft^3/s (252 m^3/s). Oxbow lakes on the forested flood plain indicate that natural meander cutoffs are of the neck type.

The White River in Indiana (fig. 30) is an example of a wide-bend point-bar stream on a flood plain, formerly forested, that has been mostly cleared for agricultural purposes. Point bars on the White River appear wider on the 1966 photograph because of the low flow which is well below normal stage. On both photographs, exposed point bars are considered wide, especially when the parts sparsely covered with vegetation are included. Concentric scrolls on the point bars are emphasized by the vegetation. On the 1937 photograph, old scrolls on the flood plain, marking the positions of old point bars, are visible because of favorable soil moisture conditions. The maximum amount of bank recession, at the outside of bends, is about 300 ft (91 m) for the 29-yr period, which is considered high for a stream of this size. Note the almost continuous cut banks at the outside of bends. Points a, b, c, and d are reference points identifiable on both photographs. Note that the center meander, terminating near point b, has changed to a compound meander during the period between the photographs. Historical evidence indicates that the lateral migration rate of the river has greatly increased since white settlement and clearing of the flood-plain forest.

Type C: Braided, Point-bar Stream

Bars--Point bars, lateral bars, and mid-channel bars. Locally or generally braided, but has a continuous thalweg. Thalweg sinuous or meandering, may be fairly stable in position or may wander, that is, shift drastically in position during floods. Markings on point bars tend to be irregular and not concentric. Bars may be of sand, gravel, or cobbles.

Width variability--Random variation or wider at bends.

Sinuosity--Main channel (as distinguished from thalweg) is straight or sinuous, more rarely meandering.

Other properties--Cut banks general. Sand-bed streams of this type usually occur in regions of grassland, rather than forest. Natural cutoffs, if they occur, are usually of chute type.

Engineering significance --Potentially very high rate of lateral erosion. Rapid movement of thalweg may cause alinement problems and bypassing of bridge. Chute cutoffs of bends may occur rapidly and cause alinement problems. Potentially deep scour at thalweg, particularly if bed is silt or sand. Transport of bed load (sand, gravel, or cobbles) probably exceeds transport of suspended load.

Figure 30. Sequential aerial photographs of a wide-bend point-bar stream, White River at Edwardsport, Ind. A, Photograph on Sept. 20, 1937; B, Photograph on Oct. 7, 1966. (From U.S. Dept. of Agriculture.)

Examples--Braided point-bar streams adequately illustrated by an aerial photograph or map in Vols. II and III are as follows: Deer Creek, Calif. (Site 16); Salinas River, Calif. (Site 73); Washita River, Okla. (Sites 116 and 120); Tazlina River, Alaska (Site 153); White River, S. Dak. (Site 186); Mississippi River, Mo. (Site 228).

The Cimmaron River in Oklahoma (fig. 31) is a braided point-bar stream that had a high degree of lateral instability during the period 1936-67. The Cimarron has erodible banks and is in a semi-arid grassland region; this degree of instability is unlikely for streams in forested regions. In both photographs of figure 31, the point bars are wide and, with lateral bars, form an almost continuous strip of unvegetated or sparsely vegetated ground along the thalweg. Markings on the point bars are irregular and have a braided aspect. The thalweg is sinuous and braided, and it wanders with respect to the general trend of the main channel; its position changed drastically during the 31-yr period between photographs. Bank recession at the sharp bend between points b and c on the 1967 photograph amounted to about 450 ft (137 m) for the 31-yr period, which is very high for a stream of this size. There is no tree cover along the bankline, only grass. The small bridge on the Cimarron, about midway between points a and c on the 1937 photograph, was protected by a timber pile retard; the loss of the center span is apparently due to scour at the piers rather than to lateral erosion.

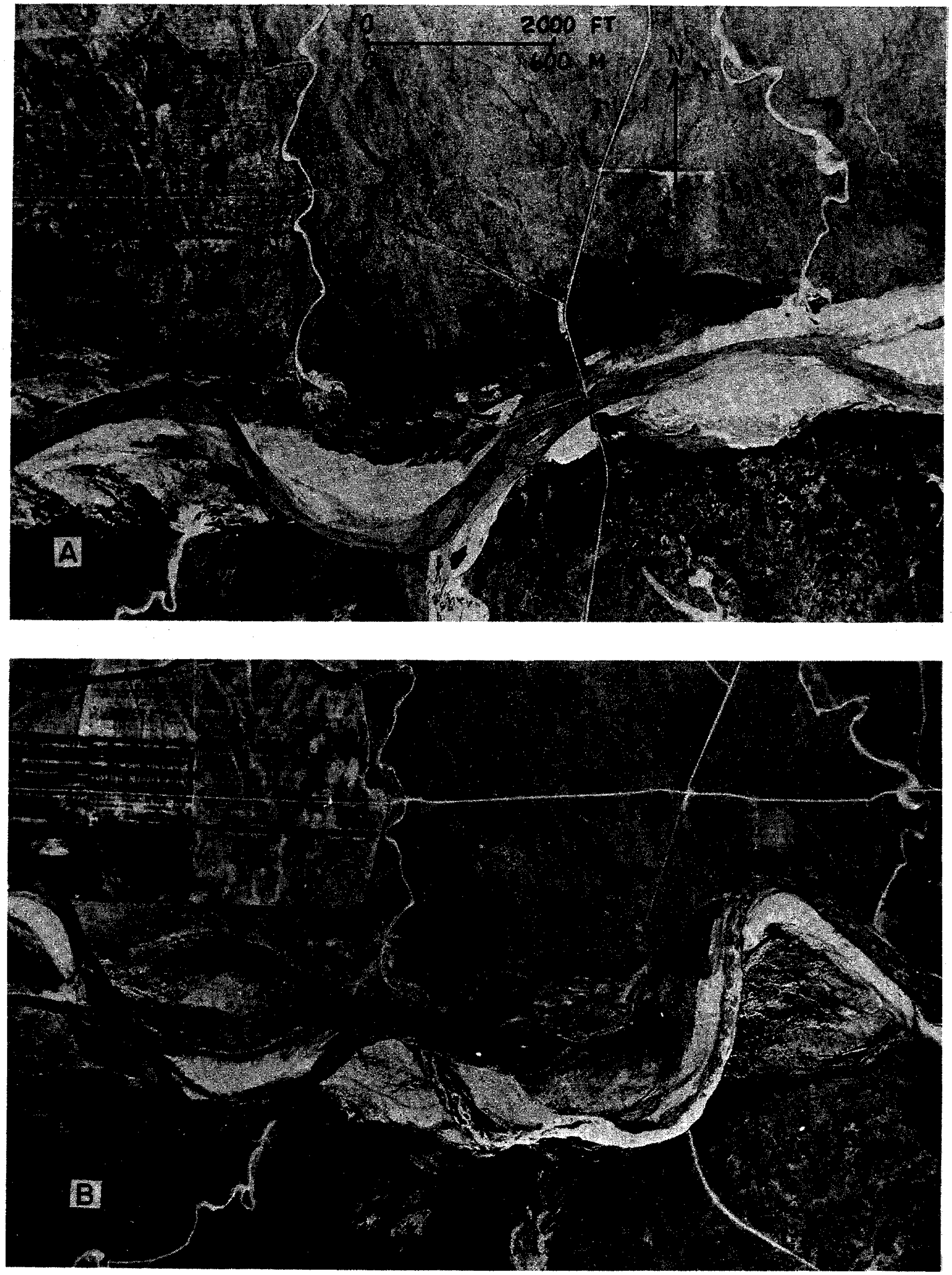

Figure 31. Sequential aerial photographs of a braided point-bar stream, Cimarron River near Mocane, Okla. A, Photograph on Oct. 15, 1936; B, Photograph on May 23, 1967. (From U.S. Dept. of Agriculture.)

The Arkansas River in Colorado (fig. 32), provides a good example of irregular braided markings characteristic of bars on braided point-bar streams. Note also the braided aspect of the channel. The very angular bend to the right of the point bar is distorted because the stream encounters resistant bedrock along its downstream bankline; and the highway beyond is protected by the bedrock. At extreme right, the US-71 bridge has lost a center span. No information on this failure was obtained, but it is apparently due to local or general scour at the piers.

The Middle Fork Koyukuk and the Hammond Rivers in Alaska are braided point-bar streams that are also anabranched, at the locality shown in figure 33. Just upstream from their confluence, both rivers are crossed by the Alaska pipeline (uppermost cleared strip) and the access road for the pipeline. Both streams have coarse bed material (gravel, cobbles) and steep channel slopes. The countermeasures consist of embankment spurs and spur dikes. The embankment spurs range in length from about 200 to 2,000 ft (60 to 600 m) and are built of coarse sediment from the flood plain. A typical spur is 45 ft (14 m) wide at the base, 10 ft (3 m) wide at the top, and rises at least 8 ft (2.5 m) above the flood plain. The base is flanked by a wide launching apron of riprap and the head is round and heavily riprapped. The spur dikes are more nearly straight, with curved ends, than elliptical. In general, the countermeasure design is typical of that used in Alberta. Although the streams look very unstable, the rates of lateral erosion and thalweg shift tend to be slow. The number and size of the countermeasures represents a high degree of precaution, at great expense.

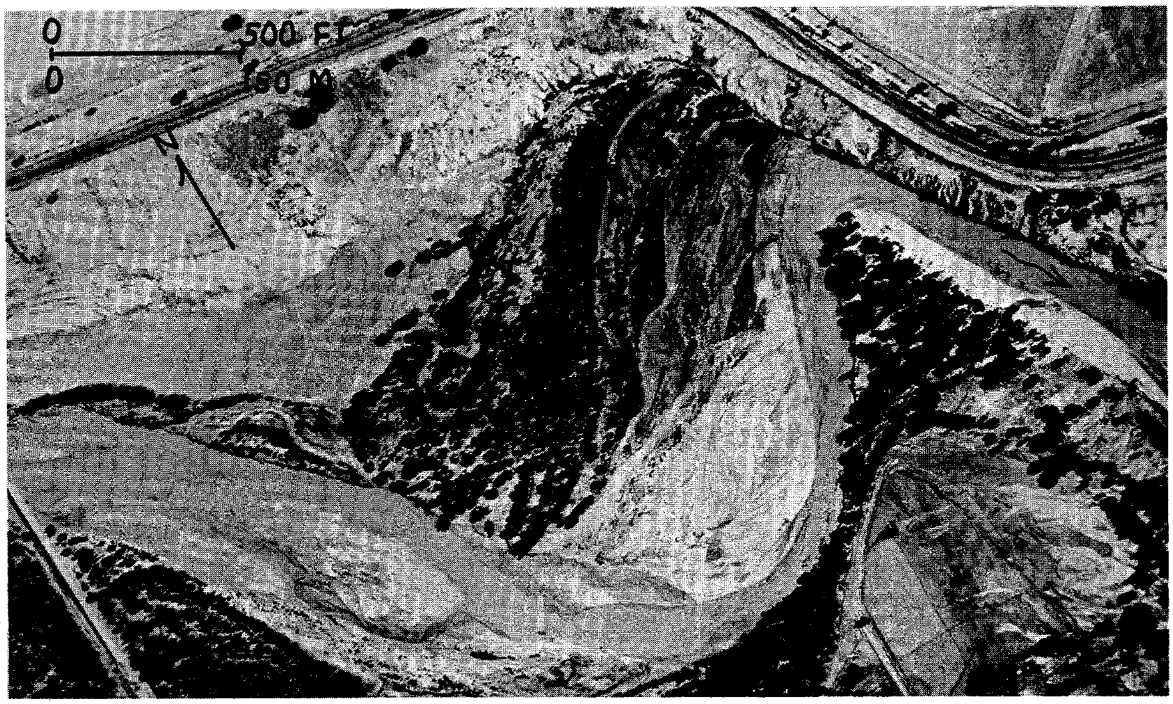

Figure 32. Aerial photograph in 1955 of a braided point-bar stream, Arkansas River between Rocky Ford and Ordway, Colo. (From Colorado Dept. of Highways.)

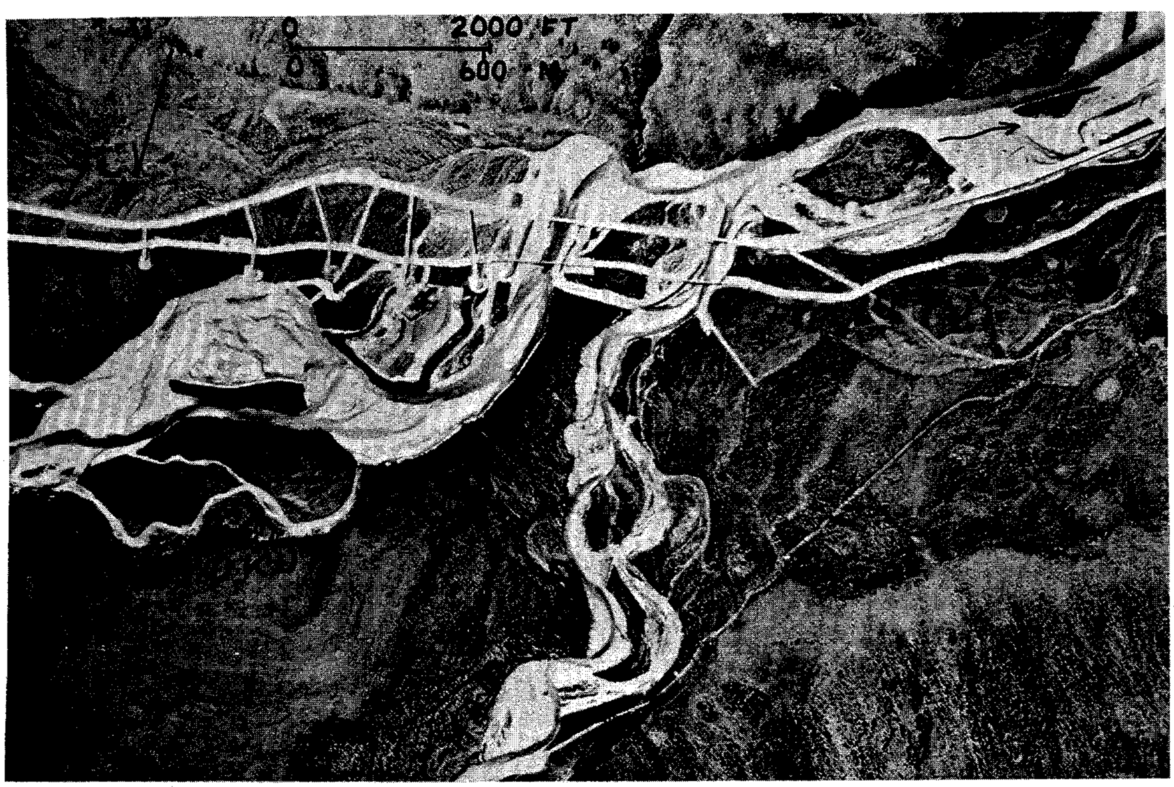

Figure 33. Aerial photograph of braided point-bar streams that are also anabranched, Middle Fork Koyukuk and Hammond Rivers near Wiseman, Alaska. (From Air Photo Tech, July 12, 1977.)

Type D: Braided Stream, Without Point Bars

Bars and islands--No point bars at bends in main channel. Many mid-channel and lateral bars, flow may be complexly divided. Scattered small islands; or islands may be more numerous than bars.

Width variability--Usually random variation, more rarely equiwidth or wider at bends.

Sinuosity--Straight or sinuous.

Other properties--Bankline tends to be irregularly scalloped, with cut banks at indentations.

Engineering significance--Channel tends to be wide and shallow, requires long bridge unless confined by suitable countermeasures. Lateral erosion rates low to moderate, but point of erosion not predictable. Banks erodible, bankline at abutments requires protection. Braids shift at each high flow, and unexpected depth of scour may occur where braids join to form a deep channel. Load transported mainly as bed load, either sand, gravel, or cobbles.

Examples--Braided streams adequately illustrated by an aerial photograph or map in Vols. II and III are as follows: Eel River, Calif. (Site 17); Canadian River, Okla. (Site 51); Bronco Creek, Ariz. (Site 54); Little Colorado River, Ariz (Site 63); South Santiam River, Oreg. (Site 126); Red Deer River, Alberta (Site 150); Lowe River, Alaska (Site 154); Niobrara River, Nebr. (Site 243): North Platte River, Nebr. (Site 244); South Fork Powder River, Wyo. (Site 275).

The Middle Loup River in Nebraska, at the locality shown in figure 34 is classed as a braided stream without point bars, although irregular point bars are formed under some conditions of flow. In figure 34A, the Middle Loup resembles the Cimarron (fig. 31), but it is more distinctly braided and much more laterally stable. The greater degree of braiding is attributed to a higher ratio of bedload to suspended load, although both are sand-bed streams. The greater stability is attributed to a uniform discharge (severe floods are unknown on the Middle Loup) and to the greater degree of braiding, which tends to divide the erosive power of the thalweg. Maximum bank recession for the 12-yr period is about 125 ft (40 m) and this occured only at a few local places. At low flow (fig. 31B) the thalweg is barely discernible. The small-scale meandering of individual braids occurs in sand-bed streams, but rarely in gravel-bed streams. The bars at bends in the thalweg (fig. 31A), which are irregular both in outline and in surface markings, are only marginally identifiable as point bars.

The Delta River (fig 35) is a braided stream that transports large quantities of coarse bedload supplied by many glaciers in the surrounding mountains. It has no flood plain; the channel is bordered by steep valley sides.

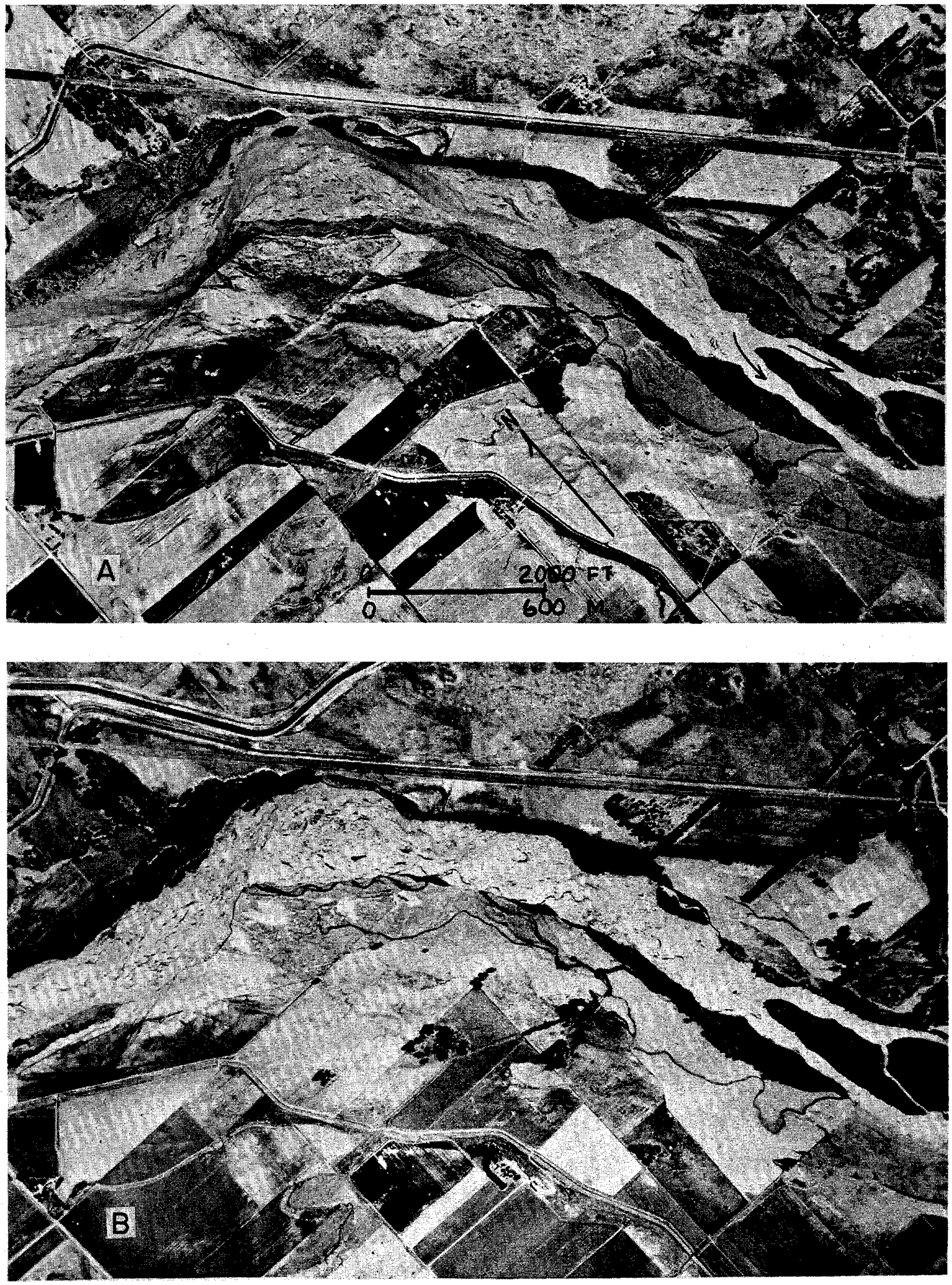

Figure 34. Sequential aerial photographs of a braided stream, Middle Loup River at St. Paul, Nebr. A, Photograph on May 27, 1957; B, Photograph on Sept. 7, 1969. (From U.S. Dept. of Agriculture.)

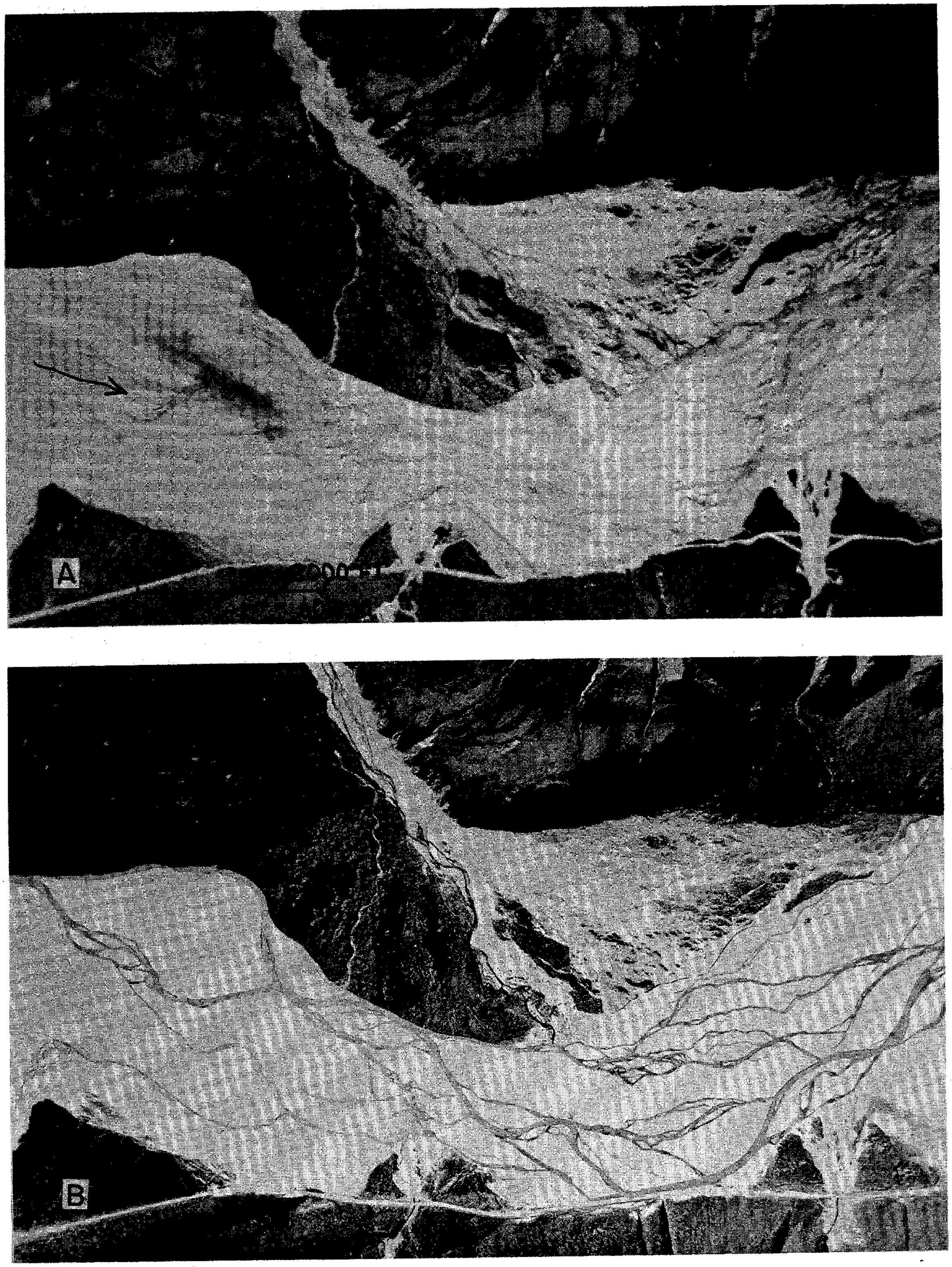

Figure 35. Sequential aerial photographs of a braided stream and of alluvial fans, Delta River near Black Rapids, Alaska. A, Photograph on August 26, 1949, (From U.S. Air Force); B, Photograph on Sept. 19, 1969, (From U.S. Bureau of Land Management.)

Where tributaries enter the channel, alluvial fans are built into the channel. The large alluvial fan at center has been built into the channel by outwash from a glacier. Smaller fans on the opposite side of the valley are crossed by the Richardson Highway. These fans are like those in arid region mountains, and they present similar engineering problems. Channels on the lower part of the fans shift from time to time, and crossings are usually placed near the apex of the fan. At many such crossings in Alaska, dikes are built to form a "V" upstream from the bridge abutments, and extending to either side of the tributary valley.

The braiding of the Delta River is characteristic of streams transporting coarse bedload. There are no point bars along the side of the channel, although transient point bars may be built within the channel by a meandering braid. Note the shift of braids during the 20-yr period between photographs, and the scalloping of banks by the lateral erosion of individual braids. There is no highway crossing on the Delta River, but similar streams have been confined at a crossing by suitable countermeasures.

Type E: Anabranched Stream

Bars and islands--Flow is distinctly divided into channels separated by large islands, which are usually covered with permanent vegetation. Anabranches are likely to be locally braided. Point bars are likely at bends in anabranches.

Width variability--Applies only to individual anabranches, which may be equiwidth, wide-bend, or random.

Sinuosity--Stream as a whole, as well as individual anabranches, may be straight, sinuous, or meandering.

Engineering significance--A long bridge is required unless the stream is crossed at a local point where it is not anabranched. If there are two or more anabranches at a crossing site, suitable countermeasures will permit design of a shorter bridge. If two bridges are used, percent of total flow at each bridge may not be predictable. Stability of anabranches differs greatly on different streams, and should be assessed as though an anabranch were an individual stream.

Examples--Anabranched streams adequately illustrated by an aerial photograph or map in Vols. II and III are as follows: Stony Creek, Calif. (Site 7); Snake River, Idaho (Site 40); Klickitat River, Wash. (Site 138); Red Deer River, Alberta (Site 151); Oldman River, Alberta (Site 152); North Fork Chena River, Alaska (Site 157); Cossatot River, Ark. (Sites 162 and 163).

The Yellowstone River (fig. 36) at Billings is an anabranched gravel-bed stream. The flow is divided from place to place by large vegetated islands, and the anabranches tend to meander. The anabranching habit of this stream is attributed partly to ice jams, which form in a main channel and divert flow

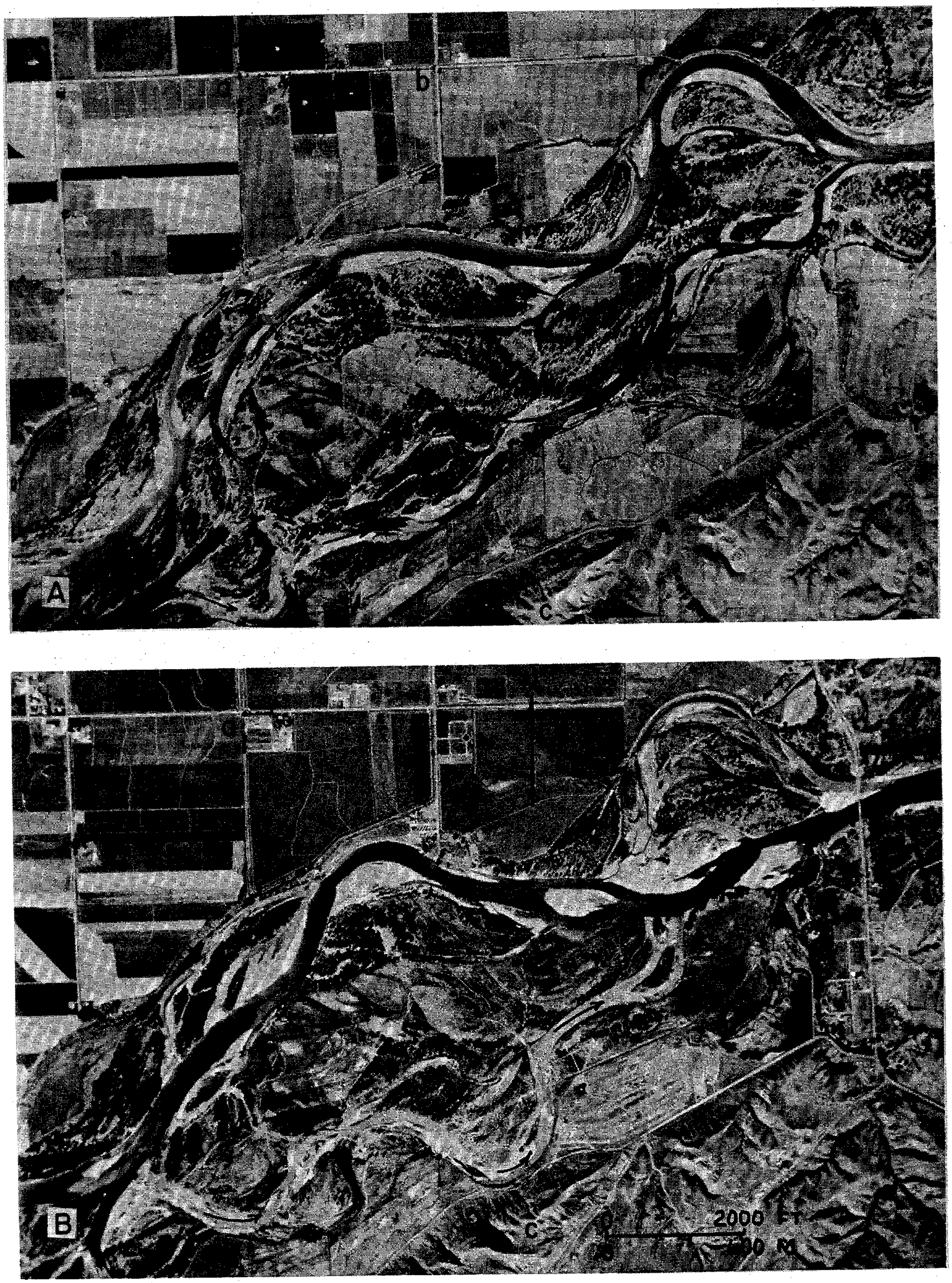

Figure 36. Sequential aerial photographs of an unstable anabranched stream, Yellowstone River at Billings, Mont. A, Photograph on Aug. 29, 1940 (From U.S. Dept. of Agriculture); B, Photograph on Aug. 27, 1969 (From U.S. Geological Survey.)

into side channels. Individual anabranches resemble wide-bend point-bar streams or braided point-bar streams, and they tend to have a rapid rate of lateral migration. Bank recession for the 19-yr period was about 300 ft (91 m) at the left bank between points a and b, which are identifiable on both photographs. The opposite anabranch, near point c, eroded laterally by about 950 ft (290 m) at the prominent bend shown in figure 36B. This is a very rapid rate for a river of this size (average discharge about 6,500 ft^3/s (184 m^3/s).

In figure 36B, the SR-416 crossing of the Yellowstone is visible at right of center. This crossing was not included in the present study, but it is apparent that a large meander loop was cut off to improve flow alinement, and the right bank upstream and downstream from the bridge is revetted.

Relations Between the Major Stream Types

Probable relations between the stream types are shown in figure 37. In the sequence of types A through D, the point bars first become wider, then deteriorate as the stream becomes braided and the thalweg less distinct, and finally disappear as the thalweg becomes braided and discontinuous. Genetic factors involved in the change from one channel type to the next are also shown in figure 37. Anabranched streams may fall at different places in this sequence, depending on how the anabranching originates. For example, equiwidth streams may anabranch on a delta or deltaic plain; and braided streams may become anabranched in a downstream direction because of a change in valley slope or in sediment load.

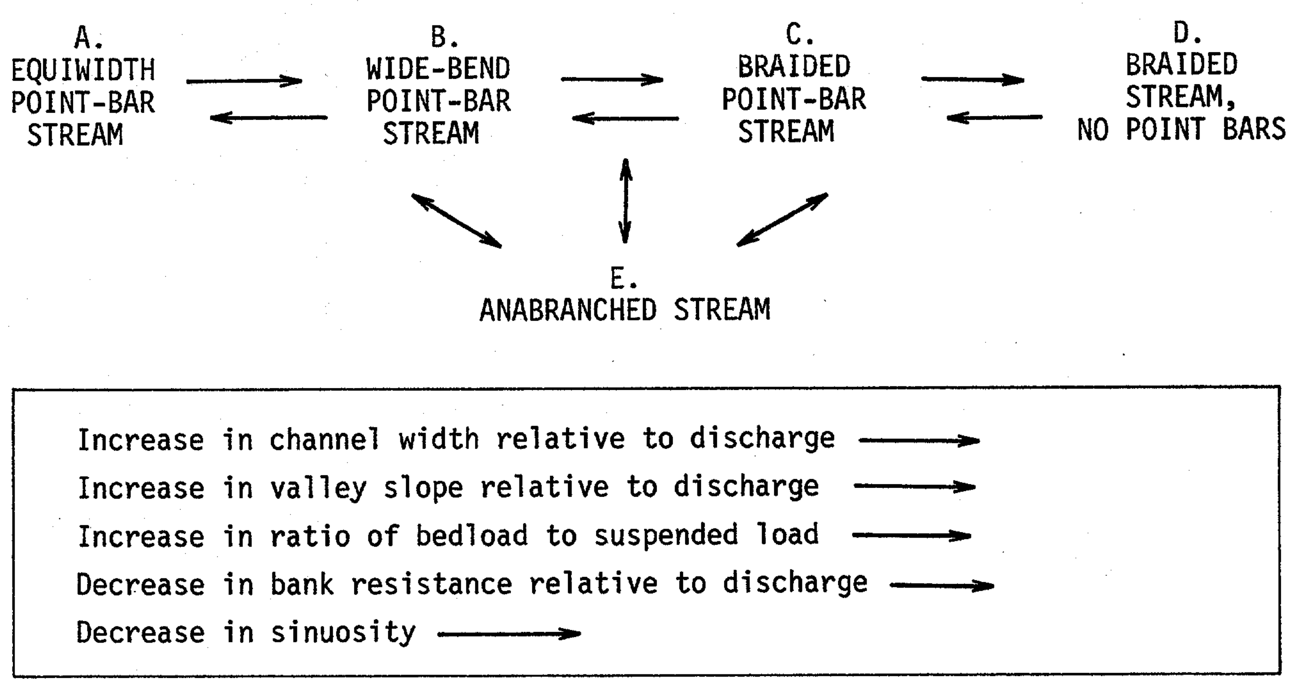

Figure 37. Probable relations between the major stream types.

MAN-INDUCED GEOMORPHIC FACTORS

Since the works of man are changing the face of the earth, man must be regarded as a geomorphic or landscape-forming agent. Some works have had rather far-reaching consequences on streams and have caused, or contributed to hydraulic problems at bridges. Construction of a bridge and approach embankments may also have geomorphic consequences, but these are unlikely to be far reaching. For purposes of this report, hydraulic problems resulting from the interaction of bridge and stream are discussed in the chapter on bridge factors. Of the non-highway works, only those that have had fairly rapid and assignable consequences are considered here, because the consequences may be very slow and subtle. For example, many channels in western U.S. are degrading by the migration of channel scarps ("headcuts"). Such degradation may be due to a combination of natural and man-induced factors.

General (Non-highway) Channel Alteration

Straightening and other alteration of natural channels, mainly to improve drainage for agricultural purposes, has been widely done in the U.S. since about 1900. Most of the straightened channels have not been maintained but have been allowed to deteriorate. Many of the straightened channels have degraded, and degradation is usually accompanied by widening of the channel. Degradation is attributed to increase in channel slope that results from a shortening of channel length. Lateral erosion is attributed to clearing of vegetation during channelization, to instability of raw artificial banks, and to wandering of the low-water thalweg.

There is some evidence that degradation, if it is to occur as a consequence of channel alteration, will be most rapid during a period shortly following the alteration and will thereafter occur at a decreasing rate. For example, Yearke (1971) measured degradation following channel straightening on the Peabody River in New Hampshire. He found that the major degradation occurred within the first year after straightening and that successively smalleramounts occurred in subsequent years. On the Homochitto River at Doloroso, Mississippi (Site 282), the river course to the Mississippi River (downstream from the US-61 bridge) was shortened from about 17 miles to about 9 miles in 1938. Degradation began at the bridge in 1944, reached 16.5 ft (5 m) by 1945, and increased an additional 4.5 ft (1.3 m) during the period 1945-57. Between 1957 and 1975, the channel aggraded about 2 ft (o.6 m). But degradation that begins in the lower reaches of a stream may require a substantial length of time to progress upstream. For example, the main wave of degradation on the Homochitto River did not reach Rosetta, Mississippi (Site 174) which is about 16 mi (26 km) upstream, until the period 1949-66.

Degradation of a main channel may lead to degradation of tributaries, whether these have been altered or not. For example, straightening and consequent degradation on the North Sulphur River in Texas led to degradation and hydraulic problems at a bridge on Merrill Creek, a tributary to the North Sulphur River (Site 148).

Hydraulic problems attributed either directly or indirectly to non-highway channel alteration are documented at 23 study sites (table 23). Alteration in nearly all cases involved channel straightening, but only dredging was involved at Site 205 (Mississippi River in Minnesota). At seven sites, problems are attributed directly to channel degradation (Sites 52, 148, 174, 205, 227, and 282). At most other sites, problems result mainly from lateral erosion and degradation. Where degradation on a sand-bed stream of moderate or large size is very severe, as on the Homochitto River in Mississippi (Sites 174, 282), no countermeasure may be effective. On smaller streams, a check dam will prevent progressive degradation at the bridge, but scour downstream from the check dam may be a hazard. Rapid lateral erosion accompanying degradation is also difficult to control, but permeable steel spurs (Sites 178, 179), riprap bank revetment (Site 37), and rock retards (Sites 226, 227), have apparently performed satisfactorily.

Clearing of Natural Vegetation

The great importance of natural vegetation in maintaining channel stability has been previously discussed in this chapter. The lateral stability of most streams in the U.S., particularly in regions where agriculture or lumbering is practiced, has very probably been affected by the clearing of natural vegetation. Because this clearing has occurred more or less gradually over the past hundred years, the magnitude of the effect at a particular crossing site is difficult to assess. Four consequences of clearing on bridges are distinguished (table 23) and sites where each consequence is probable are listed. Accumulation of debris at a bridge is attributed to lumbering (Sites 17, 264, 268), or to recent clearing of flood plains for agricultural purposes (Sites 168, 261, 262). Clearing along streambanks for agricultural purposes has contributed to lateral erosion at Sites 4, 180, 181, 213, 220, 239, and 261. At four sites (163, 184, 212, and 214), patchy clearing of forested flood plains has led to non-uniform distribution of overbank flow entering the bridge waterway, and consequent hydraulic problems. At one site (266), channel aggradation is attributed to an increased sediment supply from ground cleared by lumbering; and this problem is to be expected at some bridges in mountainous regions like the Pacific Northwest.

Mining of Sand or Gravel

If sand or gravel is removed from an alluvial channel in quantities that represent a substantial percentage of the bedload in transport, the channel will probably degrade. In addition, removal of gravel from pits or trenches in or along the stream may result in a change in flow alinement at the bridge. Sites illustrating these consequences of removal are listed in table 23. In some states, operators may legally continue to remove sand or gravel despite clear evidence of the consequent damage at a bridge. No specific countermeasures are available, beyond those applying to degradation and change of alinement from other causes. Use of both check dams and bank revetment is described in Site 85.

Table 23. Study sites at which hydraulic problems are attributed to works of man

Work	Geomorphic or hydraulic consequence	Site No.
General (non-highway) channel alteration	Channel degradation. . .	37, 42, 52, 148, 159, 174, 176, 178, 179, 180, 181, 205, 212, 218, 227, 242, 246, 261, 282.
	Lateral erosion.	37, 52, 148, 178, 179, 180, 181, 200, 212, 222, 226, 227, 241, 242, 246, 261.
Clearing of natural vegetation	Debris accumulation at bridge.	17, 168, 261, 262, 264, 268.
	Lateral erosion.	4, 49, 180, 181, 200, 213, 220, 239, 261, 262.
	Non-uniform flow distribution	163, 184, 212, 214.
	Aggradation.	266.
Sand or gravel mining	Channel degradation. . .	1, 7, 85, 90, 123, 125, 168, 193, 198.
	Flow alinement change. .	56, 177, 192.
Dam and reservoir	Channel degradation. . .	7, 24, 39, 88, 90, 170, 173, 238, 247.
	Lateral erosion.	169, 170, 171, 173.
	Wave erosion	167, 267.

Mining in an upland area may cause aggradation of channels, which are then subject to degradation after the mining ceases. On the Yuba River in California at Site 24, the channel has degraded about 27 ft (8 m) since the existing bridge was built in 1913. This degradation followed about 81 ft (25 m) of aggradation that occurred during the period of hydraulic mining for gold (about 1869-1900). Despite the large amount of degradation, the piers have been protected by countermeasures, as described in the case history.

Dams and Reservoirs

The effects of dams and reservoirs on a stream are complex and have not been thoroughly investigated. Downstream from a reservoir, channel degradation is to be expected because of removal of sediment load. This effect has been documented for many streams. The total amount of degradation is difficult to predict; if a sand-bed channel becomes armored with gravel, the amount may be small. On gravel-bed streams, aggradation may occur downstream from the dam because the flow releases are insufficient to transport gravel brought in by tributary streams. As pointed out by Kellerhals, Church, and Bray (1976, p. 822), channel avulsions, which can present a serious threat to many engineering structures, are associated with most aggrading situations. Rapid lowering of river stage may result in severe bank slumping from pore-water pressures in the banks. However, the more general effect of reservoirs is probably to lessen hydraulic problems at bridges, both by reduction of flood peaks and a reduction of lateral erosion rates. An increase in stream stability has been attributed to reservoirs by Keeley (1971, p. 129) for the North Canadian River in Oklahoma and by Brice (1977, p. 47) for the Sacramento River in California.

Sites at which hydraulic problems are attributed to reservoirs are listed in table 23. On the Trinity River in Texas (Site 39), problems with scour and lateral erosion occurred because the bridge was located too near the reservoir outlet. On Dardanelle Reservoir in Arkansas (Site 167) a high approach embankment slumped because of saturation from the reservoir, and some wave erosion of the embankment occurred. In his analysis of the failure of the I-29 bridges at the Big Sioux River crossing (Iowa-South Dakota) Anderson (1966) reports an unexpected effect of river regulation: The regulated Missouri River, to which the Big Sioux is a tributary, was at a relatively low stage when the Big Sioux was in flood. This resulted in an increase in surface slope of the Big Sioux and a consequent increase in water velocity, which contributed to the failure of the bridges.

SOME GUIDELINES FOR ASSESSMENT OF GEOMORPHIC FACTORS

1. In locating a crossing site or planning countermeasures at a bridge, look for man-induced factors that may lead to problems such as aggradation, degradation, or lateral erosion. These include recent or projected dams and reservoirs, large-scale agricultural clearing of flood plains, sand or gravel mining, and channelization projects. If stream under consideration is tributary to a larger stream, consider also the possible effects of man-induced changes on the larger stream. To determine whether degradation or aggradation is currently occurring, check history of bed-level changes in relation to older bridges or gaging-station datums.

2. Acquire recent aerial photographs of the site and, for comparative purposes, older aerial photographs. For information on acquisition of older photographs, contact the National Cartographic Information Office, U.S. Geological Survey, 507 National Center, Room 1-C-107 Sunrise Drive, Reston, Va. 22092, Telephone (703) 860-6045. Using the methods described in this chapter, make an assessment of stream stability and behavior. Note items to be observed in the field.

3. Complete assessment by studying site in the field using the recent aerial photographs as a guide. Streams are best observed at low flow and during a season when vegetal cover is at a minimum. Look particularly for indications of recent bank erosion and development of bars.

CHAPTER 5

BRIDGE FACTORS

Bridge factors are characteristics of the bridge structure and the aproach roadway that relate to hydraulic problems and countermeasures. Also included as bridge factors are channel alterations made specifically for bridge or other highway purposes. In this chapter, bridge factors are summarized for the study sites of Vols. II and III, with particular regard to factors that contributed to hydraulic problems.

General Descriptive Factors

General descriptive factors for bridges at study sites are given in tables 24 and 25. Most bridges at study sites were built after 1940, and only a small percentage were built before 1920. Increasing attention has been given to hydraulic design during the past few decades, and newer bridges are undoubtedly less subject to hydraulic problems. Most of the bridges at the study sites are in the range of 101-500 ft (31-152 m) in length, and only a small percentage are less than 100 ft (30.5 m). The predominance of spill-through abutments (table 25) reflects the design trend of the past few decades. Nearly all of the bridges at study sites cross streams. Tidal estuaries are represented by only five sites. At two of these (Sites 164 and 269), scour occurred long after bridge construction, probably because of a shift in the thalweg.

Factors Contributing to Problems

Location of crossing--More hydraulic problems were attributed to crossing location at or near a bend (table 26) than to any other bridge factor. The hazards of such a location are well known to bridge engineers and are discussed in chapter 4. Placement of a crossing at a bend may be due to a predetermined highway alinement or to lack of a more suitable site; or a bend may migrate downstream to a crossing site that was originally placed on a straight reach. The choice of specific countermeasures depends on factors such as sharpness of the bend, stability of the banks, and availability of materials; but in general flow-control measures are more suitable for control of stream alinement than are bank revetments. If the countermeasure can be applied before the alinement problem develops to a serious stage, money and trouble will likely be saved.

Tributary streams or anabranches (bypass channels) that enter near a crossing are a potential hazard. For example, if the entering stream is transporting coarse bed material, it may build an alluvial fan that will divert flow at the bridge (Sites 44 and 177). A braid or anabranch that is poorly alined with the bridge opening,or that requires a long bridge, may be successfully diverted (Site 258) or blocked.

Table 24. Construction date and length of bridge at study sites

CONSTRUCTION DATE

Before 1920

Sites 17, 21, 24, 44, 51, 73, 139,209, 240.

1920-1940

Sites 2, 8, 11, 14, 16, 23, 25, 27, 29, 31, 32, 36, 37, 41, 45, 50, 53, 71, 90, 101, 103, 163, 171, 173, 179, 184, 207, 208, 210, 211, 213, 215, 217, 222, 223, 224, 225, 227, 234, 236, 237, 238, 239, 245, 252, 253, 254, 258, 266, 269, 280, 281, 282.

1941-1960

Sites 1, 4, 8, 10, 20, 25, 26, 27, 28, 29, 35, 38, 39, 43, 47, 52, 60, 70, 71, 88, 97, 103, 116, 122, 125, 127, 134, 138, 143, 148, 150, 153, 154, 159, 164, 165, 166, 172, 174, 178, 186, 192, 196, 198, 200, 201, 204, 205, 206, 212, 216, 218, 220, 221, 226, 235, 241, 242, 243, 244, 246, 262, 263, 265, 267, 270, 272, 273, 275, 276, 278.

1961-1977

(all other sites)

BRIDGE LENGTH

20-100 ft (6-30.5 m)

Sites 2, 14, 20, 25, 29, 31, 32, 36, 41, 45, 74, 100, 134, 166, 172, 224, 237, 245, 248, 254, 277.

101-500 ft (31-152 m)

Sites 3, 5, 6, 8, 9, 10, 11, 12, 13, 16, 23, 25, 26, 27, 28, 33, 34, 37, 38, 42, 43, 48, 50, 52, 53, 54, 55, 56, 57, 58, 60, 61, 68, 69, 70, 85, 88, 89, 90, 91, 95, 97, 99, 123, 125, 132, 138, 141, 143, 146, 148, 150, 151, 152, 153, 154, 155, 157, 159, 162, 165, 167, 168, 173, 176, 178, 180, 181, 182, 183, 184, 186, 193, 194, 195, 197, 198, 200, 206, 208, 209, 212, 214, 217, 220, 226, 227, 233, 235, 236, 239, 240, 241, 242, 243, 246, 250, 252, 257, 260, 261, 262, 264, 265, 266, 267, 270, 272, 273, 274, 275, 276, 279.

501-1000 ft (31-305 m)

Sites 1, 7, 21, 24, 30, 40, 46, 51, 115, 120, 122, 124, 126, 130, 131, 133, 144, 145, 156, 158, 160, 174, 192, 195, 201, 204, 210, 211, 213, 218, 219, 221, 223, 225, 234, 238, 244, 247, 258, 259, 263, 269, 280.

Greater than 1000 ft (305 m)

Sites 17, 35, 39, 44, 47, 49, 51, 63, 73, 75, 126, 130, 139, 140, 164, 169, 170, 171, 177, 185, 188, 202, 205, 207, 215, 222, 228, 268, 281, 282.

If a stream is crossed near a confluence with a large stream, problems may result. Although the effects of backwater on water-surface elevations at a crossing are routinely considered in bridge design, there may be other effects; for example, aggradation resulted from backwater at Site 71. If water-surface elevation on the large stream is relatively low during a flood at the crossing, scour may result from an increase in water-surface slope and velocity on the tributary stream (Anderson, 1966).

Where two bridges cross a stream in close proximity, the downstream bridge is subject to hazards of flow disturbance, change in flow alinement, and possibly, increased water velocity. On the Snake River in Idaho (Site 40) scour at the piers of a new downstream bridge was attributed to flow disturbance from the piers of an older bridge upstream. The piers at the two bridges are oriented at different angles to the flow. On the Homochitto River in Mississippi (Site 174) drift accumulation at an upstream railroad bridge may have contributed to loss of the highway bridge by deflection of flow. On Nichols Creek in Mississippi, parallel approach embankments led to problems with overbank flow. In Neill (1973, p. 39), breaching of a bridge approach is attributed to effects of an upstream bridge on flow alinement. In Culbertson, Young, and Brice (1967, p. 24) failure of the I-6 bridge on Bijou Creek in Colorado is tentatively attributed to velocity increase from accumulation of debris at upstream bridges.

Piers or pile bents--The effects of pier skewness, in contributing to local scour by effectively increasing pier width and in reducing the effective waterway opening, are well known to bridge engineers. The difficulty lies in knowing the flow direction with which the pier should be oriented. The flow direction of a stream may shift with stage, with shifts in the thalweg or channel with time, or with diversion of overbank flow by approach embankments. In table 27, the 16 study sites at which pier skewness caused, or contributed to, hydraulic problems are listed. This is a small percentage of the total number of sites, and it indicates that pier skewness is not a major cause of hydraulic problems at bridges. At five sites (16, 17, 44, 124, and 174) skewness is attributed to shift of the thalweg or channel. At five other sites (34, 47, 68, 155, and 233), skewness is attributed to the angle at which overbank flow entered the bridge waterway. At four sites (2, 260, 265, and 272), piers are skewed to the alinement of flow in the main channel.

The hydraulic problem resulting from pier skewness was, in most cases, local scour at a pier. Riprap or underpinning was used as a countermeasure, mostly with satisfactory results. Where skewness is specifically related to the angle of overbank flow, spur dikes were used mostly with satisfactory results.

Although a single round pier shaft cannot be skewed to flow, skewness effects can result from multiple round columns in a pier or pile bent. At Site 263, the bridge waterway opening was effectively reduced by round-column bents skewed to flow. Consequent bank erosion in the waterway led to internal slope failure, which cracked some bent columns.

Table 25. Abutment type and kind of water body crossed at study sites

ABUTMENT TYPE

Vertical, full height

Sites 5, 8, 11, 14, 16, 17, 20, 21, 23, 25, 26, 27, 29, 31, 32, 35, 43, 44, 53, 55, 90, 100, 125, 141, 182, 194, 201, 202, 224, 236, 237, 239, 240, 241, 243, 244, 245, 253, 254.

Spillthrough

(all other sites)

WATER BODY CROSSED

Flood plain (relief bridge)

Sites 22, 146, 163, 172, 184.

Tidal estuary or bay

Sites 164, 201, 202, 268, 269.

Lake or reservoir

Sites 167, 187, 267.

Stream

(all other sites)

Table 26. Crossing factors contributing to hydraulic problems at study sites

CROSSING,

At bend

Sites 3, 9, 23, 31, 33, 36, 43, 61, 63, 68, 97, 99, 116, 123, 125, 130, 131, 133, 144, 145, 155, 169, 170, 171, 173, 175, 176, 178, 180, 181, 182, 185, 188, 192, 193, 194, 195, 197, 199, 207, 208, 209, 210, 211, 215, 216, 220, 223, 225, 227, 234, 235, 236, 238, 239, 240, 245, 248, 254, 257, 260, 264, 265, 273, 281.

At tributary or anabranch entrance

Sites 34, 44, 177, 196, 258, 279.

At confluence

Site 71

Near another bridge

Sites 40, 50, 174, 185.

Near reservoir outlet

Site 39.

Skewed crossing

Sites 14, 176, 194, 204, 212, 217, 265, 272, 280.

A distinction is made here between skewness of the crossing relative to the stream or its flood plain (bridge skew), and skewness of the piers relative to flow (pier skew). This distinction is discussed more fully in chapter 6. Where approach embankments cross a flood plain at a sharp angle of skew, overbank flow may be strongly diverted along the upstream-trending embankment, causing erosion of the embankment and scour at the bridge where the diverted flow enters the channel. This occurred at Site 204 and, to a lesser degree, at Sites 212 and 280. A spur dike is an effective countermeasure, and embankment spurs provide additional protection for the embankment. Consideration should be given to orientation of the spur dike parallel with the valley trend, rather than with the stream. Ponding of flow may occur at the downstream-trending embankment, although no examples of this were found in our study. Bridge skew may also contribute to lateral stream erosion at an embankment or abutment (Sites 14, 176, 217, 265).

Table 27. Pier and abutment factors contributing to hydraulic problems at study sites

Factor	Number of site where problem occurred
Pier skewed to flow	2, 16, 17, 38, 40, 44, 47, 68, 124, 155, 174, 233, 260, 263, 265, 272.
Square nose of pier or pier footing	2, 122, 165, 219.
Spacing between piers	58, 103, 168.
Pier width constricts waterway	4, 49.
Pier located near toe of abutment fill-slope	10, 133, 165, 183.
Cofferdam or falsework left at pier	17, 150, 238.
Abutment encroaches on channel	12, 17, 21, 26, 27, 28, 29, 32, 60, 61, 154, 156, 243, 276.
Abutment skewed to flow	233, 272.

Hydraulic problems attributable to the shape of pier or footing are not easy to isolate, but four probable sites are listed in table 27. On the McKenzie River in Oregon (Site 122), where local scour at a pier is attributed in part to a square-nosed footing, both grout and riprap were used as a countermeasure. On Hollywood Creek in Arkansas (Site 165) scour at an abutment is attributed both to the square shape of the pier column and to its placement adjacent to the toe of the abutment fill slope. On the Tombigbee River in Mississippi (Site 219) turbulence at a square pier nose was probably a contributing factor in bank erosion.

Short bridge spans, with consequent close spacing between piers in a channel, sometimes cause hydraulic problems. However, problems attributable to this cause were identified at only three sites (table 27). At Site 58, the spacing between piers is 35 ft (11 m) and accumulation of drift at the piers restricted the bridge waterway. At Site 103, where the spacing was 31 ft (9 m), bridge failure from pier scour may be partly due to the spacing. At Site 168, where the pier spacing is only 19 ft (6 m), drift accumulation at one end of the bridge has led to scour at the other.

The effects of pier width in constricting a bridge waterway are reported at Sites 4 and 49, and this problem is discussed more fully in Chapter 6.

Although the effects of pier location near the toe of an abutment fill-slope were identified at only four sites (table 5.4), this may be a fairly common cause of erosion problems. Turbulence at a pier can cause erosion of an adjacent abutment fill-slope.

Construction cofferdams or falsework left in place at a pier may serve as a scour countermeasure if they are cut off at bed elevation, but may contribute to scour if they project into the flow (Sites 17, 150, and 238).

Abutments--A distinction is drawn here between constriction of channel flow by abutments, and constriction of channel and overbank flow by abutments and approach embankments. An abutment that encroached on the stream channel is a potential source of hydraulic problems. At Site 276, for example, scour at piers is attributed to constriction of the lower part of the waterway by abutment fill-slopes. At other sites (12, 21, 28, 29, 32), encroachment of abutments contributed to erosion either of abutment sill-slopes or of streambanks. Where abutments encroach on a wide braided channel (Sites 17, 27, 60, 61, 154, and 243), countermeasures for protection of the abutment may require special attention.

Skew of abutments may contribute to a reduction of effective waterway opening, but at only two sites (233, 272) was this identified as a contributing factor in hydraulic problems.

Approach roadway--Constriction of flow by approach-roadway embankments affects the contraction ratio (bridge opening ratio), as discussed more fully in chapter 6. At many sites where hydraulic problems are attributed to flow constriction by approach embankments (table 28), problems occurred during floods that exceeded the bridge design flood. The most common problem is general scour in the bridge waterway, commonly accompanied by bank erosion near the bridge. The countermeasures that have proved to be effective are riprap or other revetment at abutments or piers, and in some cases, spur dikes.

Whether the flow is constricted by approach embankments or not, flood flow diverted along the upstream side of the embankments may cause problems where it enters the bridge waterway. Spur dikes have proved to be effective countermeasures, (see also chapters 3 and 6), and embankment spurs may also be desirable. Where the embankments are in the channel of a wide stream, riprap may be needed along the embankment (Site 154).

Superstructure--Hydraulic problems at a few sites are attributed to characteristics of the bridge superstructure. Vertical clearance is regarded as low if it is inadequate to pass debris at the design flood level. An example of this is provided by Site 25, at a bridge built in 1935. At Site 71, clearance was reduced by aggradation and increased by the measure of raising the bridge superstructure. At Site 11, a new bridge was built with a clearance considered too low, but the railings were designed to minimize flow resistance. At Site 14, clearance was increased by excavation of the channel, which has apparently been effective. No further information on the practice of increasing clearance by excavation was obtained in this study, but its effectiveness would seem to depend on an increase in velocity in the bridge waterway, as compared with velocity in the unconfined channel. Otherwise, the excavated section of the bridge waterway would quickly fill with bed material.

During flood submergence of all or part of the superstructure, downstream displacement of the spans may occur by a combination of buoyant and dynamic forces, if the spans are not properly secured (Bradley, 1973, p.45). One example of such failure (Site 148) was found.

Table 28. Approach roadway factors contributing to hydraulic problems at study sites

Factor	Number of site where problem occurred
Constriction of flow	7, 14, 26, 27, 28, 30, 31, 32, 35, 37, 38, 42, 43, 46, 48, 50, 53, 54, 123, 143, 146, 154, 156, 162, 172.
Diversion of flood flow	27, 35, 45, 46, 50, 56, 154, 204, 260, 280.

Channel alterations--Channel alterations associated with hydraulic problems at study sites (table 29) include straightening, enlarging, and diversion of braids or anabranches. At most of these sites, the problems occurred during floods exceeding the design flood. At Sites 45, 48, and 53, increased slope from channel straightening probably contributed to general scour in the bridge waterway. At Sites 95, 99, and 125, the artificial channel proved to be unstable and countermeasures were required for stabilization. At Site 134, the concrete lining of an artificial channel failed because of hydrostatic pressures. At Site 177 a tributary, relocated to enter upstream from the bridge, degraded and built a fan into the main channel. At Site 213, a meander was cut off and the relocated channel migrated laterally at the bridge. At Site 20, a channel that was artificially shaped to a trapezoidal cross-section gradually eroded to a more natural parabolic shape. Artificial channels with raw and unvegetated banks are likely to require bank protection. Methods for establishing vegetation on banks are given by Normann (1975).

Clearing of vegetation along a channel near a bridge is probably a factor in bank erosion at many sites, but is difficult to isolate from other factors. The effect of such clearing is most apparent at crossings on forested flood plains (Sites 46, 53, 211).

Table 29. Channel alterations contributing to hydraulic problems at study sites

Kind of alteration	Number of site where problem occurred
Straightening	45, 48, 53, 95, 99, 125, 134, 213
Enlarging	7, 20
Diverting	7
Clearing of vegetation	46, 53, 211

Chapter 6

FLOW FACTORS

EVALUATION OF FLOW FACTORS

The design of bridges and countermeasures to prevent damage from streamflow requires assessment of certain factors, here called "flow factors," that characterize streamflow and channel conditions at the bridge site. The importance of flow factors in the bridge design process is influenced by, among other things, the importance of the bridge and by land use on the flood plain. Flow factors, some of which may also be regarded as geomorphic factors, are listed below:

1. Water level or stage
2. Discharge
3. Flood frequency (recurrence interval)
4. Duration of flooding
5. Water velocity
6. Floating debris
7. Source of flow (streamflow at site represents either natural conditions, effects of artificial inflow or diversions, or regulated flow)
8. Channel geometry
9. Channel slope
10. Presence of existing features
11. Suspended sediment and bed material
12. Channel alinement

These flow factors are related to the bridge design process in several ways, and changes in one factor may subsequently affect several other factors. There are instances where it is desirable to modify the channel, flood plain, amount of flow, or flow alinement at a bridge site. Knowledge of existing flow factors can be used as a guideline to estimate the effects of a proposed site modification. Hydraulic concepts and design procedures required for adequate consideration of these factors are summarized in table 30.

The determination of the design flood at a bridge is usually based on historic or observed flow data. Sometimes these data are available at the site, but more often they are estimated from rainfall records , or transferred by statistical methods from a streamflow gaging station or group of stations nearby (Waananen and Crippen, 1977). The water level (stage) associated with the design flood is also estimated from historical data or computed from channel conditions as measured at the bridge site. Estimates of these flow factors are used in the derivation of most of the other flow factors used in the design of a bridge or countermeasure. To illustrate, a lack of water level or discharge data at a bridge may give misleading design criteria if unexpected or temporary flow conditions, such as backwater, occur. Also, flow factors determined at a bridge site are affected by geomorphic changes. For example, several factors will change if a meander cutoff occurs in the vicinity of a bridge site. The occurrence of man-induced or natural channel changes presents a problem to the bridge designer because hydraulic conditions used in the design of the bridge may change with time. An understanding of the various factors applicable at a bridge site is needed if the bridge or countermeasure design is to be adequate for its intended purpose (Blodgett and Stiehr, 1974).

FACTORS FOR HYDRAULIC ANALYSIS OF STUDY SITES

In the determination of bridge, geomorphic, and flow factors unique to each bridge site, data-processing procedures were standardized to provide uniformity of results. Factors describing bridge, geomorphic, and flow conditions at study sites, and the assumptions made in the derivation of these factors, are discussed in this section.

Bridge Factors

Bridge length--This factor refers to the total distance across the main channel between abutments. The length of relief bridges, if present, is not included.

Foundation type--The bridge foundation is one of two basic types, either spread-supported or piling-supported. At some bridges, both types of foundations are used. Spread footings may rest on alluvium or bedrock. Piling footings are either driven piles or bored caissons, and provide support by end bearing or friction bearing.

Skewness--The definition of bridge skew angle requires clarification, because the effect of the bridge crossing angularity may be insignificant as compared with the effect of piers skewed to flow. The skewness, or angularity, of a bridge is the acute angle subtended by a line normal to the main axis of a bridge and a line parallel to the main axis of streamflow (fig. 38).

The skewness of a bridge pier is also referenced to the main axis of streamflow (fig. 38), but the pier skewness may not be equivalent to the bridge skew. If the piers are alined with the flow, the pier skew (fig. 38) is zero, but if the piers are normal to the main axis of the bridge, the skewness of the bridge and piers will be the same.

To obtain the normal waterway opening, the angle of bridge skew should be considered separately from pier skew.

Referring to figure 38, the relation to determine the normal waterway opening of a bridge, including the width of piers, is:

$$b = L \cos \phi \qquad (1)$$

Table 30. Flow and geomorphic factors in bridge and countermeasure design

Flow factor	Purpose of data to define flow factor	Relation of flow factor to design (with site numbers of selected case history examples)
1. Water level or stage	Document water levels for existing channel conditions. Relate water level to discharge. Determine backwater (Bradley, 1973; Blodgett and Stiehr, 1974).	Clearance requirements of bridge (8,9,25,69,71,99,250). Height of bridge, approach embankment and countermeasures (9,18,41,49,99,201,267). Size of bridge to prevent excessive backwater caused by bridge constriction (99). Type of bridge structure and approach embankment if bridge submergence or approach overtopping is expected (11,18,19, 22,34,132,201).
2. Discharge	Determine magnitude of flood events (Blodgett and Stiehr, 1974). Evaluate distribution of flows in channel and flood plain (Bradley, 1973; Blodgett and Stiehr, 1974). Determine relation of discharge to water level.	Design flood discharge (8,9). Design flood can be compared to historical events on the basis of flow quantity (8,9,11,46,48,54,101,250). Bridge size and waterway requirements (4,7,8,9,12,16,20,28,29, 30,31,35,45,46,48,49,61). Need for overflow bridges or road overflow (5,22,41,46,100). Bridge height (by use of discharge-water level relation)(69).
3. Recurrence interval (flood frequency)	Determine recurrence interval of design flood (Water Resources Council, 1976; Waananen and Crippen, 1977). Determine recurrence interval of other flood events (Blodgett and Stiehr, 1974).	Flood discharge and related stage for subsequent use in bridge design (8,9,99,260). Enable relation of design flood to historical event on basis of interval of time (237,272). Enable relation of design and historic floods to meet legislative requirements.
4. Duration of flooding	Estimate length of time water surface is above a base level (Blodgett and Stiehr, 1974).	Requirements to prevent excess pore-water pressure (134, 167, 217). Height of bridge, approach embankment, countermeasures. (Inundation for short periods of time acceptable; long periods unacceptable)(18,19,22,100,223).
5. Velocity	Determine Froude number, possibility of scour.	Bridge size by comparing existing flow velocities to anticipated values at bridge constriction (53). Bridge location and countermeasures for unstable channel boundary conditions (8,28,51). Bridge size and pier type to prevent scour caused by bridge constriction (4,11,23,26,28,45,53,61,154,155).
6. Floating debris	Estimate size of debris (Blodgett and Stiehr, 1974). Estimate source (anchor ice, logs, trees) (Neill, 1973).	Clearance requirements of bridge (6,8,9,14,165). Pier shape and footing type (1,8,60,115,132,168,258,268). Bridge size and spans to prevent debris blocking bridge waterway (27,58,70,88,89,155,157,168,262,266).

Table 30. Flow and geomorphic factors in bridge and countermeasure design (continued)

Flow factor	Purpose of data to define flow factor	Relation of flow factor to design (with site numbers of selected case history examples)
7. Source of flow	Evaluate changes in distribution of flow at bridge site (Blodgett and Stiehr, 1974). Evaluate changes in flow patterns associated with artificial inflow, such as storm drain outfall, dam failure, or new dams. Determine changes in sediment discharge rates (Porterfield, Busch, and Waananen, 1978).	Height of bridge, approach embankment, and countermeasures (41). Size and location of main channel and overflow bridges (57, 138,204). Countermeasures to prevent scour, degradation, and lateral erosion (27,39,160,167,173,184,238,267). Type of pier and abutment footing.
8. Channel geometry	Determine channel area, wetted perimeter, conveyance, distribution and depth of flow, velocity, Froude number, boundary shear stress (Chow, 1959). Document existing channel size and shape.	Location and size of main channel and overflow bridges (57, 68,218). Part of channel conveying flow (46,48,151,153). Length of piers and location (1,8,11). Size, shape, and alinement of proposed channel based on existing channel size (7,9,13,20,45,53,91,92,99,101,220, 240,243).
9. Channel slope	Determine channel capacity. Determine boundary shear stress (Nece, 1974, and Volume II of this report). Determine existing channel gradient (Blodgett and Stiehr, 1974).	Size of bridge waterway to maintain channel capacity for design discharge (8,13,57,99,262). Countermeasures to prevent degradation, scour, and lateral erosion (1,8,13,39,45,48,85,89,124,125,134,148,174,277,282).
10.* Presence of existing features	Evaluate effect on channel capacity. Evaluate effect on flow alinement and distribution. Estimate effect on channel sediment transport rates.	Length and location of bridge (features upstream or downstream from site may affect capacity of channel)(37, 39,40,41,42,44,50,52,55,56,85,146,254). Need for countermeasures to maintain desired flow alinement or capacity (42,44,50,55,99,125,194,196,261). Need for countermeasures to prevent scour (39,52,56,177,197, 198,201).
11. Suspended sediment and bed material	Estimate availability of bed material for transport (Normann, 1975). Estimate roughness coefficient in computation of channel capacity (Mannings n)(Barnes, 1967).	Size of bridge required to convey design discharge or prevent damage by aggradation (57,71). Type of bridge pier and abutment footings (8,11,31,32,44,51,73). Type and location of piers to prevent abrasion and impact damage (1,3,8,24,60,74,127,132,140). Countermeasures to stabilize channel boundaries (1,7,8,17, 21,23,39,126,164).
12. Channel alinement	Document existing channel alinement (Brice, 1977).	Bridge and approach location relative to channel and flood plain (7,14,101,157,163,177,182,216,234). Countermeasures to stabilize channel location (7,13,21,33,36, 43,45,63,85,92,95,99,116,120,126,130,131,133,169,170,180, 192,211,225,226,233). Pier shape and location (7,38,99,126,228,263). Type of pier and abutment footings (11,51).

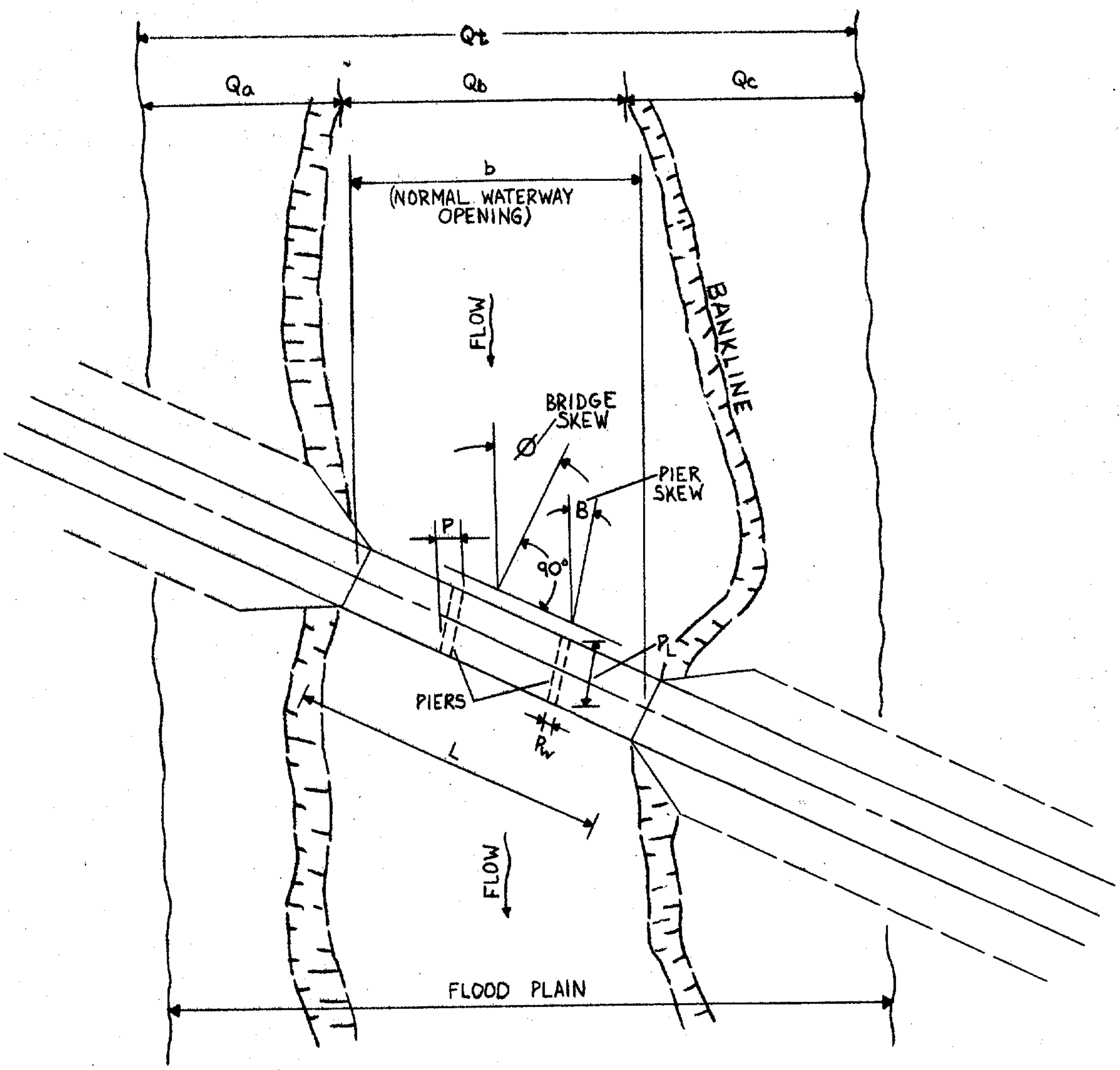

Figure 38. Definition sketch of a skewed crossing and overbank flow.

To determine the effective width of an individual bridge pier,

$$P = P_L \sin \beta + P_w \cos \beta \qquad (2)$$

In instances where single round piers are used or pier skewness angles are zero, indicating the piers are alined with the waterway, the effective pier width is:

$$P = P_w \qquad (3)$$

The net waterway opening at a bridge with skewness would be:

$$b_n = [L \cos \phi - \sum_{O}^{N} P] \tag{4}$$

where: $P = P_w$ for single round piers or piers alined with the waterway.

$P = P_L \sin \beta + P_w \cos \beta$ for each pier skewed to the waterway.

$N =$ Number of piers in waterway.

Pier shape--Because of the diversity of pier shapes, it was necessary to develop a classification of the shapes (fig. 39). In some cases, piers classified as pile bents are constructed as several rectangular or round columns in a bent, but are founded on a pile cap or spread footing. The classification was therefore based on the pier configuration as exposed to the streamflow.

Classification	Shape	Description
Pile bents	HHHH	Series of H piles
	□□□□	Series of square piles
	○○○○	Series of round or octagonal piles
	○ ○	Pair of round columns
	□ □	Pair of square columns
Web piers		Pair of round columns with web
		Pair of rectangular columns with web
Wall piers		Round nose pier
		Pointed nose pier
		Square nose pier
Cylinder piers	○	Large cylindrical column, one column per pier

Figure 39. Classification of pier shapes.

Abutment type--Bridge abutments are classified as spillthrough or vertical. A spillthrough abutment has a fill-slope on its streamward side (fig. 40). A vertical (full height) abutment usually has wingwalls but no fill-slope on its streamward side.

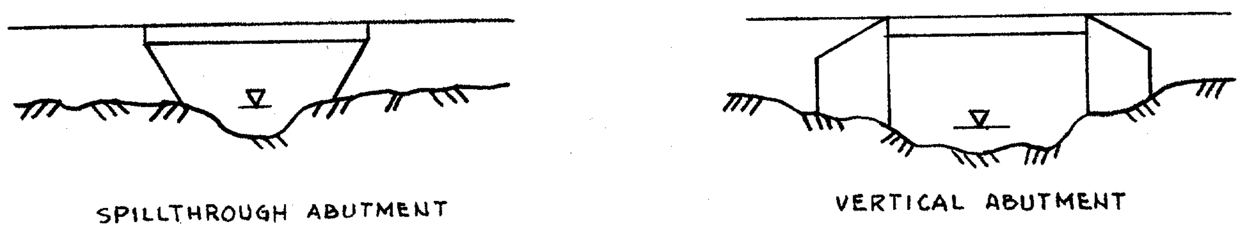

Figure 40. Classification of abutment types.

Geomorphic Factors

Channel slope--Most channel slopes were determined from topographic maps, but surveyed stream profiles were available for some sites. Where channel slope could not be determined from topographic maps, the valley slope was used. Channel slope is based on the difference in streambed elevation between two points and the horizontal distance as measured along the stream centerline. Valley slope is based on the difference in streambed elevation between two points and the horizontal distance as measured along a straight line parallel with the valley axis.

Bed material--The size of bed material was obtained from data shown on bridge plans or based on visual observations of the site. At some sites, bed material samples had been collected for other purposes and the particle size had been analyzed. The median grain size (D_{50}) is used as an expression of the particle-size distribution.

Flow Factors

Discharge--Streamflow data associated with hydraulic problems or countermeasures at a bridge site were usually determined from records obtained at Geological Survey gaging stations. At some sites, indirect measurements of floods, or results of special studies such as bridge site analyses, were available. When possible, the discharge used in design of a bridge was included for comparison with flood events associated with hydraulic problems.

Recurrence interval--The recurrence interval of floods related to hydraulic problems or countermeasures at a bridge site was determined from streamflow records or historic data that indicate the relative magnitude of a flood event. The statistical analysis was based on a series of annual maximum floods so the computed flood recurrence interval is the average interval of time within which the given flood will be equalled or exceeded as an annual maximum.

The analysis of the flood data was generally based on procedures described by the U.S. Water Resources Council (1976).

Depth--The average depth of flow at a bridge was computed by the relation:

$$\overline{Y} = A/L \tag{5}$$

where A is average cross-sectional area of stream normal to flow, and L is length of bridge adjusted for skewness. The hydraulics of bridge openings were determined at the most constricted location in each channel, generally at the downstream side of a bridge (Matthai, 1967).

Velocity--The average velocity of flow through bridge constrictions is computed by the relation:

$$\overline{V} = Q/A \tag{6}$$

where Q is discharge of stream and A is cross-sectional area of stream at the downstream side of the bridge. The gross waterway area was used in computing the average velocity of flow, without subtracting the area of piers; this is the procedure followed by Bradley (1973). At those sites where debris accumulation reduced the effective size of a waterway, and sufficient data were available, a separate computation of flow velocity was made to illustrate the effect of debris accumulation on the waterway area.

Eccentricity--Bridges and approach embankments located on streams with wide flood plains may constrict the overbank discharge during large floods (fig. 38). If the main channel is near one edge of a flood plain, overbank flow is generally forced laterally along the approach embankment for a distance in order to return to the main channel. A measure of this type of flow constriction was computed as described by Bradley (1973):

$$e = 1 - \frac{Q_a}{Q_c} \tag{7}$$

where Q_a is less than Q_c; or:

$$e = 1 - \frac{Q_c}{Q_a} \tag{8}$$

where Q_c is less than Q_a. If the bridge opening is skewed and one side of the flood plain is completely blocked (Q_a or Q_c equals zero), the value of e becomes 1.

Contraction ratio--The effect of a constricted bridge opening and of approach embankments on the passage of main channel and overbank flow (fig. 38) is measured by the bridge contraction ratio (Bradley, 1973):

$$M = \frac{Q_b}{Q_t} \qquad (9)$$

where Q_b is unconstricted discharge. According to Matthai (1967), the contraction ratio is computed by the relation:

$$M = 1 - \frac{Q_b}{Q_t} \qquad (10)$$

Contraction ratios given herein were computed according to Bradley, and the value of the ratio is, therefore, one if there is no contraction of flow.

Pier area--The area of bridge piers or pile bents occupying the waterway was determined at the downstream side of the bridge. Normally, the piers and pile bents do not occupy more than about 3 percent of the waterway. The effect of bridge piers and pile bents on the flow area at the bridge is described by Bradley (1973) and is measured by the relation:

$$j = \frac{A_j}{A} \qquad (11)$$

where j is ratio of pier area to gross area of waterway, A_j is area of piles or piers in the waterway, and A is the cross-sectional area of the stream.

Bridge submergence--During extreme floods, or when bridge waterways are blocked by debris, submergence of a bridge may occur. The amount of submergence (t) is measured as the depth of water above the "low steel" or bottom of the bridge stringer (fig. 41). The effects of bridge submergence on flow are described by Bradley (1973).

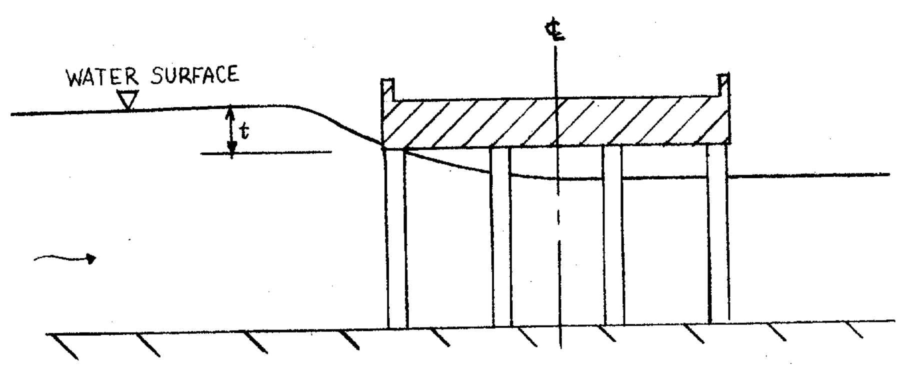

Figure 41. Definition sketch of a bridge submergence.

Froude number--The Froude number, a measure of the specific energy of streamflow, was computed by the relation:

$$F = \frac{\overline{V}}{\sqrt{g\overline{Y}}} \tag{12}$$

where $\overline{V}$ is mean velocity of flow, g is the gravitational constant, and $\overline{Y}$ is average depth of stream. Flow conditions were determined at the downstream side of bridges since this is generally the most constricted location.

Bed shear stress--The bed shear stress was computed by the relation:

$$\tau_o = \gamma\overline{Y}S_o \tag{13}$$

where γ is specific weight of water, $\overline{Y}$ is average depth of flow, and S_o is the slope of the water surface or the channel bed. The bed shear stress was determined at the constricted cross section, which is assumed to be at the downstream side of the bridge. Average depths of flow were used in the relation, rather than hydraulic radius of the channel, because mean depth was more readily determined. For channel shapes most common in nature, the mean depth and hydraulic radius are approximately equal and errors introduced by use of mean depth are negligible. Theoretically, the slope of the energy grade line, S_f, is used in the determination of bed shear stress. However, data available at the bridge sites were usually inadequate for determination of the energy grade line, and the channel bed slope, S_o, was therefore used as an approximation in equation (13).

FLOW FACTORS FOR FLOODS AT STUDY SITES

Study sites at which hydraulic problems involving local or general scour were documented, and for which adequate data for hydraulic analysis were obtained, have been placed in Vol. II and the results of analysis are given in table 31. The purpose of the tabulation is to indicate the range in values of flow factors, such as velocity and bed shear stress, from one site to another. This range reflects differences in flood magnitude and in geomorphic and hydraulic characteristics of the sites.

Bridge factors in relation to geomorphic and flow factors--The bridge factors listed in table 31 are those considered to be most important in hydraulic analysis and in assessment of the effects of adverse geomorphic and flow factors. The length of the bridge indicates the size of channel crossed and length tends also to be related to bridge cost. Foundation requirements for modern bridges are more stringent than for older bridges, and the susceptibility to damage by scour has been lessened. Spillthrough abutments are now in general use because they are less expensive than vertical abutments, but they usually require protection by countermeasures.

["--" indicates factor is not applicable or hydraulic data are inadequate for computation. See "List
Ft^3/s multiplied by 0.0283 equals m^3/s. Lb/ft^2 multiplied by 4.88 equals kg/m^2. "Flow reg"

	Bridge factors								Geomorphic factors	
					Pier		Abutment			
Site number and location	Year bridge built	Length of bridge (ft) (L)	Type of foundation for pier and abutment	Skewness: bridge ϕ, pier β, (degrees)	Pier shape	Nose shape	Type	Wing wall angle (degrees)	Channel slope (ft/ft) (S_o)	Bed material and D_{50} (mm)
1-Cache Cr at I-505 nr Madison, Calif.	1959	599	Piles	ϕ=18 β=0	Wall	Round	Spill through	--	0.00111	Gravel (40)
2-Cascade Cr at SR-89 nr South Lake Tahoe, Calif.	1935	39	Spread	ϕ=0 β=30	Wall	Square	Vert.	0	.0727	Gravel, cobble, boulders
3-N. Yuba R at SR-49 nr Goodyears Bar, Calif.	1964	312	Spread	ϕ=45 β=0	Cyl.	Round	Vert.	0	.0050	Bedrock, cobble, gravel
4-Sacramento R at SR-162 at Butte City, Calif.	1948	693	Piles	ϕ=8 β=0	Wall & guide fenders	Pointed	Spill through	--	.00029	Medium sand (0.30)
5-Salmon Cr at FS road 20N01.6 nr Potter Valley, Calif.	1975	106	Piles	0	Wall	Round	Vert.	45	.0050	Sand, gravel
6-Smokehouse Cr at FS road 20N01.6 nr Potter Valley, Calif.	1975	106	Piles	0	Wall	Round	Vert.	45	.0050	Sand, gravel
7-Stony Cr at I-5 nr Orland, Calif.	(Prior to construction of bridge)								.0022	Sand, gravel, cobble (3.3)
	1966	934	Piles	ϕ=30 β=0	Bents	Round	Spill through	--	.0022	do.
8-Salmon Cr at SR-49 nr Sierra City, Calif.	1964	252	Spread	ϕ=60 β=0	Wall	Round	Vert.	0	.050	Cobble, boulder, bedrock
9-Trinity R at SR-3 nr Coffee Cr, Calif.	1966	404	Piles	ϕ=45 β=0	Wall	Round	Spill through	--	.011	Gravel, cobble
10-Rabbit Cr at SR-45 at Murphy, Idaho	1956	40	Spread	ϕ=5 β=10	Wall	Round	Spill through	--	.027	Sand, gravel
11-Canadian R at SR-65 nr Sanchez, NM	1928	336	Spread	0	Wall	Pointed	Vert.	45	.0026	Gravel, cobble

[1]Discharge should be a reliable estimate. Value computed using inadequate data.

and flow factors for selected floods

of Abbreviations and Symbols" for definition of symbols. Feet multiplied by 0.305 equals meters.
indicates flow is regulated]

Flow factors for selected floods												
Date of flood or event (month, year)	Discharge (ft^3/s) (Q)	Recurrence interval (years) ($R.I.$)	Average conditions at bridge cross section: Depth (ft) ($\bar{Y}$)	Average conditions at bridge cross section: Velocity (ft/s) ($\bar{V}$)	Bridge eccentricity (e)	Bridge contraction ratio (M)	Ratio of area of piers to gross area (j)	Bridge submergence (ft) (t)	Froude number (F)	Bed shear stress (lb/ft^2) (τ_o)	Remarks
1-65	38,000	22	9.5	9.3	0	1	0.04	0	0.53	0.66	Flow reg.
1-73	21,300	5	8.8	5.6	0	1	.03	0	.33	.61	
12-64	323	--	1.5	7.0	0	1	.07	0	1.0	6.8	
12-64	44,000	40	19.6	11	0	1	.08	0	.44	6.1	
3-49	69,900	2	24.6	4.7	0	1	.11	0	.17	.45	Discharge in main channel only; flow regulated
12-51	78,800	4	25.4	4.9	0	1	.11	0	.17	.46	
1-74	84,300	12	29.6	4.4	0	1	.09	0	.14	.54	
Design	10,500	--	8.2	13	--	--	.03	--	.80	2.6	
Design	3,850	--	4.4	8.8	--	--	.03	--	.74	1.4	
1964	18,700	--	5.3	5.7	.70	.28	.05	0	.44	.73	Flow reg.
1967	10,800	--	3.6	5.4	.70	.28	.09	0	.50	.49	Flow reg.
1970	12,500	--	6.8	2.4	--	--	.04	0	.16	.93	Flow reg.
1974	15,200	--	5.0	3.9	--	--	.04	0	.31	.69	Flow reg.
12-64	[1]19,200	--	9.1	23	0	1	.05	0	1.4	28.3	
1-74	26,500	40	9.0	11.1	0	1	.02	0	.655	6.2	
Design	14,000	--	8.6	6.6	0	1	.02	0	.40	5.9	
6-62	3,640	--	4.9	14.9	0	1	.052	0	1.2	8.3	
6-65	126,000	50	24	16	0	1	.03	1.7	.58	3.9	

Site number and location	Bridge factors: Year bridge built	Length of bridge (ft) (L)	Type of foundation for pier and abutment	Skewness: bridge ϕ, pier β, (degrees)	Pier: Pier shape	Pier: Nose shape	Abutment: Type	Abutment: Wing wall angle (degrees)	Geomorphic factors: Channel slope (ft/ft) (S_o)	Bed material and D_{50} (mm)
12-Walnut Cr at I-25 nr San Antonio, NM	1964	132	Piles	0	Bent	Round	Spill through	--	0.0163	Gravel
13-Hutton Cr at I-5 nr Hilt, Calif.	1974	128	Spread	ϕ=26 β=0	Wall	Round	Spill through	-	.030	Sand, gravel
16-Deer Cr at SR-99 nr Vina, Calif.	1921	459	Piles	0	Wall	Round	Spill through	--	.0040	Sand, gravel
17-Eel R at SR-1 at Fernbridge, Calif.	1911	2,405	Piles	0	Wall	Pointed	Vert.	0	.0010	Sand, (0.19)
20-Red Clover Cr at FS road nr Portola,	1954	52	Piles	ϕ=0 β=74	--	--	Vert.	90	.0031	Sand, gravel
21-Sacramento R at SR-32 nr Chico, Calif.	1908	580	Piles	0	Cyl.	Round	Vert.	90	.00032	Sand, small gravel
23-S.F. Kings R at SR-180 nr Hume, Calif.	1938	122	Spread	ϕ=50 β=0	Wall	Square	Vert.	LB=40 RB=10	.0337	Gravel, cobble, bedrock
24-Yuba R at SR-20 nr Smartsville, Calif.	1913	685	Piles	0	Wall	Pointed	Spill through	--	.00234	Sand, gravel
25-Brockliss Slough at SR-88 nr Minden, Nev.	1935	76	Piles	ϕ=45 β=0	Bent	Round	Vert.	20	.00286	Sand
	1952	130	Piles	ϕ=45 β=0	Wall	Round	Spill through	--	.00286	Sand
26-Carson R at Dayton Lane at Dayton, Nev.	1951	132	Spread	0	Wall	Round	LB=vert. RB=spill through	90	.0033	Gravel, cobble
	1956	212	Pile	0	Wall	Round	Vert.	90	.0033	do.

[2]Flood frequency was determined at Eel River at Scotia gaging station and is comparable to frequencies at Fernbridge. Flow in main channel estimated to be 610,000 ft^3/s.

and flow factors for selected floods (continued)

Date of flood or event (month, year)	Discharge (ft^3/s) (Q)	Recurrence interval (years) (*R.I.*)	Average conditions at bridge cross section		Bridge eccentricity (e)	Bridge contraction ratio (M)	Ratio of area of piers to gross area (j)	Bridge submergence (ft) (t)	Froude number (F)	Bed shear stress (lb/ft^2) (τ_o)	Remarks
			Depth (ft) ($\bar{Y}$)	Velocity (ft/s) ($\bar{V}$)							
8-72	15,600	50	7.9	23.9	--	--	0.02	0	1.5	8.0	
3-74	600	20	2.2	10.3	0	1	.06	0	1.2	4.1	
12-37	23,800	60	11.2	4.7	--	--	.09	2.2	.25	2.8	
12-64	18,800	25	5.9	7.0	--	--	.05	0	.51	1.5	
3-74	11,900	6	5.5	6.3	--	1	.06	0	.47	1.4	
12-64	[2]800,000	350	21.4	19.0	0	1	.17	0	.72	1.3	
12-64	3,460	20	7.5	9.2	--	--	0	0	.59	1.46	
3-67	1,260	--	7.0	4.1	--	--	--	0	.27	1.4	
12-70	2,280	--	8.7	5.2	--	--	--	0	.31	1.7	
2-58	150,000	--	27.3	9.4	0	1	.07	0	.32	.55	
1-74	158,000	--	27.8	9.7	0	1	.07	0	.32	.56	
11-50	11,500	5	10.8	9.0	0	1	.035	0	.48	22.7	
12-66	13,600	10	10.6	10.8	0	1	.04	>0	.58	22.3	
11-50	109,000	--	14.5	14.5	0	1	.08	0	.67	2.1	Flow reg.
12-64	179,000	--	16.0	20.9	0	1	.07	0	.92	2.3	Flow reg.
12-37	1,300	30	3	3.3	0	.5	.032	>1	.34	.54	
12-55	2,000	100	4	3.8	0	1	.015	0	.33	.71	
12-55	30,000	70	12	18.9	.7	.5	.04	--	.96	2.5	
2-63	21,900	40	8	12.9	0	1	.04	--	.80	1.7	

Site number and location	Bridge factors: Year bridge built	Length of bridge (ft) (L)	Type of foundation for pier and abutment	Skewness: bridge ϕ, pier β, (degrees)	Pier: Pier shape	Pier: Nose shape	Abutment: Type	Abutment: Wing wall angle (degrees)	Geomorphic factors: Channel slope (ft/ft) (S_o)	Bed material and D_{50} (mm)
27-E.F. Carson R at SR-56 nr Gardnerville,	1936	103	Spread	ϕ=20 β=0	Wall	Round	Vert.	90	0.0050	Coarse gravel
	1956	173	Piles	ϕ=15 β=0	Wall	Round	Vert.	90	.0050	do.
28-Humboldt R at Airport Rd. nr Lovelock, Nev.	1950	132	Piles	0	Wall	Square	Spill through	--	.00090	Sand, gravel
29-Rocky Slough at SR-88 nr Minden, Nev.	1934	35	Piles	0	Bent	Round	Spill through	--	.00286	Sand, gravel
	1957	64	Piles	0	Wall	Round	Vert.	90	.00286	do.
30-Chemung R at LR-08066 nr Sayre, Pa.	1972	700	Piles	ϕ=22 β=0	--	Round	Spill through	45	.00073	Gravel, cobble
31-Conestoga Cr at SR-23 nr Churchtown, Pa.	1925	60	Spread	0	Wall	Pointed	Vert.	45	.00030	Gravel, cobble
	1973	60	Spread & piles	ϕ=20 β=0	--	--	Vert.	45	.00030	do.
32-Conestoga Cr at SR-10 nr Morgantown, Pa.	1929	80	Spread	0	Wall	Pointed	Vert.	45	.00180	Sand, gravel
	1973	60	Spread	0	(no piers)		Vert.	45	.00180	do.
33-Muncy Cr at SR-220 at Hughesville, Pa.	1973	232	Piles	ϕ=30 β=0	Wall	Round	Spill through	--	.0038	Coarse gravel
34-Muncy Cr at SR-147 nr Muncy, Pa.	1972	210	Spread on bedrock, piles	ϕ=8 β=0	Wall	Round	Spill through	--	.0023	Gravel, cobble
35-N.B. Susquehanna R at SR-4 nr Nanticoke, Pa.	1954	2,740	--	ϕ=39 β=0	Wall	Round	Vert & spill through	45	.00027	Gravel, cobble

and flow factors for selected floods (continued)

Date of flood or event (month, year)	Discharge (ft^3/s) (Q)	Recurrence interval (years) ($R.I.$)	Average conditions at bridge cross section: Depth (ft) ($\overline{Y}$)	Velocity (ft/s) ($\overline{V}$)	Bridge eccentricity (e)	Bridge contraction ratio (M)	Ratio of area of piers to gross area (j)	Bridge submergence (ft) (t)	Froude number (F)	Bed shear stress (lb/ft^2) (τ_o)	Remarks
12-37	11,000	30	9	11.9	0	0.6	0.08	>0	0.70	2.8	
11-50	10,000	30	9	10.8	0	.6	.08	--	.63	2.8	
12-55	17,000	100	--	--	0	.6	.08	--	--	--	
2-63	12,000	50	10	6.9	0	1	.012	0	.38	3.1	
12-64	7,000	25	8	5.0	0	1	.012	0	.31	2.5	
5-52	3,500	--	15	2.8	.5	.5	.02	0	.13	.84	Flow reg.
12-55	1,000	100	--	--	--	--	--	--	--	--	
2-63	900	50	4	3.5	0	1	.03	--	.31	.7	New bridge built
6-72	192,800	385	15	6.5	--	.90	<.02	--	.30	.68	
9-75	127,500	50	14.0	12.0	.74	.60	.02	0	.57	.64	
6-72	10,500	>100	10	--	--	--	.05	>0	--	.19	
9-75	1,470	4	4.0	6.1	0	.7	0	0	.54	.07	
6-72	7,770	>100	10.6	9.1	0	.8	.04	--	.49	1.2	
9-75	1,120	4	4.0	4.7	0	.9	--	0	.41	.45	
9-75	14,900	35	11	5.7	0	.9	.034	0	.30	2.6	
9-75	26,400	200	16.4	7.7	--	.8	.04	2.2	.34	2.4	
6-72	345,000	300	23.9	5.3	.60	.75	.08	--	.19	.40	

Site number and location	Bridge factors: Year bridge built	Length of bridge (ft) (L)	Type of foundation for pier and abutment	Skewness: bridge ϕ, pier β, (degrees)	Pier: Pier shape	Pier: Nose shape	Abutment: Type	Abutment: Wing wall angle (degrees)	Geomorphic factors: Channel slope (ft/ft) (S_o)	Bed material and D_{50} (mm)
36-N.B. Fishing Cr at SR-16 nr Central, Pa.	1932	58	Spread	0	--	--	Vert.	0	.0166	Gravel, cobble
	1973	140	Spread	ϕ=45 β=20	Wall	Round	Spill through	--	.0166	do.
37-Fishing Cr at SR-190266 at Light-street, Pa.	1938	162	Spread	--	Wall	Pointed	Vert.	--	.00173	Gravel
	1974	232	Piles	ϕ=7 β=0	Wall	Round	Spill through	--	.00173	do.
38-Saline R at US-70 nr Dierks, Ark.	1951	461	Spread	0	Wall	Square	Spill through	--	.0018	--
39-Trinity R at AT&SF RR nr Lavon, Texas	1951	589	Piles	0	Wall	Pointed	Spill through	--	.001	--
41-Tombigbee R relief opening No.2 at US-45 nr Aberdeen, Miss.	1934	100	Piles	0	Bents	Round	Spill through	--	.00014	Silt, sand
42-Fishing Cr at SR-487 nr Orangville, Pa.	1973	240	Piles	ϕ=49 β=0	Wall	Round	Spill through	0	.0025	Gravel, cobble
43-Lycoming Cr at US-15 nr Williamsport, Pa.	1955	248	Piles	ϕ=30 β=0	Wall	Round	Vert.	45	.0022	--
44-N.B. Susquehanna R at SR-93 nr Berwick, Pa.	1905	1,520	Piles, bedrock	ϕ=0 β=30	Wall	Pointed	Vert.	0	.000615	Sand, gravel, cobble
46-Fourche La Fave R at SR-28 nr Bluffton, Ark.	1967	504	Spread & piles	ϕ=21 β=21	Wall	--	Spill through	--	.0007	Gravel
47-Bull Mountain Cr at SR-25 nr Smithville, Miss.	1952	1,050	Piles	0	Bent	Round	Spill through	--	.00080	Sand

[3]Mean velocity computed assuming no road overflow.

and flow factors for selected floods (continued)

Flow factors for selected floods											
Date of flood or event (month, year)	Discharge (ft^3/s) (Q)	Recurrence interval (years) ($R.I.$)	Average conditions at bridge cross section: Depth (ft) ($\overline{Y}$)	Average conditions at bridge cross section: Velocity (ft/s) ($\overline{V}$)	Bridge eccentricity (e)	Bridge contraction ratio (M)	Ratio of area of piers to gross area (j)	Bridge submergence (ft) (t)	Froude number (F)	Bed shear stress (lb/ft^2) (τ_o)	Remarks
6-72	8,500	100	8.7	16.7	--	--	0	2.0	1.0	9.0	([3])
9-75	6,400	55	7.8	5.5	--	--	.03	--	.35	8.1	New bridge built
6-72	31,600	>100	12	13	0.6	0.5	.08	>1	.66	1.3	
9-75	29,300	95	12.7	10	.6	.75	.06	0	.49	1.4	New bridge built
5-61	52,100	50	13	9.0	--	.80	.030	0	.44	1.5	
5-68	59,200	75	--	--	--	--	--	--	--	--	
1-69	28,100	12	--	--	--	--	--	0	--	--	
5-57	39,000	--	10.2	6.3	--	--	--	--	.35	.64	
12-71	4,000	--	2.8	2.5	--	--	--	--	.26	.18	
3-73	5,060	65	10.7	4.8	--	--	.04	2.9	.26	.09	
3-75	4,360	--	5.9	8.0	--	--	.04	0	.58	.05	
9-75	26,100	100	13.8	10.8	--	.5	.05	0	.51	2.2	
6-72	34,800	85	17	9.5	.75	.7	.05	>0	.41	2.3	
9-75	23,800	25	15	6.8	.75	.9	.06	0	.31	2.1	
6-72	355,000	>100	26	9.3	.83	.92	.036	0	.32	1.0	
9-75	258,000	50	21	8.0	0	.95	.037	0	.31	.81	
12-71	72,000	50	20.3	7.0	--	.53	--	--	.27	.89	Main channel flow
6-74	60,000	25	--	--	--	--	--	--	--	--	
3-55	32,000	--	8.6	3.6	--	--	--	0	.22	.43	
3-73	36,000	--	13.5	2.5	--	--	--	--	.12	.67	

Table 31. Properties of bridges

Site number and location	Bridge factors: Year bridge built	Length of bridge (ft) (L)	Type of foundation for pier and abutment	Skewness: bridge ϕ, pier β, (degrees)	Pier: Pier shape	Pier: Nose shape	Abutment: Type	Abutment: Wing wall angle (degrees)	Geomorphic factors: Channel slope (ft/ft) (S_o)	Bed material and D_{50} (mm)
48-Beaver Cr at I-55 nr Tangipahoa, La.	1965	162	Piles	0	Bent	Square	Spill through	--	0.0059	Sand
49-E. Pearl R at I-10 nr Bay St. Louis, Miss.	1966	4,982	Piles	0	Wall with fenders	Square	Spill through	--	.00022	Sand
50-Nichols Cr at US-45 nr Aberdeen, Miss.	1934	200	Piles	0	Bents	Round	Spill through	--	.00014	Silt, sand
51-S. Canadian R at US-75 and US-270 nr Calvin, Okla.	1919	760	Spread	--	Web	Round	Vert.	--	--	Sand
	1966	1,225	Piles & spread	ϕ=19 β=19	Bent	Round	--	--	.00087	do.
	1977	1,260	Spread	ϕ=19 β=19	Web	Round	Spill through	--	.00087	do.
52-Pigeon Roost Cr at SR-305 nr Lewiston, Miss.	1950	352	Piles	0	Bents	Square	Spill through	--	.00070	Sand, clay
53-Bowie Cr at US-84 nr Collins, Miss.	1937	240	Piles	ϕ=30 β=0	Bents	Round	Vert.	None	.0013	Silt
54-Bronco Cr at US-93 nr Wickieup, Ariz.	1961	190	Piles	ϕ=30 β=0	Wall	Round	Spill through	--	.033	Sand, gravel
58-Osborne Wash at SR-172 nr Parker, Ariz.	1971	269	Piles	0	Bents	Round	Spill through	--	.0076	(0.4)
72-San Benito R at US-101 nr Chittenden, Calif.	1931	722	Piles	0	Wall	Pointed	Spill through	0	.0021	Sand

[4] No flooding in 1965. Discharge is for comparison with 1974 channel conditions. Total flow during 1974 flood was 21,400 ft^3/s.

[5] Total flow during 1955 flood, 106,000 ft^3/s, and 123,000 ft^3/s during 1973 flood.

and flow factors for selected floods (continued)

Flow factors for selected floods											
Date of flood or event (month, year)	Discharge (ft^3/s) (Q)	Recurrence interval (years) (*R.I.*)	Average conditions at bridge cross section: Depth (ft) ($\overline{Y}$)	Average conditions at bridge cross section: Velocity (ft/s) ($\overline{V}$)	Bridge eccentricity (e)	Bridge contraction ratio (M)	Ratio of area of piers to gross area (j)	Bridge submergence (ft) (t)	Froude number (F)	Bed shear stress (lb/ft^2) (τ_o)	Remarks
1965[4]	15,400	--	11.4	8.8	--	--	0.04	--	0.46	4.2	Before channel scour
1974	15,400	-	17.7	5.6	--	--	.03	4.0	.23	6.5	After channel scour
4-74	57,000	50	44.5	5.1	--	1.0	.25	--	.13	.61	
3-55	[5]18,100	40	18.0	2.5	--	--	--	--	.10	.16	Flow through bridge opening
3-73	[5]24,000	65	14.0	4.3	--	--	--	--	.20	.12	Do.
5-50	174,000	--	--	--	--	--	--	--	--	--	Largest flow of record
10-70	130,000	--	11.3	12.6	--	--	.04	0	.66	.61	New bridge built
10-70	94,100	--	15.1	9.8	--	--	--	0	--	--	New bridge built; measured discharge
4-69	22,800	7	11.6	7.4	--	--	.07	0	.38	5.1	
4-73	21,000	5	11.7	5.3	--	--	.06	--	.27	5.1	
4-74	15,000	100	24.6	2.4	--	--	--	0	.09	2.0	
8-71	73,500	--	21.6	22.2	0	--	.03	0	.84	44.5	Some road overflow
Design	14,000	50	9.1	6.1	--	--	.03	0	.36	4.3	
9-76	7,710	35	4.7	6.7	.83	.23	.03	0	.54	2.2	
12-55	7,460	10	5.5	2.1	0	1.0	.06	6.4	.16	.72	Flow reg.

Site number and location	Bridge factors: Year bridge built	Length of bridge (ft) (L)	Type of foundation for pier and abutment	Skewness: bridge ϕ, pier β, (degrees)	Pier: Pier shape	Pier: Nose shape	Abutment: Type	Abutment: Wing wall angle (degrees)	Geomorphic factors: Channel slope (ft/ft) (S_o)	Bed material and D_{50} (mm)
73-Salinas R at US-101 at Soledad, Calif.	1936	1,530	Piles	ϕ=30 β=30	Wall	Square	Spill through	--	0.0011	(0.42)
74-Ramshorn Cr at FS rd 39N86 nr Trinity Center, Calif.	1975	90	Spread	ϕ=15 β=0	(no piers)		Spill through	--	.0341	Gravel, cobble
88-Henrys Fork at SR-88 nr Rexburg, Idaho	1961	299	Piles	ϕ=30 β=0	Bent	Round	Spill through	--	.00021	Sand, gravel
90-Boise R at Fairview Ave at Boise, Idaho	1933	382	Spread	ϕ=30 β=30	Wall	Pointed	Vert.	0	.0033	Sand, gravel
91-Boise R at US-95 nr Parma, Idaho	1961	425	Piles	ϕ=45 β=0	Bent with webs	Round	Spill through	--	.00080	Gravel
97-Little Salmon R at US-95 nr New Meadows, Idaho	1975	168	Piles	ϕ=55 β=0	None		Spill through	--	.0039	Gravel, cobble
99-Pine Cr at I-90 at Pinehurst, Idaho	1964	105	Piles	ϕ=19 β=0	Pile bent	Round	Spill through	--	.00713	--
	1977	125	Piles	ϕ=19 β=0	Wall	Round	Spill through	--	.00713	--
101-Musselshell R at SR-200 at Mosby, Mont.	1929	436	Spread	ϕ=30 β=0	Wall	Round	Spill through	--	.0013	Sand, silt
	1972	375	Piles	ϕ=45 β=45	Wall	Pointed	Spill through	--	.0023	Sandy, silt
125-Cow Cr at I-5 nr Azalea, Ore.	1948	250	Spread	ϕ=50 β=0	Pile bent	Square	Spill through	--	.0054	Sand, gravel
139-Columbia R at SR-173 at Brewster, Wash.	1967	1,235	--	0	Wall	Round	Spill through	--	.00045	--

and flow factors for selected floods (continued)

Flow factors for selected floods											
Date of flood or event (month, year)	Discharge (ft^3/s) (Q)	Recurrence interval (years) ($R.I.$)	Average conditions at bridge cross section: Depth (ft) ($\overline{Y}$)	Average conditions at bridge cross section: Velocity (ft/s) ($\overline{V}$)	Bridge eccentricity (e)	Bridge contraction ratio (M)	Ratio of area of piers to gross area (j)	Bridge submergence (ft) (t)	Froude number (F)	Bed shear stress (lb/ft^2) (τ_o)	Remarks
2-69	106,000	--	9.0	10.0	--	1.0	0.05	0	0.59	0.62	Flow reg.
2-69	82,500	--	10.8	6.4	--	1.0	.05	0	.34	.74	Measured discharge
Design	4,000	--	5.4	11.3	--	--	--	0	.86	11.5	
6-76	69,300	--	24.3	9.5	0	1.0	.030	5.6	.34	.32	Flow reg.
4-43	21,000	--	7.2	7.8	--	--	.14	0	.51	1.48	Flow reg.
Design	20,600	--	8.9	7.2	--	--	.04	--	.43	.44	Flow reg.
6-74	8,100	--	5.1	5.2	--	--	.03	--	.41	.25	Flow reg.
Design	3,600	--	8.7	4.3	--	--	--	0	.26	2.1	
1974	8,500	85	7.4	13.3	0	1.0	.06	1.5	.86	3.3	
Design	8,500	--	7.5	11.1	0	1.0	.02	0	.71	3.3	New bridge built
1944	18,000	21	--	--	--	--	--	--	--	--	
5-75	11,100	9	9.6	3.6	0	1	.046	0	.20	1.3	Do.
Design	8,260	--	10.1	4.8	--	--	.06	--	.27	3.4	
1-74	10,400	>200	11.5	5.1	--	--	.06	--	.27	3.9	
1956	542,000	--	45	10.4	0	--	.02	0	.27	1.3	Flow reg.

Flow velocity--The range in velocity of floodflow from one site to another is indicated in table 31. In the derivation of mean velocity, the waterway area was determined at the downstream side of the bridge and may not be representative of the size of the natural channel. All of the sites for which velocities greater than 15 ft/s (4.5 m/s) are entered (sites 8,11, 12, 17, 24, 26, 36, and 54) were in extreme floods, of recurrence intervals ranging from 50 to at least 350 years. The Eel River at Fernbridge, Calif., (site 17), for which a mean velocity of about 19 ft/s (5.7 m/s) was computed, is upstream from a tidal estuary and is subject to tidal effects. A comparison of flow velocity in relation to channel slope at different bridge sites, indicates that mean flow velocities greater than 10 ft/s (3 m/s) are not uncommon at the lower range of channel slopes, as well as the higher.

Area of piers--The ratio of pier area to bridge waterway or gross area indicates the proportion of the bridge opening that is obstructed by piers. For the sites with piers analyzed, this ratio ranges in value from 0.012 to 0.25. A review of the case histories indicates that problems of local scour are more common where the ratio exceeds a value of about 0.10. In general, the incidence of local scour at piers is related to the size and number of piers, degree of pier skewness to floodflow, and the spacing of piers (span length).

Range of Froude numbers--The effect of gravity on the flow of a stream is represented by the ratio of inertial forces to gravitational forces. The Froude number represents this ratio (equation 12), and values greater than about 0.8 are an indication of critical flow conditions. Flows above critical (supercritical flow) are usually associated with a reduction in water-surface levels below normal depth for the same discharge; with the occurrence of a hydraulic jump, as manifested by a standing wave that is usually inconspicuous in natural channels; and with high flow velocity, turbulence, bed and bank scour, and suspension of bed material.

Values of the Froude number in table 31 range from 0.09 (site 53) to 1.5 (site 12). Values greater than 0.8 were computed for 10 sites. A review of case histories for the sites in table 31 indicates, however, that hydraulic problems such as scour are not necessarily restricted to flow conditions characterized by high Froude numbers (see, for example, sites 20 and 48). At site 48, scour to a depth of about 14 ft (4 m) occurred during a flood in 1974, even though the Froude number was only 0.46. Determination of the Froude number at this site was based on the channel size prior to the flood.

Range in bed shear stress--During floods, the bed shear stress in a channel increases in proportion to the depth of flow and the velocity, and the component of velocity is related to increases in energy gradient. The increase in shear stress causes instability of the channel boundary, and if the bed and bank material are not of sufficient size or degree of cohesion, scour and lateral erosion result. If unstable conditions at the bridge site are expected, countermeasures may be needed to prevent changes in channel size, shape, or location.

Design of bridges and countermeasures to prevent scour and lateral erosion requires a knowledge of the bed shear stress that may occur at the site. To counter excessive shear stresses, estimates of the erosive resistance of natural channel boundaries and proposed linings of riprap or other material are needed.

The relation of shear stress to bed material size in natural channels at selected bridge sites is shown in figure 42. Numbers on the plot refer to the case history site number (table 31). At several sites, the median particle size (D50) has been determined. For most sites, the median diameter of the bed material is unknown, and plotting positions were placed at the midpoint of the size (sand, gravel, or other) classification. The shear stress (τ_o) was obtained from table 31, and in instances where shear stresses were determined for several floods, only the maximum value was used in figure 42.

Analysis of figure 42 indicates that the bed shear stress at almost all sites exceeds the critical shear stress (τ_c) criteria developed by the U.S. Bureau of Reclamation (1962) and the Federal Highway Administration (1974) for stable channel conditions. This situation is the result of several factors such as:

1. Most bridge sites included in the study had a history of hydraulic problems representative of unstable channel boundaries.

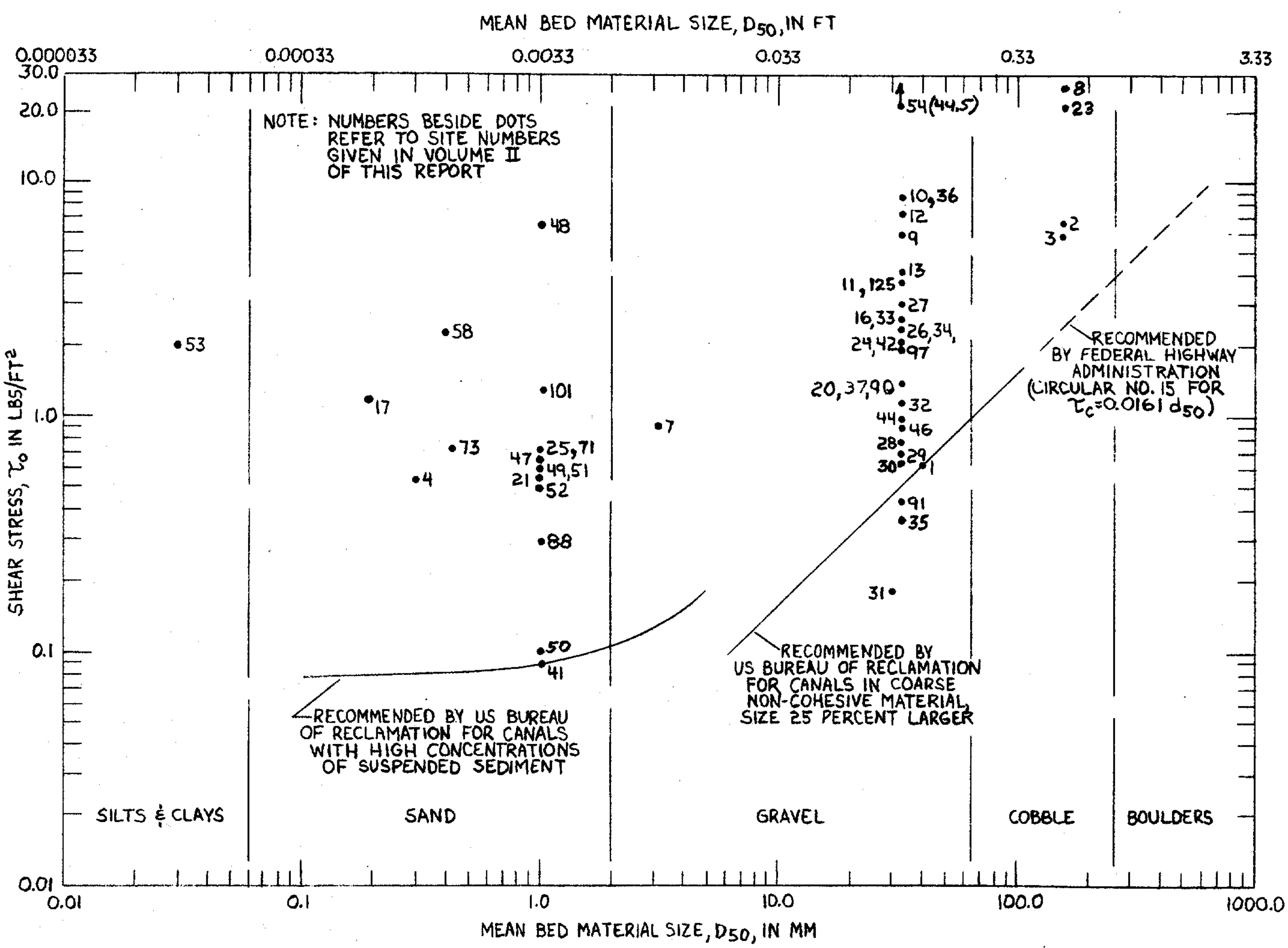

Figure 42. Relation of shear stress to bed material at selected bridge sites.

2. In many cases, shear stresses were computed for channel conditions and large flood events with high recurrence intervals.

3. The shear stresses were calculated assuming uniform flow instead of varied or unsteady flow conditions.

4. The size of bed material observed at a site may represent material deposited during low flow conditions. With higher flows, the smaller material generally moves into suspension, leaving coarser bed material which is more resistant to erosion and higher shear stresses. This condition is known as armoring.

5. The criteria in use for estimating stable channel conditions are based on average conditions and may not consider adequately such flow factors as high velocities that occur near the channel bed during floods.

One of the difficulties in applying the critical shear stress relations (fig. 42) to the design of stable channels is the lack of definition of these relations for higher values of shear stress that apparently occur in natural stream channels.

EFFECTS OF BRIDGES ON FLOW FACTORS

The construction of a bridge and approach embankment may alter in some way existing flow and geomorphic factors of a stream. Alteration of existing flow factors occurs because it usually is not practical to construct a bridge without placement of piers, abutments, or the approach embankment on the flood plain or in the main channel. For some bridge sites included in this study, sufficient quantitive data were assembled when preparing case histories of the bridge to analyze the effect of bridge and countermeasure design on flow factors. The rest of this chapter describes in detail several hydraulic problems and countermeasures that may be encountered in the river environment.

Flow Constriction Related to Inadequate Bridge Size

The case history of Red Clover Creek near Portola, Calif. (site 20), illustrates the type of channel change that may occur if the capacity of the bridge waterway is less than the capacity of the natural channel. A 52-ft (16-m) long bridge was built in 1954 across the main channel with no piers in the waterway. The approach embankments extend 500 ft (152 m) on the left bank and 150 ft (46 m) on the right bank, blocking overflow on the flood plain. The creek is a slow-moving mountain stream that meanders on a flat valley underlain by cohesive materials. Vegetation grows along the channel bank near the bridge but does not significantly affect the flow. At the time of bridge construction, the channel was reshaped to a trapezoid, and cobble riprap was placed on both banks. Following floods between 1954 and 1967, the channel size under the bridge increased 13 percent to compensate for the reduction in flow area caused by the bridge constriction. After 1967, there has been little change in the channel size even though large floods occurred in 1970 and 1974. The variation in flow factors at the bridge between 1954 and 1975 for an assumed discharge is given in table 32.

In this instance, assuming channel changes are related to the December 1964 peak, the size of the main channel increased 13 percent even though the average velocity of flow at the bridge for this flood was less than 10 ft/s (3.0 m/s). Also, the bed shear stress was not significantly larger (15 percent) than values of shear stress for flows before and after the December 1964 flood. The observed channel changes at this site illustrate:

1. Channel grading and improving did not prevent subsequent enlargement of the channel size by a flood of 20-year recurrence interval. The channel bed surveyed in 1967 and 1975 was parabolic in shape, indicating the trapezoidal channel built at the time of bridge construction was not a stable configuration.

2. A 13 percent increase in channel size is not large, but in this case became an important consideration because the area of scour was at the abutments.

3. Following the period 1954-67, when the channel size increased, only minor changes occurred, suggesting the channel geometry had stabilized--possibly caused by armoring of the bed material.

4. Absolute values of flow factors developed to indicate channel stability are not always indicative of the response of a stream to the bridge and associated countermeasures. The channel shape at a particular site is the result of a complex interrelation of streamflow, sediment transport and other geomorphic factors, and the bridge structure.

Table 32. Variation in flow factors for Red Clover Creek near Portola, Calif., between 1954 and 1975 (site 20)

Date	Water surface elevation (ft)[1]	Width of channel at water surface (ft)	Flow factors			
			Area (ft^2)[2]	Average velocity (ft/s)[3]	Froude number (F)	Bed shear stress (lbs/ft^2)[4]
1954[5]	5,344	40	262	[6]5.4	[6]0.34	[6]1.27
Dec. 1964[7]	5,346.3	50	377	9.2	.59	1.46
1967	5,344	40	297	[6]4.4	[6].28	[6]1.44
1975	5,344	40	296	[6]4.4	[6].29	[6]1.43

[1]Feet multiplied by 0.305 equals meters.
[2]Ft^2 multiplied by 0.093 equals m^2.
[3]Ft/s multiplied by 0.305 equals m/s.
[4]Lbs/ft^2 multiplied by 4.88 equals kg/m^2. Bed shear stress computed using channel slope of 0.0031 from topographic maps.
[5]After construction.
[6]Assumed discharge of 1,300 ft^3/s.
[7]Maximum flood occurring during period 1954-75, discharge 3,460 ft^3/s, flow factors computed on basis of channel size in 1954 and assuming no road overflow.

Flow Constriction Related to Approach Embankment

The case history for Fourche La Fave River at SR-28 near Bluffton, Ark., (site 46) provides data illustrating the effects of a bridge constriction on overbank flow. Flooding in December 1971 inundated the flood plain on both banks of the main channel (J. N. Sullavan, written commun., 1972). During this flood (discharge 75,000 ft^3/s), serious bank scour occurred on the left bank about 35 ft (11 m) upstream from the bridge. This scour is attributed partly to return flow from the left bank flood plain and partly to main channel flow deflected toward the left bank by a bend in the river upstream from the bridge. The distribution of flow through the bridge during the December 1971 flood indicates that flow from the left-bank flood plain occupied 150 ft (49 m) of the 504-ft (154-m) long bridge opening. Also, because of poor flow alinement at the junction of the left-bank overflow and main channel flow near the left-bank abutment, about 70 ft (21 m) of the bridge opening on the left bank (not part of the 150 ft) was not used to transfer flows downstream (fig. 43).

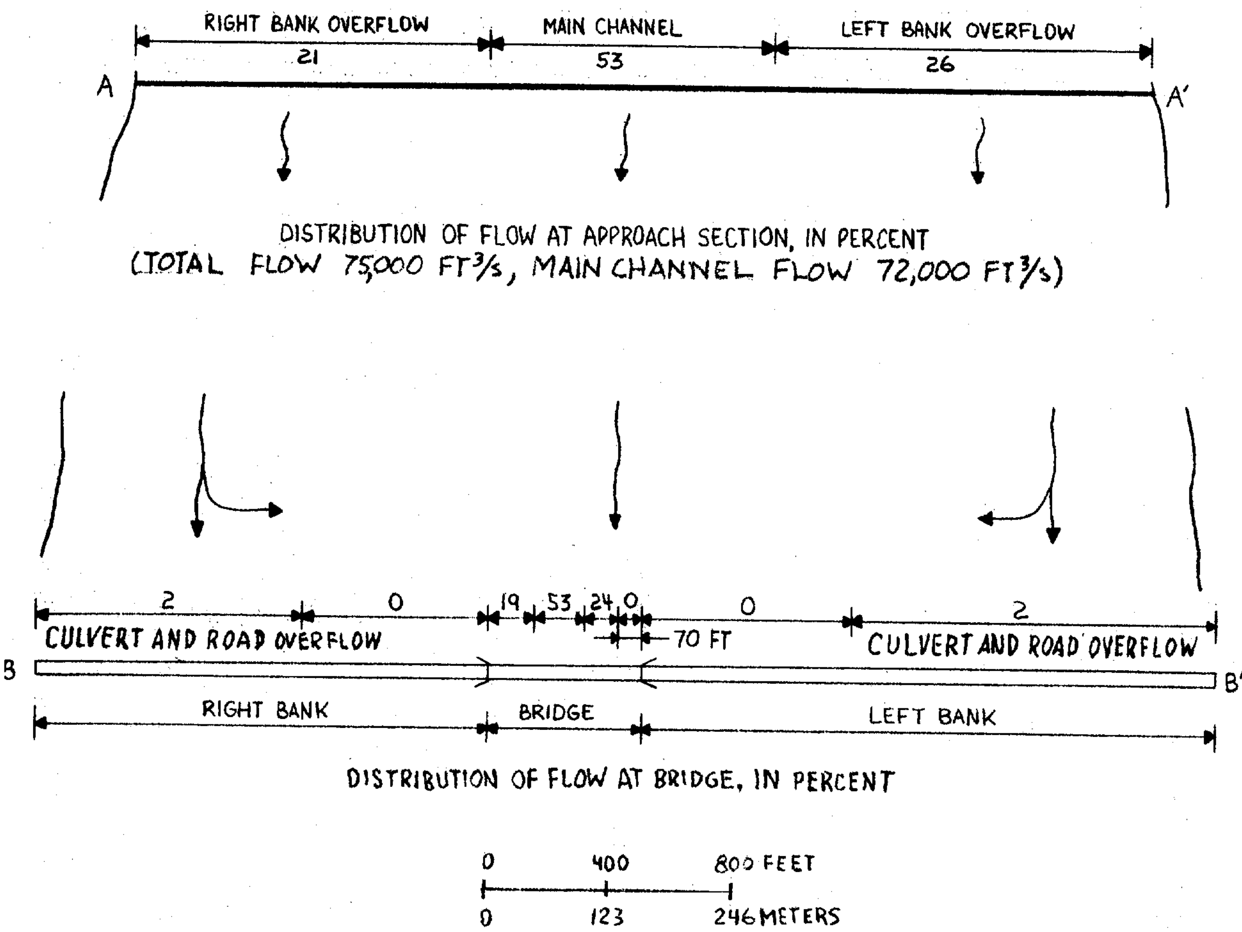

Figure 43. Distribution of flow, Fourche La Fave River at SR-28 crossing near Bluffton, Ark. (Site 46) during flood of December 10, 1971.

Bank erosion also occurred downstream from the bridge constriction. Approximately 43 percent of the total flow was forced to leave the flood plain and pass through the bridge constriction. As a result, flows formerly on the flood plain and temporarily constricted at the bridge could not be contained in the main channel and subsequently re-entered the flood plain downstream from the bridge on both banks of the stream. Local areas of bank erosion at the points of re-entry then occurred.

In 1973, a spur dike about 175 ft (53 m) long and covered with rock riprap was installed on the left bank upstream from the bridge abutment. This dike was extended downstream 255 ft (78 m) to prevent erosion of the left bank when flows re-enter the flood plain. The spur dike prevented the problem of erosion on the left bank during the 1974 flood (discharge 60,000 ft^3/s or 1,700 m^3/s), but erosion occurred on the right bank downstream from the bridge as it had in 1971. These two countermeasures, however, were only partially successful during floods of the magnitude experienced at this site because the bridge opening is inadequate in size. The spur dike and its extension served primarily to transfer the hydraulic problem further downstream and away from the bridge. Possible solutions to this type of problem would be the construction of a longer bridge across the main channel or provision of overflow bridges on the flood plain.

Flow Constriction Related to Large Piers

The case history for the Sacramento River crossing at SR-162 near Butte City, Calif. (site 4), provides data giving depths of scour resulting from a bridge constriction of the main channel. At the bridge site, overbank flow is not contracted because the right-bank flood plain is crossed by an approach trestle to the bridge. Streambed material at the site is medium size sand (D_{50} = 0.30 mm) for an unknown depth below the channel bed. The streambed level observed in 1946 was used as the base for determining later changes in the streambed.

The percentage of the waterway occupied by the piers would normally not be considered significant, but by the end of the 1946-74 period, an average depth of scour of 5.3 ft (1.6 m) (table 33) was recorded at the bridge though it had been as much as 7.5 ft (2.3 m) in 1969. The maximum depth of scour in the channel below the 1946 streambed level was 13 ft (4 m). Much of the scour is not adjacent to the bridge piers, indicating that local turbulence around the piers was not a dominant factor. Between 1954-74, about 30 ft (9 m) of lateral erosion occurred on the right bank. There are, however, only minor amounts of channel bed degradation outside the immediate area of the bridge. Measurements of the channel bed in 1945 and 1975 at a location of about 300 ft (91 m) upstream from the bridge showed scour of about 1 ft (0.3 m) occurred at the deepest part of the cross section, which may be related to channel changes at or downstream from the bridge. For a discharge of 7,000 ft^3/s (198 m^3/s), the water level was 1 ft (0.3 m) lower in 1975 than 1945. The observed changes in the channel at the upstream cross section are considered minor and indicate the large amounts of scour measured at the bridge are not caused by degradation of the channel.

Table 33. Hydraulic properties of Sacramento River at Butte City, Calif. (site 4)

Date of cross-section measurement	Water-surface elevation (ft)[1]	Discharge (ft^3/s)[2]	Average velocity (ft/s)[3]	Net cross-section area (ft^2)[4]	Average depth of scour since measurement of 11-25-46 (ft)
1942-46 (average value)	86.6	64,000	5.8	11,100	0
11-25-46 (assuming piers in place)	86.1	64,000	6.3	10,100	--
2-5-52	87.1	64,000	5.6	11,500	2.0
2-19-54	87.6	64,000	5.5	11,700	1.3
12-28-55	87.6	64,000	5.3	12,100	2.9
12-29-64	87.8	64,000	4.9	13,000	6.1
12-22-69	87.8	64,000	4.7	13,600	7.5
1-18-74	87.7	64,000	4.7	13,500	6.0
1-22-74	87.3	64,000	4.8	13,300	5.3
Cross section of channel 300 feet upstream from bridge					
11-8-45	71.7	7,010	1.9	3,700	--
11-20-75	70.7	7,010	1.9	3,620	--

[1]Feet multiplied by 0.305 equals meters.
[2]Ft^3/s multiplied by 0.028 equals m^3/s.
[3]Ft/s multiplied by 0.305 equals m/s.
[4]Ft^2 multiplied by 0.093 equals m^2.

The primary flow factor causing scour and lateral erosion of the Sacramento River at the site is the presence of large piers which cause a lateral constriction of the waterway and induce turbulence. The piers required to support the swing span and adjacent approach spans in the main channel occupy about 9 percent of the original channel (measured in 1946) at a bankfull discharge of 64,000 ft^3/s (1,812 m^3/s).

The limit of channel scour may not have been reached (table 33) by 1974. The cross-sectional area showed signs of stabilizing (fig. 44), but the maximum depth of scour below the 1946 channel bed (which is greater than the average depth of scour in table 32) was still increasing in 1974. Coarse bed material may have acted as an armor and restricted the amount of scour. Also, several floods occurred after construction of the bridge. Annual floods exceeding 64,000 ft^3/s (1,812 m^3/s) occurred five times during the 7-year period after construction of the bridge.

A countermeasure that may be used to stabilize a channel subject to extensive scour caused by large piers in the waterway is the placement of rock riprap on the streambed (sites 17 and 124).

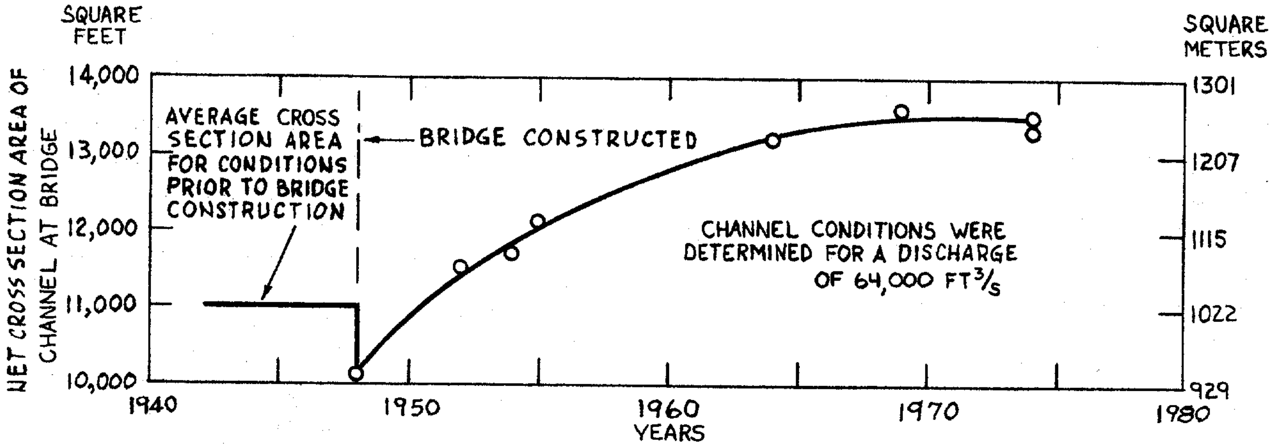

Figure 44. Variation in net channel area before and after bridge construction, Sacramento River at SR-162 at Butte City, Calif.

Flow Constriction Related to Skewed Piers

The effect of skewed bridge piers and a channel bend on flow is illustrated by data obtained during the January 1978 flood on Butte Creek near Gridley, Calif. (a full case history of this site is not available). Two wall-type bridge piers skewed about 19 degrees to the main channel flow (fig. 45) affected the flow pattern for about 37 ft (11 m) of the 150-ft (46-m) long bridge. In terms of the channel cross section at the bridge, flow in about 36 percent of the total area was affected by the skewed piers which tended to cause a shadow effect (fig. 45) with eddies and flow turbulence.

Changes in the velocity of flow, measured at the downstream side of the bridge, reflect the effect of the skewed piers. The velocity of flow decreased near the left side of piers 1 and 2 (fig. 45), and near station 90, velocities went from positive to negative, indicating reverse flow and eddies in the current.

A plot of cumulative discharge (fig. 45), expressed in percent, for the discharge measurement graphically illustrates the effects of a bridge located at a channel bend with skewed piers. Flatter slopes of the cumulative discharge plot indicate locations of reduced flow velocity and discharge. The cumulative discharge plot indicates about 74 percent of the bridge waterway is conveying 98 percent of the total flow. In terms of distance across the 150-ft (46-m) long bridge, the plot indicates 98 percent of the total flow is conveyed within 86 ft (26 m) of the left-bank abutment.

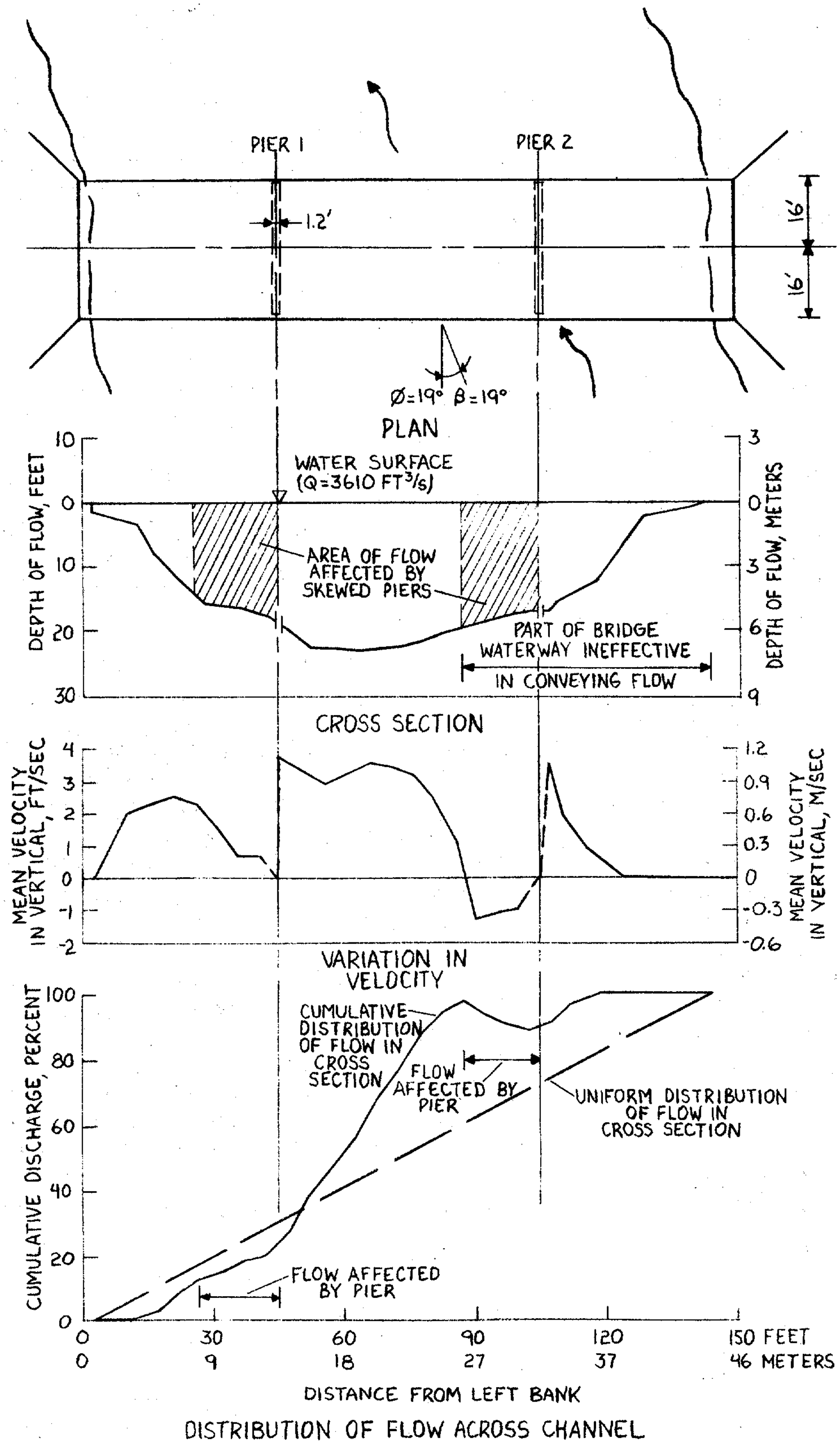

Figure 45. Effect of skewed wall-type piers on flow at downstream side of bridge, Butte Creek at Gridley-Colusa road crossing near Gridley, Calif.

EFFECTS OF COUNTERMEASURES ON FLOW FACTORS

Countermeasures, just as the construction of a bridge and approach embankment, may alter existing flow and geomorphic factors of a stream. In fact, the purpose of flow-control countermeasures is to alter flow and geomorphic factors for protection of the bridge and approach embankments. Examples of the effect of countermeasures on flow factors are given below.

Spur Dikes

The Tombigbee River relief bridge 2 at US-45 near Aberdeen, Miss. (site 41) is an example of channel bed scour at a flow constriction. At this site, the source of flow is overbank flow from the Tombigbee River, and the degree of flow contraction on the left and right sides of the relief opening are about equal. Although the amount of flow contraction, M, (equation 9) was not determined, measurements of flows on March 18 and 21, 1973, indicate that the channel bed scoured a maximum of about 6 ft (2 m), and the size of the waterway increased from 201 ft^2 (19 m^2) to 677 ft^2 (63 m^2) (fig. 159, Vol. II). After the 1973 flood, two spur dikes were built to improve the distribution of flow at the bridge.

Data obtained at the site during floods in 1955 and 1975 illustrate the effectiveness of spur dikes in improving flow conditions at a bridge waterway (fig. 46). Discharge measurements of floods on March 23, 1955 (discharge, 5,280 ft^3/s or 147 m^3/s), and March 15, 1975 (discharge, 4,360 ft^3/s or 122 m^3/s), were used to compare flow patterns before and after construction of spur dikes. For the comparison, measured velocities of the March 1975 measurement, which had the same water-surface elevation as the March 1955 measurement, were adjusted by increasing the incremental flow velocities in proportion to the difference in discharge of the two measurements.

Prior to construction of spur dikes at both abutments, the discharge per unit of width in the center part of the bridge waterway was 68.8 ft^3/ft (6.4 m^3/m). After spur dike construction late in 1973, data obtained during the March 1975 flood showed the discharge per unit of width in the center part of the waterway increased to 73.4 ft^3/ft (6.8 m^3/m), indicating improved hydraulic efficiency of the bridge opening. Associated with the improved conditions, flow velocities within about 5 ft (1.5 m) of the bridge abutments were about 2 ft/s (0.6 m/s) (fig. 46). To summarize, the spur dikes increased the flow capacity of the bridge and decreased the potential for scour at the abutment.

Jack Fields

In 1944, 250 steel jacks were placed on the right bank of the Colorado River at the US-90 crossing at Columbus, Texas (site 133) to prevent further lateral erosion and eventual damage to the bridge abutment. The effect of the jack field is illustrated by the significant reduction in maximum flow velocities near the right bank (fig. 47). For a discharge of 60,000 ft^3/s (1,699 m^3/s), the maximum flow velocity between piers 6 and 7 was reduced from

8.4 ft/s (2.6 m/s) to 5 ft/s (1.5 m/s). Field surveys at the site between 1944 and the mid-1970's indicate the jack field has been effective in preventing further bank erosion, and suspended material is depositing in the jack field causing restoration of the bank.

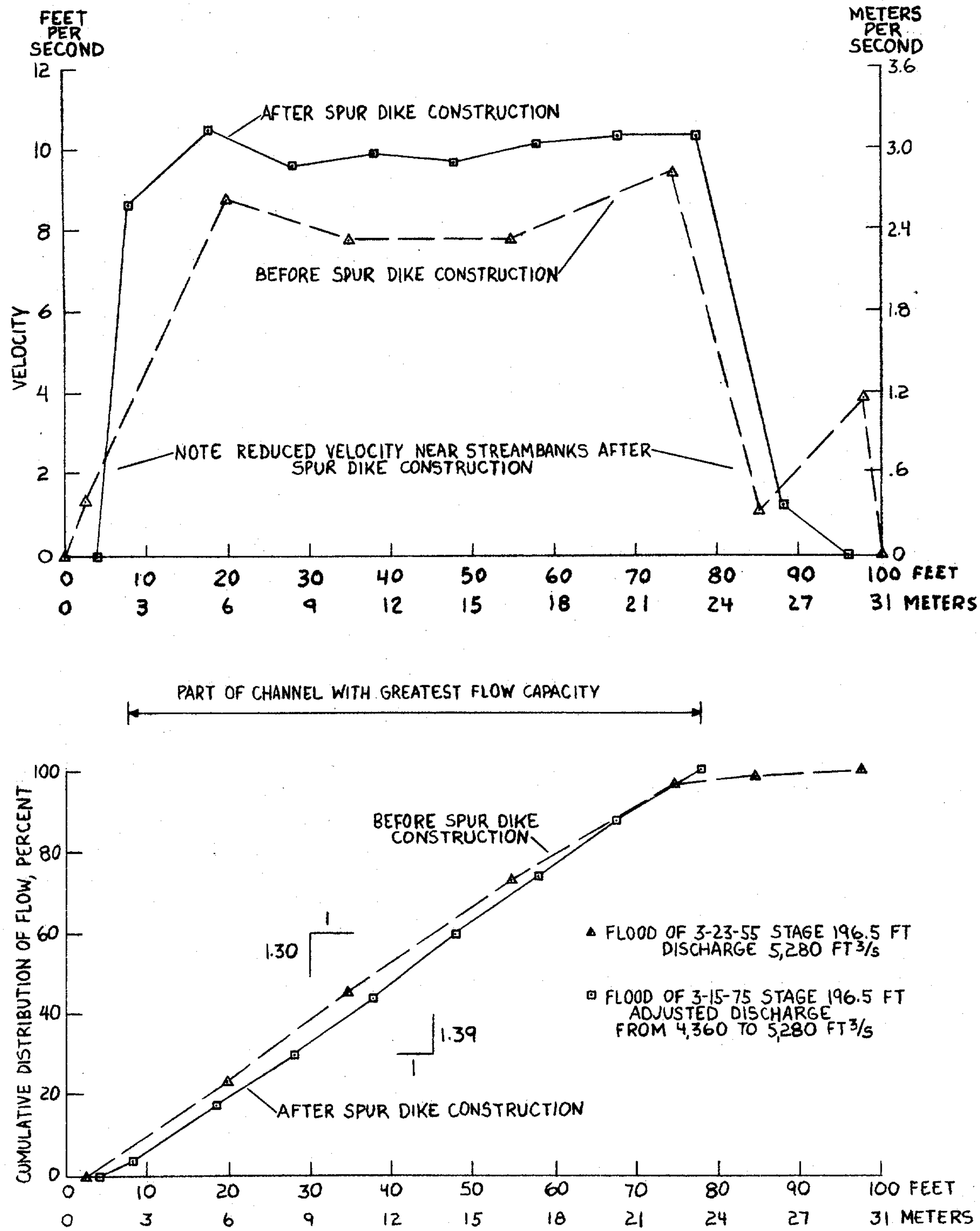

Figure 46. Improvement in flow distribution and velocity in bridge waterway after construction of spur dikes, Tombigbee River relief bridge 2 near Aberdeen, Miss. (Site 41).

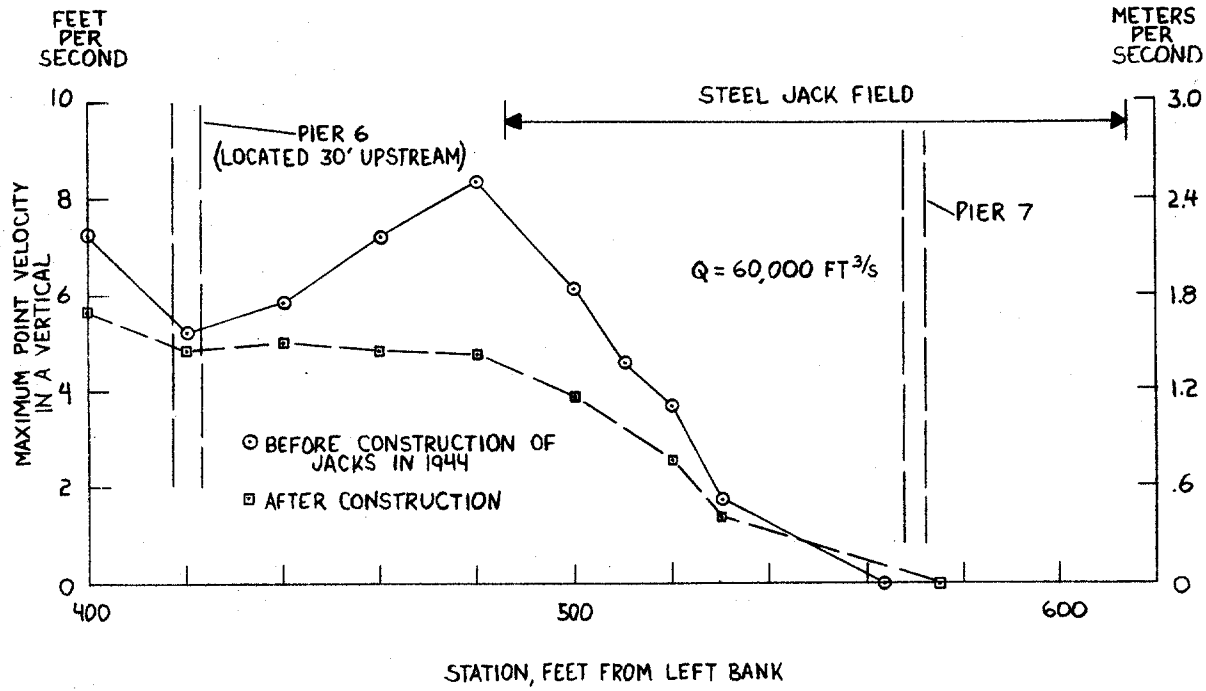

Figure 47. Effect of steel jacks on velocity of flow near the right bank, Colorado River, US-90 at Columbus, Tex. (Site 133).

Channel Relocation

At the SR-200 crossing of the Musselshell River at Mosby, Mont. (site 101), a new bridge was built in 1972 about 150 ft (46 m) upstream from the old bridge. Associated with the new bridge, the main channel was realined and excavated through a previous overflow channel and extended about 750 ft (229 m) upstream from the original bridge. The banks of the new channel and bridge abutments were lined with rock riprap and a spur dike built on the right bank to prevent lateral erosion and to direct flow back to the main channel.

Flooding in 1975 (Recurrence Interval about 9 years) caused no damage to the bridge. The effectiveness of the new channel in conveying floodflows efficiently was determined by a discharge measurement made during a flood in 1977. A comparison of measured and estimated flow factors for the old and new channels is given in table 34. Although some channel scour or lateral erosion normally occurs when channels are realined and straightened, flood stages and velocities measured after the channel improvement were less than anticipated, and the channel bed at the bridge showed a small amount of aggradation between 1972 and 1975.

Table 34. Comparison of measured and estimated flow factors for the old and new channels, Musselshell River at SR-200 at Mosby, Montana (site 101)

Flow factor	Old channel (estimated)[1]	New channel Estimated[1]	New channel Measured[2]
Channel length (ft)[3]	1,240	760	--
Channel slope	0.00148	0.00250	--
Water surface elevation (ft)	2,505.5	2,506.0	2,505.65
Discharge (ft^3/s)[4]	[5]11,100	[5]11,100	11,100
Area of channel (ft^2)[6]	2,700	2,410	2,960
Mean velocity (ft/s)[7]	4.1	4.7	3.8

[1]Computed using data from bridge plans.
[2]Flow and channel geometry measured in 1977.
[3]Feet multiplied by 0.305 equals meters.
[4]Ft^3/s multiplied by 0.0283 equals m^3/s.
[5]The discharge of 11,100 ft^3/s is for comparative purposes.
[6]Ft^2 multiplied by 0.093 equals m^2.
[7]Ft/s multiplied by 0.305 equals m/s.

Table 35. Lateral movement of main channel thalweg between bridge abutments at three study sites

Location	Site number	Year	Distance to thalweg from left abutment (ft)[1]	Net change in distance since initial date (ft)
Deer Creek near Vina, Calif.	16	1951	84	0
		1962	127	+ 43
		1974	297	+213
		1976	308	+224
Snake R. near Heise, Idaho	40	1971	123	0
		1977	247	+124
Canadian R. near Sanchez, N.M.	11	1928	[2]186	0
		1943	255	+ 69
		1965	196	+ 10
		1976	73	-113

[1]Feet multiplied by 0.305 equals meters.
[2]Centerline of graded channel.

EFFECTS OF CHANGING FLOW FACTORS

Lateral Migration of Thalweg

Lateral migration of the thalweg may result in undermining of pier footings that were originally placed in more shallow water near the margins of the channel. The rate and magnitude of thalweg shift is substantial for some streams (table 35) and is therefore a significant factor to be considered in the placement of piers and the design of foundations.

Channel Degradation

Channels subject to extensive degradation require special consideration when designing countermeasures for bridge protection. Scour and degradation typically involve lowering the channel bed and, depending on the type of bed and bank material, lateral erosion. The amount of channel bed scour and degradation in a channel is associated with significant changes in flow patterns. Examples of flow factors that may cause channel changes are new diversions of flow into a stream that augment the natural discharge, floods, changes in sediment supply, or bed material size. Some of the flow factors that describe the extent of channel change are channel geometry (channel width and depth), and channel slope.

The normal water-surface profile of a stream reflects the gradient of a stream channel. If there is a local change in the level of a body of water into which a stream flows (base level) or the gradient of a stream, channel degradation or aggradation is induced in the channel upstream from the point of change. A reduction in the base level or channel bed elevation, caused by sand and gravel mining and channel clearing and alinement changes, will increase the channel gradient and erosive ability of the stream. The ultimate extent of channel change on a degrading stream is difficult to estimate if the manmade operations are continuing. The amount of channel degradation may be significant, on the order of 10 to 12 ft (3.1 to 3.7 m), based on data obtained at several bridge sites (fig. 48). The rate of degradation is affected by the amount of sand and gravel removed from the channel and amount of streamflow. In Tujunga Wash, for example, the occurrence of two large floods in 1969 caused about 14 ft of channel degradation (fig. 48) within 1 year (Scott, 1973). Degradation of Merrill Creek streambed, which is a tributary to the North Sulphur River (fig. 48), however, is attributed to channel clearing and alinement changes made prior to 1950 on the North Sulphur River about 2 miles downstream from the bridge site.

Several countermeasures may be used to protect bridges from hydraulic problems associated with degrading channels. These measures include the use of longer piling to support piers and abutments, check dams, and construction of bridges with fewer piers in the channel. Case histories 1, 85, and 148 (see Vol. II) include descriptions of countermeasures used on streams with degrading channels.

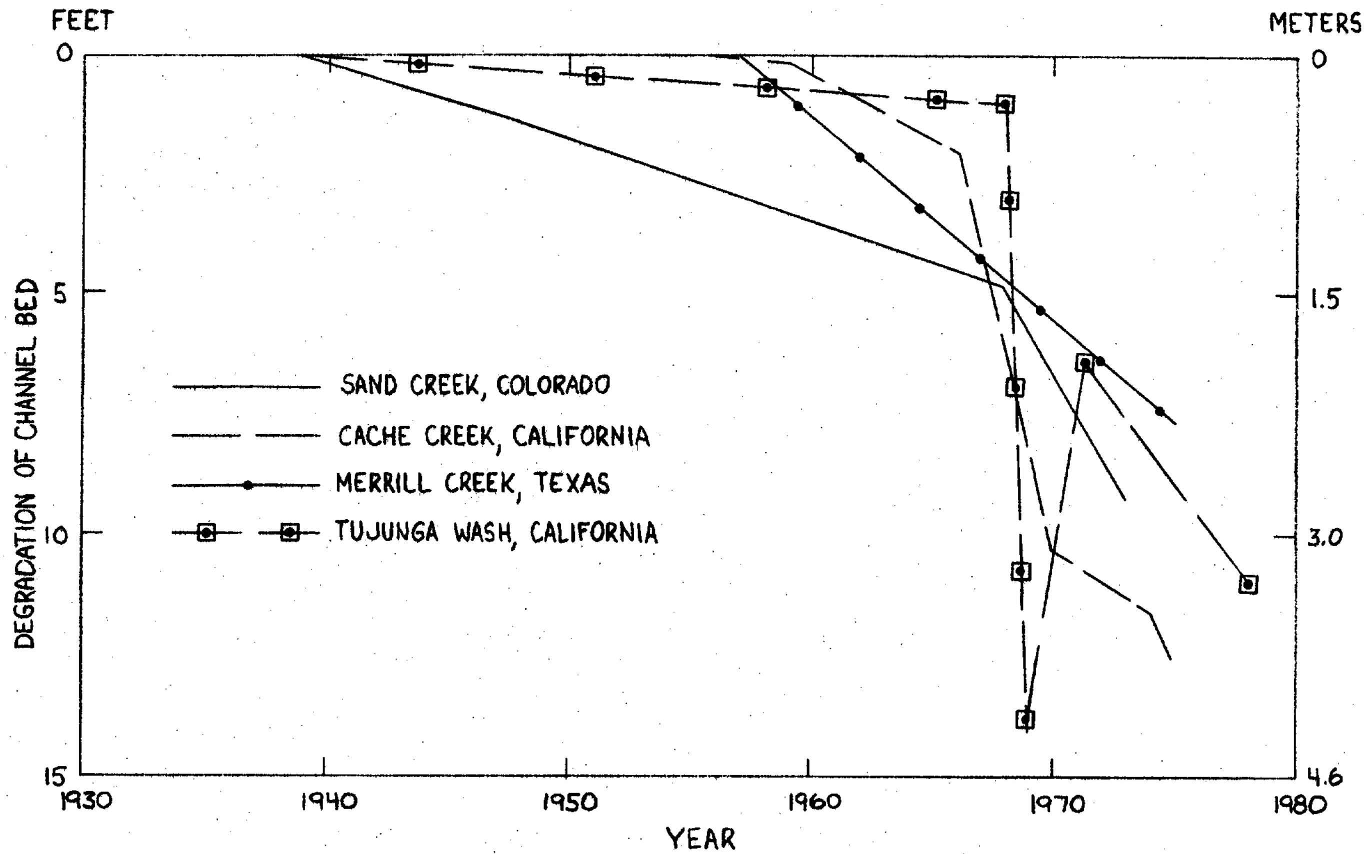

Figure 48. Channel degradation at four bridge sites, attributed to the works of man.

SUMMARY AND CONCLUSIONS

1. Guidelines suggested here for the use of countermeasures are based on case histories of 223 bridge sites in the U.S. and Canada, on interviews with bridge engineers in 34 states, and on a survey of published work on countermeasures. As an essential aspect of countermeasure use and performance, factors contributing to hydraulic problems at bridges are analyzed, with particular regard to stream properties and behavior.

2. Scour is regarded as vertical erosion of a surface, as distinguished from lateral erosion by stream (or wave) action. Hydraulic problems are attributed to local scour at about 50 sites, to general scour at about 55 sites, and to lateral erosion by stream action at about 105 sites.

3. Problems at piers occurred at about 100 sites and problems at abutments at about 80 sites.

4. The performance of a countermeasure is rated mainly on how well it served its intended function, but also with regard to any damage it sustained and to any unwanted effects it may have produced. According to the "principle of expendability", a countermeasure that serves its purpose most economically is likely to be damaged.

5. Flexible revetment, of which dumped rock riprap is the most widely used type, has a better record of performance at study sites than does rigid revetment, of which concrete pavement is the most widely used. However, concrete paving has proved satisfactory in regions where stream velocities are low, or in other circumstances where it is well protected from undermining at the toe and ends. Failures of riprap are attributed to factors already taken into account by existing guidelines. Riprap keyed in place by tamping is more stable than loose riprap, and a lesser volume of rock per unit area is required. The use of rock-and-wire mattress (wire-enclosed riprap) is most advantageous in regions of dry climate, where streambanks are high and steep, and where stones of cobble size are readily available. Concrete-filled fabric mat has proved advantageous at existing bridges where maximum efficiency of the waterway opening is critical for conveyance of flow.

6. As protection for piers, dumped rock riprap proved to be effective at most sites where it could be evaluated. There is some evidence that the riprap should be placed so that the top is below, rather than level with, the streambed.

7. Flow-control structures include spurs (jetties), retards, spur dikes, dikes, check dams, and jack fields. They serve as countermeasures by controlling the direction, velocity, or depth of flowing water.

8. Spurs are particularly useful for bank protection and control of flow alinement at bends, where they may be less expensive and more effective

than riprap revetment. Permeable spurs (supported by timber or steel pile) are used at more study sites than are impermeable spurs, and they have a generally good performance record. Impermeable spurs are more likely to cause flow disturbance and bank erosion, but are suitable for the confinement of wide shallow channels. Only straight spurs (as compared with T-head or L-head types) were encountered at study sites.

9. Permeable spurs are most effective when placed at right angles to the bank or inclined downstream. A slight upstream inclination has some advantages for impermeable spurs, particularly when they are used to confine a shallow channel. In view of the high cost of riprap and of conventional spurs, and the dominance of lateral stream erosion in hydraulic problems at bridges, new spur types are needed. Permeability, flexibility, low cost, and expendability should be incorporated into the design.

10. Retards are advantageous for control of flow alinement at sharp bends near a bridge, and for other situations in which f-ow impinges directly on a bank near a bridge. Bulkheads are advantageous for control of internal slope failure, where the slope cannot be graded to a low angle for the placement of other revetment.

11. The major use of spur dikes at study sites is the control of overbank flow where it enters the bridge waterway. We found substantial evidence that spur dikes serve this purpose well. Most of the spur dikes are of elliptical shape, and no grounds for dissatisfaction with this shape were found. In one state, spur dikes are grassed but not riprapped or otherwise revetted. Spur dikes are also used for the control of channel alinement, particularly in Canada a straight dike with curved ends has proved satisfactory for this purpose. A greater length of dike may be needed than for control of overbank flow.

12. Check dams proved effective for control of channel degradation at several sites, but a check dam is a risky countermeasure because it may induce downstream erosion.

13. In the design of crossings and the placement of countermeasures at bridges, geomorphic factors are an important consideration. These factors include natural stream properties and behavior, and possible effects of the works of man on the stream. The lateral stability of a stream is assessed from its properties as shown on aerial photographs, and a more specific assessment of rate and direction of lateral erosion is obtained by comparing past and present aerial photographs.

14. For engineering purposes, five major alluvial stream types can be distinguished, each of which has characteristic properties and modes of behavior. The equiwidth point-bar stream is the most stable of these types and the braided point-bar stream, the least stable. Fully braided streams have low to moderate rates of lateral erosion, but the point of bank erosion is unpredictable.

15. Bridge factors are characteristics of the bridge structure and the approach roadway that relate to hydraulic problems and countermeasures. Location of crossing at a bend contributed to hydraulic problems at 65 study sites, more than any other bridge factor. Skewness of the crossing is a factor in problems at 9 sites and skewness of piers to flow, at 16 sites. Pier skewness resulted mainly from a shift of the thalweg or channel, or from the direction of overbank flow. At one site, skewness effects were obtained from multiple round columns in a pile bent. There is some evidence that flow disturbance at a pier will erode an adjacent abutment fill slope. Constriction of flood flow by approach embankments led to problems (mainly general scour) at 25 sites. Channel alterations for bridge purposes were unsatisfactory at 10 sites, but at only one site were the effects of alteration found to extend beyond the bridge site.

16. Values for flow, bridge, and geomorphic factors are tabulated for flood conditions at 60 bridge sites. The flow factors include water discharge, recurrence interval of the flood, average depth, velocity, ratio of area of piers to gross area, Froude number, and bed shear stress. Problems with local scour at piers are more common where the ratio of pier area to gross waterway area exceeds about 0.10. At ten sites, the Froude number during flood exceeded 0.8 which is regarded as the lower limit of supercritical flow for field conditions. The bed shear stress at almost all sites exceeds published critical shear stress criteria for stable channels. However, flow factors at many of the sites were computed for floods of high recurrence interval, and the stresses were calculated assuming uniform rather than unsteady flow conditions.

17. Quantitative determinations of the effect of countermeasures on flow were made at two sites. At one site, a spur dike reduced water velocity near the bridge abutment about 2 ft/s (o.6 m/s). At another site, a jack field reduced the stream velocity from about 8 ft/s (2.4 m/s) to 5 ft/s (1.5 m/s).

GLOSSARY

aggradation   General and progressive upbuilding of the longitudinal profile of a channel by deposition of sediment.

alluvium   Unconsolidated clay, silt, sand, or gravel deposited by a stream in a channel, flood plain, alluvial fan, or delta.

alluvial channel   Channel wholly in alluvium; no bedrock exposed in channel at low flow or likely to be exposed by erosion during a major flood.

alluvial fan   A landform, shaped like a fan in plan view, and deposited where a stream issues from a narrow valley onto a plain or broad valley.

anabranch   Individual channel of an anabranched stream.

anabranched stream   A stream whose flow is divided at normal and lower stages by large islands or, more rarely, by large bars. The width of individual islands or bars is greater than about three times water width. The channels of an anabranched stream are more widely and distinctly separated than those of a braided stream.

armor   Artificial surfacing of channel bed, banks, or embankment slope for protection against scour and lateral erosion.

average velocity   Velocity at a given cross section as determined by dividing discharge by cross-sectional area.

avulsion   A sudden change in course of a channel, usually by breaching of the banks during flood.

backwater   The increase in water-surface elevation, relative to the elevation occurring under natural channel and flood-plain conditions, induced upstream from a bridge or other structure that artificially obstructs or constricts a channel.

bank (of channel or stream)   Lateral boundaries of a channel or stream, as indicated by a scarp or, on the inside of bends, by the streamward edge of permanent vegetal growth.

bank, left (right)   The bank on the left (right) side of a channel as viewed in a downstream direction.

bankfull discharge   Discharge that fills a channel to the height of its banks where the banks stand at flood-plain elevation. For many streams, bankfull discharge has a recurrence interval of about 1.5 years.

bar   An elongated deposit of alluvium, not permanently vegetated, within or along the side of a channel.

bridge A structure, including supports, erected over a depression or an obstruction such as a body of water, a road, or a railway, having a track or passageway for carrying traffic and a length of oopening greater than 20 ft (6 m).

bridge opening The cross-sectional area beneath a bridge that is available for conveyance of water when the water surface approaches, but does not touch, the bottom of the superstructure.

bridge waterway The area of a bridge opening available for flow, as measured below a specified stage and normal to the principal direction of flow.

bed (of channel or stream) The part of a channel not permanently vegetated, bounded by banks, over which water normally flows.

bedload Sediment that is transported in a stream by rolling, sliding, or skipping along the bed or very close to it; considered to be within the bed layer.

bed material Sediment consisting of particle sizes large enough to be found in appreciable quantities at the surface of a streambed.

bed shear (tractive force) The force per unit area exerted by a fluid flowing past a stationary boundary.

boulder A rounded or angular fragment of rock, the diameter of which is in the size range of 250 to 4,000 mm (10 to 160 in).

braid A subordinate channel of a braided stream.

braided stream A stream whose flow is divided at normal stage by small mid-channel bars or small islands. The individual width of bars and islands is less than about three times water width. A braided stream has the aspect of a single large channel within which are subordinate channels.

bulkhead A steep or vertical wall that supports a natural or artificial embankment and may also serve as a protective measure against erosion.

channel The bed and banks that confine the surface flow of a natural or artificial stream. Braided streams have multiple subordinate channels, which are within the main stream channel. Anabranched streams have more than one channel.

channelization Straightening or deepening of a natural channel by artificial cutoffs, by grading, by flow-control measures, or by diversion of flow into a parallel artificial channel.

channel pattern The aspect of a stream channel in plan view, with particular reference to the degree of sinuosity, braiding or anabranching.

check dam A low dam or weir across a channel, for the control of water stage, or velocity, or to prevent channel degradation.

cobble A rounded or angular fragment of rock, the diameter of which is in the size range of 64 to 250 mm (2.5 to 10 in).

countermeasure A measure, either incorporated into the design of a bridge or installed separately at or near the bridge, that serves to prevent or control hydraulic problems.

concrete paving Plain or reinforced concrete slabs poured or placed on the surface to be protected.

constriction A control section, such as a bridge crossing, reach of channel, or dam, with limited flow capacity, in which the discharge is related to the upstream water-surface elevation. A constriction may be either natural or artificial.

contraction The effect of a channel constriction on flow.

clay Particles, usually of clay minerals, the diameter of which is less than 0.004 mm (0.00016 in).

crib An open-frame structure filled with rocks, intended as protection for a bank or embankment.

cross-section (of channel) A section perpendicular to the trend of a channel, bounded by the bed banks and water surface. In geomorphology, the term "cross profile" is applied if either the water surface or the channel perimeter, but not both, are shown.

cutoff A natural or artificial channel that shortens the length of a stream. Natural cutoffs may occur either across the neck of a meander loop (neck cutoffs) or across a point bar (chute cutoffs).

cutoff wall A wall, usually of sheet piling or concrete, that extends from the toe of revetment down to scour-resistant material or to below the expected scour depth.

debris Material transported by the stream, either floating or submerged, such as logs or brush that may lodge against the bridge.

deflector Alternative term for "spur".

degradation General and progressive lowering of the longitudinal profile of a channel by erosion.

design flood frequency Recurrence interval of the flood discharge that the bridge opening is expected to accommodate without contravention of the adopted design constraints.

design high-water level The maximum water level that a bridge opening is designed to accommodate without contravention of the adopted design constraints.

dike An impermeable linear structure for the control or containment of overbank flow. A dike trending parallel with a river bank differs from a levee only in extending for a much shorter distance along the bank.

discharge The volume of flow of a stream per unit of time, usually expressed in ft^3/s (cubic feet per second) or in m^3/s (cubic meters per second).

drift Alternative term for "debris".

eddy current A current of water moving contrary to the main current, usually in a circular pattern, that is associated with turbulence of flow.

ephemeral stream A stream or reach of a stream that does not flow continuously for most of the year. As used here, the term includes intermittent streams, whose flow is less than perennial but more than ephemeral.

erosion The general process by which solid materials at the earth's surface are loosened or worn away and simultaneously transported.

fill-slope Side or end slope of an earth-fill embankment. Where a fill-slope forms the streamward face of a spillthrough abutment, it is regarded as part of the abutment.

filter blanket One or more layers of gravel, or layers of sand and of gravel, placed between bank material and riprap. Purpose of the blanket, which is intermediate in particle size and placed between the bank material and riprap, is to prevent erosion of the embankment fill behind the riprap.

filter cloth, plastic Cloth of woven plastic strands that serves the same purpose as a granular filter blanket.

flashy stream Stream characterized by rapidly rising and falling stages, as indicated by a sharply peaked hydrograph. Most flashy streams are ephemeral but some are perennial.

flood plain A nearly flat alluvial lowland bordering a stream, formed by stream processes, that is subject to inundation by floods.

flow-control structure Structure, either within or outside a channel, that acts as a countermeasure by controlling the direction, depth, or velocity of flowing water.

flow hazard Flow characteristics (of discharge, stage, velocity, or duration) that are associated with a hydraulic problem; or that can reasonably be considered of sufficient magnitude to cause a hydraulic problem or to test the effectiveness or a countermeasure.

foundation The supporting material upon which the substructure portion of a bridge is placed.

freeboard The vertical distance above a design stage that is allowed for waves, surges, drift, and other contingencies.

Froude number A dimensionless number that represents the ratio of gravitational to inertial forces. High Froude numbers are indicative of high flow velocity and potential for scour.

gabion A basket or compartmented rectangular container made of steel wire mesh. When filled with cobbles or other rock of suitable size, the gabion becomes a flexible and permeable block with which flow-control structures can be built.

general scour Scour in a channel or on a floodplain that is not localized at a pier, abutment, or other obstruction to flow. In a channel, general scour usually affects all or most of the channel width.

gravel Particles, usually of rock, the diameter of which is in the size range of 2 to 64 mm (0.08 to 2.5 in). The term gravel is also applied to a mixture of sizes (gravel with sand or gravel with cobbles) in which the dominant or modal fraction is in the gravel size range.

groin Alternative form for "spur".

guide bank Alternative term for "spur dike".

headcutting Channel degradation associated with abrupt changes in the bed elevation (headcut), that migrates in an upstream direction.

historical flood A past flood event of known magnitude, which may be a significant factor in the hydraulic design of a bridge.

hydraulic problem (at a bridge) An effect of streamflow, tidal flow, or wave action on a crossing, such that traffic is immediately or potentially disrupted.

incised stream A stream that flows in a well-defined channel, the banks of which are high and steep. For most streams, banks that stand more than 15 ft (5 m) above the water surface at normal stage are regarded as high.

icing Masses or sheets of ice formed on the frozen surface of a river or flood plain. When shoals in the river are frozen to the bottom, or otherwise dammed, water under hydrostatic pressure is forced to the surface, where it freezes.

instantaneous discharge Discharge at a given moment.

invert Lowest point in the channel cross section or at flow control devices such as weirs or dams.

island A permanently vegetated area, emergent at normal stage, that divides the flow of a stream. Some islands originate by establishment of vegetation on a bar, but others originate by channel avulsion or at the junctions of minor tributaries with a stream.

jack A device for flow control and protection of banks against lateral erosion, that has six mutually perpendicular arms rigidly fixed at the center and strung with wire. Kellner jacks are made of three steel struts; concrete jacks are made of three reinforced concrete beams bolted together at the midpoints.

jack field Rows of jacks tied together with cables, some rows generally parallel with the banks and some perpendicular thereto or at an angle. Jack fields may be placed either outside or within a channel.

jetty Alternative term for "spur".

local scour Scour in a channel or on a flood plain that is localized at a pier, abutment, or other obstruction to flow.

lateral erosion Erosion in which the removal of material has a dominantly lateral component, as contrasted with scour, in which the component is dominantly vertical.

levee A linear embankment outside a channel for containment of flood water.

longitudinal profile The profile of a stream or channel, drawn along the length of its centerline. In drawing the profile, elevations of the water surface or the thalweg are plotted against distance as measured from the mouth or from an arbitrary initial point.

meander loop An individual loop of a meandering or sinuous stream, lying between inflection points with adjoining loops.

meander scrolls Low concentric ridges and swales on a flood plain, marking the successive positions of former meander loops.

meandering stream Stream having a sinuosity greater than some arbitrary value, herein placed at a value of 1.25. The term also implies a moderate degree of pattern symmetry, imparted by regularity of size and repetition of meander loops.

measured scour Measured depth to which a surface is lowered by scour below a reference elevation.

median diameter Particle diameter at the 50-percentile point on a size distribution curve, such that half of the particles (by weight for samples of sand, silt, or clay and by number for samples of gravel) are larger and half are smaller.

mid-channel bar A bar, lacking permanent vegetal cover, that divides the flow in a channel at normal stage.

migration Change in position of a channel by lateral erosion of one bank and simultaneous accretion of the opposite bank.

natural levee A low ridge along a stream channel, formed by deposition during floods, which slopes gently away from the channel.

non-alluvial channel Channel wholly in bedrock which is either exposed or is likely to be exposed by erosion during a major flood.

normal stage The water stage prevailing during the greater part of the year.

perennial stream A stream or reach of a stream that flows continuously for all or most of the year.

pile bent (or pile pier) A pier composed of piles capped or decked with a timber grillage or with a reinforced-concrete slab forming the bridge sea.

point bar Alluvial deposit of sand or gravel, lacking permanent vegetal cover, in a channel at the inside of a meander loop, usually somewhat downstream from the apex of the loop.

railbank protection A type of countermeasure composed of rock-filled wire fabric and supported by steel rails or posts driven into the streambed.

reach Segment of stream length that is arbitrarily bounded for purposes of study.

recurrence interval (R. I.; return period, exceedance interval) The average time interval between actual occurrences of a hydrological event of a given or greater magnitude. In an annual flood series, the average interval in which a flood of a given size is exceeded as an annual maximum.

relief bridge Opening in an embankment on a flood plain to permit passage of overbank flow.

retard A permeable or impermeable linear structure in a channel, parallel with the bank and usually at the toe of the bank, intended to reduce flow velocity, induce deposition, or deflect flow away from the bank.

revetment Rigid or flexible armor placed on a bank or embankment as protection against scour and lateral erosion.

riffle A natural shallow extending across a streambed, at which the surface of flowing water is broken by waves or ripples. Typically, riffles alternate with pools along the length of a stream channel.

riprap In the restricted sense, riprap refers to a layer or facing of broken rock or concrete, dumped or placed to protect a structure or embankment from erosion; it also refers to the broken rock or concrete suitable for such use. The term has also been applied to almost all kinds of armor, including wire-enclosed riprap, grouted riprap, sacked concrete, and concrete slabs. Usage in the restricted sense is followed herein.

spillthrough abutment Bridge abutment having a fill-slope on the streamward side. The term originally referred to the "spillthrough" of fill at an open abutment, but is now applied to any abutment having such a slope.

spread footing A pier or abutment footing that transfers load directly to the earth.

spur A linear structure, permeable or impermeable, projecting into a channel from the bank for the purpose of altering flow direction, inducing deposition, or reducing flow velocity along the bank.

spur dike A dike extending upstream from the approach embankment at either or both sides of the bridge opening, for the general purpose of directing flow through the opening. Some spur dikes extend downstream from the bridge, as well as upstream.

stage Height of water surface above a specified datum.

stream A body of water, which may range in size from a large river to a small rill, flowing in a channel. By extension, the term is sometimes applied to a natural channel or drainage course formed by flowing water, whether it is occupied by water or not.

suspended-sediment load Amount of sediment transported in suspension by turbulent flow.

thalweg A line connecting the lowest points along the bed of a channel.

uniform flow Flow of constant cross section and average velocity through a reach of channel during an interval of time.

unit shear force (shear stress) The force, or drag, developed at the channel bed by flowing water. For uniform flow, this force is equal to a component of the gravity force acting in a direction parallel to the channel bed on a unit wetted area. Usually expressed in units of stress, lbs/ft^2 or kg/m^2.

unsteady flow Flow of variable cross section and average velocity through a reach of channel during an interval of time.

velocity The rate of motion of a stream or of the objects or particles transported therein; usually expressed in ft/s or m/s.

vertical (full-height) abutment An abutment, usually with wingwalls, that has no fill slope on its streamward side.

wandering thalweg Thalweg that wanders within a channel, shifts in position during floods, and typically serves as an inset channel that transmits all or most of the streamflow at normal or lower stages.

weephole A hole in an impermeable wall or revetment to relieve the neutral stress or pore-water pressure.

wire mesh Wire woven or welded to form a mesh, the openings of which are of suitable size and shape to inclose rock or broken concrete, or to function on fence-like spurs and retards.

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FEDERALLY COORDINATED PROGRAM OF HIGHWAY RESEARCH AND DEVELOPMENT (FCP)

The Offices of Research and Development of the Federal Highway Administration are responsible for a broad program of research with resources including its own staff, contract programs, and a Federal-Aid program which is conducted by or through the State highway departments and which also finances the National Cooperative Highway Research Program managed by the Transportation Research Board. The Federally Coordinated Program of Highway Research and Development (FCP) is a carefully selected group of projects aimed at urgent, national problems, which concentrates these resources on these problems to obtain timely solutions. Virtually all of the available funds and staff resources are a part of the FCP, together with as much of the Federal-aid research funds of the States and the NCHRP resources as the States agree to devote to these projects.*

FCP Category Descriptions

1. Improved Highway Design and Operation for Safety

Safety R&D addresses problems connected with the responsibilities of the Federal Highway Administration under the Highway Safety Act and includes investigation of appropriate design standards, roadside hardware, signing, and physical and scientific data for the formulation of improved safety regulations.

2. Reduction of Traffic Congestion and Improved Operational Efficiency

Traffic R&D is concerned with increasing the operational efficiency of existing highways by advancing technology, by improving designs for existing as well as new facilities, and by keeping the demand-capacity relationship in better balance through traffic management techniques such as bus and carpool preferential treatment, motorist information, and rerouting of traffic.

3. Environmental Considerations in Highway Design, Location, Construction, and Operation

Environmental R&D is directed toward identifying and evaluating highway elements which affect the quality of the human environment. The ultimate goals are reduction of adverse highway and traffic impacts, and protection and enhancement of the environment.

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5. Improved Design to Reduce Costs, Extend Life Expectancy, and Insure Structural Safety

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* The complete 7-volume official statement of the FCP is available from the National Technical Information Service (NTIS), Springfield, Virginia 22161 (Order No. PB 242057, price $45 postpaid). Single copies of the introductory volume are obtainable without charge from Program Analysis (HRD-2), Offices of Research and Development, Federal Highway Administration, Washington, D.C. 20590.

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