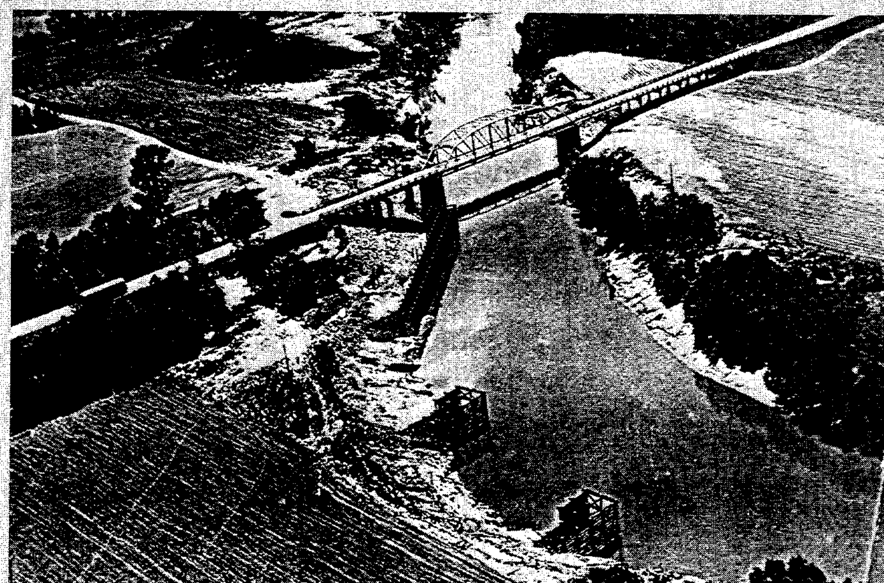
Report No. FHWA-RD-78-163

COUNTERMEASURES FOR HYDRAULIC PROBLEMS AT BRIDGES

Vol. II Case Histories for Sites 1-283



September 1978
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

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Director, Office of Research
Federal Highway Administration

NOTICE

Technical Report Documentation Page

1. Report No. FHWA-RD-78-163	2. Government Accession No.	3. Recipient's Catalog No.
4. Title and Subtitle COUNTERMEASURES FOR HYDRAULIC PROBLEMS AT BRIDGES Volume 2 - Case Histories for Sites 1-283		5. Report Date September 1978
		6. Performing Organization Code
7. Author(s) J. C. Brice, J. C. Blodgett, and Others		8. Performing Organization Report No.
9. Performing Organization Name and Address U.S. Geological Survey Water Resources Division, Mail Stop 66 345 Middlefield Road Menlo Park, California 94025		10. Work Unit No. (TRAIS)
		11. Contract or Grant No. P. O. No. 5-3-0199
12. Sponsoring Agency Name and Address Office of Research and Development Federal Highway Administration U.S. Department of Transportation Washington, D.C. 20590		13. Type of Report and Period Covered Final Report June 1975 to September 1978
		14. Sponsoring Agency Code

15. Supplementary Notes

FHWA Contract Manager - Stephen A. Gilje (HRS-42)

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

17. Key Words Bridge failure, Bank protection, Channel morphology, Open channel flow, River training, Stream erosion, Scour	18. Distribution Statement No restrictions. This document is available to the public through the National Technical Information Service, Springfield, Virginia 22161

19. Security Classif. (of this report) Unclassified	20. Security Classif. (of this page) Unclassified	21. No. of Pages 554	22. Price

Form DOT F 1700.7 (8-72)   Reproduction of completed page authorized

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.

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.

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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

INTRODUCTION

Volume II contains 224 case histories for Sites 1-283, arranged in numerical sequence by site number. Gaps in the numerical sequence represent sites that were originally considered but later omitted. Cases that were suitable for hydraulic anaylsis are given in the first 150 sites, which therefore emphasizes the hydraulic aspects of bridges and countermeasures to a greater extent than following sites. Sites having problems of local or general scour, effective countermeasures, and a record of hydraulic information were selected for more detailed analysis. According to the format used for all case histories, a site is described under the headings "Description of site", "Hydraulic problems and countermeasures", and "Discussion."

Under "Description of site" are included the site location and properties of the bridge that are relevant to the case history. Among the bridge properties documented are the bridge length and the skewness of the bridge and the piers; skewness was measured as shown in figure 1. Bridge abutments are classified as spillthrough or vertical (fig. 2). A spillthrough abutment has a fill-slope on its streamward site. A vertical (full height) abutment typically has wingwalls and no streamward fill-slope. Pier shapes are described according to the classification given in figure 3. During extreme floods, or when a bridge waterway is blocked by debris, submergence of the bridge structure may occur. The amount of submergence (t) is measured as the depth of water above "low steel", or the bottom of the bridge stringer (fig. 4). A more complete description of terms applied to the bridge and to countermeasures is given in the Glossary and in chapter 6 of Vol. I.

Also included under "Description of site" are hydrologic and geomorphic characteristics of the site. Drainage area refers to the area of the drainage basin upstream from the site, as measured on topographic maps or, for sites at or near a stream gage, as given in the gage description. Valley slope was measured, for most sites, on topographic maps. Descriptive terms applied to the stream and its valley are defined in the Glossary and discussed in chapter 4 of Vol. I.

Under "Hydraulic problems and countermeasures" is given the date of bridge construction and a chronological account of relevant events, problems, bridge repairs, and installation of countermeasures. Under "Discussion", a statement is given on the significance of case history with regard to hydraulic problems and the performance of countermeasures.

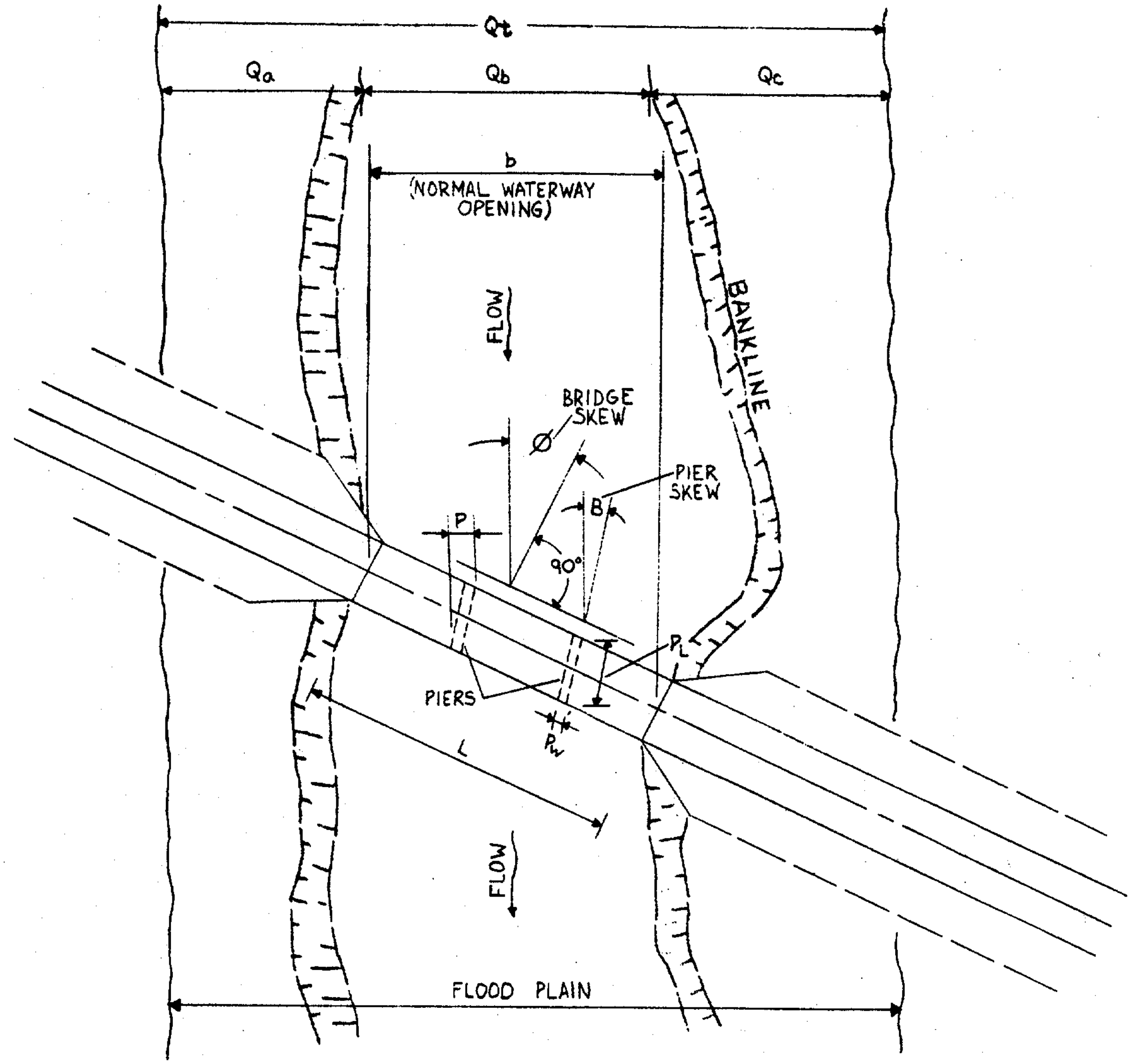


Figure 1. Definition sketch of a skewed crossing and overbank flow.

Figure 2. Classification of abutment types.

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 3. Classification of pier shapes.

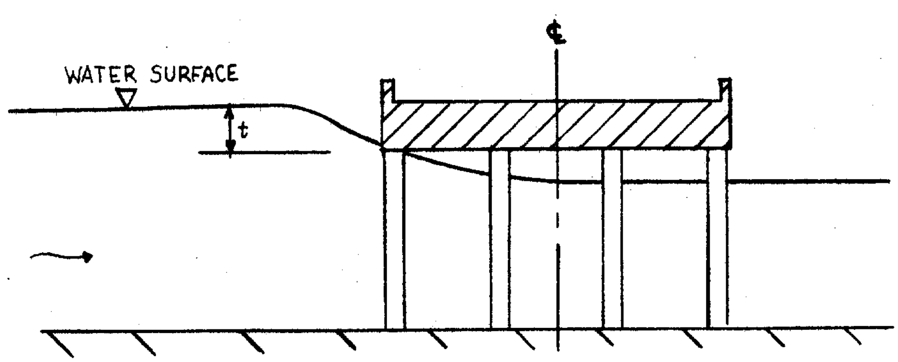


Figure 4. Definition sketch of a bridge submergence.

CASE HISTORIES

PART I

SITE 1. CACHE CREEK AT I-505 NEAR MADISON, CALIF.

Description of site: Lat 38°42', long 121°57', location as shown in fig. 5. Concrete box-girder bridge, length 599 ft (183 m), with concrete piers founded on steel piles. Piers are rectangular with rounded noses and abutments are spillthrough.

Drainage area, 1,139 mi^2 (2,950 km^2); bankfull discharge, about 40,000 ft^3/s (1,132 m^3/s); valley slope, 0.0014; channel width, about 700 ft (213 m). Stream is ephemeral, regulated, alluvial, sand-gravel bed, on alluvial fan, distinct natural levees. Channel is sinuous, locally braided, wandering thalweg, probably incised, cut banks general, silt-sand banks.

Hydraulic problems and countermeasures:

1959 Bridge built. Both abutments and embankments armored with sacked concrete. Double pipe-and-wire retard installed upstream from bridge on both banks (fig. 5).

1963 Ongoing gravel mining operations in channel caused channel degradation and subsequent scour around piers 5 and 6 (fig. 6). Exposed square footings caused obstruction to flow and caught drift (fig. 7).

1965 Flood discharge 38,000 ft^3/s (1,076 m^3/s). Retard on left bank was damaged but was effective in preventing damage to left bank abutment.

1968 Channel alinement changes caused more flow to be directed at the left-bank abutment.

1973 Flood discharge, 21,300 ft^3/s (603 m^3/s). Progressive channel changes since 1965, and floods of January 1970 and January 1973, caused left-bank pipe retard to wash out and sacked-concrete revetment on both banks was undercut. Retard was less effective because channel degradation allowed flows to undercut retard support pipes. Twenty car bodies, placed in 1973, served during the 1973 flood as temporary protection for the left-bank embankment and abutment.

1974 Restored sacked-concrete revetment at both abutments. Dug cutoff trench and placed rock riprap of 5-ton (4.5 t) maximum size at eroded area (fig. 8). Channel erosion due to gravel mining lowered average bed elevation of channel 6.3 ft (1.9 m) between 1959 and 1974 (fig. 9).

1976 New bridge parallel to existing bridge under construction. During excavation for repair of footings for old bridge, wood debris carried by the stream was found 11 ft (3.3 m) below the top of the footings at pier 4 (fig. 9). At pier 6, the streambed was 9 ft (2.7 m) below top of footing. Countermeasures were applied to prevent damage to bridge by flood flows or continued channel degradation. These measures include placing new piers with skew of 40 degrees (fig. 10); placing rock riprap on both banks (figs. 10 and 11); extending footings of the old and new bridge piers about 10 ft (3.1 m) lower, with deeper penetration of pilings (fig. 9); and placing webbing at piers to prevent lodging of drift.

Discussion: Channels subject to extensive and long-term degradation require special consideration when designing countermeasures for bridge protection. Retards were initially effective in protecting the abutment and embankment, but were later damaged when support posts were undercut by the degrading stream. Sacked concrete revetment failed when toe was undercut. Adequate keying into channel bed and at ends of revetment is required to resist stream erosion. Rectangular-shaped footings should be avoided if there is a possibility of exposure to streamflow. The sharp corners caused localized high velocities which scoured the streambed. Debris is more likely to lodge against rectangular footings and cause additional channel-bed scour. Use of old car bodies during flooding to protect the embankment and abutment was apparently effective in reducing damage to the bridge.

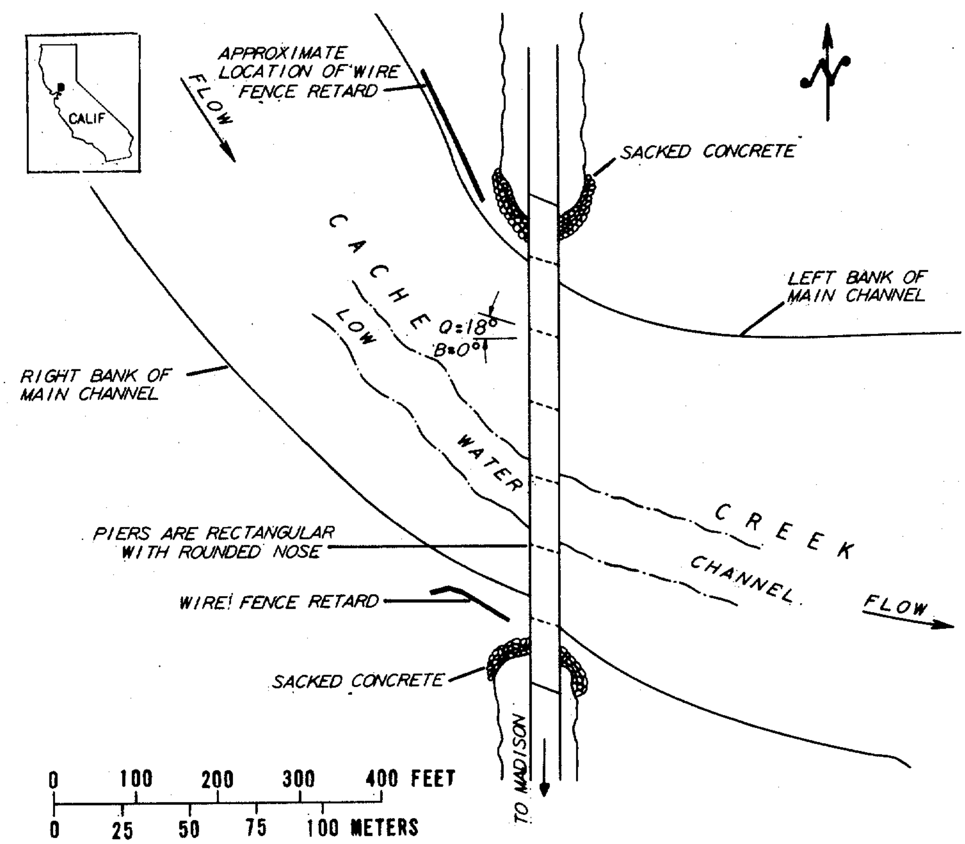


Figure 5. Plan of Cache Creek at I-505 bridge, 1959.

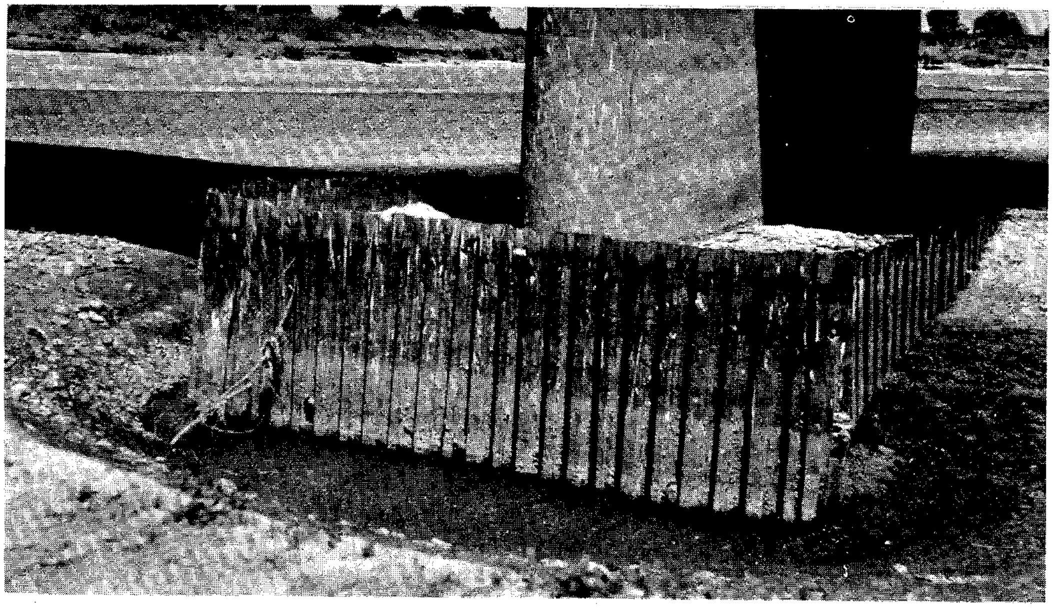


Figure 6. Exposed pier footing, I-505 bridge at Cache Creek. (From Calif. Dept. of Transportation, photograph taken prior to 1976.)

Figure 7. Debris lodged on exposed pier footings. (From Calif. Dept. of Transportation, photograph taken prior to 1976.)

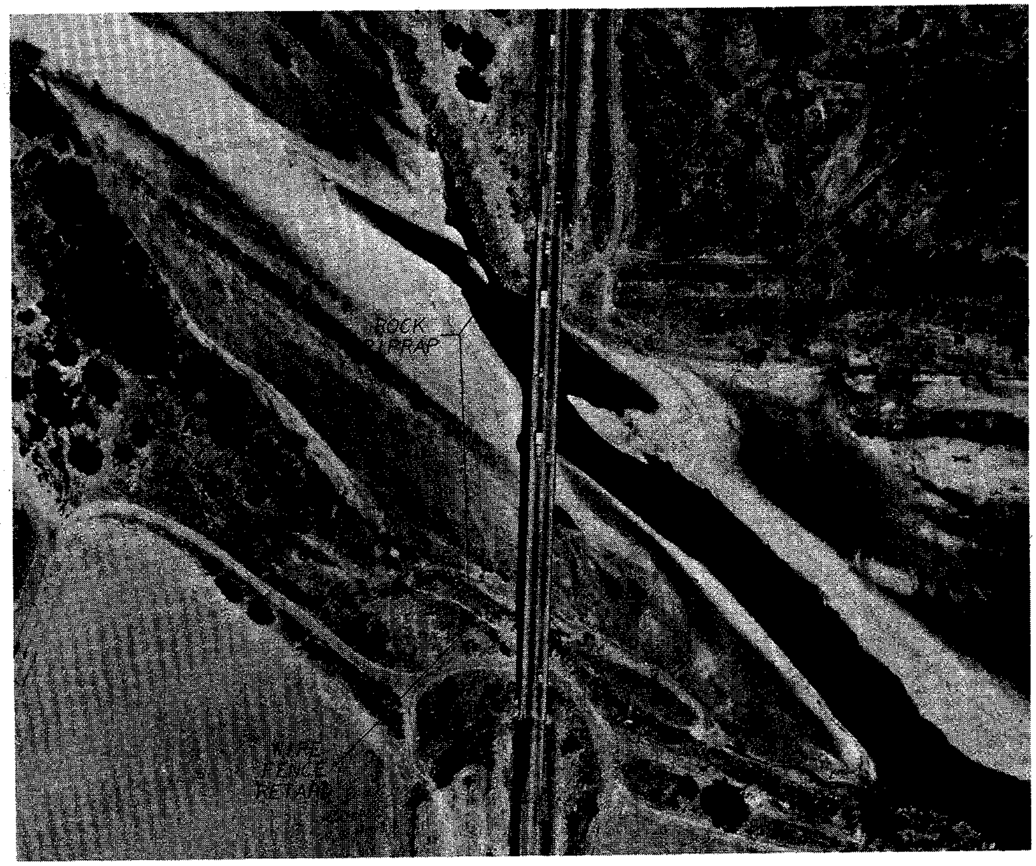


Figure 8. Cache Creek at I-505 bridge on August 16, 1974. (From Calif. Dept. of Transportation.)

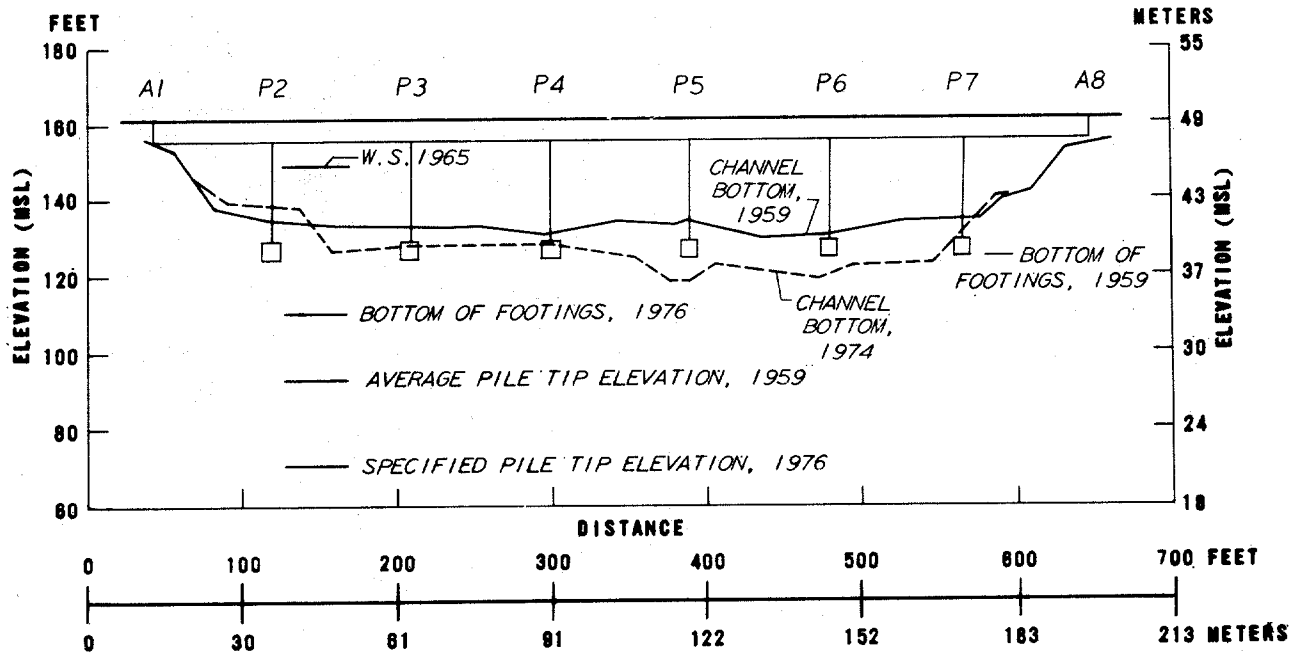


Figure 9. Cross section of Cache Creek channel at I-505 crossing.

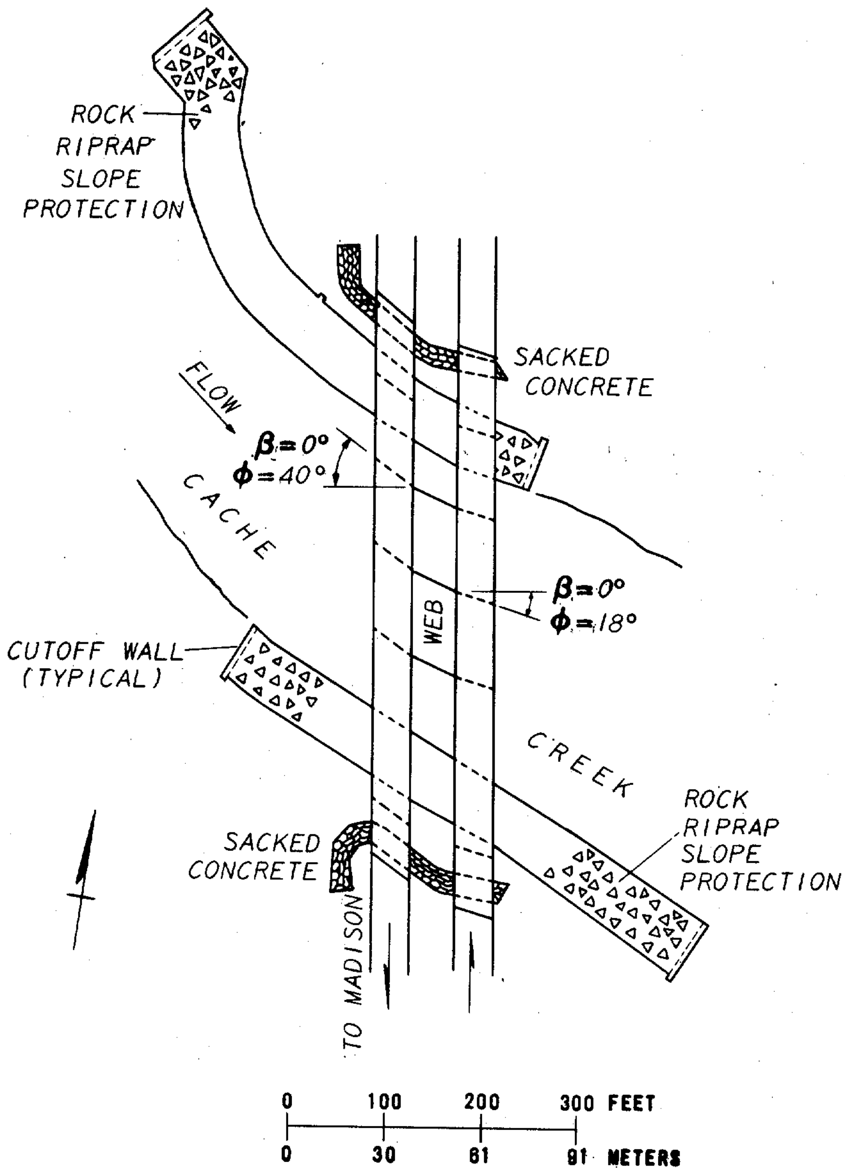


Figure 10. Plan of bridge countermeasures, 1976.

Figure 11. Placement of rock riprap along bank of Cache Creek during September, 1976. (Excavation for toe of riprap and cutoff trench in foreground.)

SITE 2. CASCADE CREEK AT SR-89 NEAR SOUTH LAKE TAHOE, CALIF.

Description of site: Lat $38^{o}57'$, long $120^{o}05'$, location as shown in fig. 12. Timber bridge with reinforced-concrete slab, rubble masonry pier and abutments. One pier evenly divides total bridge length of 39 ft (12 m) into two spans. Pier is rectangular, abutments are vertical and foundations are spread-footing type.

Drainage area, 4.6 mi^2 (11.9 km^2); channel slope, 0.073; width, 37 ft (11 m). Stream is perennial, alluvial, cobble and boulder bed, in valley of moderate relief, no flood plain. Channel is straight at bridge site.

Hydraulic problems and countermeasures:

1935 Bridge built.

1965 Flood resulting from high snowmelt runoff during winter months washed out the upstream half of the pier and scoured 2 ft (0.6 m) deep under the remaining length of the spread footing (fig. 13). Scour of the pier footing is probably related to the large angle of bridge pier skewness to flows (fig. 13). Repairs consisted of replacing the washed out half of pier with reinforced concrete with base of footing at least 1.5 ft (0.5 m) lower than old pier footing, and placing a concrete apron around the remaining rock masonry pier at bed level to prevent future flows from undermining the pier (fig. 14).

1970-71 Floods with recurrence intervals (R. I.) of about 5 years caused no damage. The countermeasure (fig. 15) is considered effective.

Discussion: An effective method of protecting spread footings on alluvium is to place a concrete apron around the piers. Top of the apron should be at or below the streambed to prevent constriction of flow and increased flow velocities.

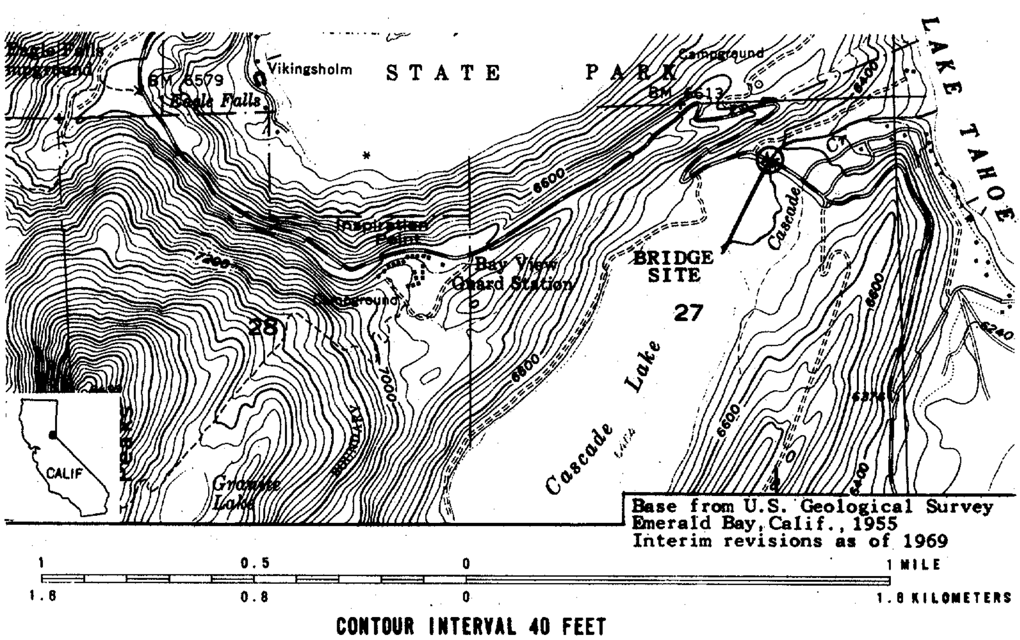


Figure 12. Location of Cascade Creek bridge site.

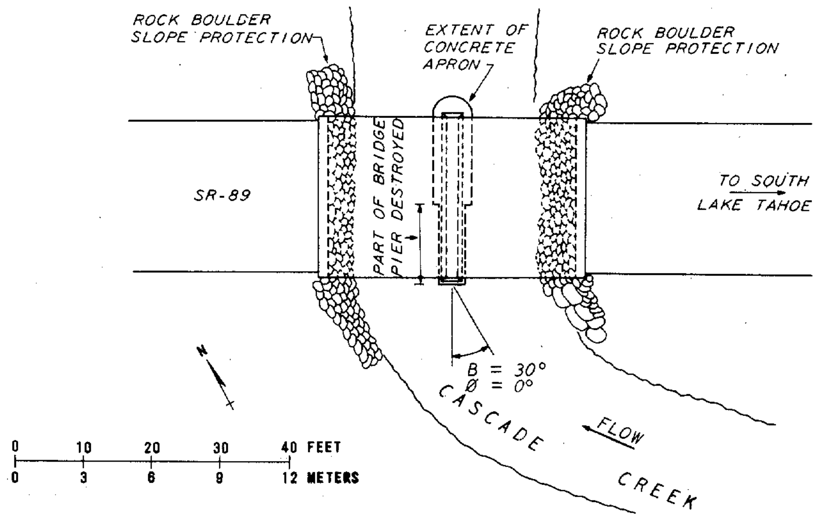


Figure 13. Plan of Cascade Creek bridge site.

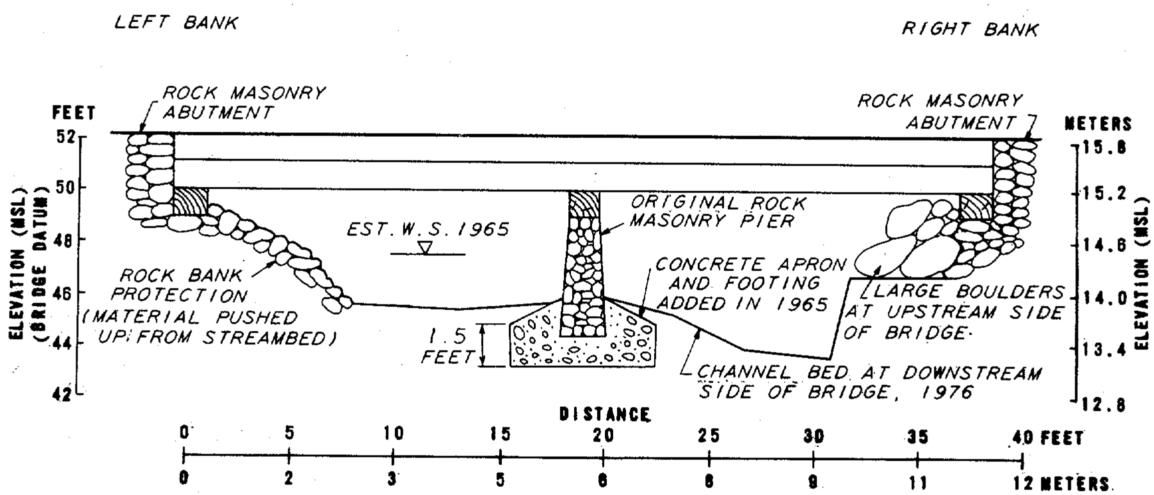


Figure 14. Cross section at Cascade Creek.

Figure 15. Concrete apron around bridge pier, 1965. (From Calif. Dept. of Transportation.)

SITE 3. NORTH YUBA RIVER AT SR-49 NEAR GOODYEARS BAR, CALIF.

Description of site: Lat 39°31', long 121°01'. Precast-concrete bridge, 312 ft (95 m) in length, concrete single-column piers. The left abutment and the three piers are on spread footings founded on bedrock. The right abutment is supported by piles. The channel is composed of bedrock on the left side, and cobble and gravel on the right side.

Valley slope, 0.0050. Stream is perennial, non-alluvial, cobble-boulder bed, in valley of high relief, no flood plain.

Hydraulic problems and countermeasures:

1964 Bridge built (fig. 16) following collapse of previous bridge during floods in 1963.

1964 Flood of December 1964, discharge 44,000 ft^3/s (1250 m^3/s), R. I. about 40 yr.

1969 Bridge inspection reports noted the exposure of aggregate over the entire circumference of the center pier. The channel bed appeared stable, and no scour of the pier footing was observed. Flow at the time of the bridge inspection in August was clear and turbulent.

1972 Abrasion of the concrete on the pier column had progressed such that reinforcing steel was exposed. Wear of the column, both in terms of depth and height, was greatest on the downstream side. The additional abrasive damage noted between 1969 and 1972 probably occurred during the January 1970 flood (discharge 21,000 ft^3/s or 595 m^3/s).

1974 By 1974, damage to the concrete pier (fig. 17) required repairs, which consisted of a reinforced-concrete collar around the damaged part of the pier (fig. 18).

1976 Inspection of the site indicates the concrete collar is performing satisfactorily.

Discussion: Addition of the concrete collar around the damaged pier will not prevent future abrasive damage, but does solve an immediate problem. The wearing damage of the pier is apparently caused by turbulent currents at the side of the pier carrying suspended material which provides the abrasive action. Depths of flow at the pier during the 1964 flood were about 28 ft (8 m) (fig. 16). This type of problem is apparently unique with large diameter single column piers. The abrasive damage noted on the face of the cylindrical pier column is significant because most of the damage was observed at the downstream face. Normally, the upstream face on a pier is most subject to impact damage. The turbulence of flows around the cylindrical pier, suspended sediment, and depths of flow are considered significant factors in the abrasive damage observed at the pier.

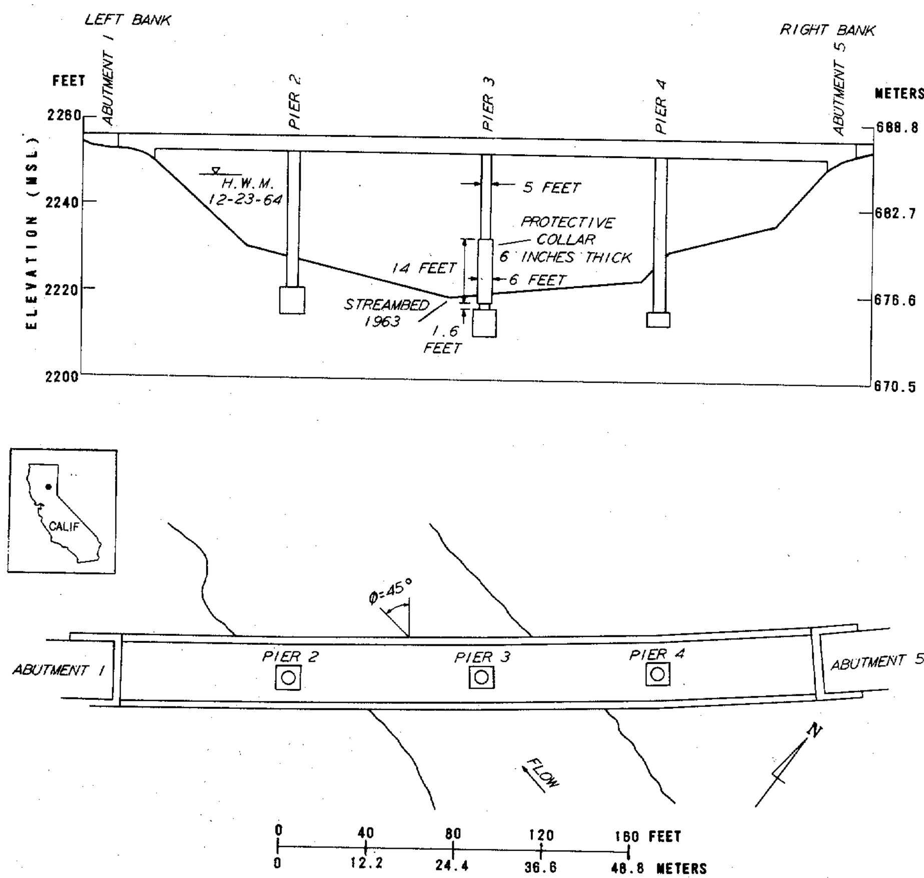


Figure 16. Plan and cross section of North Yuba River at SR-49 bridge near Goodyears Bar, California.

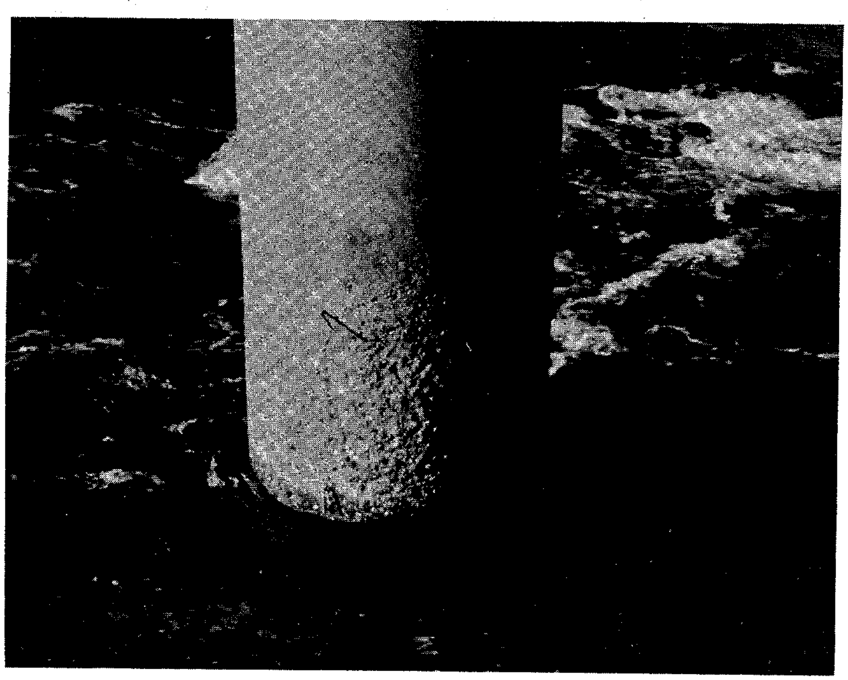


Figure 17. Abrasion at upstream face of concrete pier as photographed in 1974. (From Calif. Dept. of Transportation.)

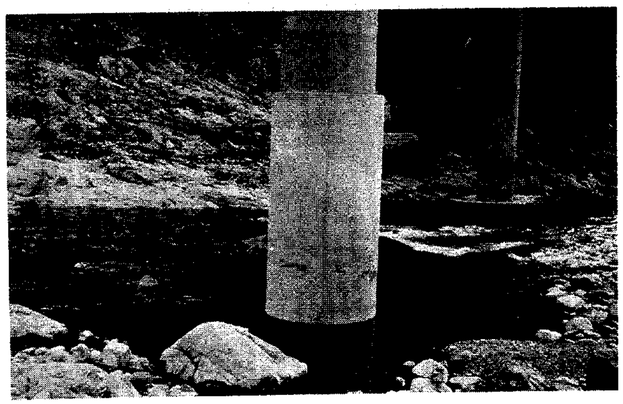


Figure 18. Downstream view in 1976 of concrete collar around pier.

SITE 4. SACRAMENTO RIVER AT SR-162 AT BUTTE CITY, CALIF.

Description of site: Lat 39°27', long 122°00', location as shown in fig. 19. Riveted steel truss swing span, one steel-truss approach span, and prestressed-concrete girder on concrete-pile approach spans across right-bank flood plain. All piers are supported on piles. Total bridge length is 4,389 ft (1,338 m) main channel span is 693 ft (211 m) in length.

Drainage area, 12,081 mi^2 (31,029 km^2); bankfull discharge, 90,000 ft^3/s; valley slope, 0.00032; channel width, about 400 ft (122 m). Stream is perennial, regulated, alluvial, sand bed, wide flood plain, natural levees. Channel is sinuous, locally braided, wider at bends, point bars, cut banks general, silt-clay banks, tree cover at less than 50 percent of bankline. Instability of channel and rapid lateral migration rate is attributed to clearing of vegetation along banks and to confinement of flow by artificial levees.

Hydraulic problems and countermeasures:

1948 Bridge built with swing span to pass river traffic. Clearance of low steel above design water level (Feb. 28, 1940, high water discharge about 160,000 ft^3/s or 4,530 m^3/s) is 9 ft (2.7 m). The center pier is protected from river traffic by a timber drawrest structure (fig. 20), and guide fenders protect the two adjacent piers (fig. 21). The piers are supported by steel piling with pile tips about 41 ft (12.5 m) below the 1946 channel bed level. At a discharge of 64,000 ft^3/s (1,812 m^3/s), the area of piers 25-28 occupy 9 percent of the waterway area as defined by cross section surveyed in 1946.

1949 Timber guide-fender on right bank collapsed during flood of March 12, 1949, (discharge in main channel and overflow 82,200 ft^3/s or 2,327 m^3/s). Sheet piles at west end of swing span, pier 26 (fig. 22), were damaged. Following flood, pier fender was replaced and bent piles straightened.

1952 High water of Dec. 29, 1951. (discharge in main channel and overflow 111,000 ft^3/s or 3,142 m^3/s) caused lateral scour of right bank under the bridge for about 140 ft (43 m) along the stream, and shoreward about 20 ft (6.1 m). Soundings indicated local scour around piers 25, 26, and 28.

1954 Lateral erosion of right bank under bridge continuing (fig. 23). Scour between bents 23 and 24 on right bank flood plain has developed due to return of overland flow from flood plain to main channel (fig. 19). The area near bents 23 and 24 was backfilled with cobble and earth (fig. 23) to prevent additional scour of the channel bank. Main channel bed scour near piers 26 and 28 and right channel bank between piers 24 and 26 continuing. About 7 ft (2.1 m) of scour between piers 25 and 26 occurred between 1946 and 1954 (fig. 23). The deepest point of scour is 80 ft (24 m) away from the edge of the drawrest at pier 27. The location of the scour area indicates the scour is probably caused by the bridge constriction rather than local turbulence around the drawrest and pier 27.

1957 Lateral erosion of right bank between piers 23 and 26 is continuing, but there is evidence of sediment deposition on the flood plain above 90 ft (27 m) elevation (fig. 23).

1974 Seven floods exceeding 100,000 ft^3/s (2,831 m^3/s) occurred between 1954 and 1974 (flood of January 18, 1974, discharge in main channel and overflow, 136,000 ft^3/s) and caused additional local scour of about 5 ft (1.5 m) between piers 24 and 28 (fig. 23). Total scour of the channel bed at the bridge between 1946 and 1974 is about 12 ft (3.7 m). Measurements of the channel upstream from the bridge between 1946 and 1974 show bed scour of less than 1 ft (0.3 m), indicating the channel is not degrading.

Discussion: The bridge spans the main channel and right bank flood plain so floodflows at the bridge are not contracted (fig. 23), yet extensive scour and lateral erosion at the bridge during the period 1948-74 have been observed. The reduction in waterway area for a discharge of 64,000 ft^3/s (1,812 m^3/s) was about 9 percent. Thus the pier size is a factor in constriction of the waterway by the bridge.

Associated with the reduction in waterway area caused by the bridge construction, however, is the corresponding lateral erosion and local scour which tended to replace the initial waterway. This case history demonstrates the need to consider the possibility of scour and lateral erosion at bridges when there is no apparent contraction from overflow returning to the channel but the size and number of bridge piers may be sufficient to contract the flows and cause scour at the crossing.

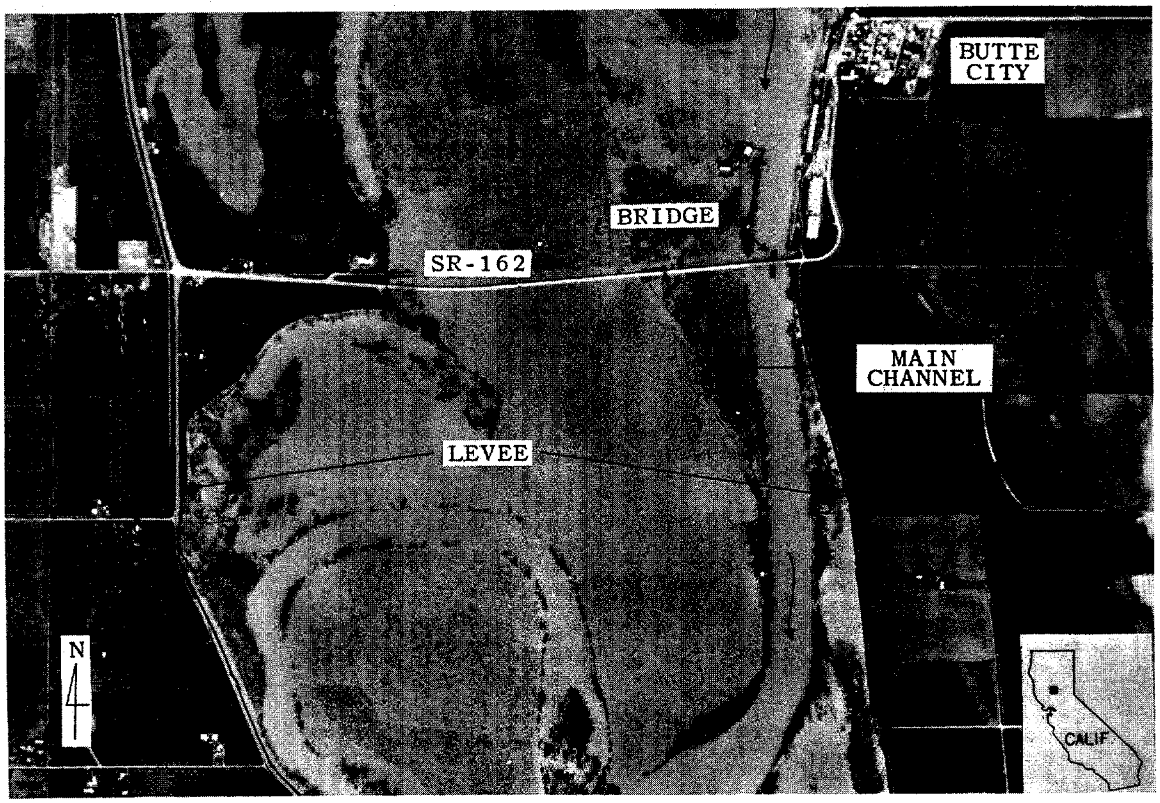


Figure 19. Aerial photograph of Sacramento River at SR-162 bridge during flood of January 19, 1974. (From Calif. Dept. of Water Resources.)

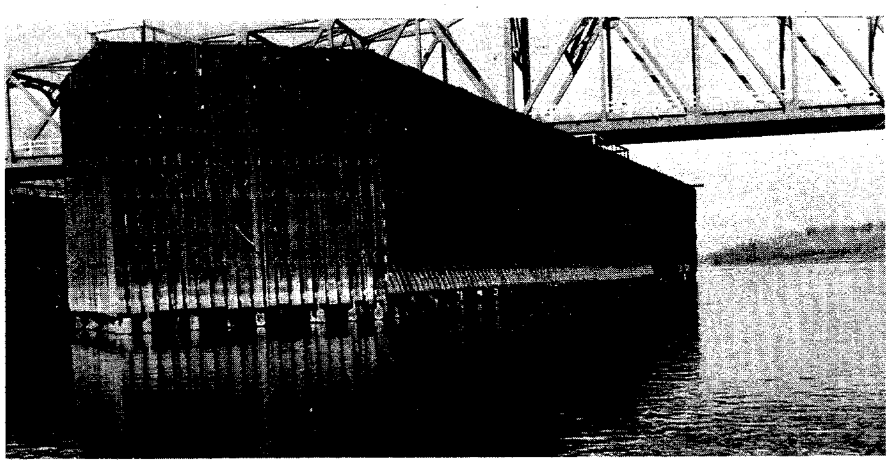


Figure 20. View upstream of timber drawrest structure, in 1977.

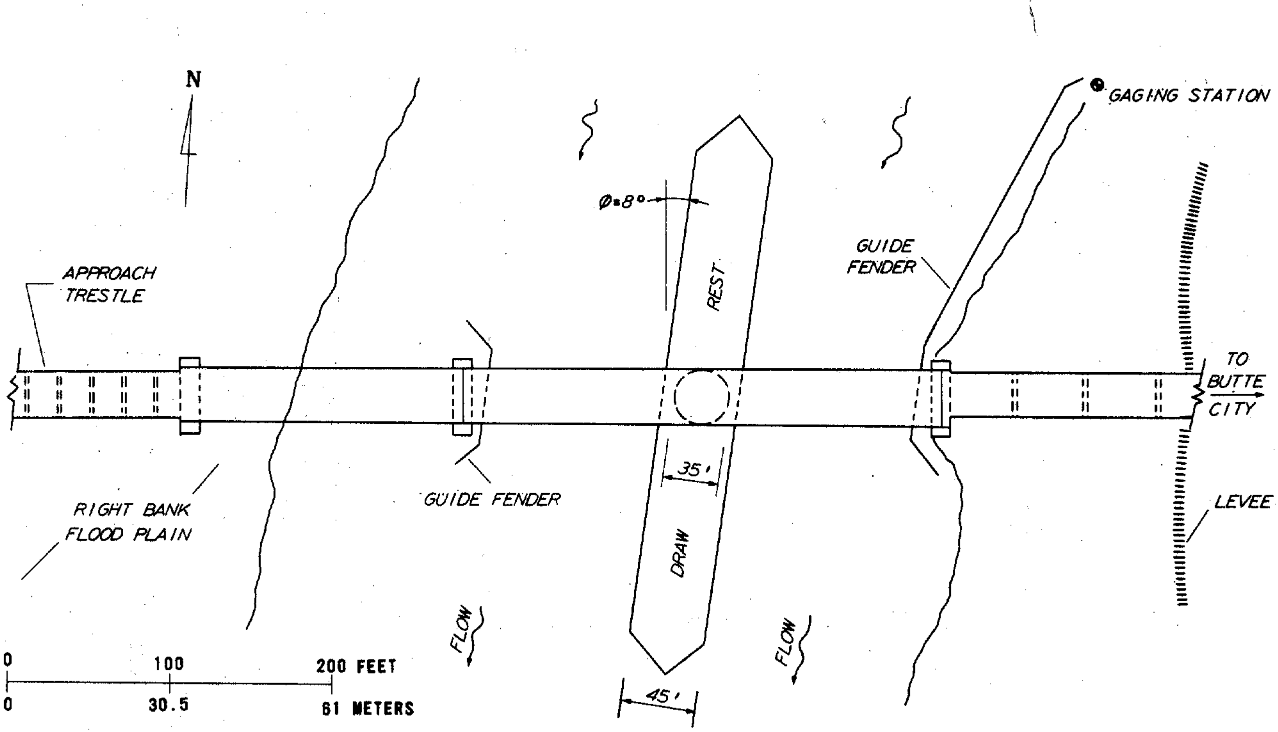


Figure 21. Plan of bridge across main channel of Sacramento River on SR-162 at Butte City.

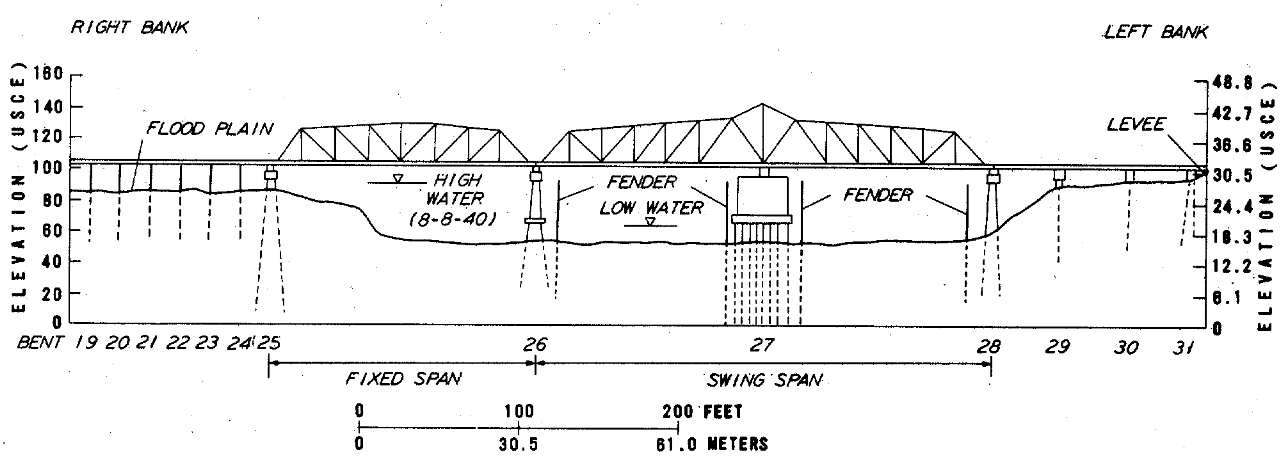


Figure 22. Cross section of bridge across main channel of Sacramento River on SR-162 at Butte City.

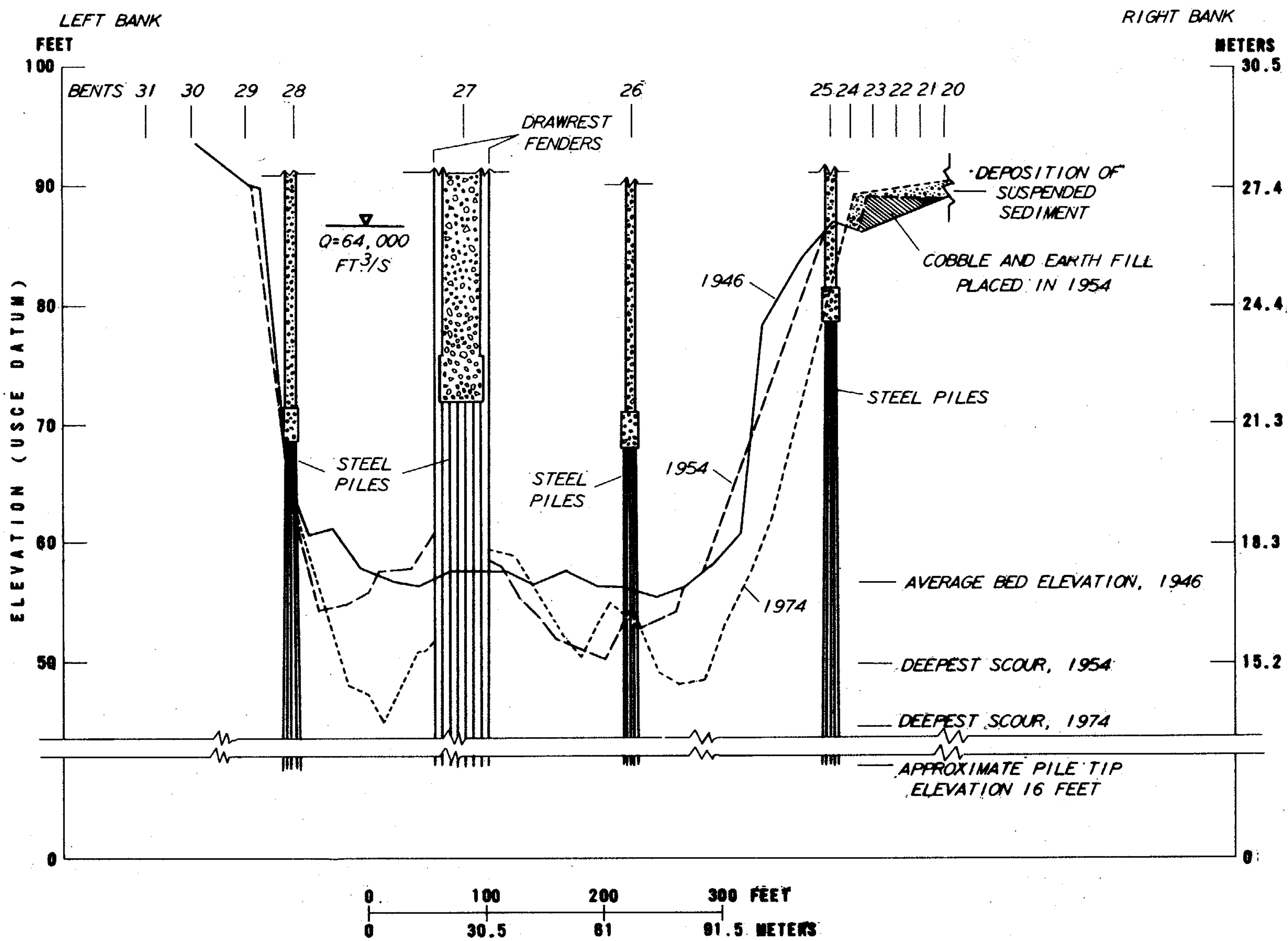


Figure 23. Cross section of main channel in 1946, 1954, and 1974.

SITE 5. SALMON CREEK AT FOREST SERVICE ROAD 20N01.6 NEAR POTTER VALLEY, CALIF.

Description of site: Lat 39°27', long 122°58', location as shown in fig. 24. Reinforced-concrete bridge, concrete wingwalls at abutments, supported on concrete piers founded on steel H-piles. Bridge spans are 32, 42, and 32 ft (9.8, 12.8, and 9.8 m), with total length of 106 ft (32.3 m). Bridge not skewed to flow.

Drainage area, 15 mi^2 (39 km^2); valley slope, 0.00052. Stream is ephemeral, alluvial, gravel bed, on alluvial fan. Channel is generally braided and generally anabranched, sparse desert shrubs along banklines. Crossing is near lower end of alluvial fan, rather than at apex.

Hydraulic problems and countermeasures:

1975 New bridge built to replace two existing structures (fig. 25). Countermeasures included placement of concrete wingwalls; use of rectangular piers with rounded noses; construction of spur dikes (figs 26 and 27) with rock-filled gabion protection; protection of embankment on downstream side of bridge with rock gabions; and regrading the streambed in the vicinity of the bridge (figs. 28 and 29).

1976 The countermeasures have not been tested by floodflow. Hydraulics of the bridge opening for design flow are given in Vol. I under "Flow Factors". Scour (headcutting) of the transition between the old and new channel bottom approximately 900 ft (274 m) upstream from the bridge (fig. 25) may be expected during future floods.

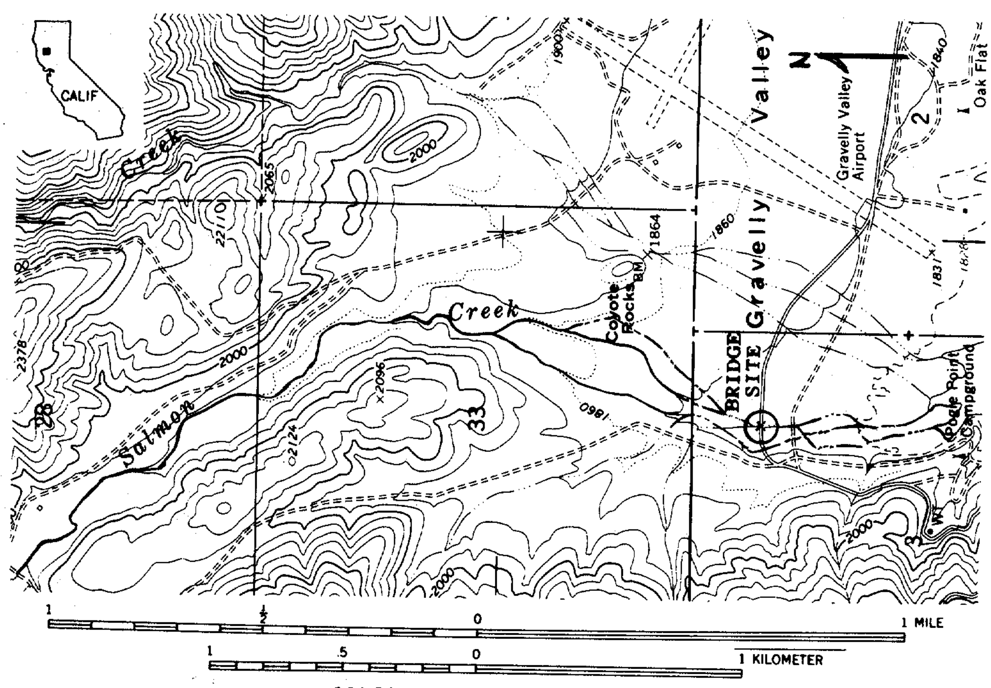


Figure 24. Map showing Salmon Creek at Forest Service road 20N01.6. (Base from U.S. Geol. Survey Lake Pillsbury, Calif., 7.5' quadrangle, contour interval 40 ft, 1967.)

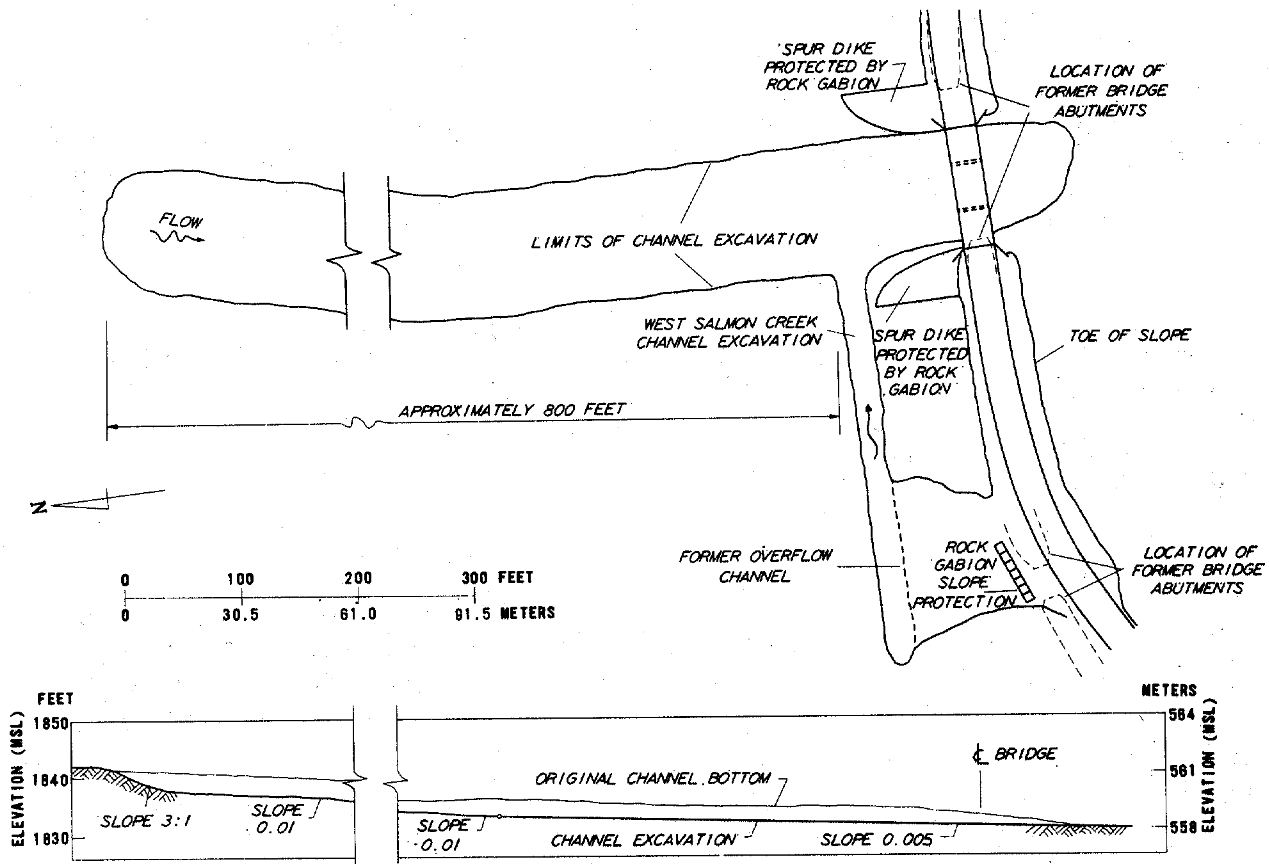


Figure 25. Plan and profile of Salmon Creek at Forest Service road 20N01.6 near Potter Valley, California.

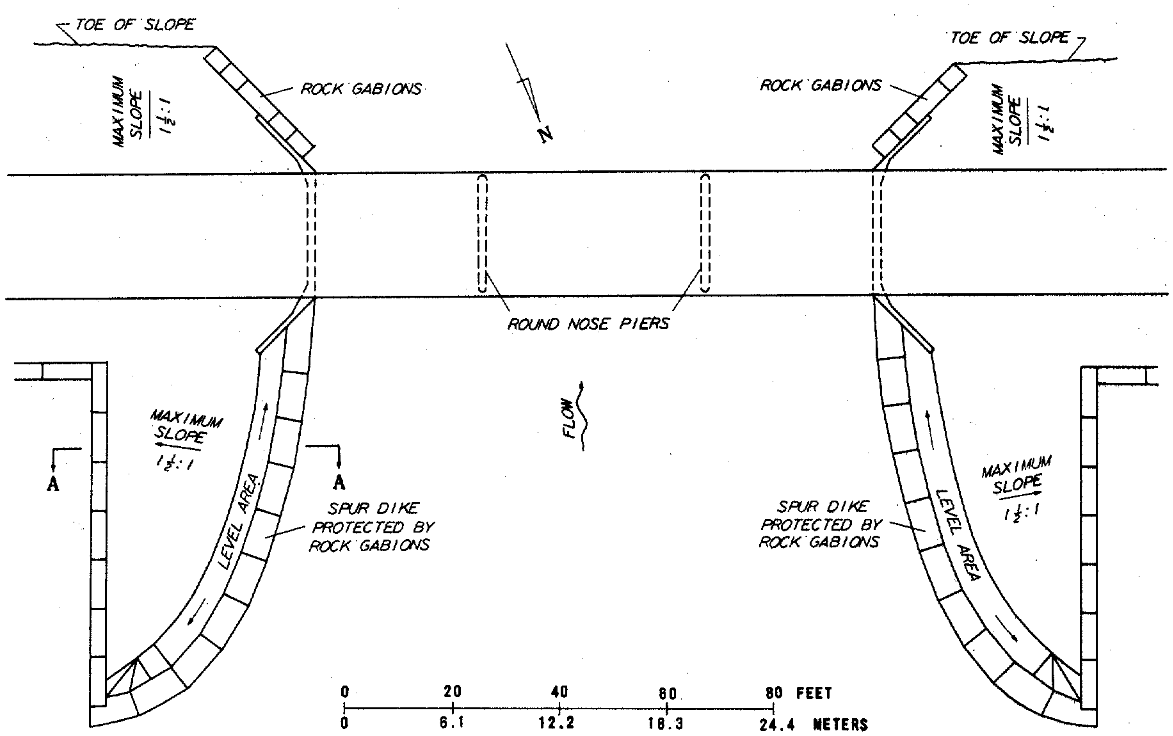


Figure 26. Plan of Salmon Creek bridge.

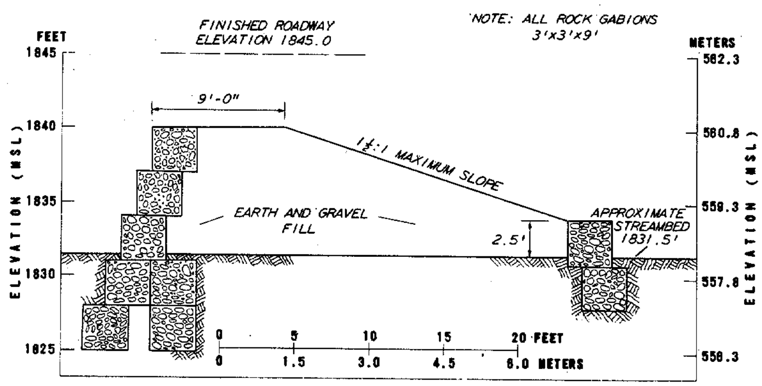


Figure 27. Section A-A, details of spur dike.

Figure 28. Downstream view of Salmon Creek bridge built in 1975. (From U.S. Forest Service.)

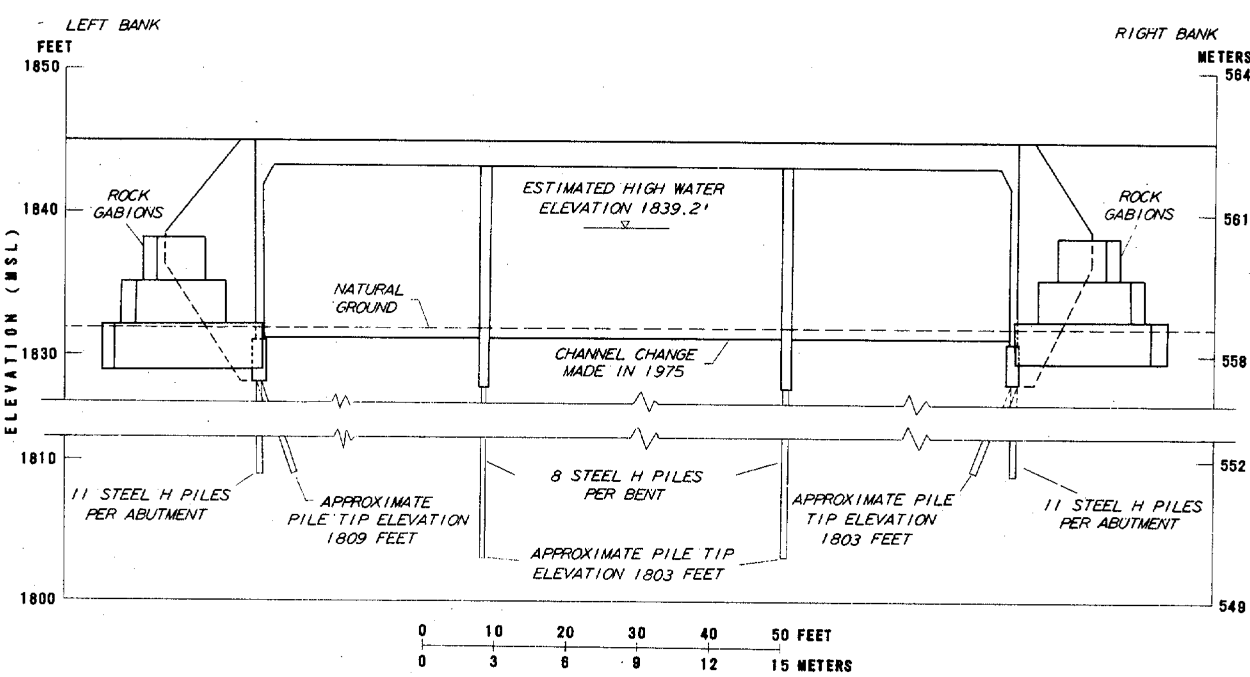


Figure 29. Section at downstream side of bridge, Forest Service road at Salmon Creek.

SITE 6. SMOKEHOUSE CREEK AT FOREST SERVICE ROAD 20N01.6
NEAR POTTER VALLEY, CALIF.

Description of site: Lat 39°27', long 122°58'. Reinforced-concrete bridge with concrete wingwalls at abutments and supported by concrete piers, founded on steel H-piles. Bridge spans are 32, 42, and 32 ft (9.7, 12.8, and 9.7 m), with total length 106.5 ft (32.5 m). No bridge skew to flow.

Drainage area, 13 mi^2 (34 km^2); valley slope, 0.0097. Stream is ephemeral, alluvial, gravel bed, on alluvial fan. Channel is generally braided and generally anabranched. Crossing is near mid-point of alluvial fan, rather than at apex.

Hydraulic problems and countermeasures:

1975 New bridge (fig. 30) built to replace two existing structures. Countermeasures included installation of rock gabion abutment protection (fig. 31); use of rock gabion along upstream side of approach embankment (fig. 32); placement of concrete wingwalls and use of rectangular piers with rounded nose; regrading the channel (fig. 30); and clearance for drift of about 3 ft (1 m) below the bridge superstructure.

1976 Countermeasures have not been tested.

Discussion: Crossings on alluvial fans are usually made either near the apex, where a single rather definite channel is characteristic, or near the base of the fan, where channels are poorly defined but discharge is reduced by infiltration or dispersion (Calif. Div. of Highways, 1970, p. 83). The crossing of Smokehouse Creek (as well as the adjacent crossing of Salmon Creek) is near the mid-point of the fan, and countermeasures have been provided to prevent erosion of the abutment fill-slopes.

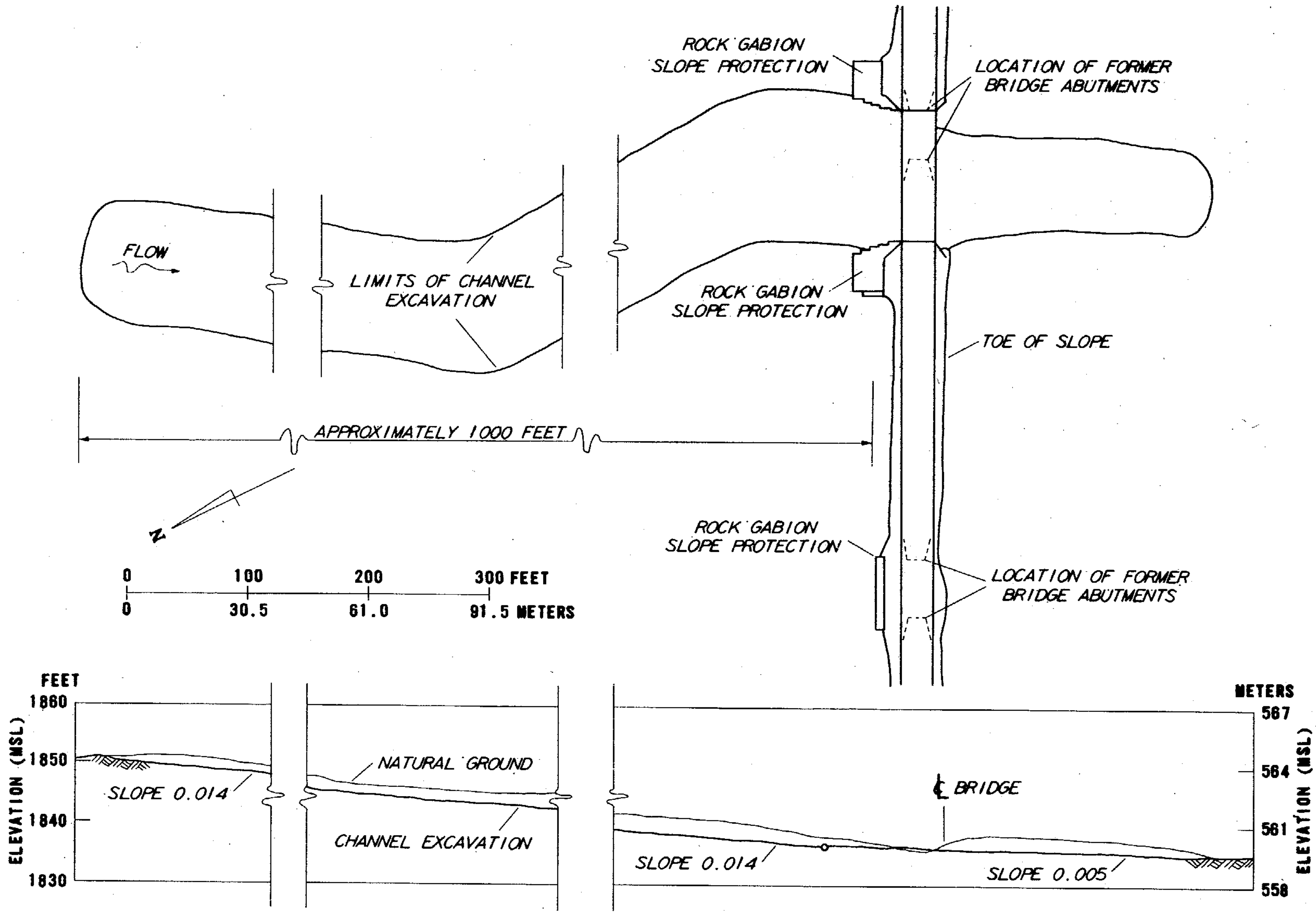


Figure 30. Plan and profile of Smokehouse Creek at Forest Service road 20N01.6.

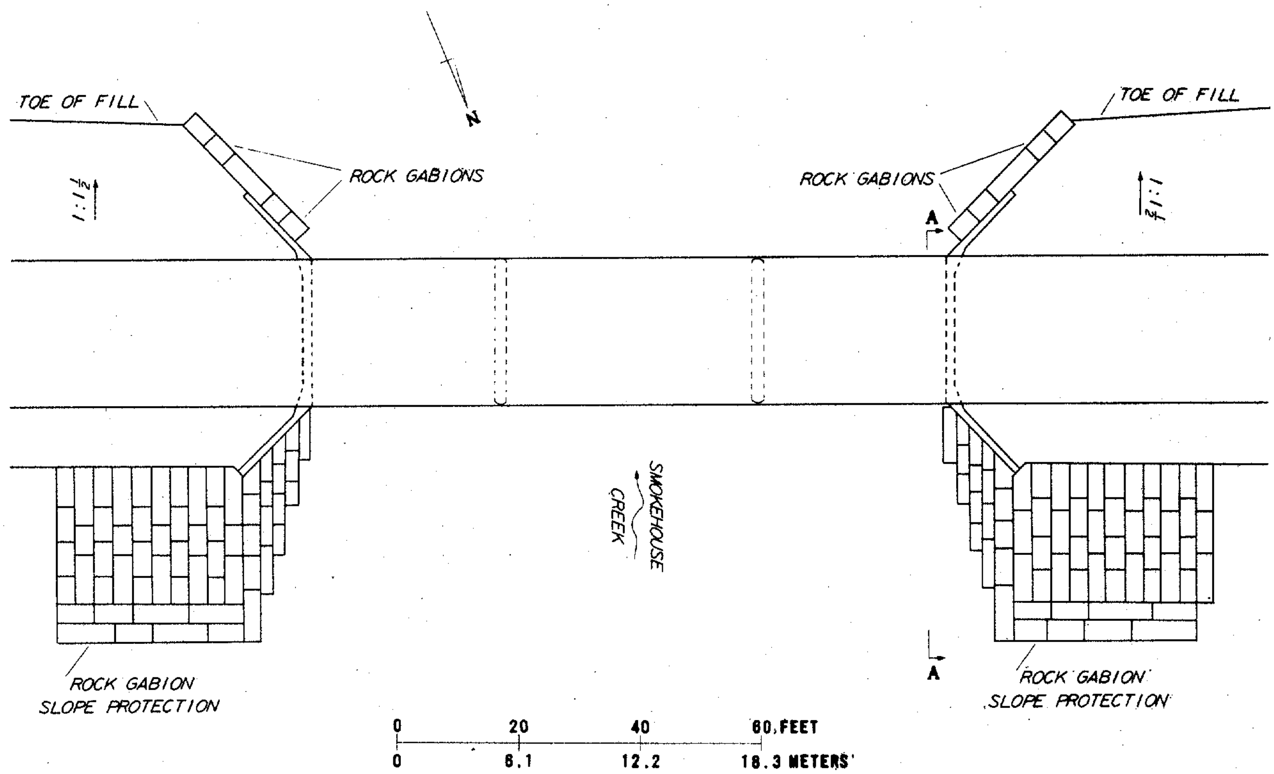


Figure 31. Details of rock gabion slope protection.

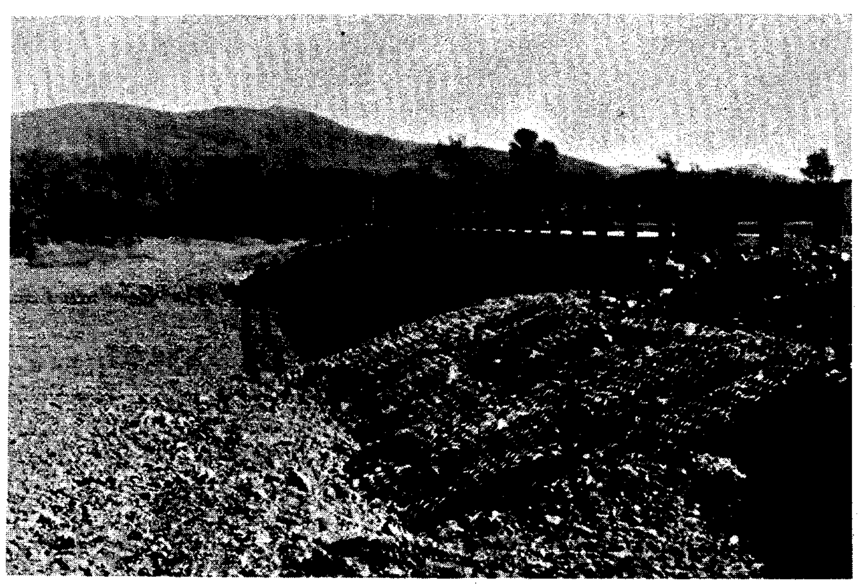


Figure 32. View of rock gabion slope protection on upstream side of right-bank approach embankment. (From U.S. Forest Service.)

SITE 7. STONY CREEK AT I-5, NEAR ORLAND, CALIF.

Description of site: Lat 39°46', long 122°12', location as shown in fig. 33. Reinforced-concrete, continuous T-beam girder bridge built in 1966, with 7 columns per bent (pile extension) on steel H-piles, and spillthrough abutments. Bridge length is 934 ft (285 m) with one span of 41 ft (12.5 m), 17 spans of 50 ft (15.2 m) and one span of 41 ft (12.5 m). Bridge skew is 30 degrees.

Drainage area, about 777 mi^2 (2,012 km^2); valley slope 0.0025; channel width, about 1,500 ft (457 m). Stream is perennial (but highly regulated, such that there is no flow during much of the year), alluvial, sand-gravel bed, on an alluvial fan, distinct natural levees. Channel is locally braided, generally anabranched, random width variation, silt-sand banks.

Hydraulic problems and countermeasures:

1964-66 Several countermeasures were installed during construction of the bridge as follows: Channel was realined upstream from bridge, and a 100-ft (30-m) wide overflow channel 700 ft (213 m) north of the bridge was blocked off; overflow was diverted through a drainage ditch to main channel upstream from bridge (fig. 34). The main channel in vicinity of bridge was excavated an average of 3 ft (1 m) below original streambed to channelize flow (fig. 35). Light-duty double pipe-and-wire fence retard (like that in fig. 36) was installed on both banks upstream from bridge. The abutments and embankments on both banks were covered with grouted rock revetment. A steel sheet-pile cutoff wall was placed around the toe of upstream side of left-bank abutment.

1967 High water of Jan. 1967 (discharge = 10,800 ft^3/s or 306 m^3/s) eroded 150 ft (46 m) of double pipe-and-wire retard on right bank (fig. 33) by undermining support posts. Retard was placed on right-bank flood plain too far above channel to be effective. Performance of left-bank retard satisfactory. General scour is attributed to constriction of floodflows, and progressive degradation is attributed to several factors, including upstream impoundments and sand and gravel removal operations. Scour and degradation exposed steel piles at five bents to damage from rust and abrasion. Exposed parts of piles were encased in a concrete jacket for a distance of 6 ft (2 m) below existing concrete columns (fig. 37). Scour at toe of right bank abutment was evidently caused by flooding during winter of 1967. Additional grouted rock was placed at toe of existing rock riprap on right bank abutment. Remains of right-bank retard was removed and a new heavy-duty double pipe-and-wire retard placed along upstream side of embankment at a lower elevation to prevent undermining of support posts. A training dike (or levee) was constructed along tne right bank of the channel (fig. 38).

1971 High water in Jan. 1969 (Q = 10,000 ft^3/s or 283 m^3/s) and in Jan. 1970 (Q = 12,500 ft^3/s or 354 m^3/s) caused additional scour around bridge pier (fig. 39) and changes in main channel flow alinement.

to the bridge. Excavated material from the main channel upstream from the bridge was placed above the right bank retard to extend training levee upstream.

1974 Two additional floods occurred following installation of countermeasures in 1967 and 1971. Floods of Jan. 1973 ($Q = 10{,}200$ ft^3/s or 289 m^3/s) and Jan. 1974 ($Q = 15{,}200$ ft^3/s or 430 m^3/s) did not cause significant erosion problems at the bridge.

Discussion: Original pipe-and-wire retards built on the right bank were too far above channel to be effective in preventing streambank erosion upstream from bridge. Modification of the main channel alinement, deepening the channel bed during original highway construction, and addition of the left-bank overflow to divert overflow from the flood plain to the main channel were factors contributing to the general streambed scour noted at the bridge.

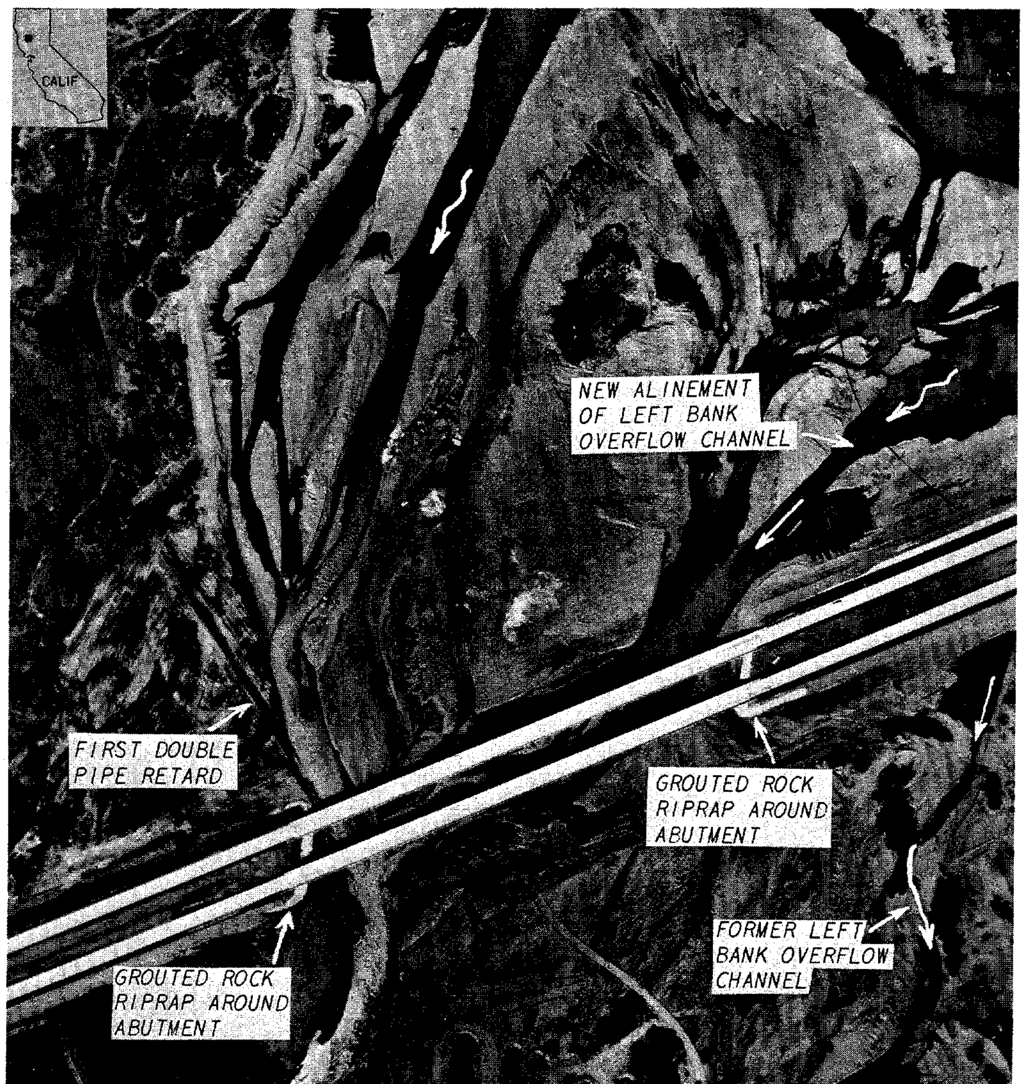


Figure 33. Aerial view of Stony Creek bridge after flood of January 1967. (From Calif. Dept. of Transportation.)

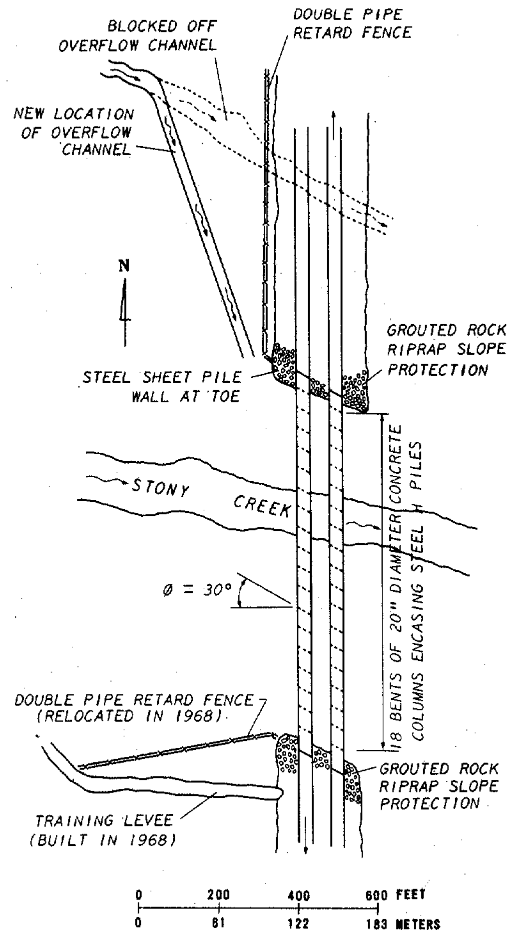


Figure 34. Plan of bridge built over Stony Creek in 1964.

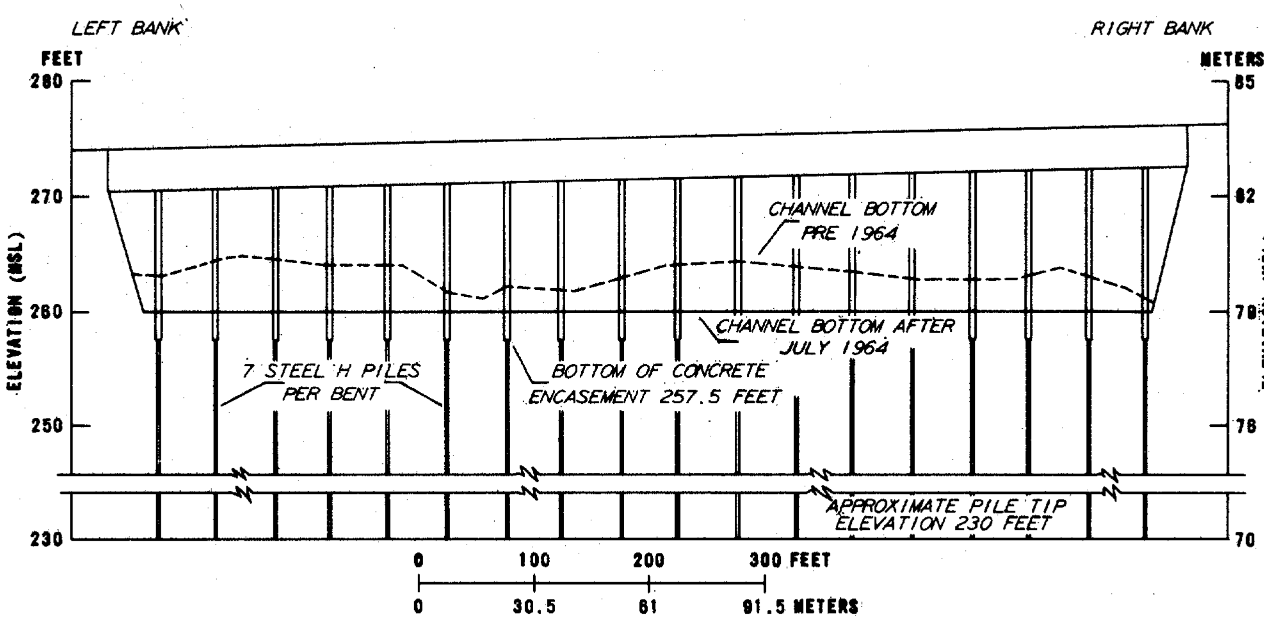


Figure 35. Cross sections of Stony Creek before and after bridge construction in 1964.

Figure 36. Light duty, double pipe-and-wire retard (center) and grouted rock revetment at approach embankment (right). Photographed in 1976.

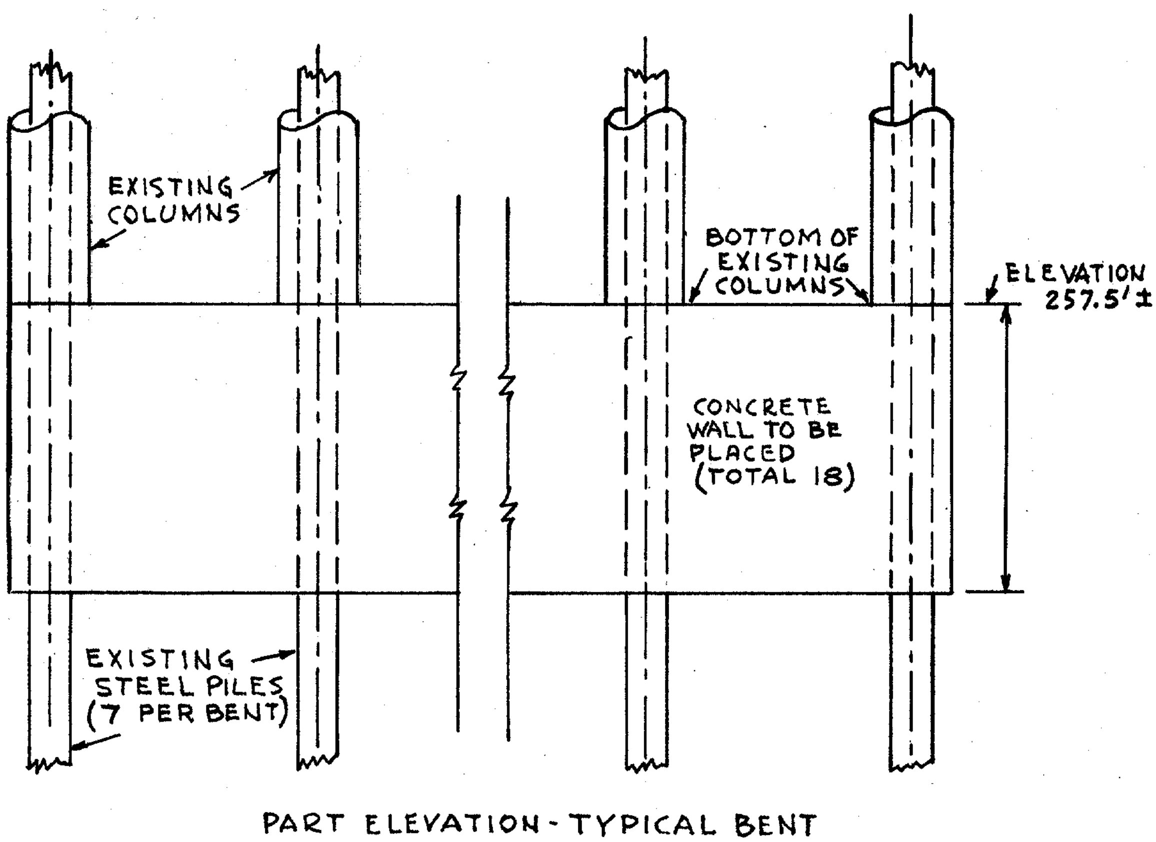


Figure 37. Repairs made in 1967 to exposed parts of steel H-piles.

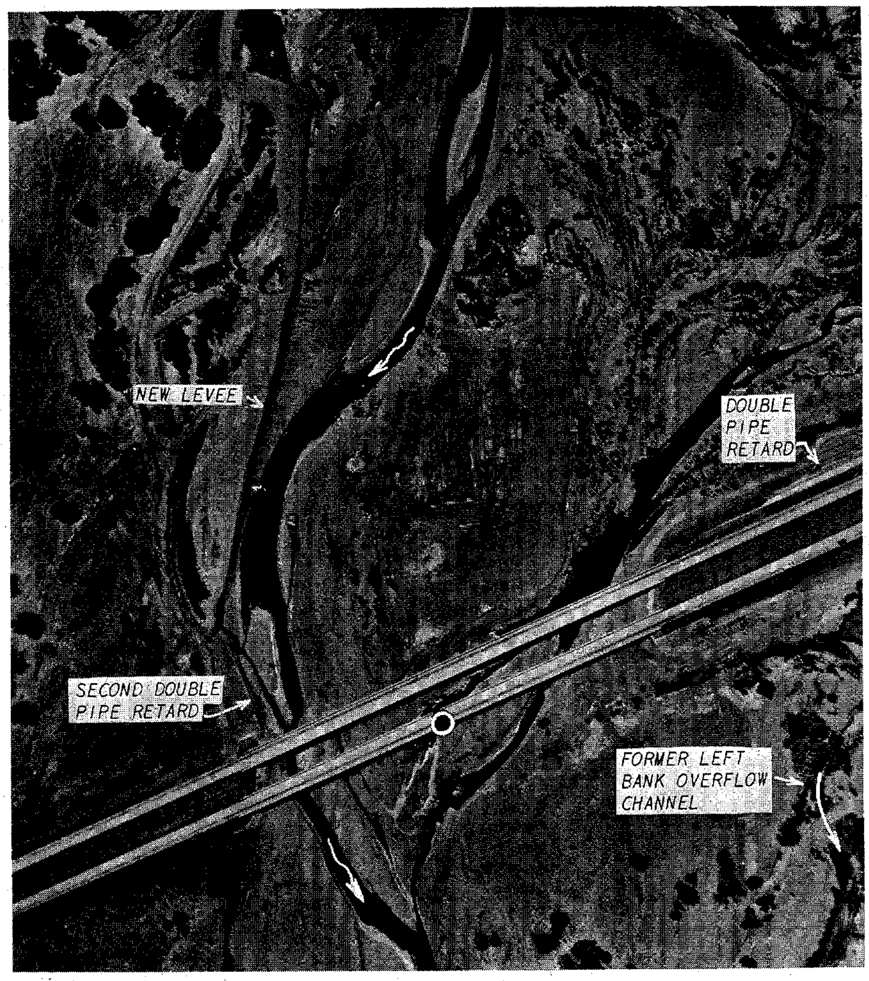


Figure 38. Aerial view of Stony Creek bridge showing extension of right-bank levee in 1970. (From Calif. Dept. of Transportation.)

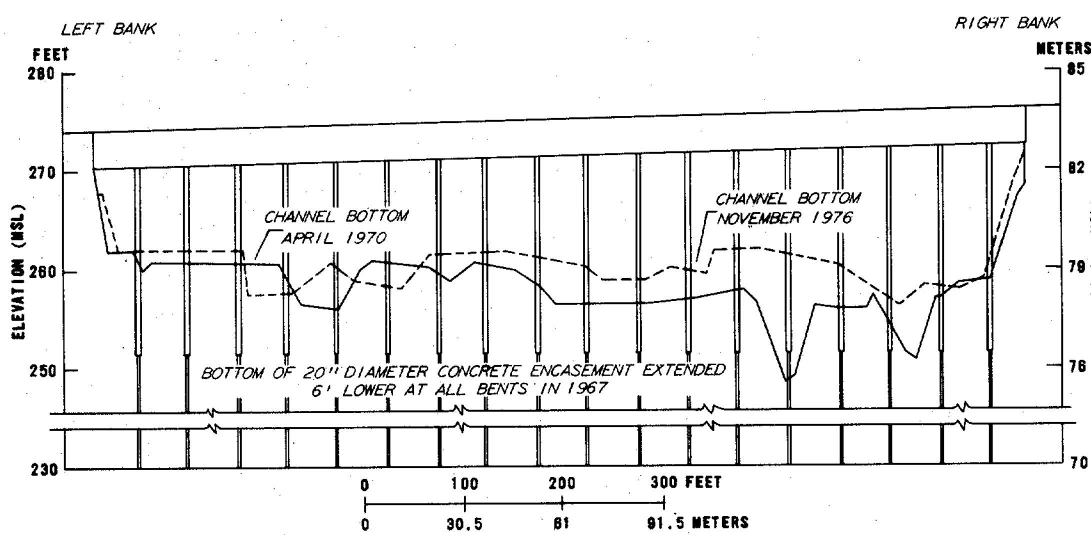


Figure 39. Cross sections of Stony Creek in 1970 and 1976.

SITE 8. SALMON CREEK AT SR-49 NEAR SIERRA CITY, CALIF.

Description of site: Lat 39°36', long 120°31', location as shown in fig. 40. Reinforced-concrete box-girder bridge, built in 1964, has reinforced-concrete piers and abutments on spread footings founded on bedrock. Total bridge length is 252 ft (77 m). Channel bed material includes boulders up to 7 ft (2.1 m) in diameter, cobbles, gravel, and areas of exposed bedrock. During floods, large amounts of floating debris, boulders, and suspended material are transported and channel scour occurs. Former bridge (fig. 41), built in 1925 and located 1/8 mi (0.2 km) upstream, required extensive repair to damaged piers and abutments after flood events.

Drainage area, 17 mi^2 (44 km^2); valley slope, 0.050; channel width, about 100 ft (30 m). Stream is perennial, semi-alluvial, boulder bed, in valley of high relief, no flood plain. Channel is sinuous, boulder banks.

Hydraulic problems and countermeasures:

1937 Flood of unknown magnitude washed out left-bank approach fill of 1925 bridge and footings of bents 2 and 3 were undermined. The upstream columns of bents 2 and 3 were damaged by battering action of rolling boulders and drift accumulated between columns. Approach fill replaced and heavy riprap protection was added on upstream side. Footings were lowered.

1950 Flood (discharge, 17,400 ft^3/s or 493 m^3/s) caused the channel to shift toward right bank and washed out approach fill and heavy riprap. Upstream columns of bents 2 and 3 were again damaged by abrasive action of debris and rolling boulders.

1951 Curtain walls were constructed at bents 2, 3, and 4 to prevent lodging of debris between columns. A 35-foot (11 m) span with abutment and wingwalls was added to the right-bank end of bridge.

Pre-1955 A protective lining consisting of a timber layer with steel plate outer lining had been placed along upstream face of bent 3.

1955 Flood (Q = 11,500 ft^3/s or 326 m^3/s) caused some erosion of right-bank approach fill, damaged the upstream faces of all bents except bent 3 by abrasion and deposited debris at upstream side of bridge (fig. 42). The steel-plate lining at bent 3 was bent and torn slightly along the sides, but did protect the concrete. The eroded right-bank approach fill was replaced and some large rocks were placed at toe of abutment.

1964 Countermeasures incorporated in the new bridge (fig. 43) included the following: The minimum span length was 35 ft (11 m) and piers were closed to prevent lodging of drift on bridge piers. Full-height concrete abutments were used to increase waterway area between bridge abutments (fig. 44). The bridge has about 8 ft (2.4 m) freeboard above the design discharge of 4,000 ft^3 (113 m^3/s) (fig. 45). The channel bed was excavated to a minimum width of 50 ft (15 m) at base (fig. 45), with a slope of 0.050 for a reach of about 250 ft

(76 m) in vicinity of bridge. The channel alinement was also modified to reduce the sharp bend at the bridge crossing. Bridge piers were constructed with a sloped upstream cutwater (fig. 46) to guide boulder movement, and armored with steel plate to prevent abrasive damage and spalling of concrete at piers.

Late 1964 — Flood, discharge 19,200 ft^3/s (503 m^3/s), caused no damage to new bridge. The flood water level was about the same as the design level because of channel excavation during bridge construction, but the flood discharge exceeded the design discharge of 4,000 ft^3/s by about 6 times.

1976 — Floods during the period 1964-76 have not caused noticeable damage to the bridge, although the bridge piers show the effects of abrasion (fig. 47). The channel in the vicinity of the bridge has not changed appreciably between 1964 and 1976.

<u>Discussion:</u> Addition of curtain walls at bents prevented lodging of debris between pier columns. Bridges across streams in mountains should be designed with clearance, abutments, and span lengths adequate to pass debris, and not constrict the channel. The use of rectangular-shaped piers with rounded noses was effective in preventing lodging of debris on bridge piers. The use of bridge piers designed with a sloped upstream cutwater and protected with steel plate effectively guided cobble and boulder movement past the bridge, and prevented abrasive damage to piers.

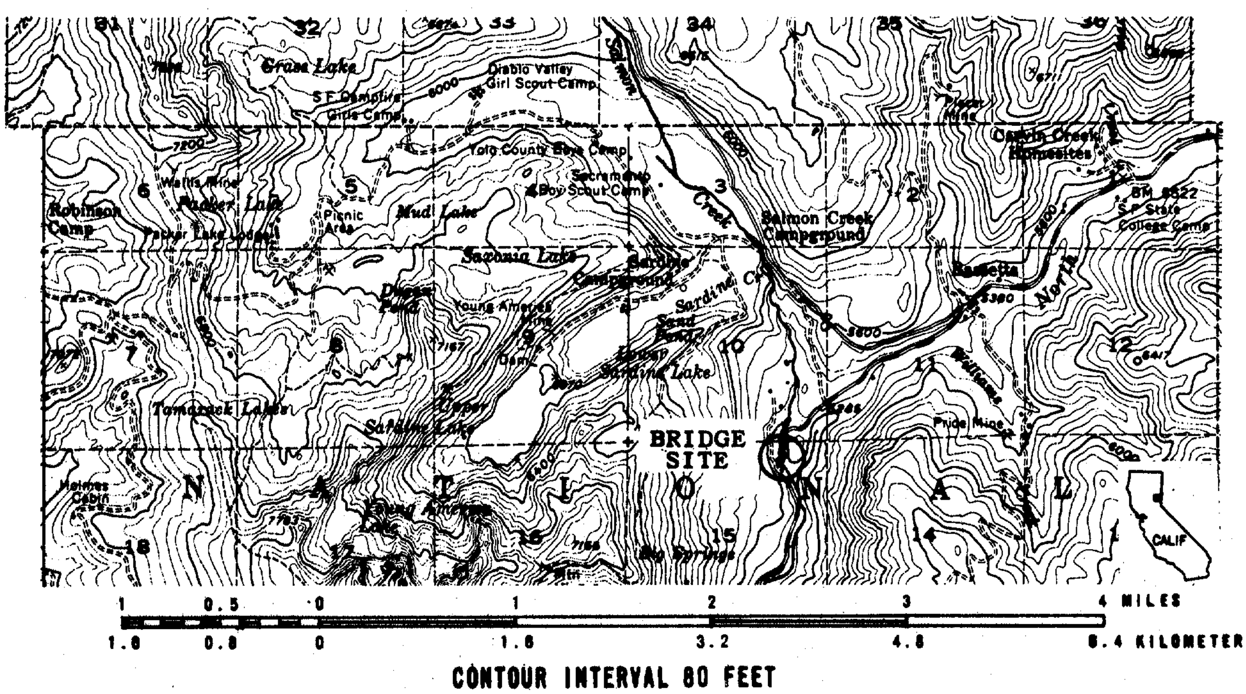


Figure 40. Location of Salmon Creek at SR-49 crossing. (From U.S. Geol. Survey Sierra City, Calif., 15' quadrangle, 1955.)

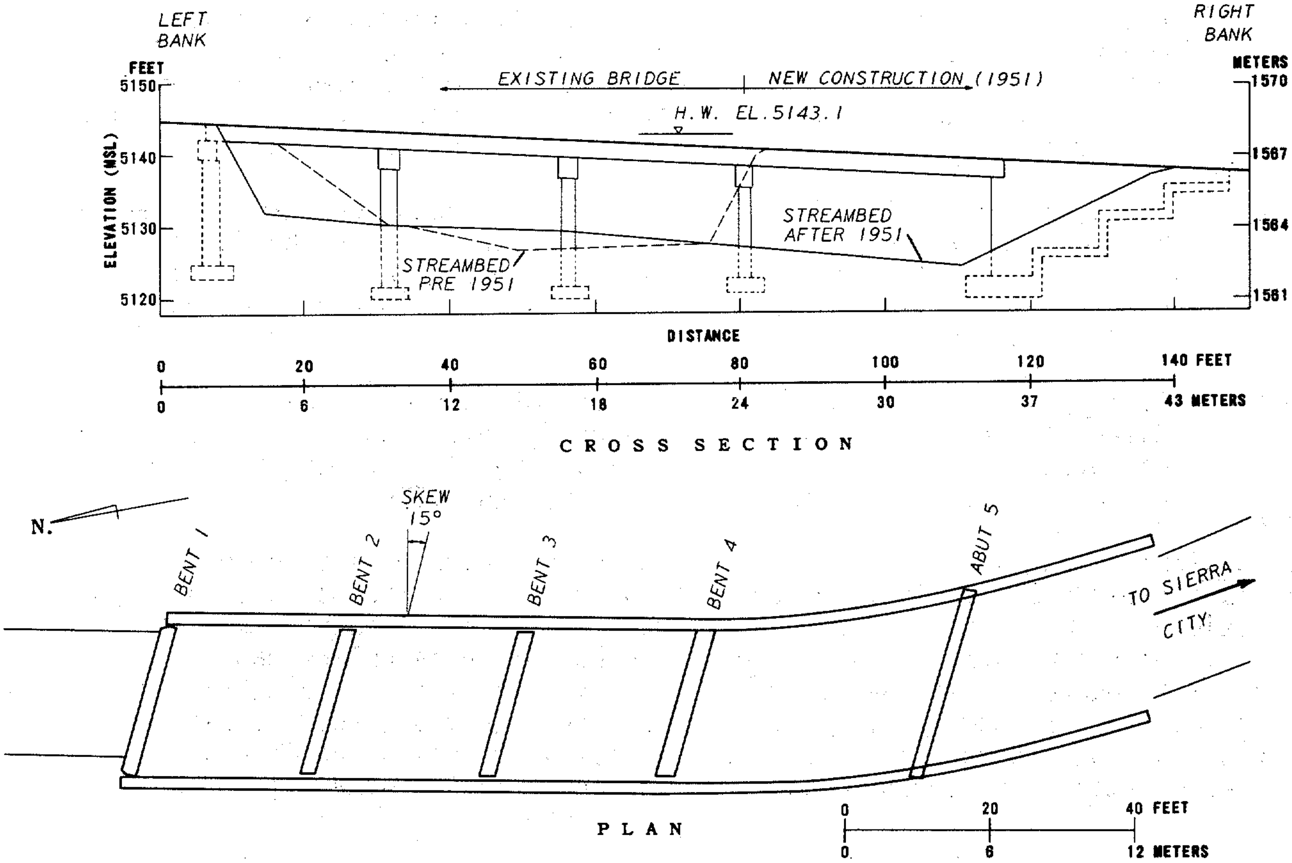


Figure 41. Plan and cross section of Salmon Creek bridge, built in 1925 and extended in 1951.

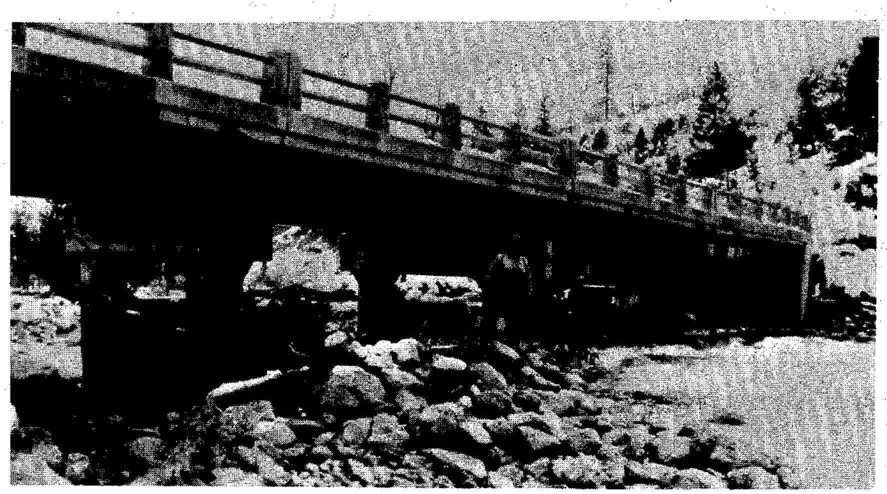


Figure 42. Accumulation of debris at upstream side of old bridge following flood of December 1955.

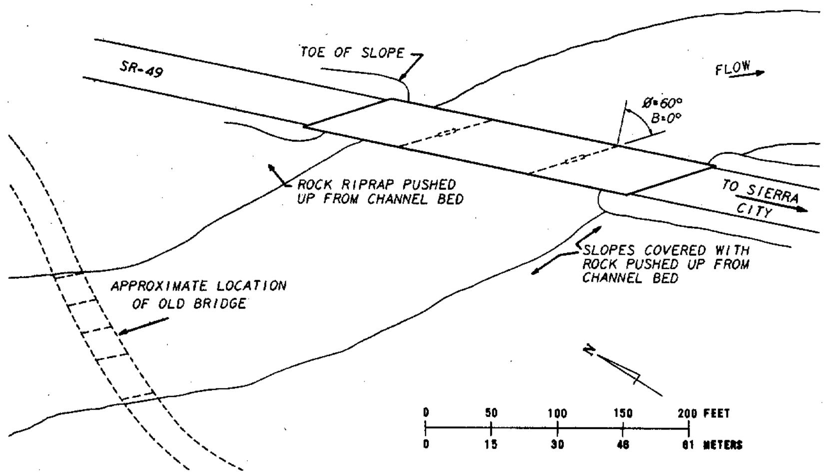


Figure 43. Plan of Salmon Creek bridge, built in 1964.

Figure 44. View downstream of Salmon Creek bridge, built in 1964.

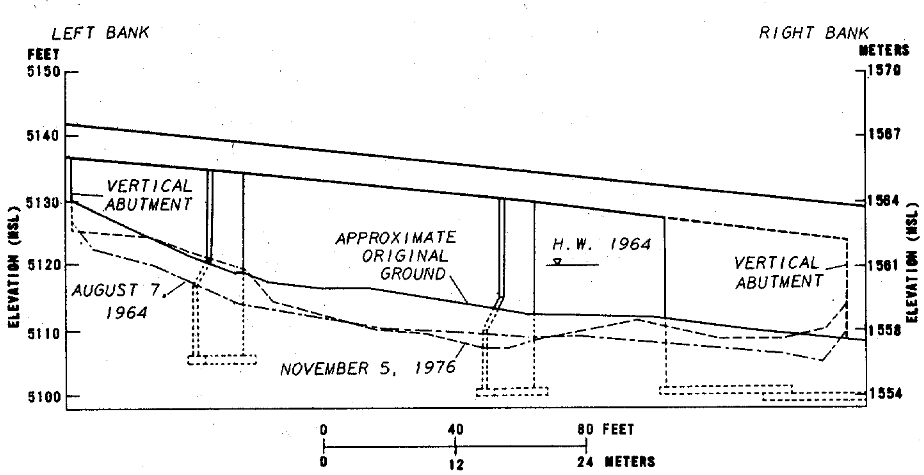


Figure 45. Cross section of bridge built in 1964.

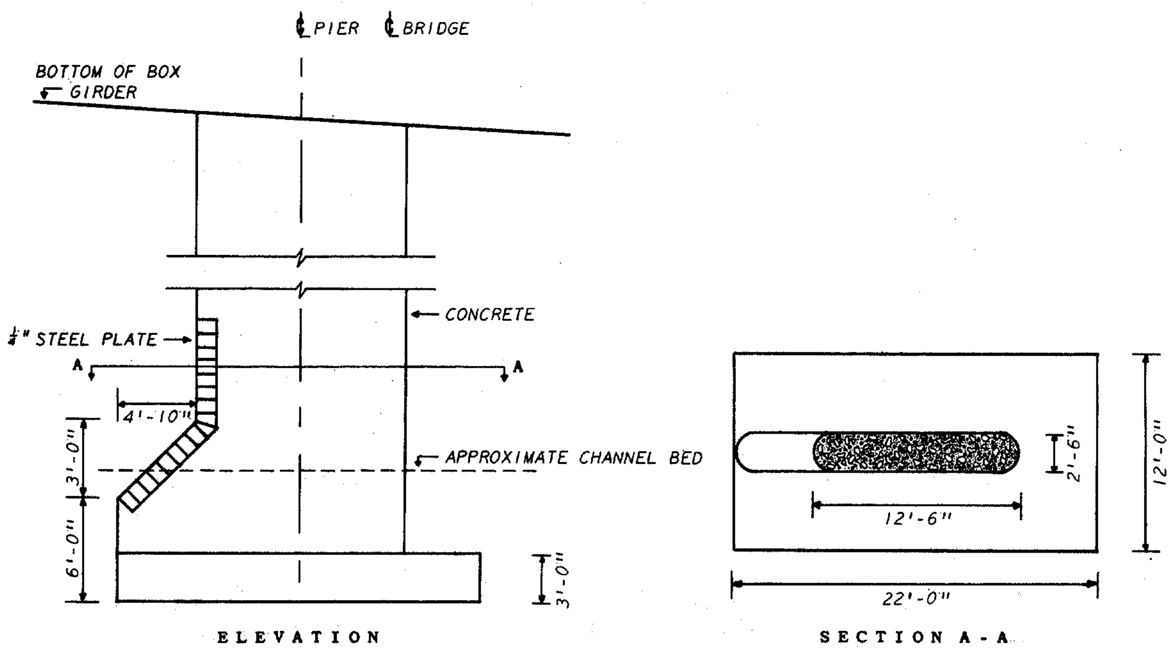


Figure 46. Detail of pier design and armoring, 1964 bridge on Salmon Creek.

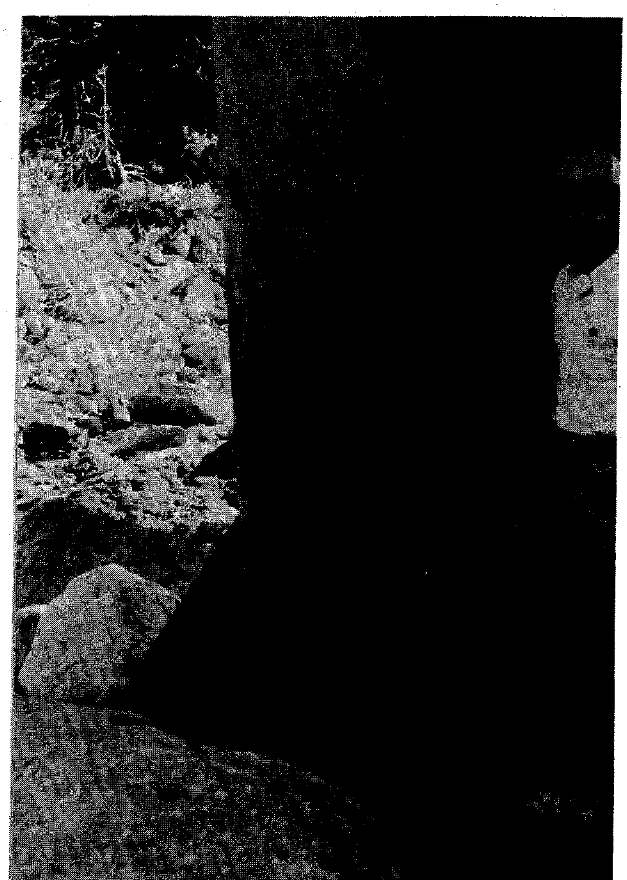


Figure 47. Abrasion of pier near right bank, as photographed in 1976.

SITE 9. TRINITY RIVER AT SR-3 NEAR TRINITY CENTER, CALIF.

Description of site: Lat 41°06', long 122°42', location as shown in fig. 48. Bridge is a 404-ft (123-m) long, concrete box-girder structure, supported on three rectangular concrete piers, with spillthrough abutments. Piers are spread-footing type.

Drainage area, 157 mi^2 (407 km^2); valley slope, 0.010. Stream is perennial, alluvial, cobble-boulder bed, in valley of high relief, narrow flood plain. Channel is sinuous, locally braided, locally anabranched, cut banks rare, sand-gravel banks.

Hydraulic problem and countermeasures:

1968 Concrete box-girder bridge built with spans of 90, 112, 112, and 90 ft (27.4, 34.1, 34.1, and 27.4 m) (fig. 49). The following countermeasures were included in the bridge design and site preparation: A dike was built across flood plain on right bank (fig. 49) to prevent overflow on right bank and damage to the approach embankment. Rock riprap was placed along upstream face of dike and at both abutments. The bridge piers were built without openings between columns to prevent lodging of drift. Pier noses were streamlined, and constructed to withstand impact of occasional large drift. The minimum clear distance between piers to prevent lodging of debris was designed to be 75 ft (23 m), as measured perpendicular to the flow direction. The vertical clearance above design discharge of 14,000 ft^3/s (396 m^3/s) was 5 ft (1.5 m). The base of pier footings located about 10 ft (3 m) below the channel bed.

1974 Flood in January 1974 (discharge of 26,500 ft^3/s or 750 m^3/s, R. I. about 40 yr) caused scour around the piers and settlement of pier 2 (fig. 50). Based on reports by California Dept. of Transportation, test borings taken after flood show general scour of about 9 ft (3 m) at pier 2 and 6 ft (2 m) at pier 4. The average general scour at the bridge was about 2 ft (0.6 m) (fig. 51). Local scour caused by turbulence at the piers was somewhat restricted because of the armoring effect of cobble present in the channel bed. Drift apparently did not contribute to observed scour at the bridge. The primary causes of general scour were considered to be the location of the bridge at the point of channel curvature, and constriction of flow by the right-bank dike (fig. 49).

1976 After the flood, a new footing for pier 2 was placed about 8 ft (2.4 m) lower than the existing footing. This is a depth of about 16 ft (5 m) below the channel bed surveyed in 1976 (fig. 51).

Discussion: Construction of the dike on the right-bank flood plain effectively forced all flow to remain in the main channel and pass under the bridge. Scour of the bridge pier during the January 1974 flood is probably associated with the additional flow (main channel plus overbank flow) forced to remain in the main channel instead of partially bypassing the bridge site. The normal capacity of the main channel to convey the flood was inadequate to handle the total discharge and scour resulted.

The flood of January 1974 was about 1.9 times larger than the design flood. The stage was about 1 ft (0.3 m) higher, giving clearance for drift of 4 ft (1.2 m) instead of 5 ft (1.5 m). The countermeasures applied at the time of bridge construction are considered effective.

Although the bridge damage occurring during the January 1974 flood is considered major, studies by Caltrans concluded only protection at pier 2 (fig. 49) was justified because additional costs to provide protection guaranteed against bridge failure were considered excessive if the channel scour continues. The rock slope protection at the dike and right-bank abutment showed no signs of failure during the 1974 flood.

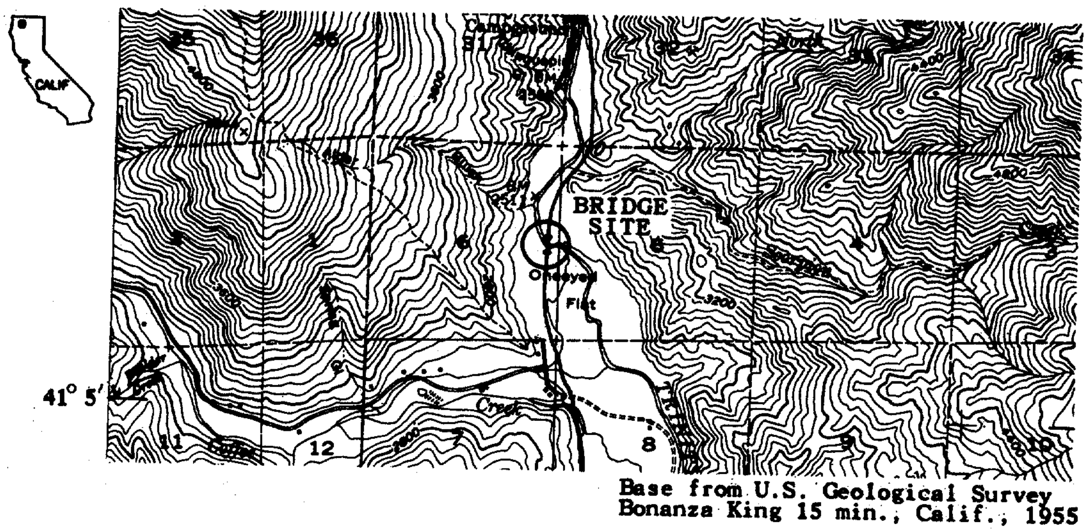


Figure 48. Location of Trinity River at SR-3 crossing.

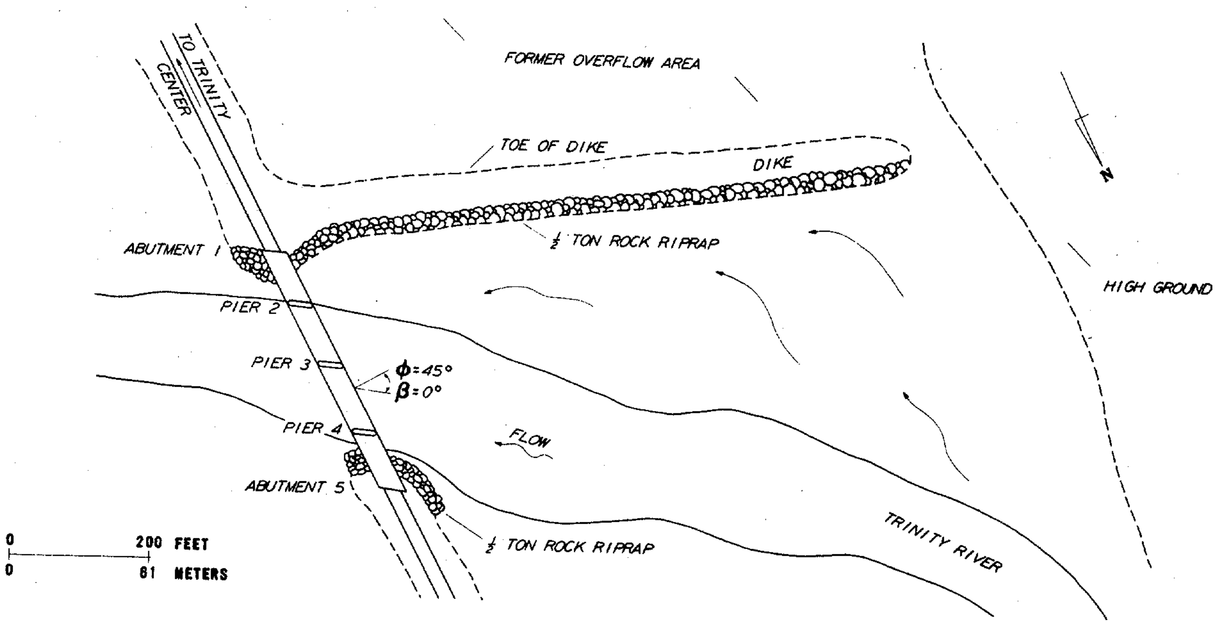


Figure 49. Plan of SR-3 crossing near Trinity Center.

SITE 261. SOUTH FORK FORKED DEER RIVER AT US-51 NEAR HALLS, TENN.

Description of site: Lat 35°57', long 89°24', location as shown in fig. 255. Dual bridges, built in 1963, had a 53-ft (16-m) main span supported by wall-type piers in the main channel, and thirty 28-ft (8-m) approach spans supported by concrete pile bents. In 1975, both bridges over the main channel were rebuilt, with 75-ft (22.5-m) main span supported by hammerhead piers. Spillthrough abutments, set well back from the main channel, were protected with sacked concrete in 1963 and have remained stable.

Drainage area, 1,038 mi^2 (2,688 km^2); bankfull discharge, 1,000 ft^3/s (28 m^3/s); width where bordered by natural vegetation, 80 ft (24 m). Stream is perennial, alluvial, sand bed, in valley of moderate relief, wide flood plain. Natural channel had a sinuousity of about 2.5, but channel has been straightened. Natural channel is equiwidth, not incised, cut banks rare, silt-sand banks. Channel was first straightened and enlarged in the 1920's by local drainage districts; but, probably because the natural flood plain forest was not cleared, the banks remained stable. In 1969, the Corps of Engineers straightened and enlarged a reach about 3 mi (4.8 km) in length downstream from the bridge, reducing the length about 20 per cent. During the past few decades, and particularly in recent years, the flood plain has been cleared of trees for agricultural purposes.

Hydraulic problem and countermeasure:

1963 Bridge built, across previously straightened reach of channel, sacked concrete at abutments.

1969 Channel enlarged and straightened downstream from bridge, by U.S. Army Corps of Engineers.

1970-71 Left bank receded an average distance of 14 ft (4 m) during this period. Peak discharge during period, 7,590 ft^3/d (215 m^3/s), R.I. of flood, 1.5 yr. Timber-pile retard built at left bank near bent 7; single row of pile with wood face planks, extending from the downstream end of bent 7 for a distance of 125 ft (37.5 m) upstream.

1971-73 Peak discharge during period 26,540 ft^3/s (751 m^3/s), 17-yr R.I. Bankfull stage occurred several times, high flows sustained for periods of weeks. Left bank continued to erode behind retard, average distance of recession, about 7 ft (2 m). Bent 7 became exposed below ground line, concrete was poured at base to prevent further erosion. Slumping from the left bank deflected flow toward the right bank, causing rapid erosion and failure of bent 8. South lane of bridge closed.

1975 Both lanes of bridge rebuilt, with new piers having deeper footings and less area normal to flow (fig. 256). Single-row, timber pile retard built along both banks in vicinity of bridge (fig. 257). A large scour hole in the center of the channel downstream from the bridge, attributed to flow constriction during bridge construction, was filled with gravel.

SITE 10. RABBIT CREEK AT SR-45 AT MURPHY, IDAHO

Description of site: Lat 43°13', long 116°33', location as shown in fig. 52. Bridge is 120 ft (36.6 m) in length, precast-concrete stringers, supported on concrete piers with spread footings on bedrock. Spillthrough abutments are supported on timber piles.

Contributing drainage area, 25 mi^2 (65 km^2); valley slope, 0.015. Stream is ephemeral, alluvial, sand bed, in valley of moderate relief, narrow flood plain. Channel is sinuous, locally braided.

Hydraulic problems and countermeasures:

1956 Bridge built with three spans of 40 ft (12.2 m). Bridge skew is 10°; pier skew is 5° (fig. 53). Channel bed was cleaned and rock riprap was placed at both abutments. A cutoff trench for rock riprap, with base about 2 ft below the streambed, was placed at the toe of abutment (fig. 54).

1962 Flood of June 19 (discharge 3,640 ft^3/s or 103 m^3/s) caused general scour in vicinity of bridge (fig. 54). To prevent additional scour near the bridge, a check dam of 18- to 24-inch (0.5- to 0.6-m) size rock riprap was placed across the channel (fig. 55). The crest of the dam was placed at the desired streambed elevation.

1976 Flood of August 6 (discharge less than 3,600 ft^3/s or 102 m^3/s) caused no general scour of the channel bed (fig. 54). The check dam built in 1962 is barely visible in the channel bed and its location is identified by the rock riprap placed on the channel banks. Much of the rock riprap placed at the abutments during bridge construction has disappeared. There is evidence the channel downstream from the dam is controlling the streambed level causing submergence of the check dam.

A new problem caused by lateral erosion is occurring at the upstream side of pier 1 (fig. 56). Lateral erosion of the channel bank at this location is chiefly caused by curvature in channel alinement at the bridge approach and presence of the pier at the toe of the abutment slope, which affects the flow pattern. Rock riprap placed on the left-bank abutment in 1956 is no longer present in sufficient amounts to protect the left channel bank and abutment.

Discussion: The check dam has stabilized the channel bed elevation in the vicinity of the bridge. Lateral erosion is now causing problems of scour at the outside of the channel bend, and additional protection of the left abutment and channel bank is needed. Bridge piers located at the toe of spillthrough abutments evidently affect flow patterns and are especially prone to local scour problems.

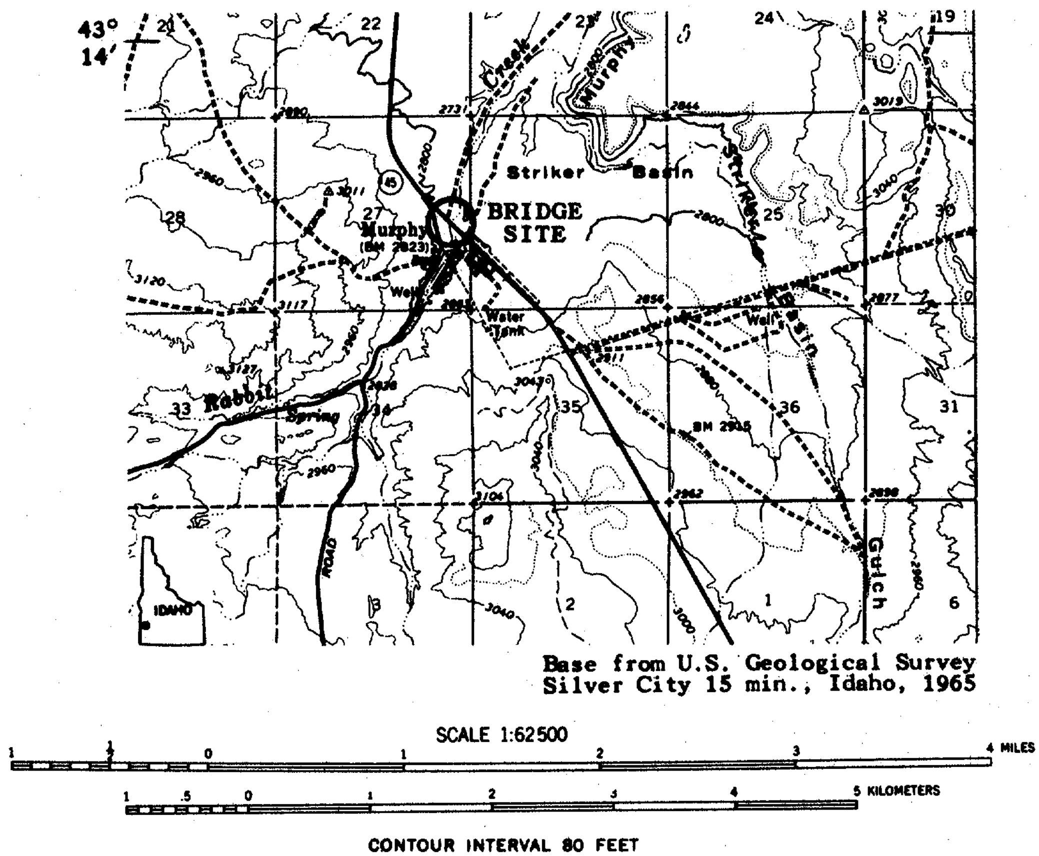


Figure 52. Location of Rabbit Creek at SR-45 at Murphy, Idaho.

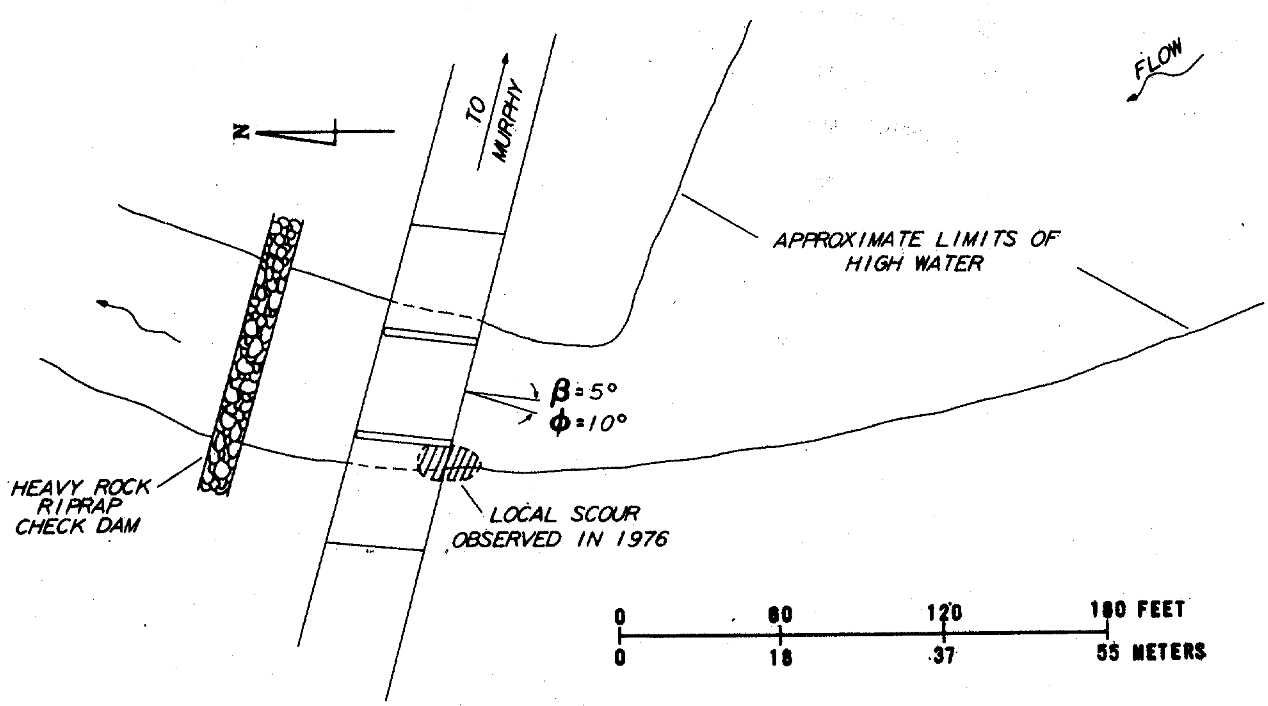


Figure 53. Plan of SR-45 crossing at Murphy, Idaho.

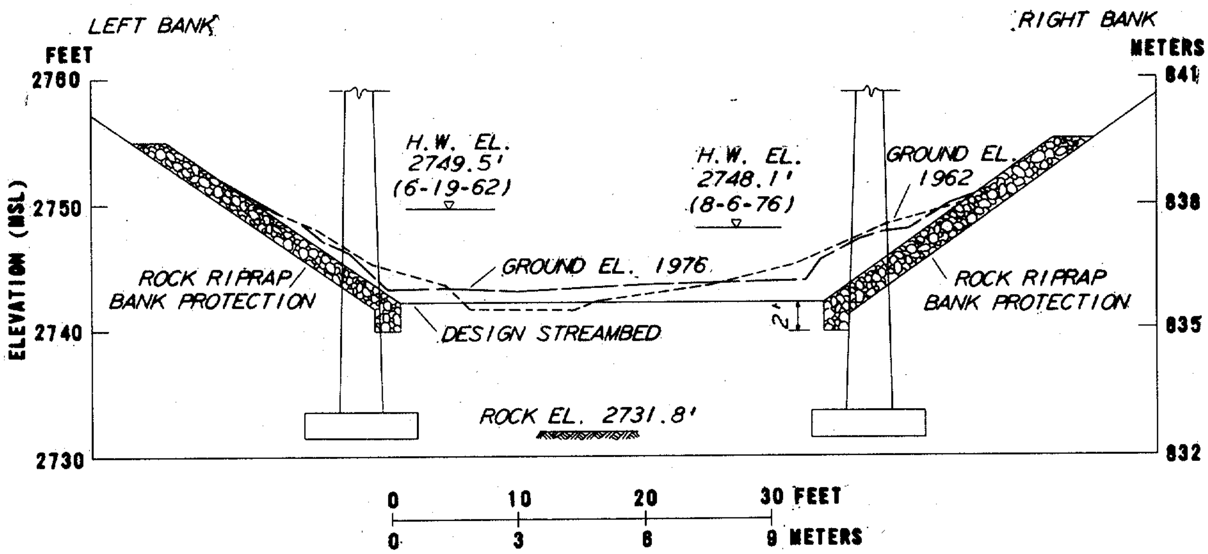


Figure 54. Changes in cross section of channel at downstream side of bridge between 1956 and 1976, Rabbit Creek.

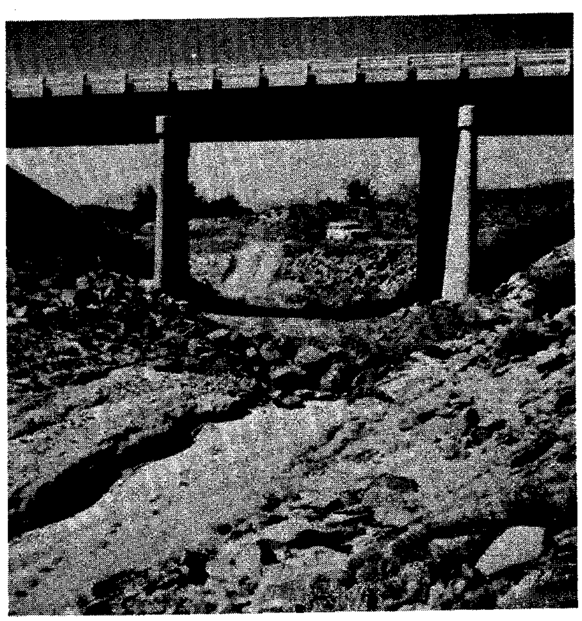


Figure 55. Placement of rock check dam downstream of bridge after flood in 1962.

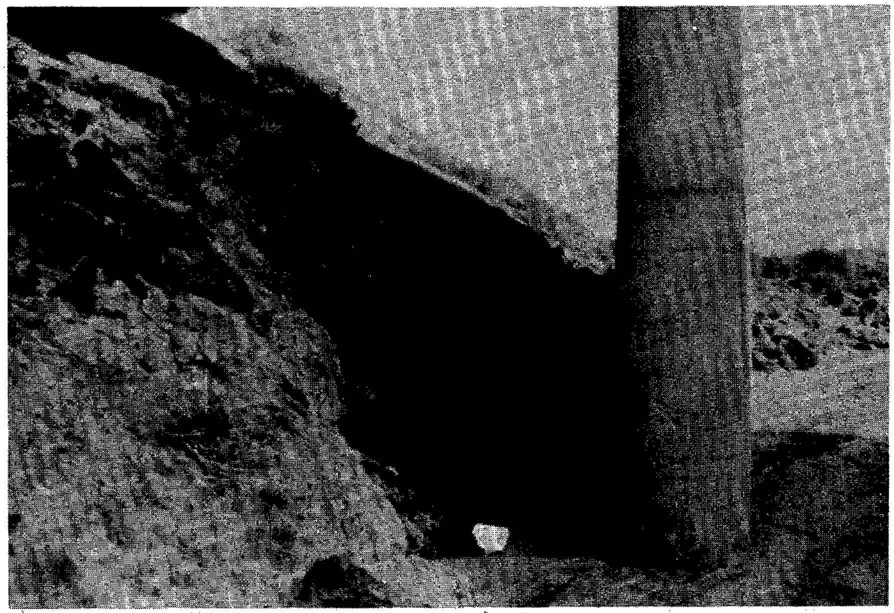


Figure 56. Local scour at upstream side of left bank pier, as observed in 1976.

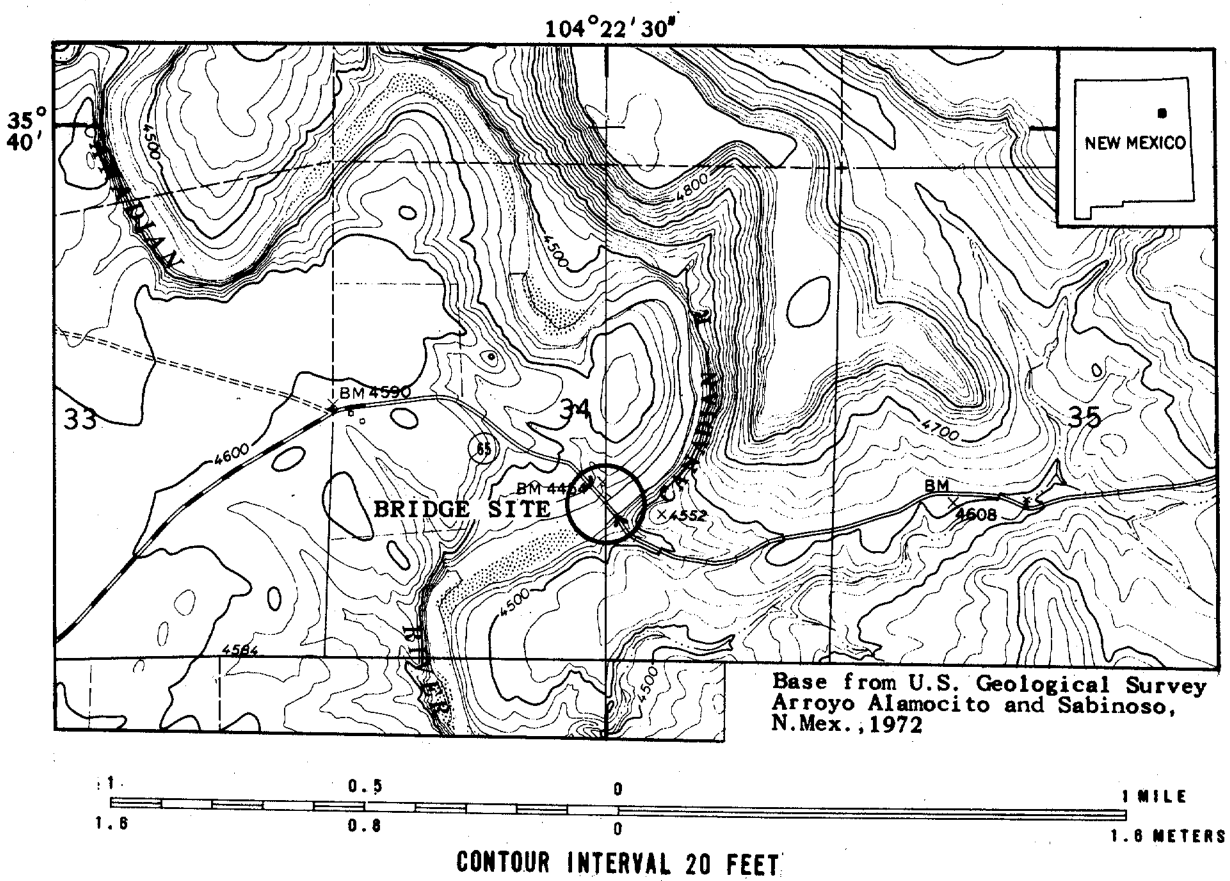


Figure 57. Location of Canadian River at SR-65 near Sanchez, New Mexico.

SITE 11. CANADIAN RIVER AT SR-65 NEAR SANCHEZ, N. MEX.

Description of site: Lat 35°39', long 104°22', location as shown in fig. 57. Bridge is 336 ft (102 m) long, steel, through-truss structure, supported on two masonry piers, vertical abutments with 45° wingwalls. After collapse of the bridge during the 1965 flood, a new concrete-and-steel bridge, 336 ft (102 m) long and supported on steel H-piles, was constructed at the same site.

Drainage area, 5,710 mi^2 (14,789 km^2); bankfull discharge, 15,000 ft^3/s (425 m^3/s); valley slope, 0.0026; width, 200 ft (61 m). Stream is perennial but flashy, non-alluvial, gravel-sand bed, in valley of moderate relief, no flood plain. Channel is meandering, meanders incised into bedrock, locally braided.

Hydraulic problems and countermeasures:

1928 Steel-truss bridge built, concrete masonry piers (figs. 58 and 59). Pier base was keyed at least 6 in (0.15 m) deep into bedrock after removal of 2 ft (0.61 m) of sand and 8 to 11 ft (2.4 to 3.4 m) of compact gravel. Abutments are vertical and founded on bedrock.

1965 Flood of June 15, 1965, Q = 126,000 ft^3/s (3,567 m^3/s), caused the left pier to topple and the two left spans of the structure were destroyed. The June 18 flood, Q = 145,000 ft^3/s (4,105 m^3/s), caused the remaining pier to topple, destroying the right-bank span. After collapse, both piers moved only a few feet downstream (fig. 60).

1966 New bridge built, supported by four steel-pile bents (fig. 61). To prevent overturning of bridge during future floods, the six steel piles at each bent were set in holes drilled 6 ft (1.8 m) deep in bedrock (fig. 62). Because existing bridge abutments were used for the new bridge, clearance of low bridge steel over the waterway was not increased. The bridge railings were therefore designed to minimize resistance to floodflow (fig. 63).

1976 No floods have occurred since 1965 to test the countermeasures.

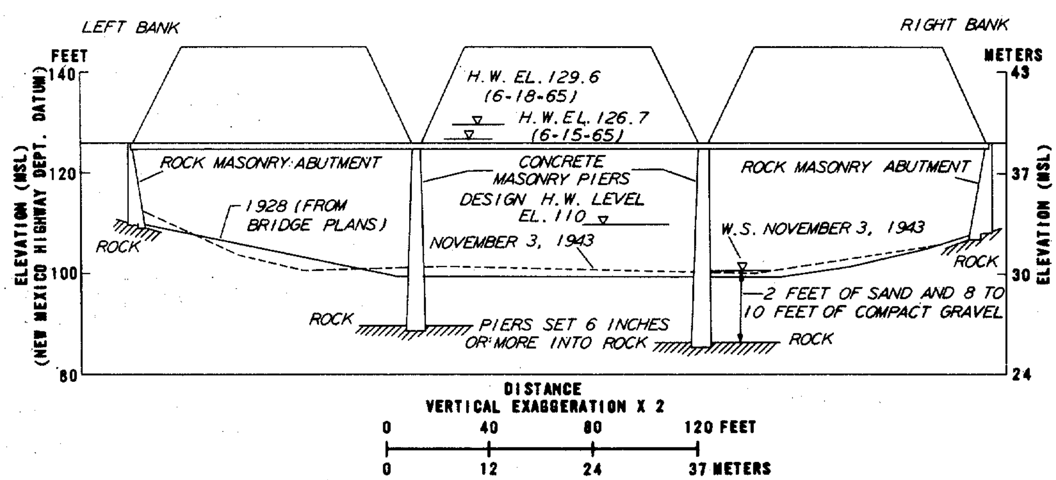


Figure 58. Cross section of Canadian River at SR-65 near Sanchez, New Mexico in 1928 and 1943.

Discussion: Bridge failure was probably due to lateral forces on the partially submerged bridge and to undermining of the piers during the June 1965 flood. A new bridge built after the flood provides the same waterway capacity but has, as countermeasures, steel piling in drilled holes in bedrock to resist lateral forces, and a bridge railing designed to minimize resistance to flow.

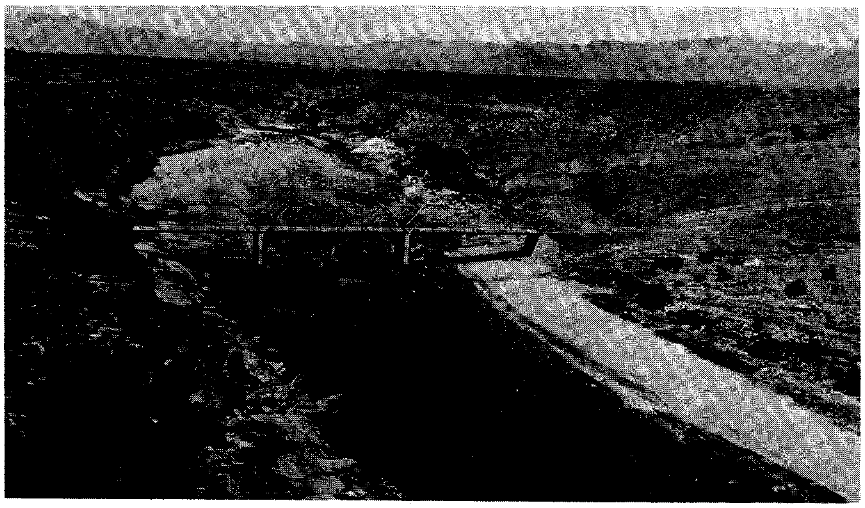


Figure 59. View downstream at bridge site prior to June, 1965. (From New Mexico Dept. of Highways.)

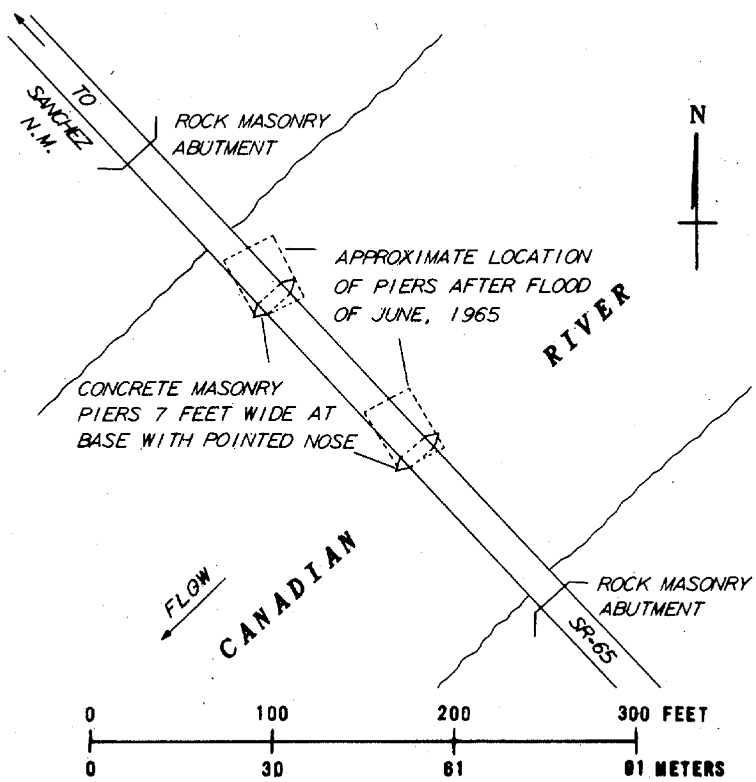


Figure 60. Canadian River at SR-65 near Sanchez, New Mexico, prior to 1965.

Figure 61. View looking upstream toward bridge from center of channel, 1976.

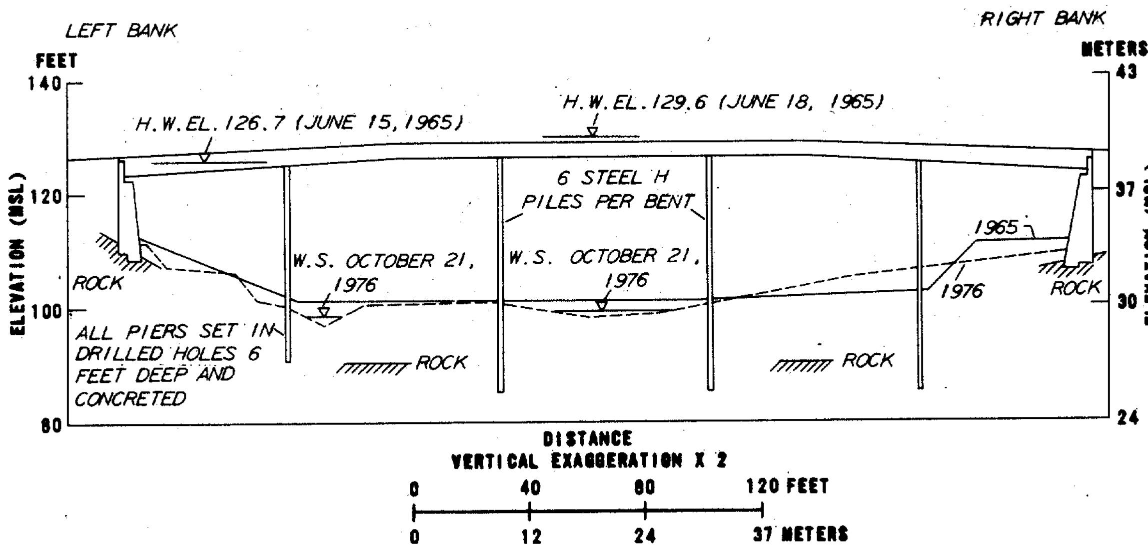


Figure 62. Cross section of Canadian River at SR-65 near Sanchez, New Mexico, in 1965 and 1976.

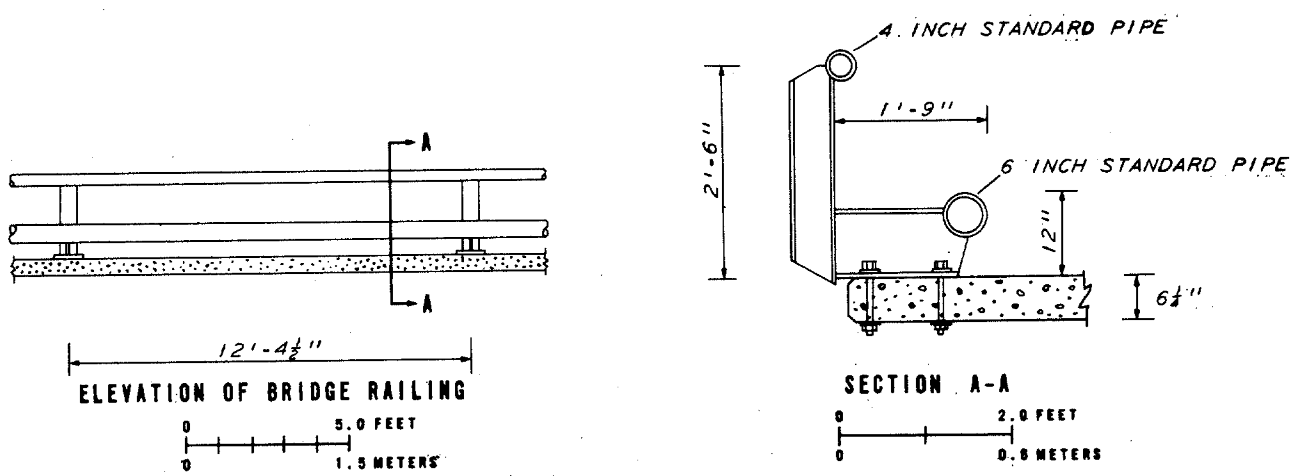


Figure 63. Details of bridge railing designed to minimize resistance to flood flow.

SITE 12. WALNUT CREEK AT I-25 NEAR SAN ANTONIO, N. MEX.

Description of site: Lat 33°55', long 107°53', location as shown in fig. 64. Dual bridges, each bridge consisting of two 41-ft (12.5-m) side spans and one 50-ft (15.2-m) midspan constructed of steel and concrete; spillthrough abutments. The bridges are supported by steel H-piles driven to a depth of about 40 ft (12 m) below the streambed.

Drainage area, 32 mi^2 (83 km^2); valley slope, 0.0163; channel width, about 500 ft (152 m). Stream is ephemeral, alluvial, gravel bed, on alluvial fan. Channel is straight, generally braided, sparse desert shrubs along bankline (fig. 65).

Hydraulic problems and countermeasures:

1964 Bridge built with abutment slopes protected by wire-enclosed rock riprap (figs. 66 and 67) under both lanes of highway, through the open median, and extending upstream in order to channelize a small left-bank tributary.

1972 Flood discharge about 16,000 ft^3/s (453 m^3/s), R. I. greater than 50 yr, occurred on August 30, 1972. This discharge is about 2.5 times the bridge design discharge. Damage to the site included scour at the base of the riprap upstream and between the bridges, causing sagging of the riprap; loss of about 75 ft (23 m) of riprap at the left bank downstream bridge abutment; and about 2 ft (0.6 m) of local scour at the pilings. The wire-enclosed riprap was not completely effective in preventing erosion of the abutment but it probably prevented damage to the bridge.

1972 Repairs following the flood consisted of replacing the eroded abutment fill and streambanks, and the destroyed wire-enclosed rock riprap (fig. 68).

1976 No significant floods have occurred since 1972. The repaired countermeasures appear in good condition.

Discussion: Lateral erosion occurred at the downstream abutments on both banks during the flood because of the bridge contraction, and is attributed to eddy action as the flow expanded to occupy the normal channel. Although not entirely effective, the wire-enclosed rock riprap probably prevented more extensive damage to the bridge.

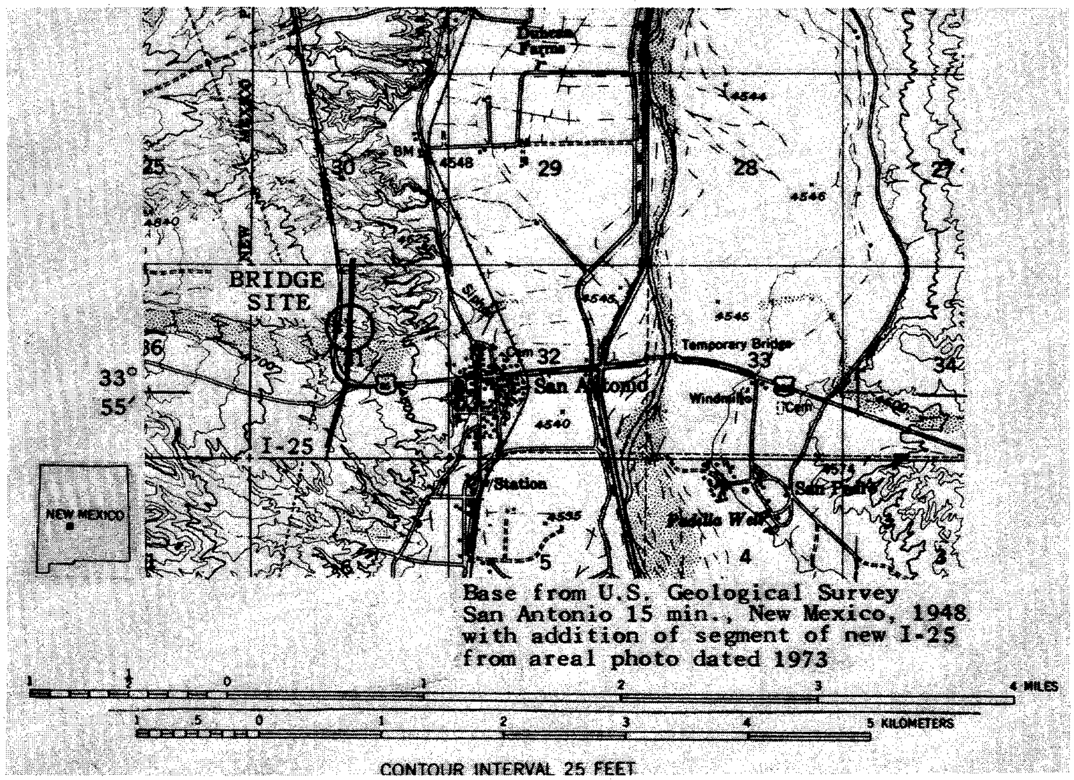


Figure 64. Location of Walnut Creek at I-25 near San Antonio, New Mexico.

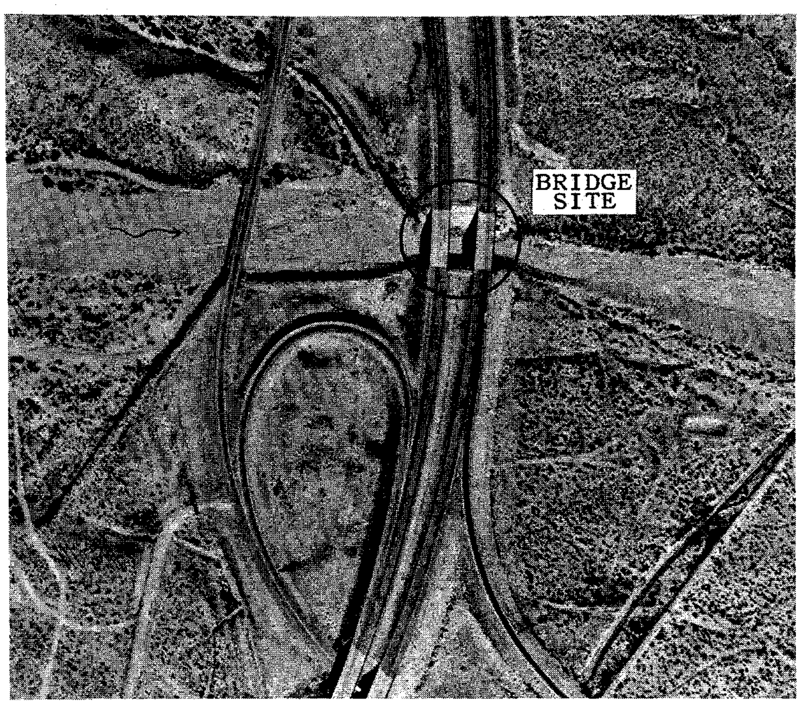


Figure 65. Aerial photograph taken in 1973 of Walnut Creek at I-25 near San Antonio, New Mexico. (From New Mexico Dept. of Highways.)

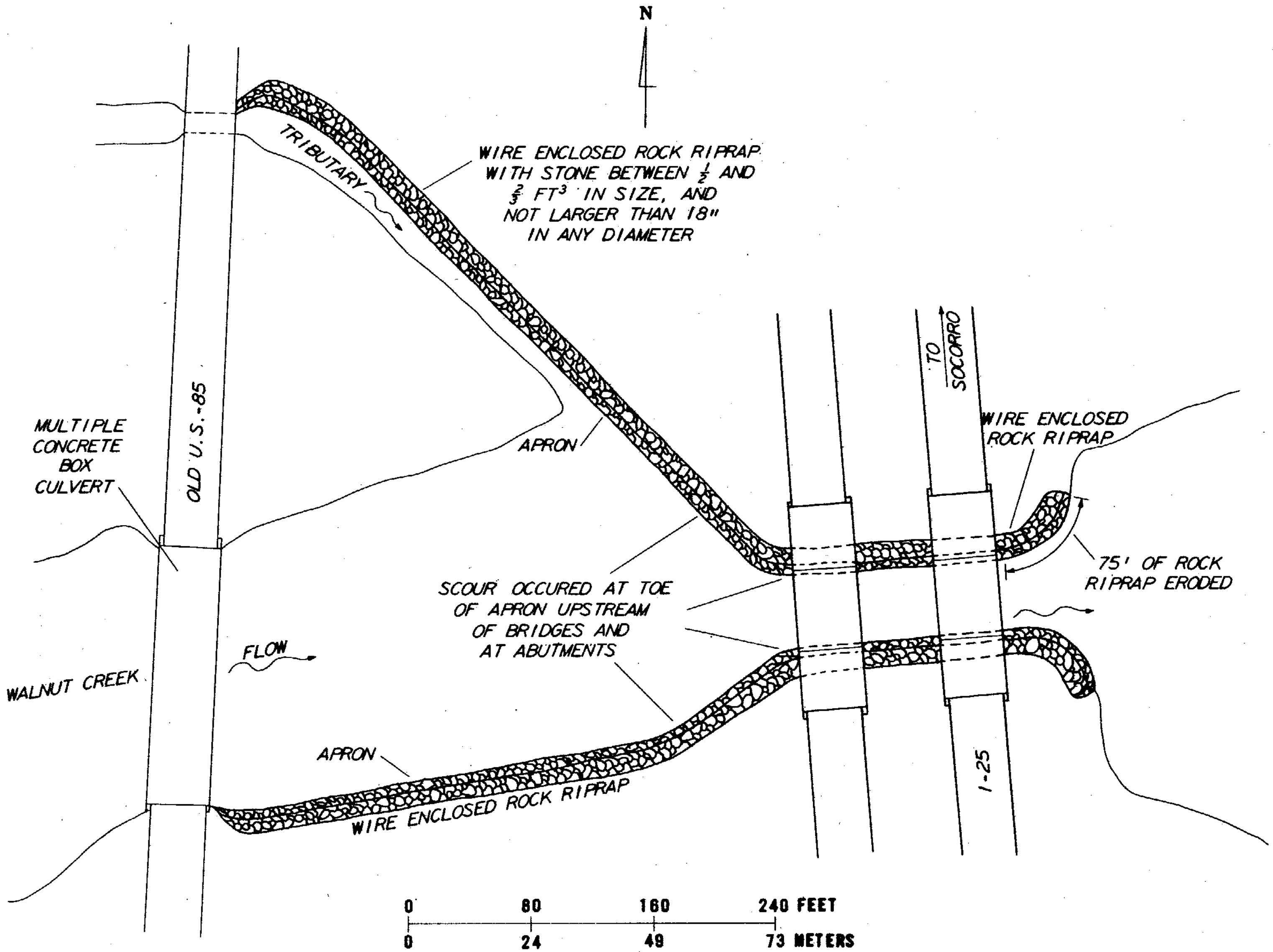


Figure 66. Plan of Walnut Creek at I-25 near San Antonio.

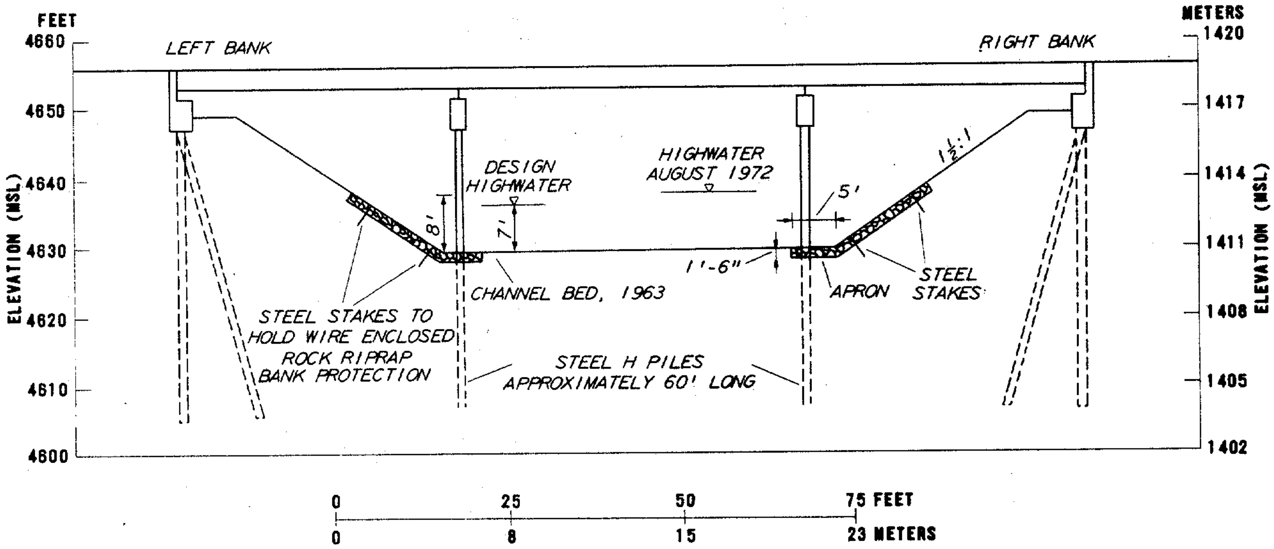


Figure 67. Cross section of upstream bridge across Walnut Creek at I-25 near San Antonio.

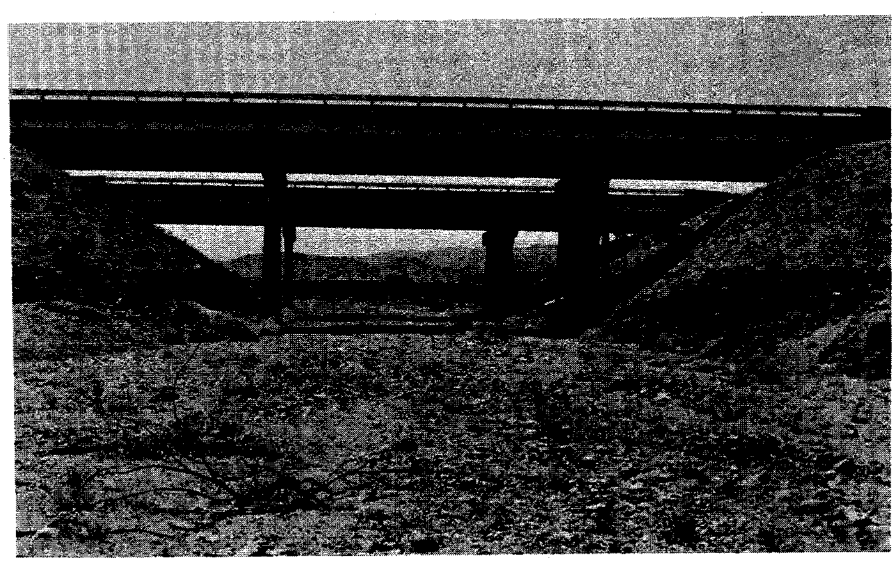


Figure 68. View upstream in 1976 of wire-enclosed riprap as repaired after the 1972 flood.

SITE 13. HUTTON CREEK AT I-5 NEAR HILT, CALIF.

Description of site: Lat 41°57', long 122°35', location as shown in fig. 69. Continuous concrete box-girder, 128 ft (39 m) long, on rectangular piers with pointed nose. Abutments are spillthrough, piers and abutments on spread footings. Channel slope is 0.030.

Drainage area, 11 mi^2 (28 km^2); valley slope, 0.030; channel width, about 20 ft (6 m). Stream is ephemeral, alluvial, cobble bed, in valley of high relief, no flood plain. Channel is sinuous, gravel banks.

Hydraulic problems and countermeasures:

1973-74 Bridge built over relocated channel, of which the bed and banks were protected with rock riprap (fig. 70). Along the outside of the channel bend, the rock riprap was further strengthened by use of concrete grout (fig. 71).

1974 Flood of March 1974, discharge 600 ft^3/s (17 m^3/s), R. I. 20 yr, did not cause damage to bridge or channel works.

1976 All countermeasures at site are in good condition, and performing satisfactorily.

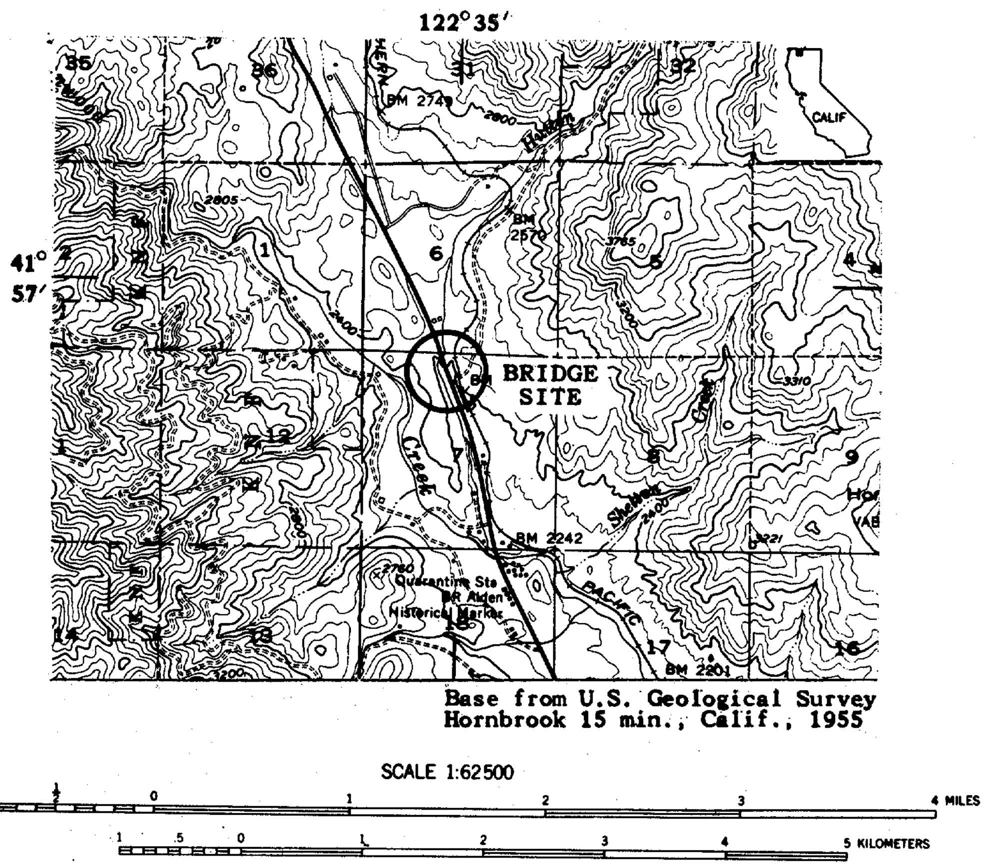


Figure 69. Location of Hutton Creek at I-5 near Hilt, California.

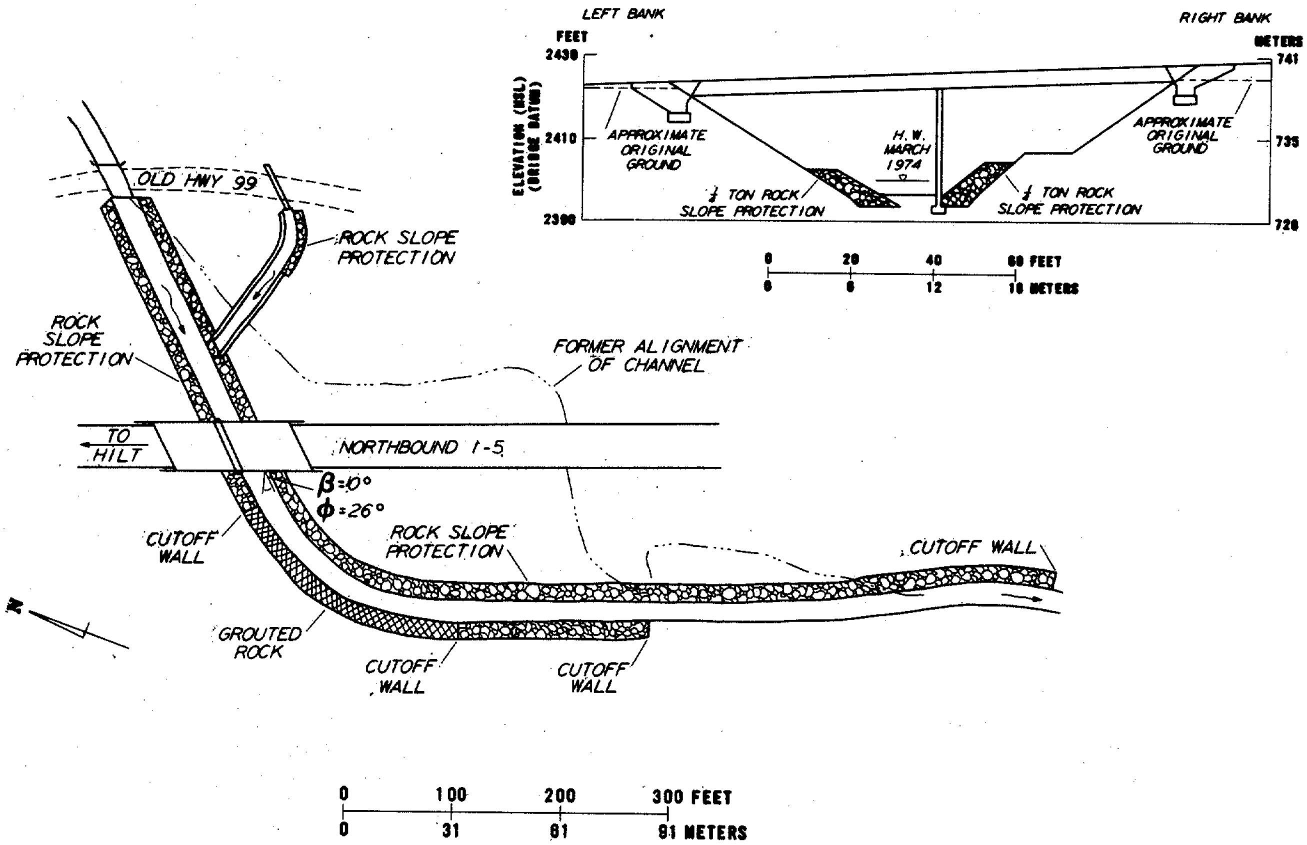


Figure 70. Plan of Hutton Creek at I-5, and (right) cross section of channel at downstream side of bridge.

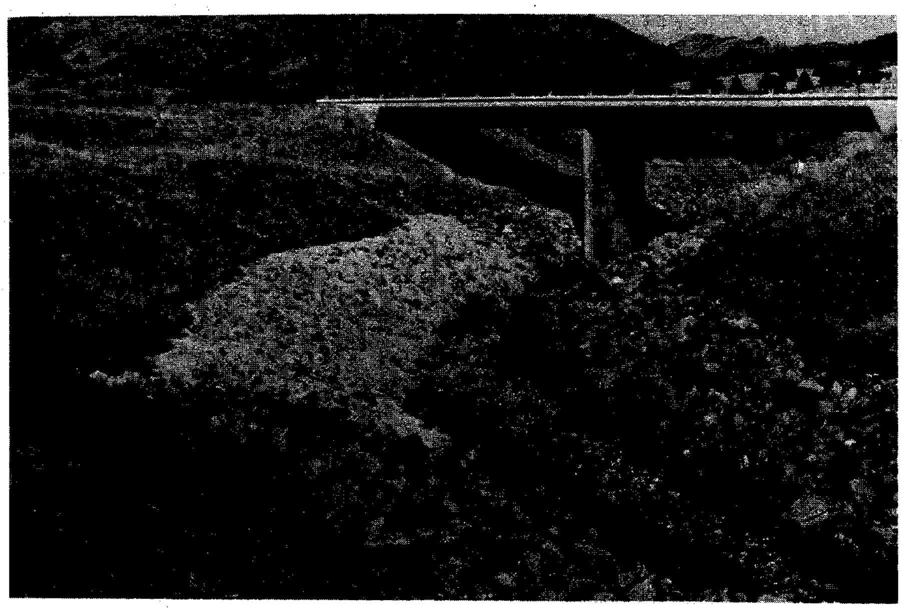


Figure 71. View upstream of channel bed and bank protection in 1976. Note grouted riprap on slope in foreground.

SITE 14. LITTLE TRUCKEE RIVER AT SR-89 NEAR HOBART MILLS, CALIF.

Description of site: Lat 39°29', long 120°14', location as shown in fig. 72. Concrete 4-girder spans, on piers consisting of a pair of rectangular webbed columns founded on spread footings. Clearance of bridge above flood stage is limited, and lodging of ice and debris at bridge is a problem. Abutments are vertical, with short wingwalls. Channel slope is 0.0107.

Valley slope, 0.012; channel width at bridge, about 75 ft (23 m). Stream is perennial, alluvial, cobble-boulder bed, in valley of moderate relief, narrow flood plain. Channel is straight, not braided at bridge but generally braided downstream, cobble-boulder banks.

Hydraulic problems and countermeasures:

1937 Bridge built (fig. 73), length 100 ft (30 m), two 30-ft (9-m) spans and one 40-ft (12-m) spans. Bridge is skewed 45° to the channel. Size of bridge and clearance above natural channel (fig. 74) was barely sufficient for floodflow, so the channel was changed in alinement and increased in size at time of construction (fig. 75).

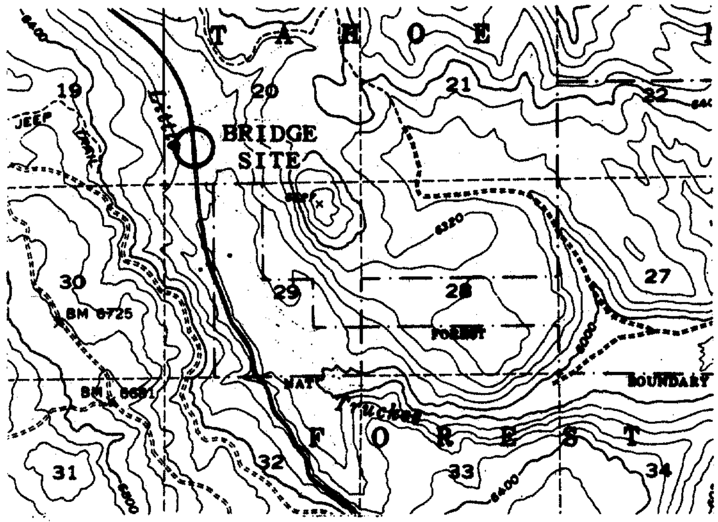
1937 Flood of December 1937 overtopped flood plain on the right bank upstream from the bridge. A part of the right-bank overflow broke through the approach embankment next to the abutment, and the approach embankment was destroyed for nearly 0.5 mi (0.8 km).

1938 Repairs to the embankment and abutment consisted of building a rubble masonry wall to face the end of approach fills near the abutments. Also, a dike (levee) armored with heavy rock riprap, was constructed along the right bank of the channel upstream from the bridge (figs. 73 and 76).

1963-65 Floods of unknown magnitude, but equal or larger in magnitude than the December 1937 flood, did not cause hydraulic problems at the bridge.

1972 In December, ice piled up at bridge (fig. 77) as a result of inadequate clearance above waterway. Ice was removed by maintenance crews using a crane.

Discussion: Construction of the armored dike to prevent overflow on the right-bank flood plain has effectively restricted flows to the main channel and prevented damage to the approach embankment. Accumulation of ice is attributed to inadequate clearance.



Base from U.S. Geological Survey
Truckee 15 min., Calif., 1955

SCALE 1:62500

Figure 72. Location map of Little Truckee River at SR-89 near Hobart Mills, California.

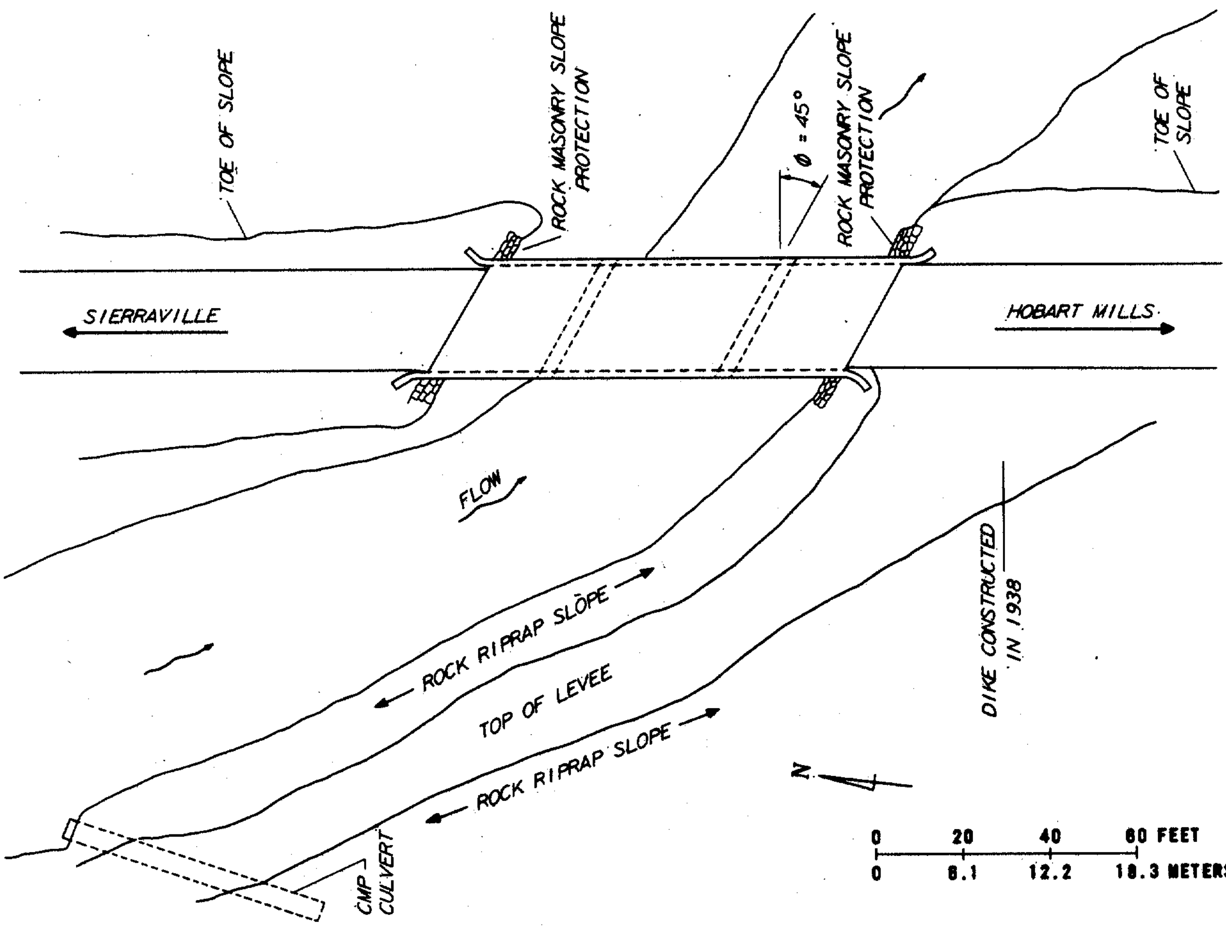


Figure 73. Plan, in 1976, of Little Truckee River crossing.

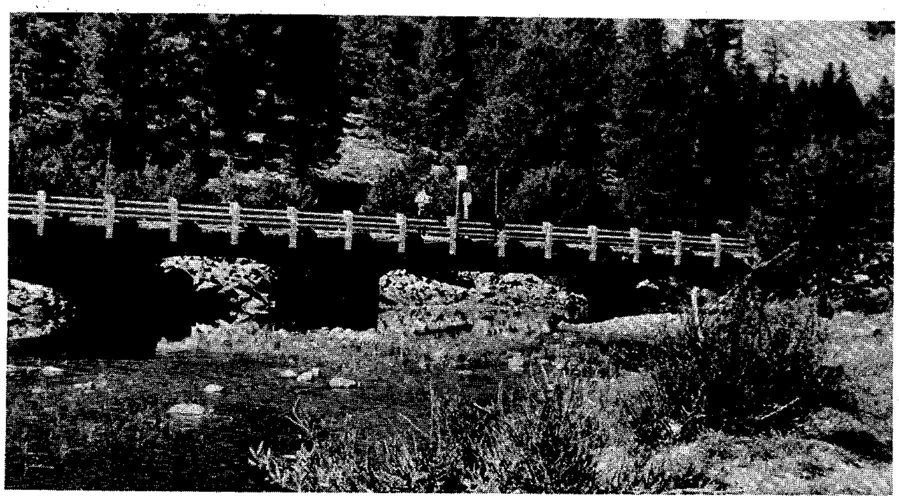


Figure 74. View upstream, in 1976, showing clearance of bridge at time of low flow.

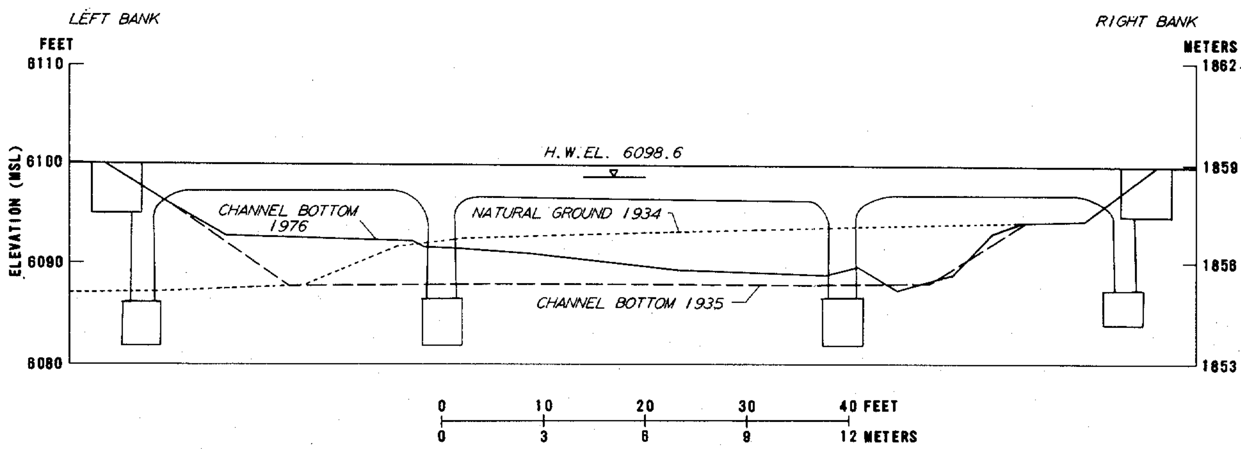


Figure 75. Channel cross sections on downstream side of Little Truckee River bridge, in 1934, 1935, and 1976.

Figure 76. Heavy rock riprap on upstream right-bank dike, in 1976.

Figure 77. Ice accumulation at upstream side of bridge on December 20, 1972. (From Calif. Dept. of Transportation.)

SITE 15. COON CREEK AT SR-65 NEAR SHERIDAN, CALIF.

Description of site: Lat 38°56', long 121°21'. Reinforced-concrete beam bridge supported by reinforced-concrete pile bents. Total bridge length is 192 ft (58 m). Bridge and pile bents are skewed 13° to flow. Abutments are spillthrough.

Valley slope, 0.00116; channel width, about 200 ft (61 m). Stream is perennial, alluvial, silt-sand bed, in valley of low relief, wide flood plain. Channel is sinuous, locally anabranched.

Hydraulic problems and countermeasures:

1974 New bridge constructed (figs. 78 and 79). To improve channel alinement with respect to the new bridge, the approach channel was relocated and broken concrete was placed along the right bank to prevent bank erosion (fig. 80). Details of the bank protection countermeasures are shown in figure 78.

1977 No floods have occurred since construction of the bridge to test the countermeasure.

Discussion: Broken concrete riprap (fig. 80) is more resistant to movement caused by flowing water if the thickness to length ratio of individual pieces is 1:3 or less (California Division of Highways, 1970). A survey of the bridge site in 1977 indicates that the channel changes made at the time of bridge construction have not caused scour or erosion problems in the channel near the bridge. The stability of the present channel is probably due to lack of floods since construction on the bridge, and the small change in the original channel gradient as a result of the channel relocation.

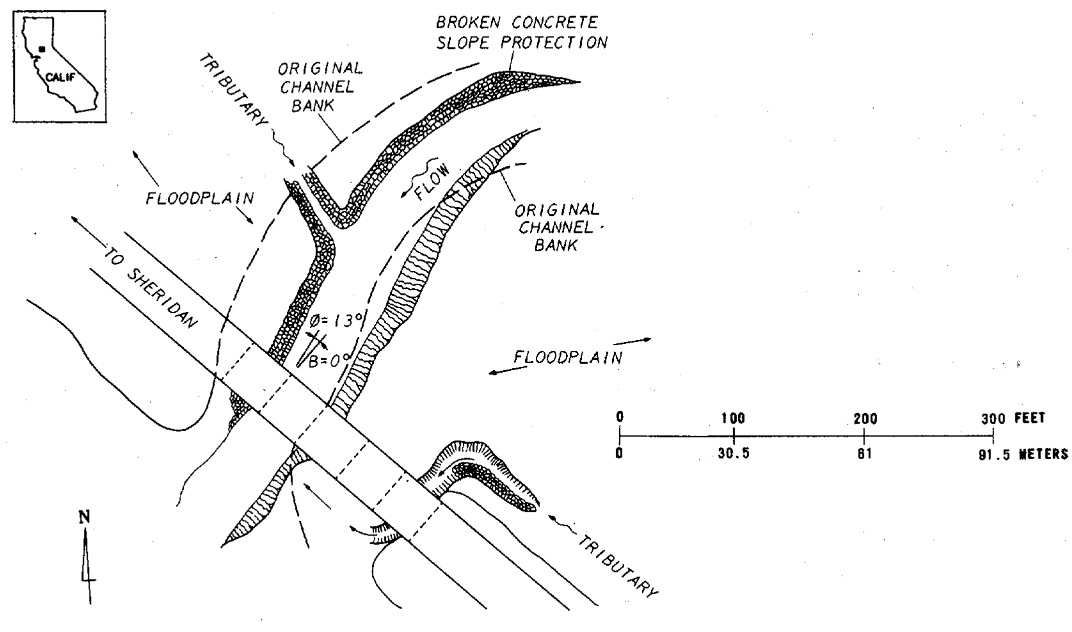


Figure 78. Plan of Coon Creek at SR-65 crossing.

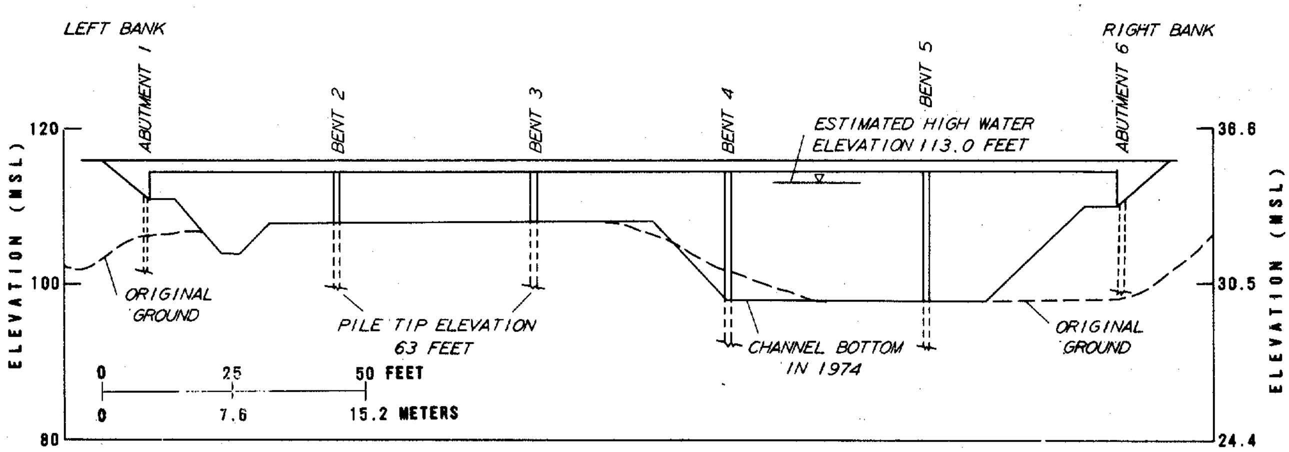


Figure 79. Cross section of Coon Creek channel, 1974.

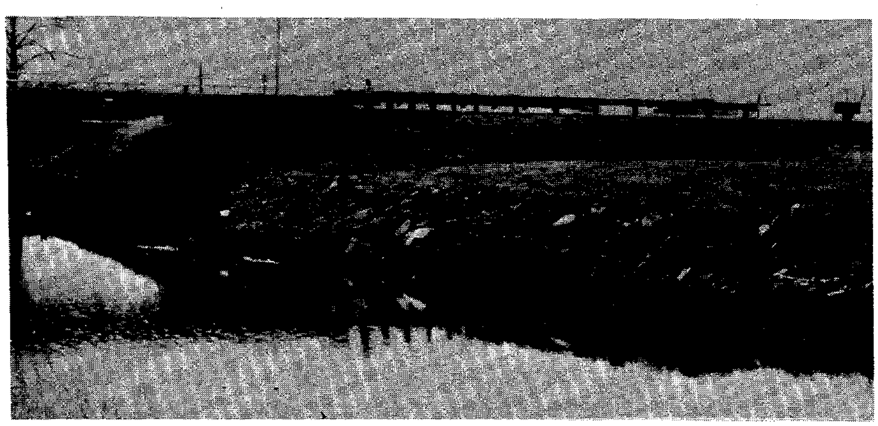


Figure 80. View in 1977 of broken concrete used to protect right bank of channel.

SITE 16. DEER CREEK AT SR-99 NEAR VINA, CALIF.

Description of site: Lat 39°57', long 122°03', location as shown in fig. 81. Reinforced-concrete, open-spandrel arch on concrete wall-type piers with rounded nose; concrete closed-end (vertical type) abutments. Bridge is 459 ft (140 m) long with 7 spans and no skew (fig. 83). Bridge was built in 1921 and widened in 1951. All piers and abutments founded on piles except pier 7 and abutment 8 (fig. 83). Channel bed consists of sand and gravel and is very susceptible to movement. Floodflows are unregulated.

Drainage area, 222 mi^2 (575 km^2); valley slope, 0.004; channel width, about 250 ft (76 m). Stream is perennial, alluvial, gravel bed, on an alluvial fan, natural levees. Channel is sinuous, locally braided, wider at bends, point bars, cut banks general.

Hydraulic problems and countermeasures:

1921 Bridge built (fig. 84).

1937 High water of December 11, 1937, (table 1) washed out 50 ft (15 m) of approach embankment fill at left-bank abutment. Following the flood, the fill was replaced, but was not protected from damage by future floods.

1946 Local scour was noted at pier 2 (fig. 83). The hydraulic problem apparently was caused by moderate flows in 1940, 1941, and 1942 (table 1). Depth of scour was 3 ft (0.9 m) below top of concrete footing at pier 2. Abrasion of the concrete footing was noted, and debris was caught on the original falsework pile stubs.

Table 1. Stage, discharge, and frequency of selected floods on Deer Creek at US-99 bridge

Date of flood	Stage [1](feet, MSL)	Discharge (ft^3/s)	Approximate recurrence interval (years)
3-26-28	--	12,200	8
12-10-37	218.6	23,800	55
2-28-40	--	21,600	40
2-10-41	--	12,700	9
2- 6-42	--	13,700	11
1- 8-53	--	10,800	7
12-22-55	209.0	12,600	9
2-24-57	--	7,470	3
12-22-64	210.7	18,800	25
1- 6-65	208.7	13,400	10
1-23-70	209.9	20,100	30
3-29-74	--	11,900	8

[1]ft^3/s multiplied by 0.0283 equals m^3/s.

1947 Scour in channel between piers 2 and 3 (fig. 83) and along left-bank levee (fig. 82) continuing. Overflow channel on right bank side of channel overgrown with brush and small trees. The channel was cleaned and leveled at the upstream side of bridge to improve passage of water.

1948 Scour was observed on both sides of pier 2 and the channel bed adjacent to the upstream side of the footing is about 6 feet below top of footing or about 2 ft (0.6 m) below the theoretical bottom of footing. Heavy riprap was placed around the upstream and downstream side of the pier 2 footing for a width of 8 ft (2 m). Riprap was also placed along the left bank of the channel for a distance of 30 ft (9 m) upstream from bridge to prevent scour from starting on the shore side of pier 2.

1953 Lateral streambank erosion of 40 to 60 ft (12 to 18 m) on the right bank of the channel occurred for a considerable distance upstream from the bridge apparently as a result of flooding in 1953 (table 1). A gravel bar has built-up the former main low-water channel near pier 3, and the main low-water channel was shifted so it hits the upstream end of pier 4 (fig. 83). Local scour 1.5 ft (0.5 m) deep has undermined the footings of piers 3 and 4. A large pothole has developed upstream from pier 4 footing about 8 ft (2 m) deep and 20 ft (6 m) wide. The bottom of the scour hole is about the same as the tip elevation of the piles. Repairs consisted of the following: (1) Placement of heavy rock backfill in pothole upstream from pier 4 up to the elevation of the bottom of the footing. (2) Diversion of the flow so it passes between piers 1 and 2. (3) Backfilled the cavities under portions of footings at piers 3 and 4 with gravel and cobble. (4) Excavated a trench 6 ft (2 m) in width and depth around pier 3, and along upstream side of pier 4. The trench was then backfilled with heavy rock riprap so that the top of the rock was about level with the top of footings. (5) The channel was graded so floodflows are distributed uniformly across the channel.

1956 Flood of December 1955 (table 1) caused scour below the footing base up to 6 ft (2 m) deep at the downstream end and along both sides of pier 2. Repairs consisted of backfilling the mined area under the footing with gravel and the remaining scoured area with heavy rock riprap, placed so the top of the rock was level with the top of the footing. Some of the large rock placed in 1953 around pier 3 had washed out, causing some spaces with depths about 4 ft (1 m) below the top of the footing. None of the scour around pier 3 was serious, and performance of the rock riprap is considered satisfactory.

1957 Flooding during the winter (table 1) caused the main channel alinement to begin a shift toward the right bank near pier 5 (fig. 83). A scour hole had developed on the upstream side of pier 5 footing, and serious scour developed on the right bank side and at the downstream end of pier 4. Depth of scour is about 4 ft (1 m) below bottom of the footing. Also, a small amount of rock riprap was lost along side of pier 2. The scour holes near piers 2, 4, and 5 were filled with rock riprap.

1965 High water of December 1964 and January 1965 (table 1) caused the channel to complete its shift from the left to right bank side of the channel (fig. 83). The main channel alinement now flowed at an angle at pier 7, and localized scour holes have developed at piers 5, 6, and 7. Depth of scour at pier 7 is about 5 ft (2 m) below the bottom of the footing, and at piers 5 and 6 the scour is near the bottom of the footing. The areas around pier 5 previously backfilled with rock are not scoured and appear stable. Repairs consisted of (1) placement of ¼ ton (0.23 t) to ½ ton (0.45 t) maximum size rock riprap in the scour areas not previously protected with rock at pier 5, and (2) at piers 6 and 7, rock was placed entirely around the pier to the top of the footing, and extending upstream about 15 ft (5 m) from the footing (fig. 85). During placement of the rock riprap, native sand and gravel was filtered in with the rock to help fill in the void under the footing.

1970 Flood during January 1970 (table 1) caused lateral erosion and undercutting of the channel right bank about 100 ft (30 m) upstream and 30 ft (9 m) downstream from the bridge. Repairs made at different times during the year at the site include: (1) Construction of a dike on the right bank upstream from the bridge to prevent further bank erosion (fig. 82). (2) Shifting the low water channel from the right bank towards the center of the bridge (spans 4 and 5, fig. 83), and lowering the elevation of the channel so it is the same elevation as the top of the old footings (fig. 85). The purpose of the work is to direct the flow away from the right bank and also increase the channel capacity. (3) Placement of gravel from the channel against the right bank and under the bridge so the span between pier 7 and abutment 8 (fig. 83) is blocked. (4) Placing additional rock riprap adjacent to piers 4, 5, and 7 to compensate for settling and movement of rock placed originally, and to protect the piers now exposed as a result of channel reshaping and lowering.

1976 Flood in 1974 (table 1) did not cause hydraulic problems at the bridge. Placement of rock riprap to prevent scour around the piers, and the lowering of the channel apparently has been successful in controlling local scour around the piers.

Discussion: Local scour around the bridge piers occurred during floods of 3- to 5-year recurrence intervals. Some rock riprap of ¼ to ½ ton (0.23 to 0.45 t) size placed in scour holes around the bridge piers were moved during floods of 10-year recurrence intervals. Depth of flow was about 10 ft (3 m). Rock riprap placed in scour holes around the piers prevented additional scour. The top of the rock was placed level with the streambed so the channel was not constricted. In this case, the bridge designers should have considered the possibility of channel alinement changes with time and consequent hazards such as undermining pier footings.

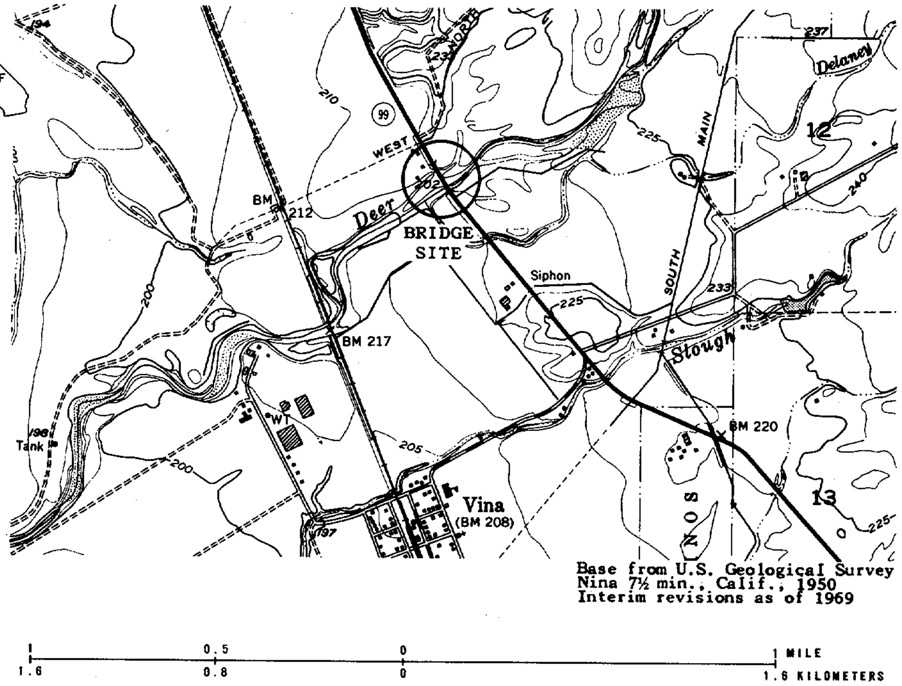


Figure 81. Location map of Deer Creek at SR-99 near Vina, California.

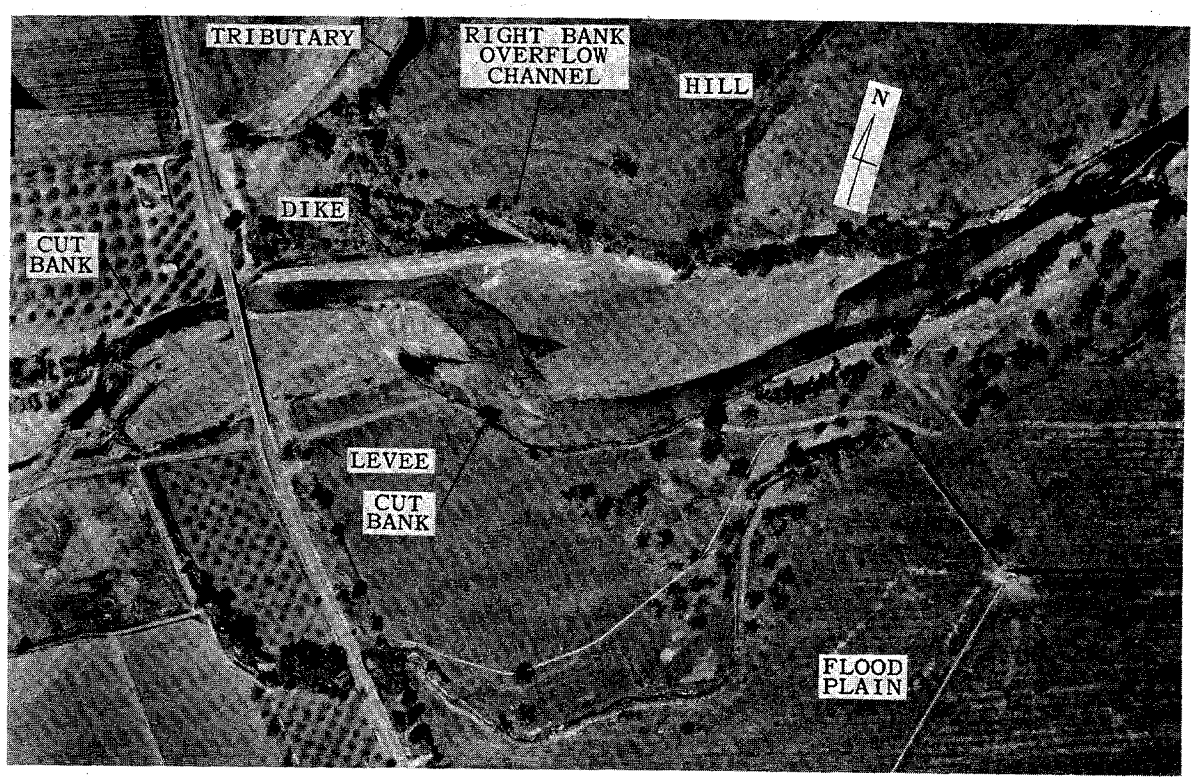


Figure 82. Aerial view of bridge site in 1970. (From Calif. Dept. of Transportation.)

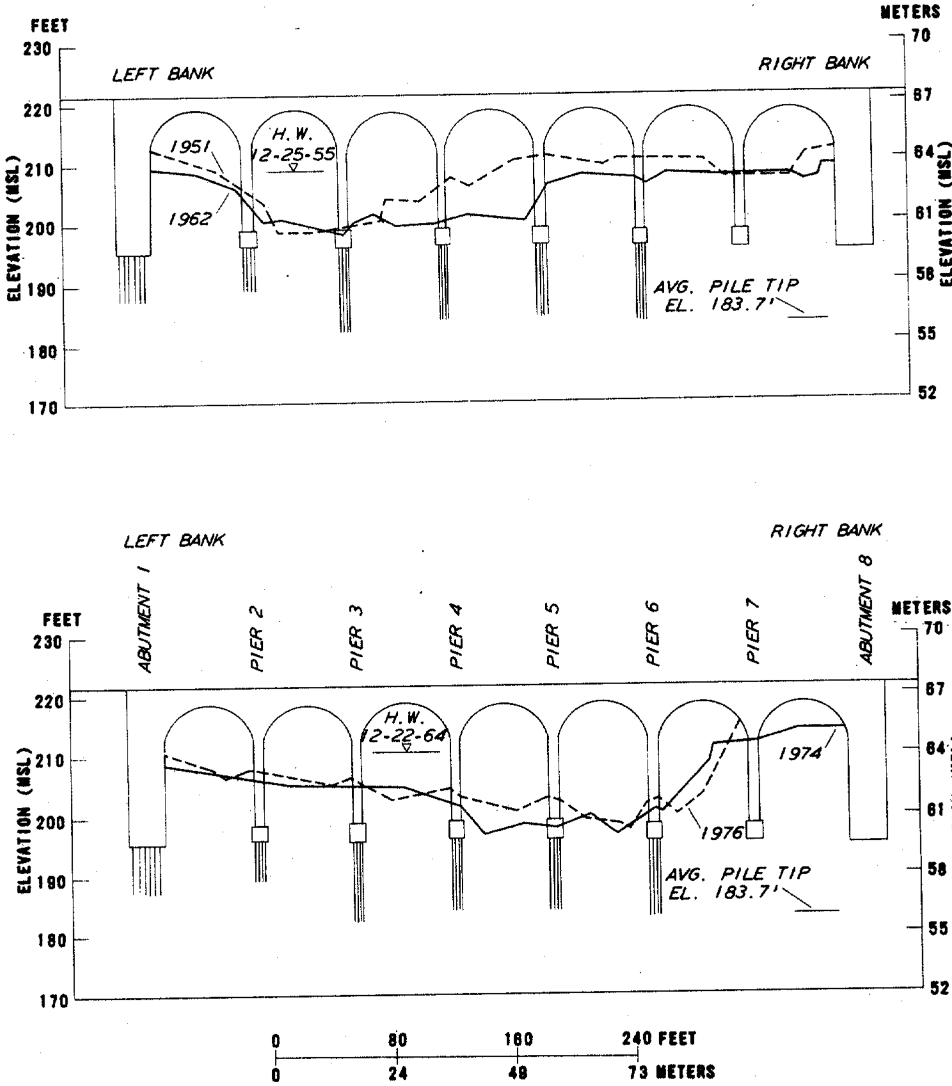


Figure 83. Cross sections of Deer Creek showing bridge built in 1921 and channel changes between 1951 and 1976.

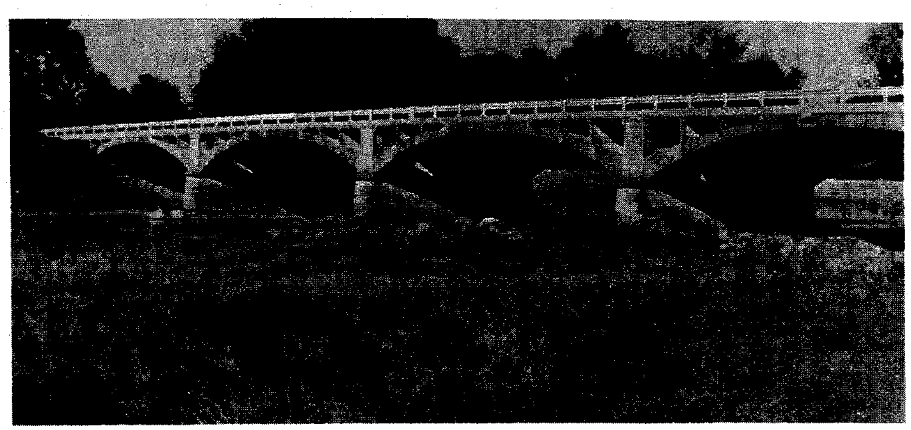


Figure 84. View upstream of Deer Creek bridge from left bank, in 1976.

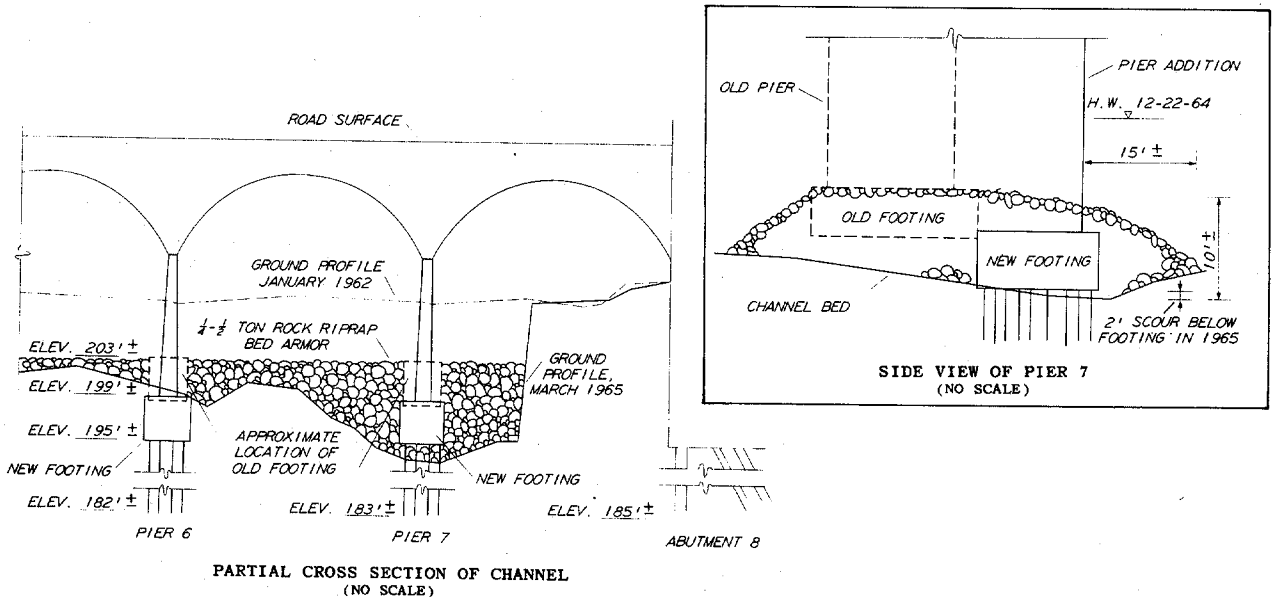


Figure 85. Details of new pier footings and rock riprap bed armor placed in 1965 to prevent pier scour.

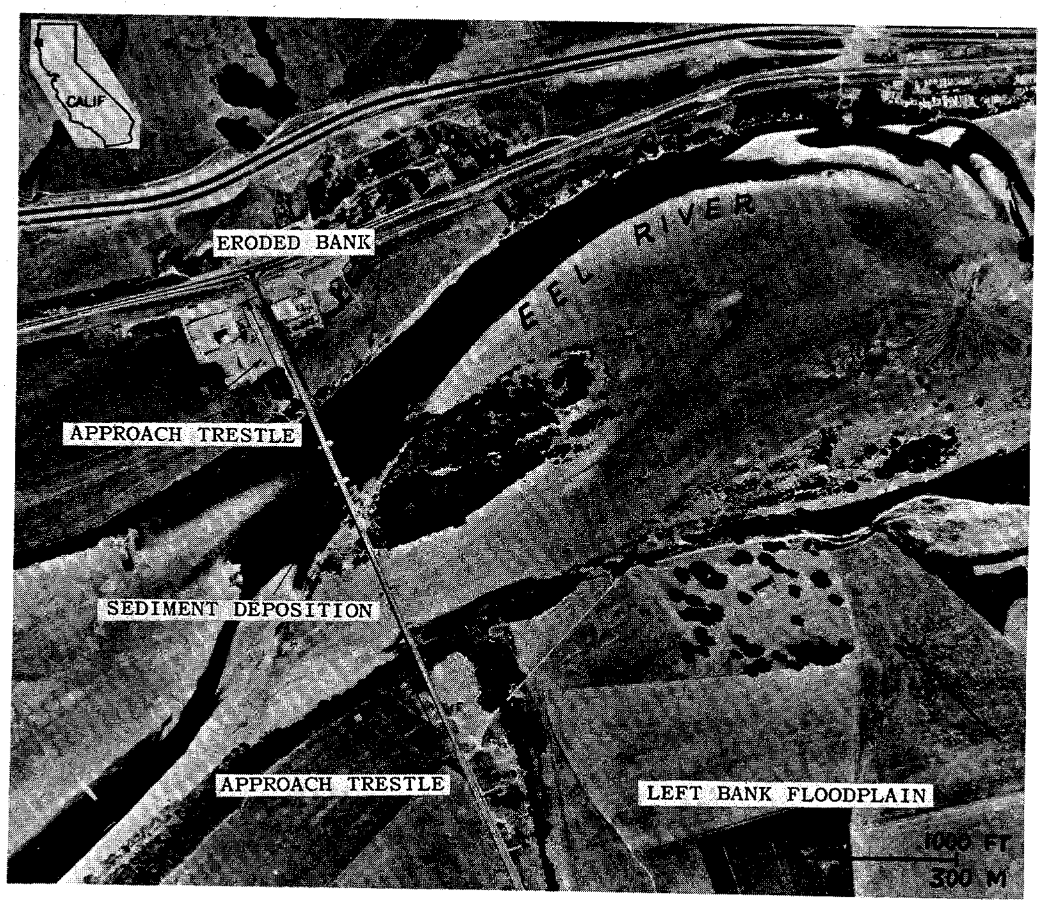


Figure 86. Aerial view in 1959 of site showing deposition of sediment downstream from piers 6 and 7, Eel River at SR-1. (From Calif. Dept. of Transportation.)

SITE 17. EEL RIVER AT SR-01 AT FERNBRIDGE, CALIF.

Description of site: Lat 40°37', long 124°12', location as shown in fig. 86. Bridge has reinforced-concrete, continuous T-girder spans on concrete piers supported by wood piles; and concrete continuous-filled spandrel arch spans. Bridge length is about 2,405 ft (733 m), no skew, with length of 1,451 ft (422 m) over main channel (fig. 86). Approach trestles are 528 ft (161 m) long on left bank and 426 (130 m) on right bank. Main channel piers are massive reinforced concrete, 21 ft (6.4 m) wide at base and placed on piles with tips near elevation -50 ft (-15 m).

Drainage area, 3,700 mi^2 (9,583 km^2); bankfull discharge, 120,000 ft^3/s (3,396 m^3/s); valley slope, 0.0014; channel width, about 2,000 ft (610 m). Stream is perennial, alluvial, sand bed (D_{50}, 0.19 mm), in valley of moderate relief, wide flood plain. Channel is sinuous, locally braided, wandering thalweg.

Hydraulic problems and countermeasures:

1911 Main channel concrete bridge with timber approach trestles built. The channel banks were protected by placement of concrete blocks to prevent channel alinement changes.

1918 Timber approach trestles on both banks replaced with concrete trestles. Scour at pier 6 to depth of 5 or 6 ft (1.5 or 1.8 m) noted when original pier footing pilecap (concrete seal) was replaced with new concrete.

1933 Inspection notes by California Dept. of Transportation report heavy riprap had been placed at upstream end of main river piers to prevent scour. The report added that deep scour was not a recent development, but rather a consequence of oblique flow and wide piers in a sand bed channel. Other measures taken were the removal of drift from piers, cutoff of exposed falsework piling around piers flush with streambed and cutting willow growth from channel to increase the flow capacity.

1948 Scour at the bridge occurred between 1933 and 1948, and required cutting off newly exposed parts of falsework piles flush with streambed. Additional drift had accumulated against the piers, caused disruption of flow, and was removed.

1955 Flood of December 1955, discharge 585,000 ft^3/s (16,567 m^3s (16,567 m^3/s), exceeded bankfull stage, for which the discharge is about 120,000 ft^3/s (3,398 m^3/s). Flows overtopped the left-bank approach embankment and damaged or destroyed about 120 ft (36.6 m) of the approach trestle. Part of the trestle was tilted out of plumb because some pile in a bent settled. Scour to depths of about 20 ft (6.1 m) occurred at the end of the approach fill, and 2 to 4 ft (0.6 to 1.2 m) of scour occurred at several points on the downstream side of the road fill. It was noted that the channel in the vicinity of the bridge had shifted toward the right bank, which had become almost vertical because of erosion.

1960 Flooding during February 1960, discharge over 350,000 ft^3/s (9,912 m^3/s), R. I. of 10 yr, caused additional channel scour. Figure 86 shows the effects of piers 6 and 7 on streamflow and the associated sediment deposition downstream from the bridge.

1961 Diver removed large tree and other drift lodged against footing of pier 6. Undermining at pier 7 was found to be about 5 ft (1.5 m) deep for about 50 percent of the footing area. The island near piers 3, 4, and 5 had increased in length, width, and height, forcing increased main channel flow between pier 7 and abutment 8. Lateral erosion of the channel right bank continuing.

1963 Flood of February 1963, discharge about 275,000 ft^3/s (7,788 m^3/s) R. I. about 5 yr, also caused scouring around bridge piers. A channel survey in June 1963 revealed that erosion was deepening the most active part of the river channel bed (between pier 7 and abutment 8, fig. 87), and building up the bed elevation throughout most of the rest of the channel.

Examination of pier 7 footing by diver in June 1963 indicated that the streambed below the pier footing is stiff blue silt and clay. Sand and fine gravel filled the bottom of the hole for an estimated depth of 3 to 5 ft (9.9 to 1.5 m). The top of the sand in the scour hole was 12 ft (3.1 m) below the footing bottom. During periods of flooding, the sand probably moved out, resulting in scour depths 16 ft (4.9 m) below the footing or 21 ft (604 m) below the normal streambed at the deepest point. About 90 percent of the footing bottom was undermined, and the only part not undermined was at the downstream end. The timber piles were in good condition, but many of the upstream piles showed effects of abrasive action. Parts of two trees, transported as drift and up to 2 ft (0.6 m) in diameter, were lodged along the right bank row of pilings.

Repairs to the footing of pier 7 consisted of placing gravel fill under and around the footing, and protection of the gravel fill with rock riprap (fig. 88).

1964 Flood of December 1964, and largest since construction of the bridge (table 2) caused several hydraulic problems at the bridge. (1) The channel bed around pier 7 (fig. 87) scoured, and scour occurred at abutment 8. (2) The right bank of the channel eroded for a distance of about 1,000 ft (304.8 m) each way upstream and downstream from bridge. (3) Long drift punched a hole through the upstream side of the concrete wall at abutment 8. (4) Deep holes scoured around the pile bents under the left bank approach spans. This scour was probably due to local conditions at the bridge, such as turbulence of flow around the pile bents.

1965 Some of the repairs and construction of countermeasures following the flood consisted of: (1) Placing steel sheet-piling protection around 3 sides of abutment 8 (fig. 89). (2) Replacing rock riprap in the scour area at pier 7 (fig. 90). (3) Filling scour holes around pile bents of the left-bank approach trestle, and protection of the fill with a blanket of rock riprap.

Table 2. Selected flood stages and discharges of Eel River at SR-1 at Fernbridge, Calif.

Date	Stage, in feet above mean sea level	Approximate discharge (ft^3/s)
12-11-37	29.0	305,000
2- 6-42	25.2	220,000
12-22-55	[1]32.8	585,000
2- 8-60	--	373,000
2- 1-63	--	275,000
12-22-64	29.7	[2]800,000
1-24-70	--	338,000
1-16-74	--	422,000

[1] Apparently affected by backwater from tide.
[2] Main channel discharge about 610,000 ft^3/s (17,275 m^3/s).

1966 Rock riprap placed around pier 7 in 1965 had settled and was covered with silt and sand. A few rocks were reported exposed along the sides of the pier.

1974 Lateral erosion and scour problems previously noted at the bridge are not considered significant. Field investigations report recent erosion of the channel near pier 1, indicating the main channel is shifting towards the left bank.

During the period 1965-74, floods in 1970 and 1974 (table 2) were among the largest recorded since 1937, except for the floods in 1955 and 1964. Assuming the 1970 and 1974 floods were a good test of countermeasures installed in 1965, it appears the pier footing protection is performing satisfactorily.

Discussion: Placement of large (2-ton or 1.8-t) rock riprap around the bridge piers did not prevent scour. The riprap did not occupy a significant part of the channel. Placement of rock riprap so that the top of the rock is level with the streambed (fig. 90) appears to prevent scour better than rock riprap placed above the bed (fig. 88). Protection of piling under pier foundations by use of steel piling appears satisfactory if pile tips are below the level of scour. Wide pier footings appeared to cause a higher degree of scour around the pier than smaller footings.

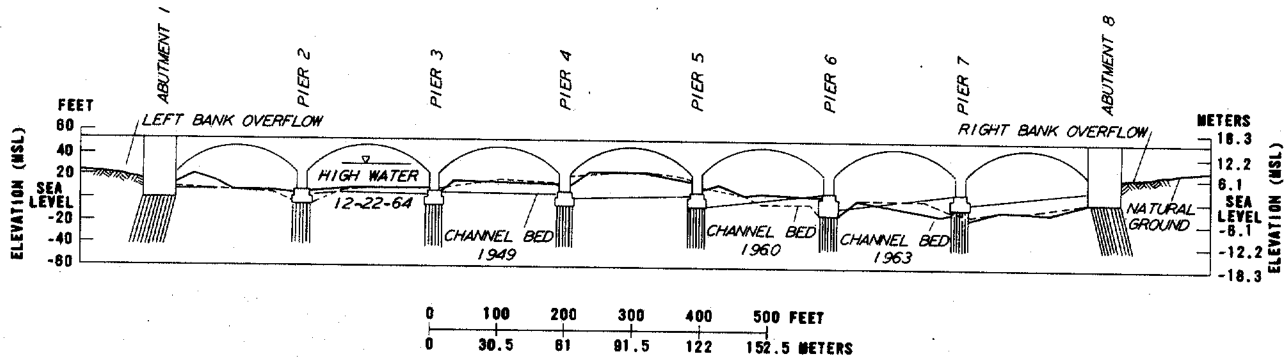


Figure 87. Cross section of main channel of Eel River in 1949, 1960, and 1963.

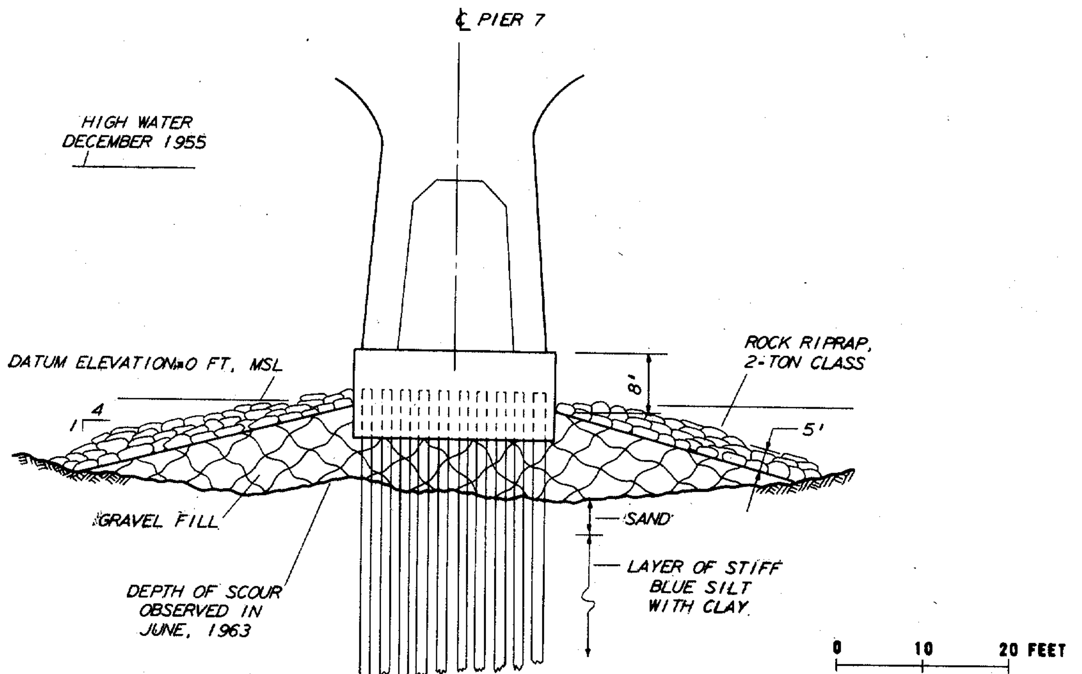


Figure 88. Protection of pier 7 in 1963 to prevent additional scour.

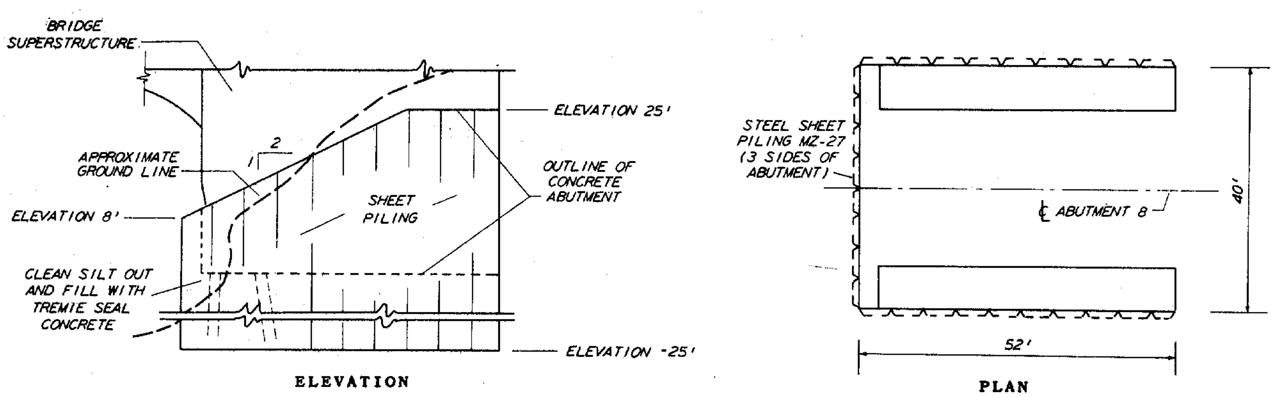


Figure 89. Details of countermeasures installed in 1975 to protect abutment 8 from scour.

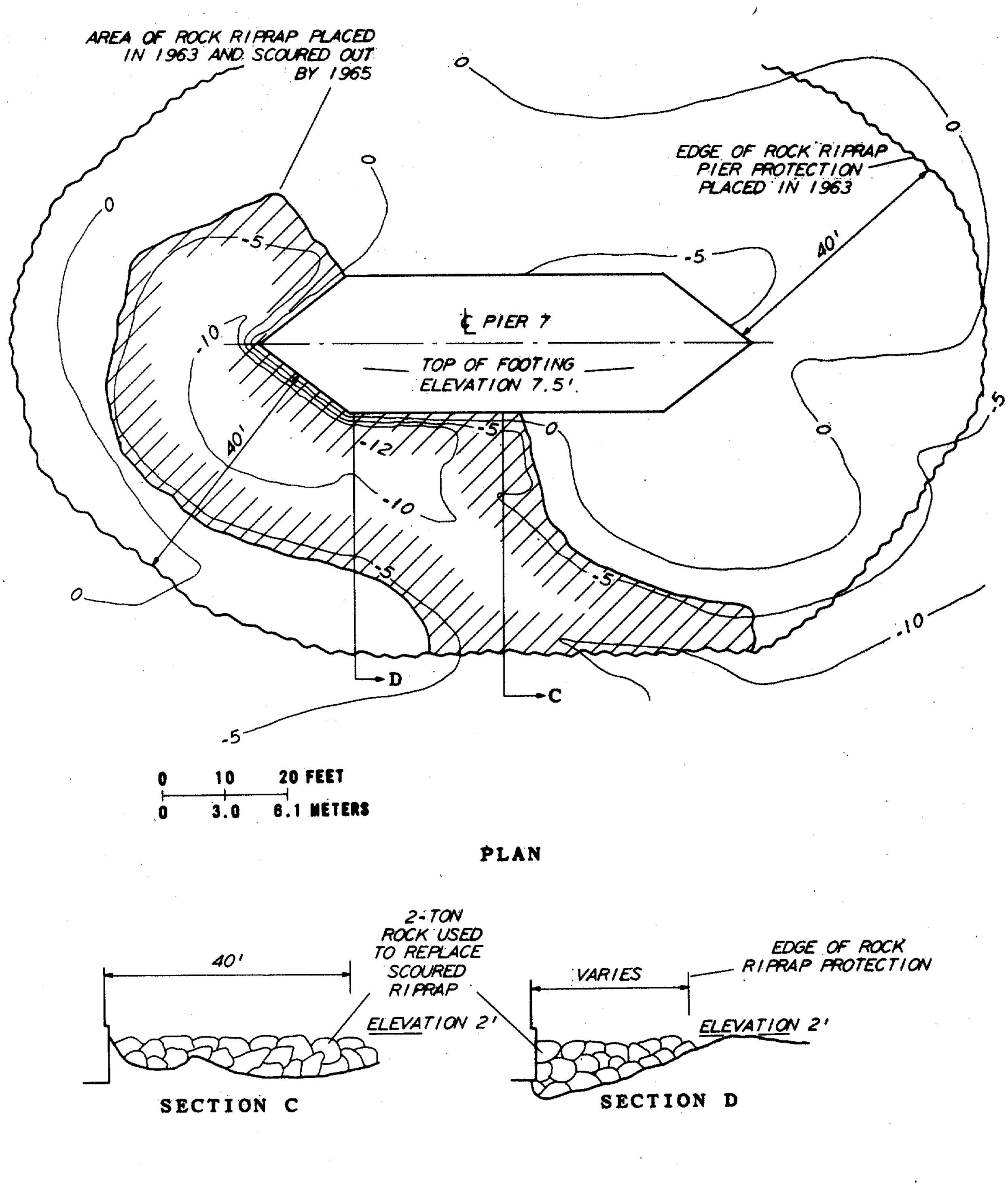


Figure 90. Plan of scour hole at pier 7 as surveyed in 1965; and details of rock riprap replacement.

SITE 18. FEATHER RIVER OVERFLOW AT GRIDLEY-OROVILLE ROAD NEAR GRIDLEY, CALIF.

Description of site: Lat 39°22', long 121°37', location as shown in fig. 91. Roadway across left-bank flood plain is subject to overtopping during floods and is protected by sacked-concrete revetment.

Hydraulic problems and countermeasures:

Date unknown — Part of the approach embankment across the left-bank flood plain of the Feather River is designed to be overtopped during flooding. The embankment is protected by sacked concrete placed on the upstream and downstream side of the road (fig. 92). The downstream toe of the embankment is further protected from erosion by use of an apron, acting as an energy dissipator, that extends about 4 ft (1.2 m) from the toe of fill.

1964 — Left bank overflow during flood of December 23, 1964, overtopped the road embankment to a depth of about 2.5 ft (0.8 m). No damage to the embankment was reported (fig. 93).

Discussion: The success of this countermeasure is due to protection of the downstream side of the embankment with an apron of sacked concrete, which prevents undermining of the embankment toe.

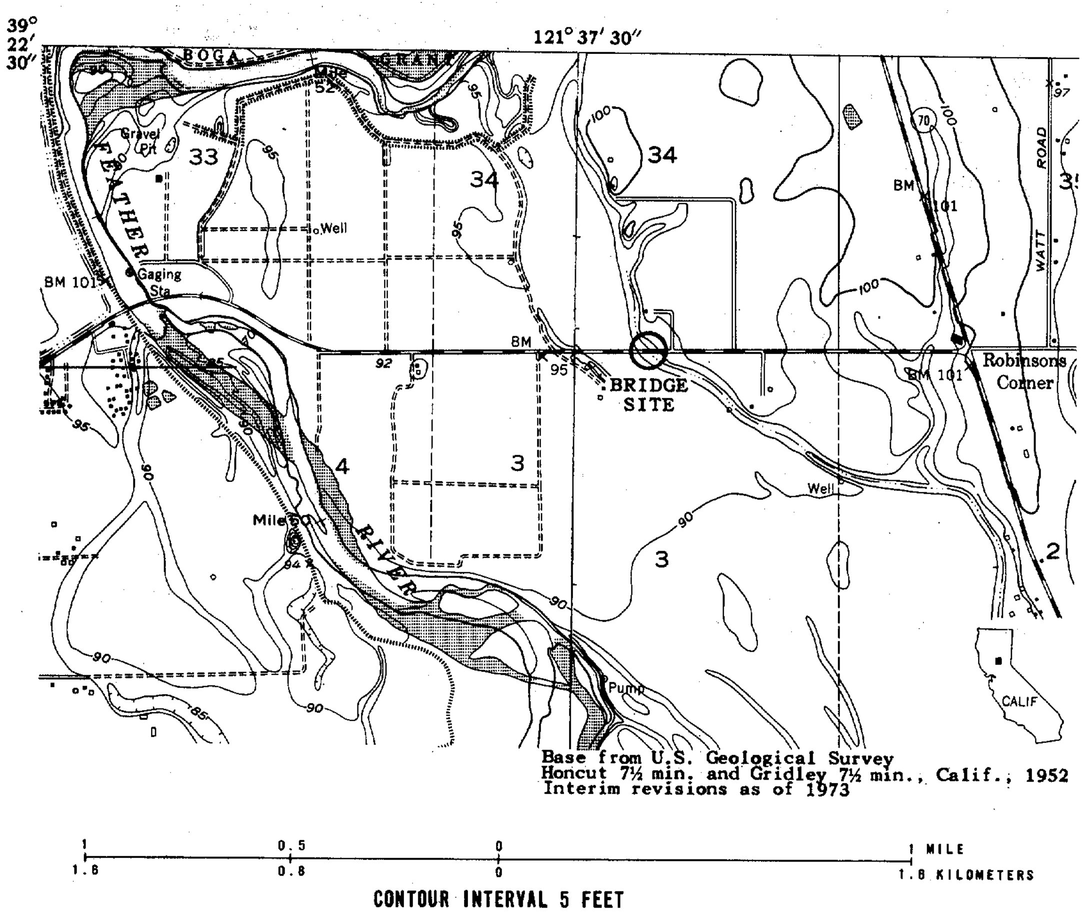


Figure 91. Location of Feather River overflow at Gridley-Oroville road near Gridley, California.

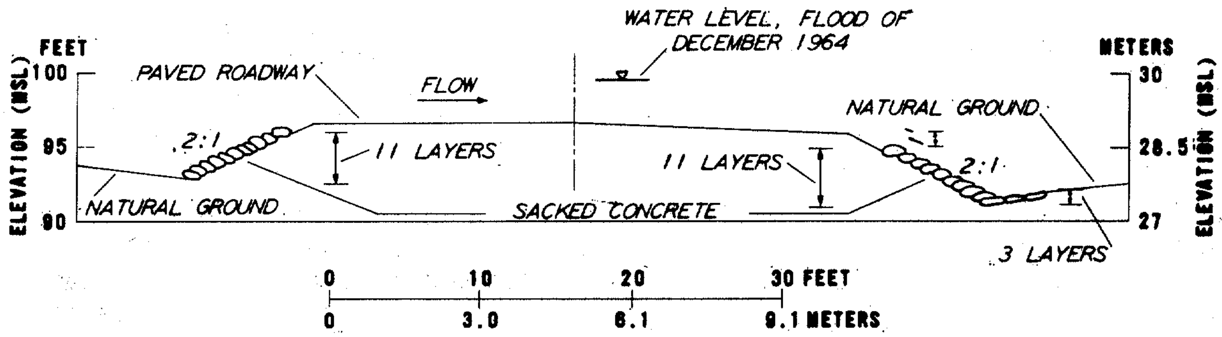


Figure 92. Cross section of road embankment designed for overflow.

Figure 93. Sacked-concrete embankment protection on downstream shoulder of roadway, in 1976.

SITE 19. ROCK CREEK OVERFLOW AT COUNTY ROAD NEAR NORD, CALIF.

Description of site: Lat 39°48', long 121°57', location as shown in fig. 94. A road embankment across right bank flood plain, subject to overtopping during floods, is protected by a concrete apron built as an integral part of the roadway.

Hydraulic problems and countermeasures:

Date unknown — Concrete roadway and shoulder protection was placed and later extended (fig. 95). Flows over the road have not damaged the road embankment, but scour of the flood plain downstream from the road is evident (fig. 96).

Discussion: The road embankment protection (fig. 97) is performing satisfactorily, but an energy dissapator is needed downstream from the road to prevent damage to the road shoulder and scour of the flood plain.

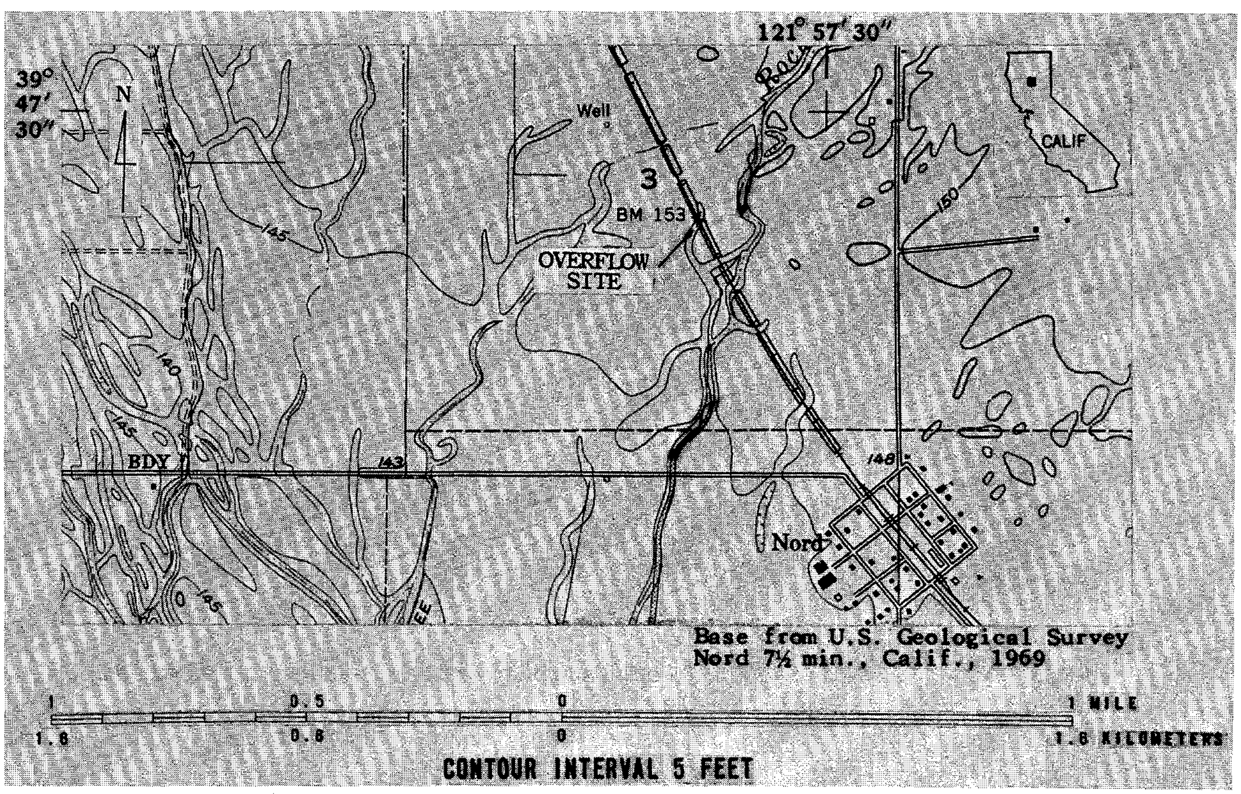


Figure 94. Location of Rock Creek at county road near Nord, California.

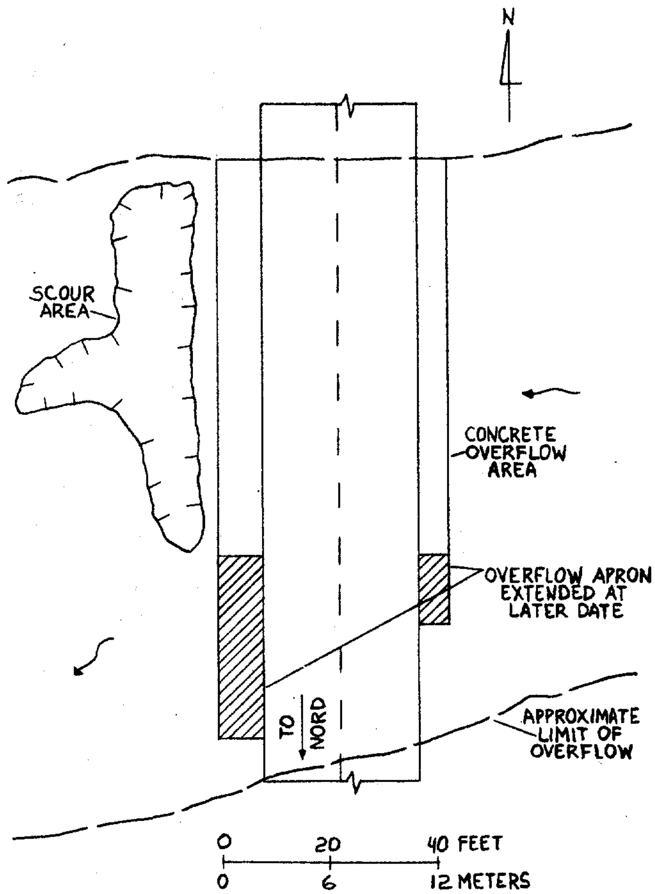


Figure 95. Plan of road embankment designed for overflow.

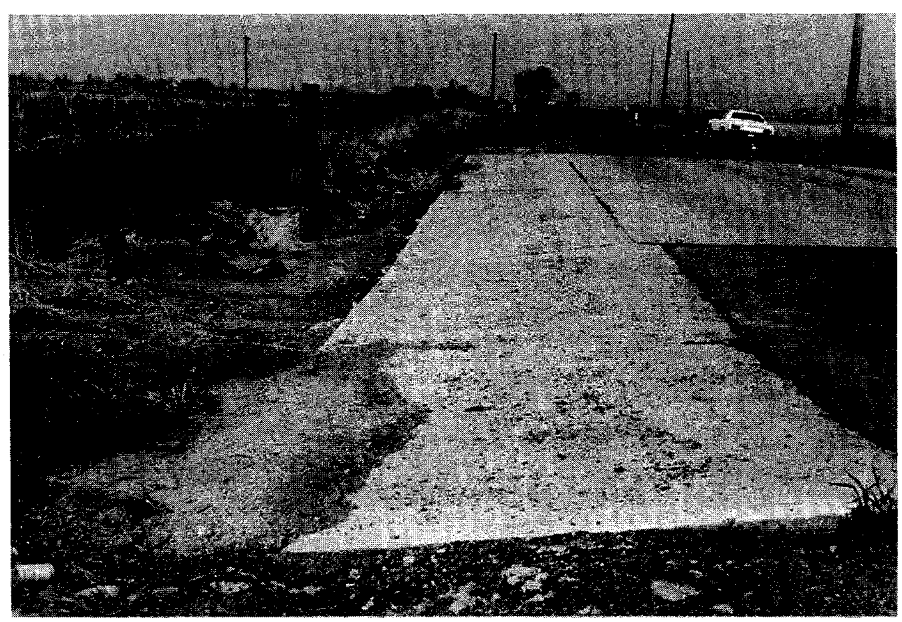


Figure 96. Concrete apron protection at downstream side of road overflow section, in 1977.

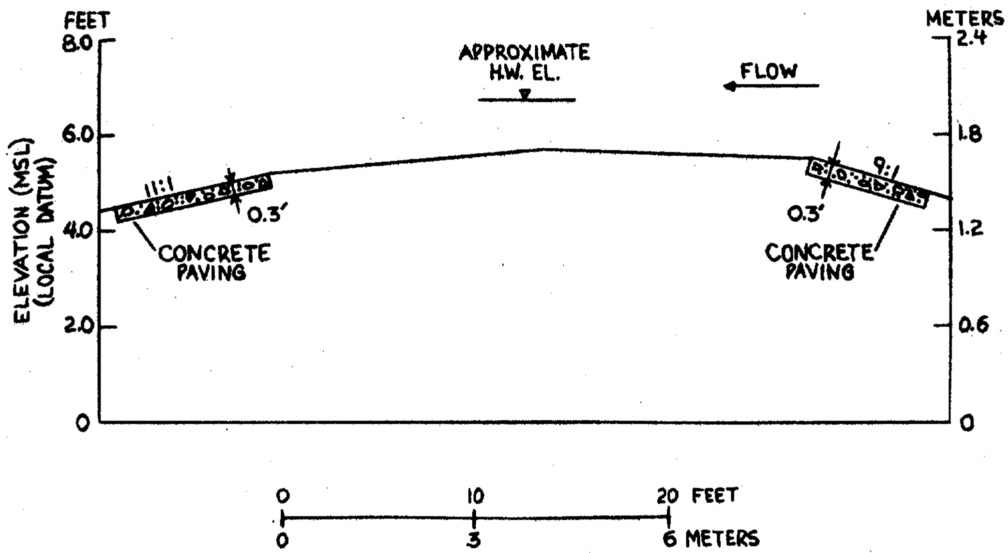


Figure 97. Cross section of road embankment designed for overflow.

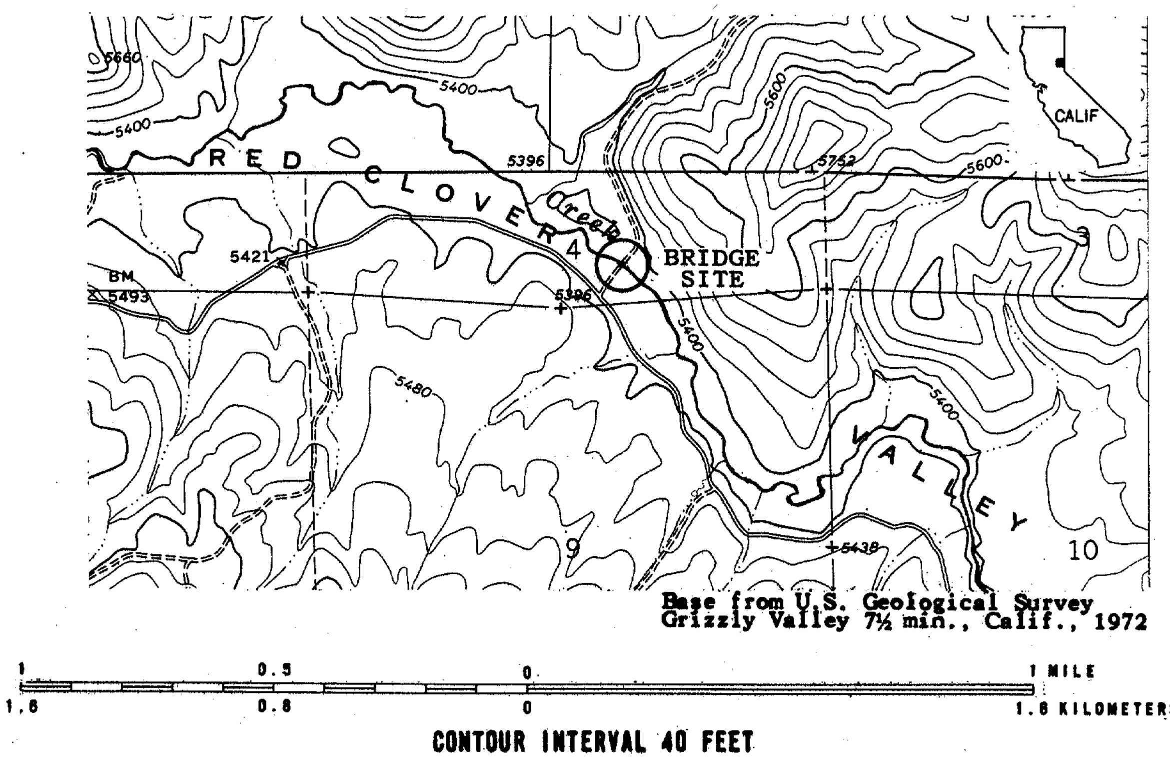


Figure 98. Location of Red Clover Creek at forest service road near Portola, California.

SITE 20. RED CLOVER CREEK AT FOREST SERVICE ROAD NEAR PORTOLA, CALIF.

Description of site: Lat 39°58', long 120°31', location as shown in fig. 98. Steel girder bridge, length 52 ft (16 m) with concrete vertical abutments founded on treated-timber piles. There are no piers. Abutment wingwall angles are 90° downstream, and 74° at the upstream side of bridge (fig. 99).

Drainage area is 87.9 mi^2 (22.8 km^2). The creek is a slow mountain stream which meanders through a flat valley.

Hydraulic problems and countermeasures:

1954 Bridge constructed (fig. 100). To improve the waterway capacity the channel bed was widened and shaped as a trapezoid (fig. 101), and cobble riprap was placed on both banks for scour protection.

1967 High water during March 1967 contributed to general scour and lateral erosion of channel. This erosion caused the channel bed to change from a trapezoidal to parabolic shape (fig. 101).

1970 Flood of January 1970 caused additional general scour of channel at the bridge.

1975 A survey of the channel in the spring indicates the magnitude of channel change between 1967 and 1975 (figs. 101 and 102). Of significance is the change in channel shape between 1954 and 1967. This change is indicated by the variation in hydraulic factors (table 3):

Table 3. Variation in hydraulic factors for Red Clover Creek between 1954 and 1975.

Hydraulic factors assuming water surface at elevation 5,344 ft (1629 m)				Assumed discharge of 1,300 ft^3/s (37 m^3/s)		
Date	[3]Area (ft^2)	Convey-ance	Width of Channel at water surface [4](ft)	Velocity (ft/s)	Froude number (F)	[1]Bed Shear stress [5](lbs/ft^2)
[2]1954	262	30,454	40	5.4	0.34	1.27
1967	297	36,105	40	4.4	.28	1.44
1975	296	36,256	40	4.4	.29	1.43

[1] Channel slope 0.0031 from topographic maps.
[2] After construction.
[3] ft^2 multiplied by 0.093 equals m^2.
[4] ft multiplied by 0.305 equals m.
[5] lbs/ft^2 multiplied by 4.88 equals kg/m^2.

The channel width at the crossing remained constant at 40 ft (12 m) between 1954 and 1975, but the area of the waterway increased about 13 percent between 1954 and 1967. If the channel size as determined in 1967 and 1975 represent a condition of equilibrium, then construction of the sacked-concrete bank protection in 1975 (fig. 103) was not needed as a countermeasure.

Discussion: The channel, which was excavated to a trapezoidal shape and subjected to flow contraction, reverted to its natural size and parabolic shape. The period of time required for the change in channel shape to occur depended on the number and magnitude of flood events. At this bridge site, a stable channel shape was reached about 13 years after construction of the bridge. Significant changes in the channel occurred even though the Froude number for flood flows at the bridge cross section were only about 0.3.

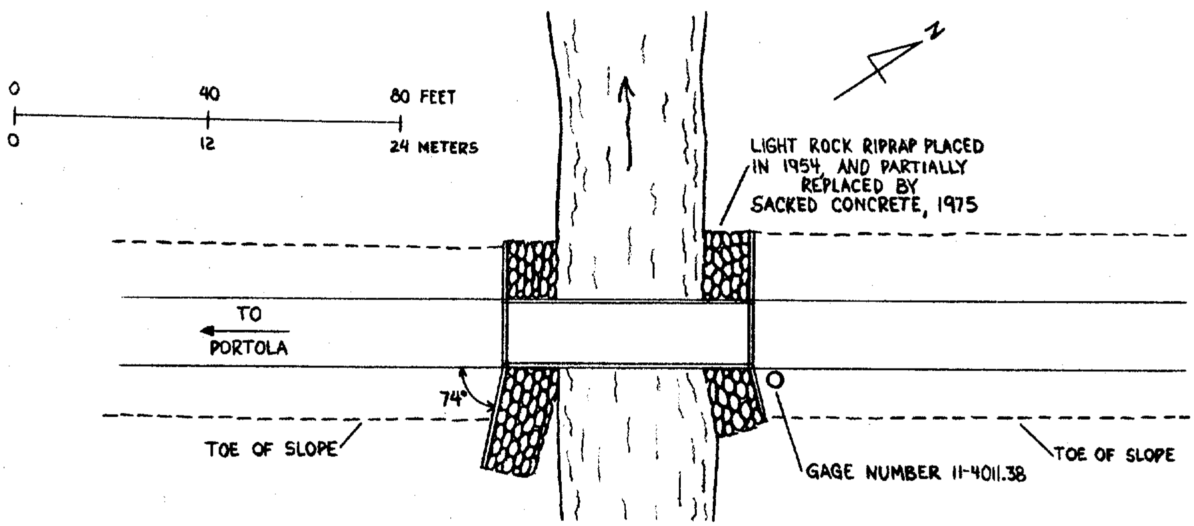


Figure 99. Plan of Red Clover Creek bridge.

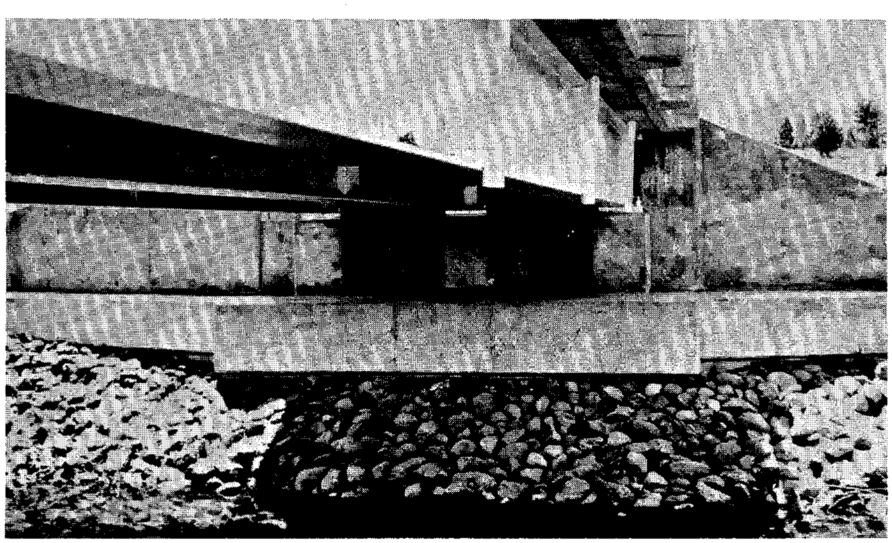


Figure 100. View of left-bank abutment with cobble riprap bank protection placed in 1954. (From U.S. Forest Service.)

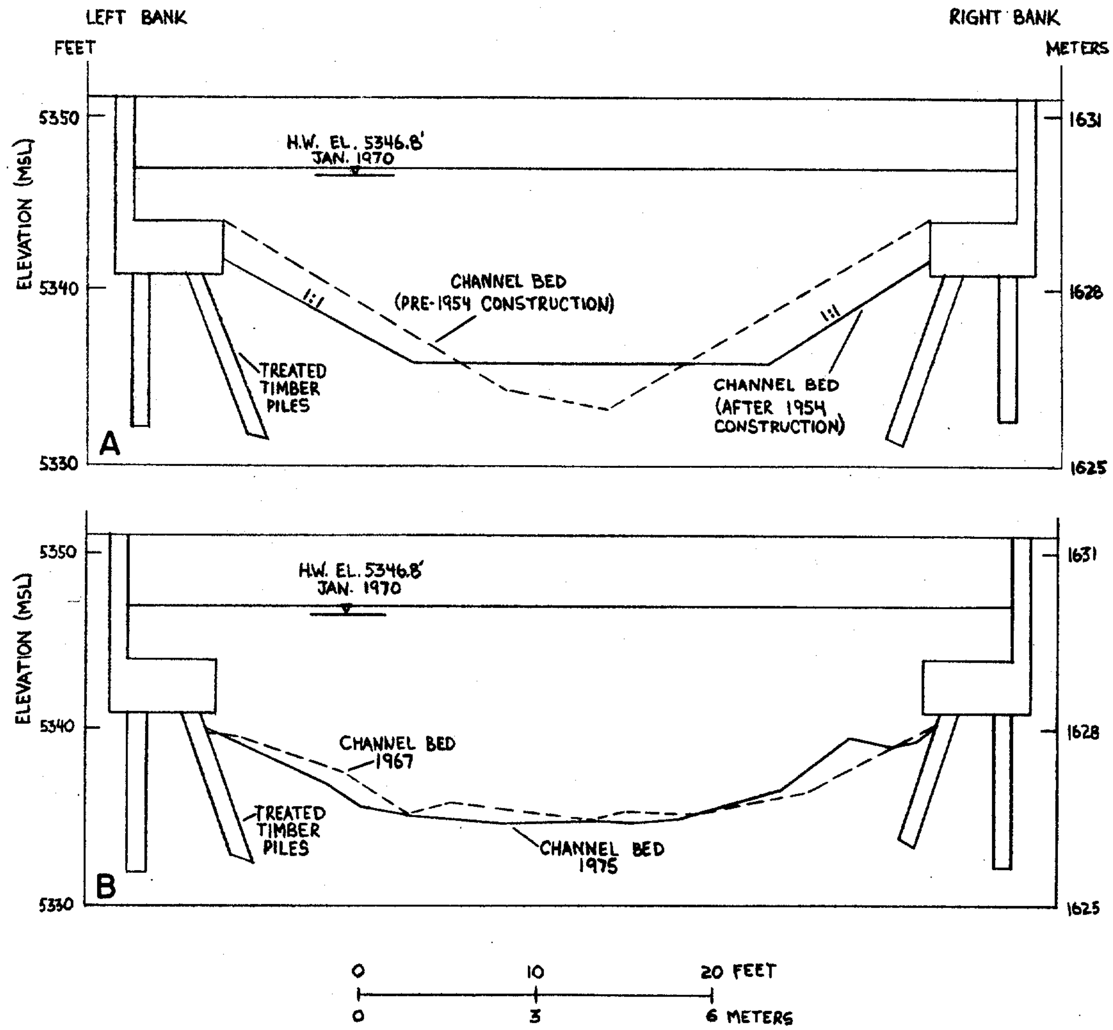


Figure 101. Cross sections of channel. A, before and after bridge construction in 1954. B, in 1967 and 1975.

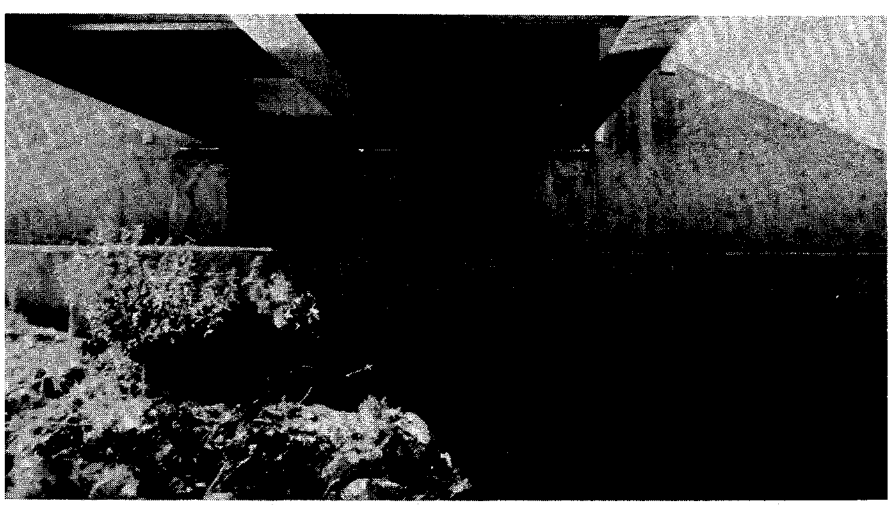


Figure 102. Erosion at left bank bridge abutment by 1975.

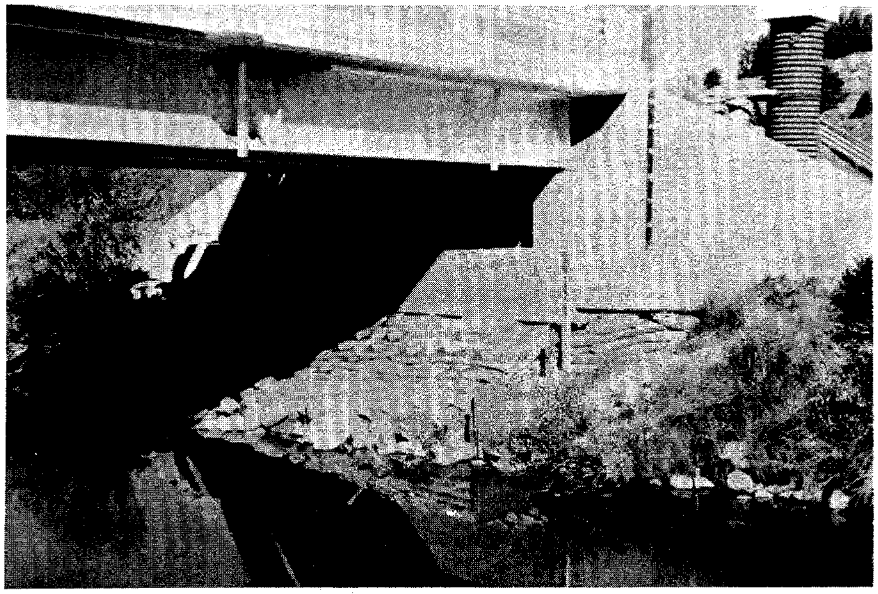


Figure 103. Placement of bags of premixed concrete at right bank abutment in 1975, to prevent lateral erosion. (From U.S. Forest Service.)

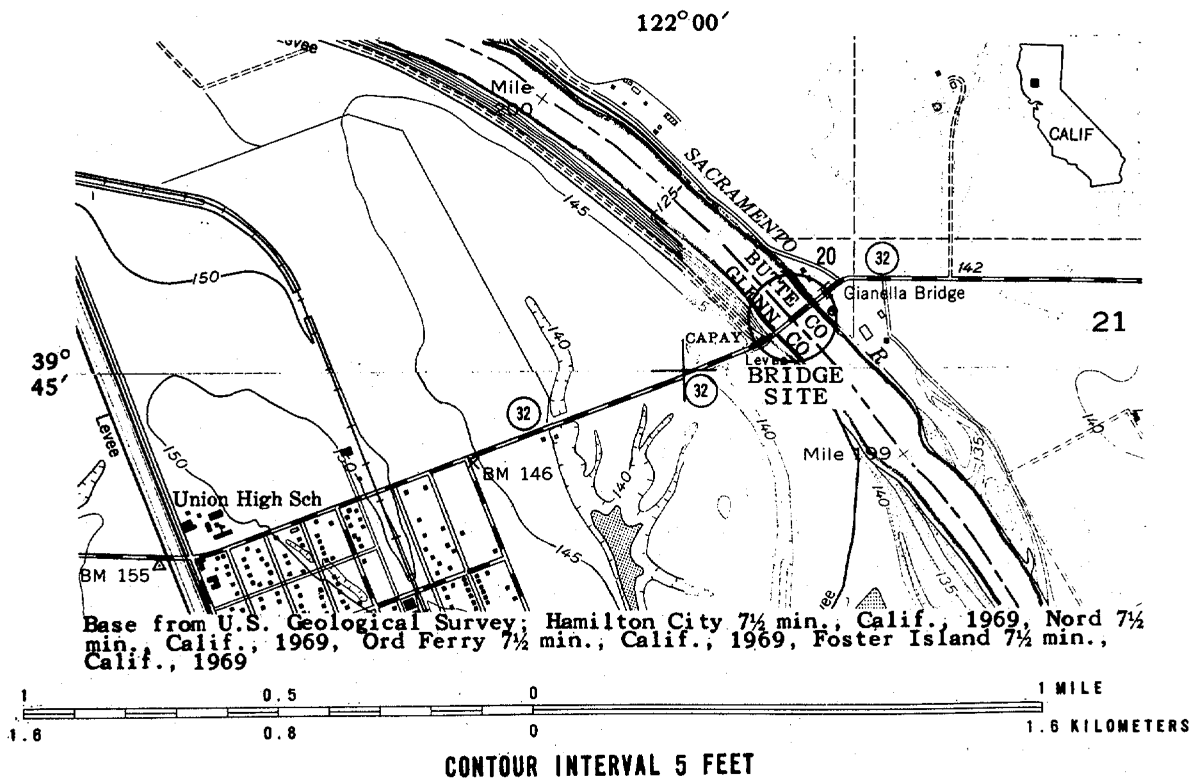


Figure 104. Location of Sacramento River at SR-32 near Chico, California.

SITE 21. SACRAMENTO RIVER AT SR-32 NEAR CHICO, CALIF.

Description of site: Lat 39°45', long 122°00', location as shown in fig. 104. Steel truss swingspan, length 311 ft (94 m), on massive concrete piers supported on timber piles; steel-truss approach spans, each 133 ft (40.5 m) long. Total bridge length, 580 ft (177 m). Flow normal to bridge.

Valley slope, 0.0006; channel width, 500 ft (152 m). Stream is perennial, regulated, alluvial, gravel bed, wide flood plain. Channel is sinuous, locally braided, wider at bends, point bars, cut banks general, silt sand banks, tree cover at less than 50 percent of bankline. Rapid lateral migration of channel (average of about 15 ft or 4.5 m per year) is attributed mainly to past clearing of natural vegetation along river for agricultural purposes.

Hydraulic problems and countermeasures:

1908 Bridge built with center pier designed to support swingspan mechanism and with cutwater shape to prevent debris lodging by use of a concrete fender (fig. 105), which is an integral part of the center pier.

1940 Prior to 1940, a slope protection of large articulated concrete blocks, held together by heavy steel cable, was placed around the right-bank abutment. Other large concrete blocks were placed as riprap around the piers. During flooding in the winter of 1940 some of the blocks were undermined and left hanging by the cables with no support underneath. No repairs were considered necessary at the time.

1949 Considerable erosion at the left-bank abutment and downstream wingwall (retaining wall) occurred. Recommended repairs included placement of riprap and pile protection to stabilize the wingwall.

1956 Flood of December 1955, caused further erosion at the left bank abutment. Eddies generated at the end of the retaining wall caused washout of some of the approach embankment behind the wingwall. The approach fill behind the retaining wall was replaced using cobble.

1958 During the flood of February 1958, (table 4), erosion occurred at the upstream side of the left-bank approach embankment to the extent of cutting into the roadway (fig. 106). For temporary protection, old car bodies and cobble were dumped into the eroded area. Later during the flood, the left bank levee 200 ft (61 m) upstream from the abutment was destroyed, and the downstream left-bank wingwall failed, allowing the approach embankment to slough down. Depths of scour around the left bank abutment from the original bed level in 1908 were about 19 ft (5.8 m) (fig. 106). Steel sheet-pilings were then placed on both sides of the abutment and approach to protect the embankment.

Following the February 1958 flood, permanent repairs and countermeasures for erosion control were planned. Choice of a remedy was reached by a process of elimination. The main channel is nearly constant in width and could not be constricted by jetties. A retard structure would collect drift. Replacement of the concrete wingwalls would require extension, placing on piles, and deep foundation. The best protection against erosion would guide the stream past the abutment without eddies and reduce stream velocities near the toe of the abutment. The countermeasure consisted of a timber-pile deflector wings (retards) built 75 ft (23 m) upstream and 50 ft (15 m) downstream from the abutment (fig. 107).

1956-76 Floods during the period caused no damage to the left bank abutment or countermeasure and the timber pile deflector performed satisfactorily. A comparison of major flood events before and after construction of the countermeasure are given in table 4. No frequency of event data are available because of upstream regulation.

Table 4. Selected flood stage and discharge data for Sacramento River at SR-32 near Chico, Calif.

Flood date	Stage ft (USED datum)	Main channel discharge (ft^3/s)
1938	150.7	not available
Feb. 1940	150.5	" "
1956	146.4	" "
Feb. 1958	149.3	150,000 (estimated)
----------------Countermeasure installed----------------		
Dec. 1964	149.6	151,000 (estimated)
Jan. 1970	150.8	156,000
Jan. 1974	149.6	158,000

Discussion: The center pier with special cutwater design is apparently effective in preventing the lodging of debris at the bridge. Depth of scour at the face of the left-bank abutment during flooding was 19 ft (6 m) below original streambed level. Use of timber-pile deflector wings has effectively protected the abutment from lateral erosion and local scour during flooding.

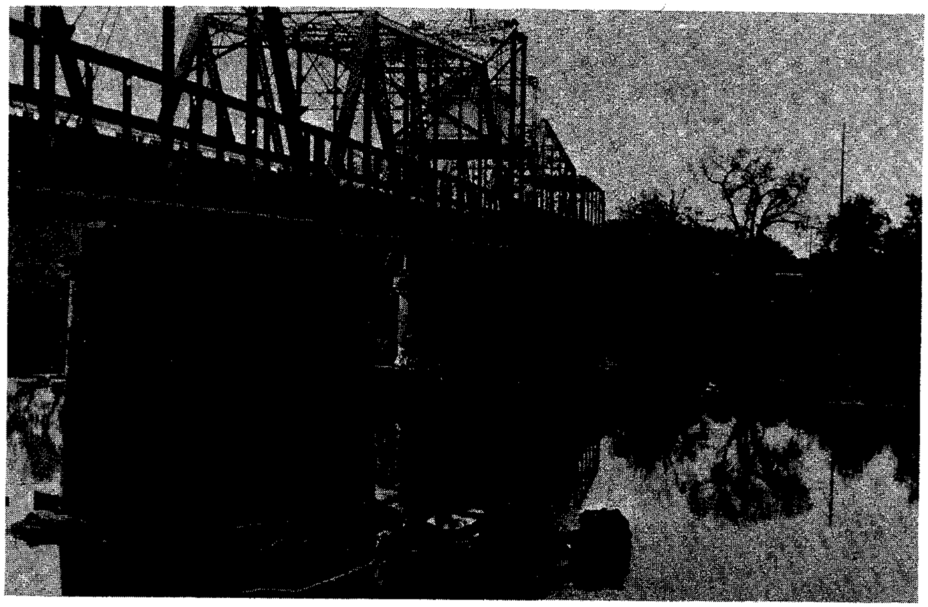


Figure 105. View of center pier with concrete fender debris deflector, in November 1976.

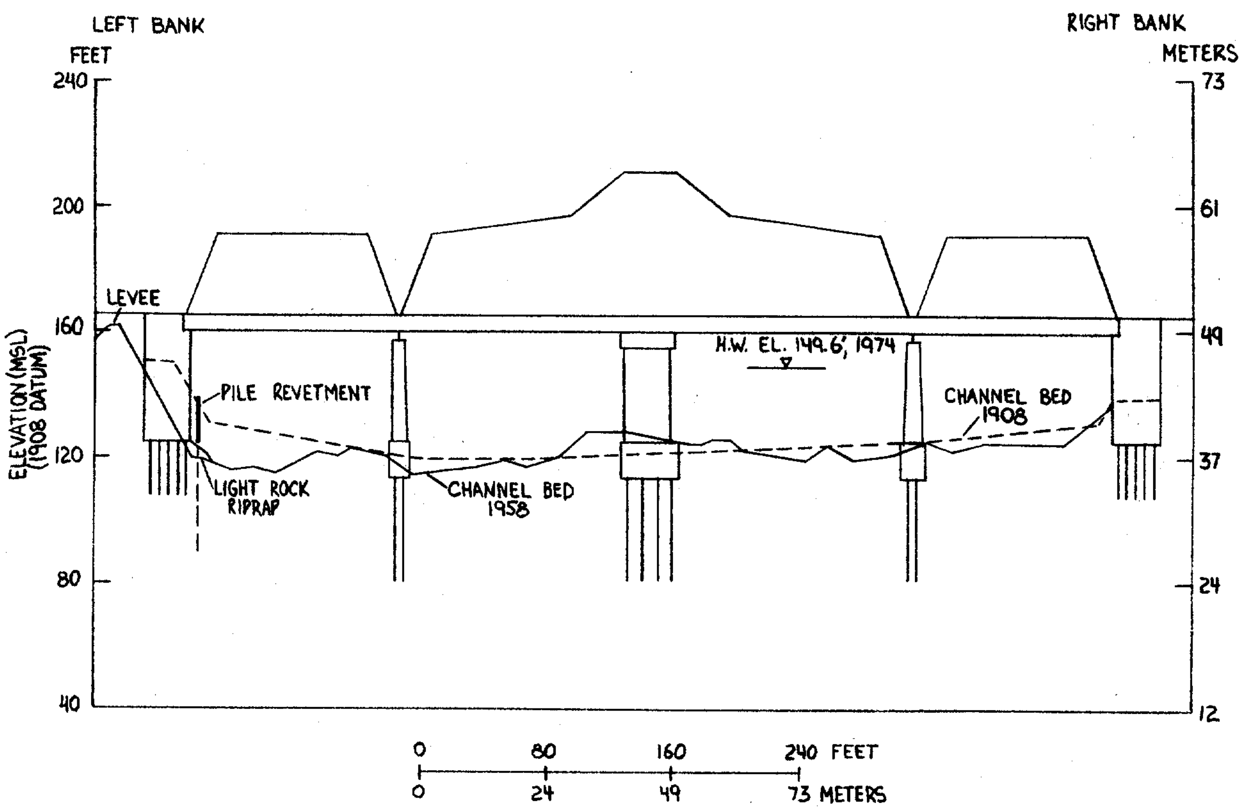


Figure 106. Cross sections of Sacramento River at SR-32 near Chico, California.

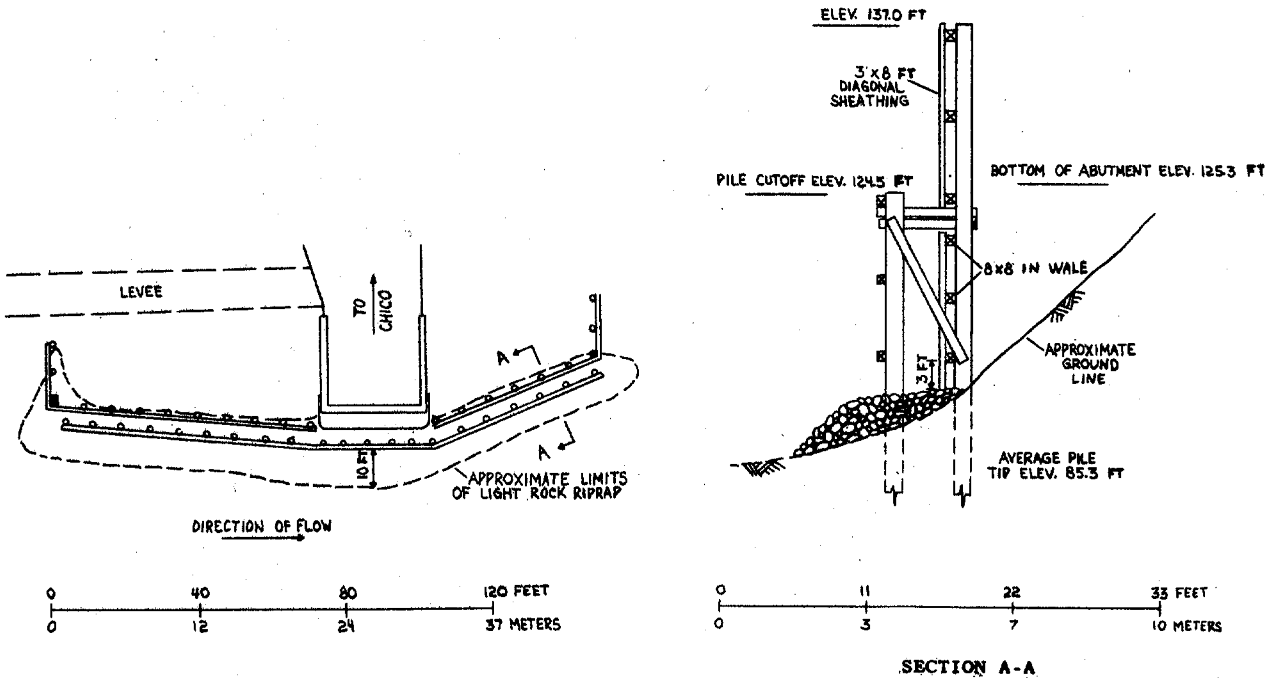


Figure 107. Plan (left) and cross section (right) of timber pile deflector.

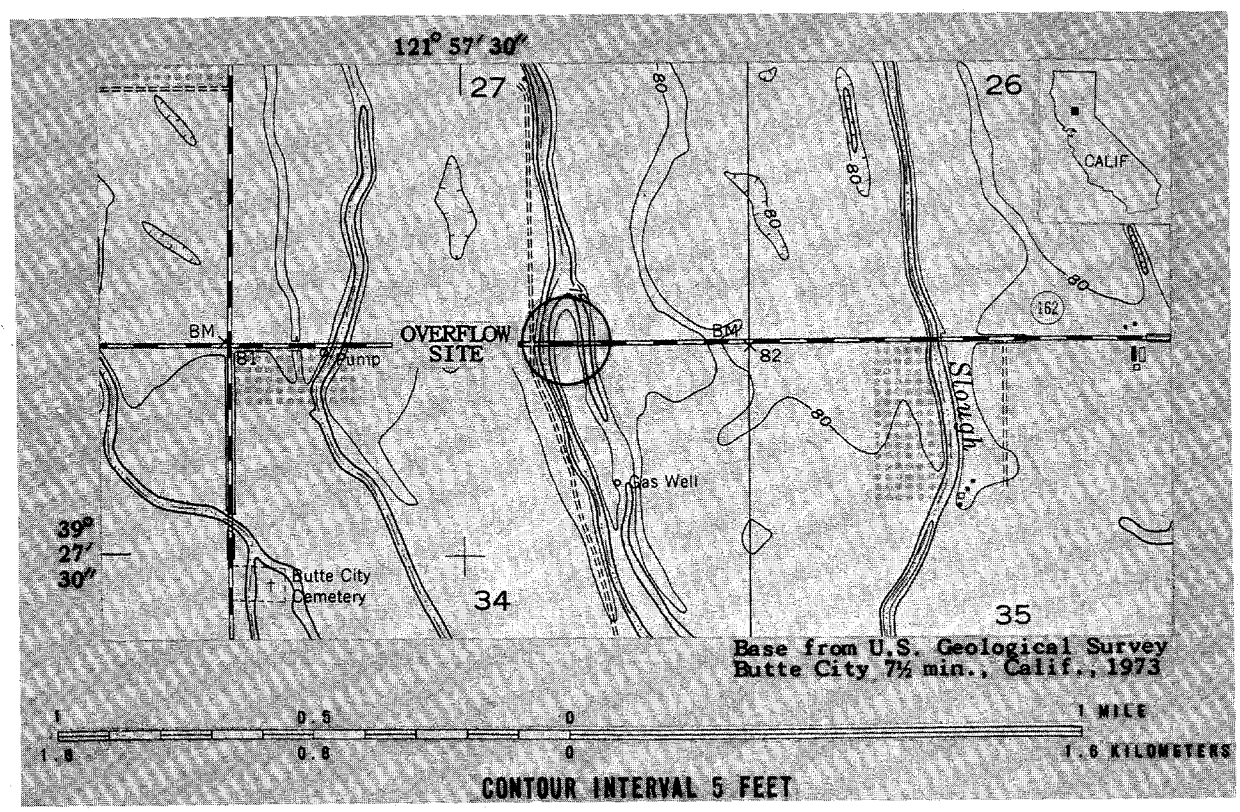


Figure 108. Location of Sacramento overflow at SR-162 near Butte City, California.

SITE 22. SACRAMENTO RIVER OVERFLOW AT SR-162 NEAR BUTTE CITY, CALIF.

Description of site: Lat 39°28', long 121°57', location as shown in fig. 108. Road embankment across left-bank flood plain, and subject to overtopping during floods, is protected by a concrete apron placed on downstream side of road embankment.

Hydraulic problems and countermeasures:

1970 During the January 1970 flood (R. I. about 25 yr) damage of the approach embankment caused by road overflow is associated with several factors. During the initial period of road overflow, there was a large differential between water surface levels upstream and downstream from the road. As water passed over the road, scour of the downstream road shoulder occurred as flows dropped over the embankment. This is a typical flow situation, and is the condition commonly associated with embankment damage caused by road overflow. Additional scour occurred in the downstream drainage ditch which acted as a plunge pool and energy dissipator for the road overflow.

Scour and lateral erosion of both the upstream and downstream approach embankment shoulders occurred because flood flows were diverted from an upstream-downstream direction by drainage channels. The transverse flows in these channels tended to cause lateral erosion and scour because steep gradients in the drainage channels caused high velocities as the flows were conveyed toward the main channel. (Water levels on the flood plain are usually higher than levels in the main channel). Until there was sufficient overflow on the flood plain to cause road overflow, all flows were diverted down the upstream drainage channel. The amount of this flow was greatly in excess of the design capacity of the drainage channel. Considerable turbulence of flow in the downstream drainage channel occurred when flows already in the channel were augmented by road overflow entering the channel in both vertical and lateral directions.

1974 Part of the approach embankment across the left-bank flood plain of the Sacramento River (fig. 108) is designed for overtopping during floods. The embankment (fig. 109) is protected by a concrete apron that acts as an energy dissipator (fig. 110).

1976 Countermeasure untested. Design of the energy dissipator was based on flow conditions occurring during the January 1970 flood.

Discussion: Approach embankment damage caused by road overflow during the January 1970 flood (R. I. about 25 yr) was associated with differential water levels upstream and downstream from the road, and excess flows in drainage channels along the embankment. A concrete apron was built along the downstream side of the road embankment to prevent scour by flows that overtop the roadway.

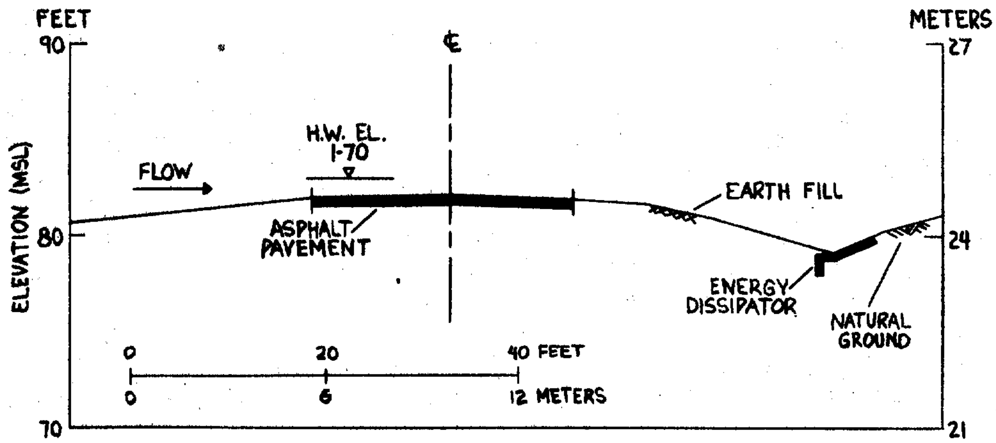


Figure 109. Cross section of road embankment.

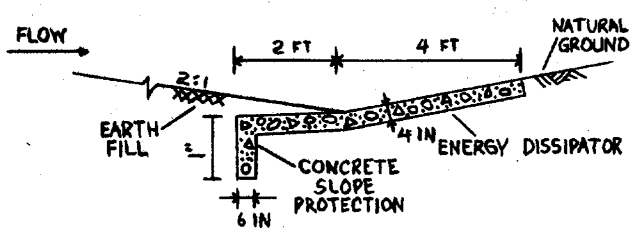


Figure 110. Energy dissipator located on downstream side of road.

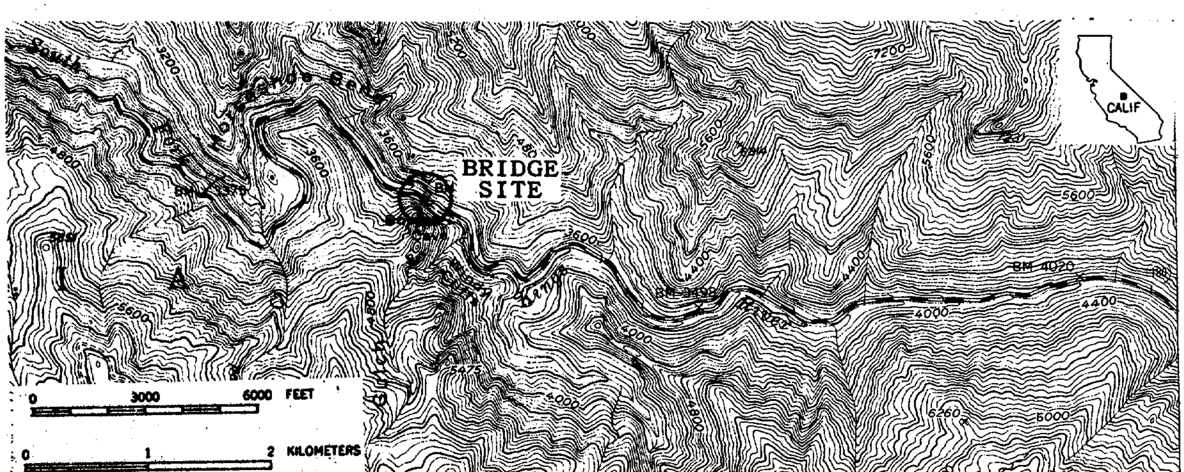


Figure 111. Location of S.F. Kings River at SR-180 near Hume, California.

SITE 23. SOUTH FORK KINGS RIVER AT SR-180 NEAR HUME, CALIF.

Description of site: Lat 36°49', long 118°49', location as shown in fig. 111. Bridge has continuous steel stringers on stone masonry pier and closed end vertical abutments. Piers and abutments are placed on spread footings founded on large granite boulders and bedrock. Bridge length is 122 ft (37 m) with two equal length spans. Bridge is skewed 50 degrees and piers skewed 11 degrees to the stream. Stream channel is composed of large boulders and bedrock with some gravel.

Drainage area, 470 mi^2 (1,217 km^2); valley slope, 0.034. Stream is perennial, non-alluvial, boulder bed, in valley of high relief, no flood plain.

Hydraulic problems and countermeasures:

1938 Bridge built (fig. 112). A concrete apron 2 ft (0.6 m) wide and 1 ft (0.3 m) thick was placed around center pier and left-bank abutment to act as a buffer against impact of large boulders.

1939 A 2-ft (9.6-m) deep scour hole at downstream toe of left-bank abutment developed.

1940 New cutoff wall was built to prevent further scour at the left-bank abutment.

1941 General scour about 3 ft (0.9 m) developed beneath concrete apron at bridge.

1942 A scour hole 3 ft (0.9 m) developed beneath upstream end of center pier.

1943 The scour hole at the upstream end of the center pier was 5 ft (1.5 m) deep, and a new scour hole 4.5 ft (1.4 m) deep has developed at the downstream end of the left bank abutment.

1947 Repairs to scour holes consisted of placing boulders in scour holes.

1951 Flood of November 18, 1950, washed out part of the left-bank approach fill and undermined the right bank side of the center bridge and pier (fig. 113). Water overtopped the bridge. Repairs included backfilling the approach embankment and filling the undermined pier footing. The scoured parts under the bridge piers and abutments were filled with concrete and the entire channel bed near the bridge was paved for a distance of 56 ft (17 m), 10 ft (3 m) upstream and 20 ft (6 m) downstream from bridge.

1956 Flood of December 23, 1955, washed out both approaches to bridge, and overtopped the bridge deck. Recurrence interval of flood was about 20 yr. Portions of channel bed near bridge scoured and downstream end of pier was undermined (fig. 112). Several methods of preventing further undermining of the bridge piers and abutments were considered, including lowering the channel bed to increase the channel capacity. The countermeasure used was to continue placing concrete, as needed, around footings as the channel degraded.

1961 A scour hole of the concrete apron at the downstream end of the center pier developed and was filled with tremie concrete.

1966 Flood of December 1966 washed out the right bank approach roadway and caused some scour of the channel bed concrete paving. Repairs consisted of replacing the embankment fill material.

1968 Scour at downstream end of center pier observed, and concrete paving in channel also breaking up. Tremie concrete placed in scour areas.

1969 Flood of June 1969 caused no damage to tremie concrete placed in 1968.

1970 The scour holes in the channel bed concrete apron have filled with huge rocks. Natural fill by the stream has raised the level of the channel (fig. 113) to the level of the remaining concrete bed movement. This condition may be caused by a new channel control located downstream from the bridge.

1971 Channel still filled with rock deposited during the 1969 flood.

1976 Channel appears stable since 1969 (fig. 114) and the problem of local scour around the pier footing has diminished.

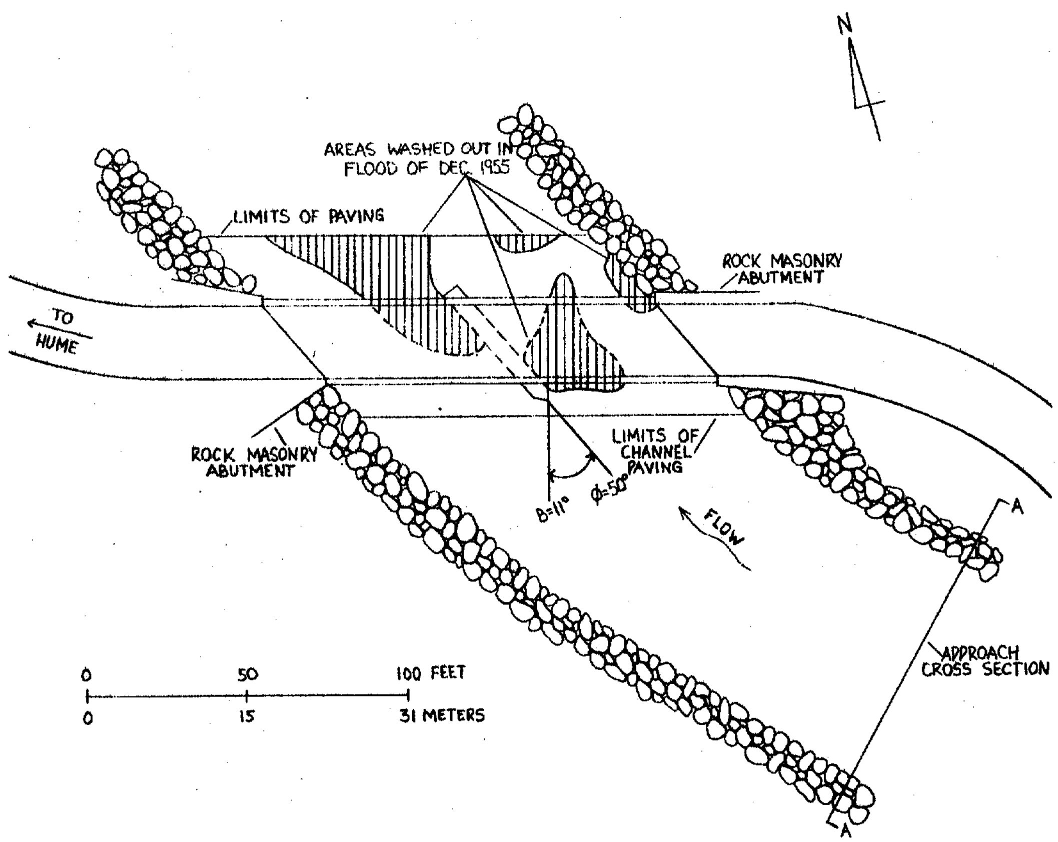


Figure 112. Plan of S.F. Kings River at SR-180 crossing.

Discussion: Use of concrete to fill scour holes and protect the channel bed from scouring appears to be satisfactory. Because the stream has high average velocities (11 ft/s or 3.4 m/s), the concrete fill material will require constant maintenance and repair. Placement of bridge abutments and piers on spread footings resting on large boulders does not insure the footing will be stable. Bed material deposition at the bridge may be the result of a channel control located downstream from the bridge. This control may change in the future, and increased general bed scour or degradation at the bridge may again occur.

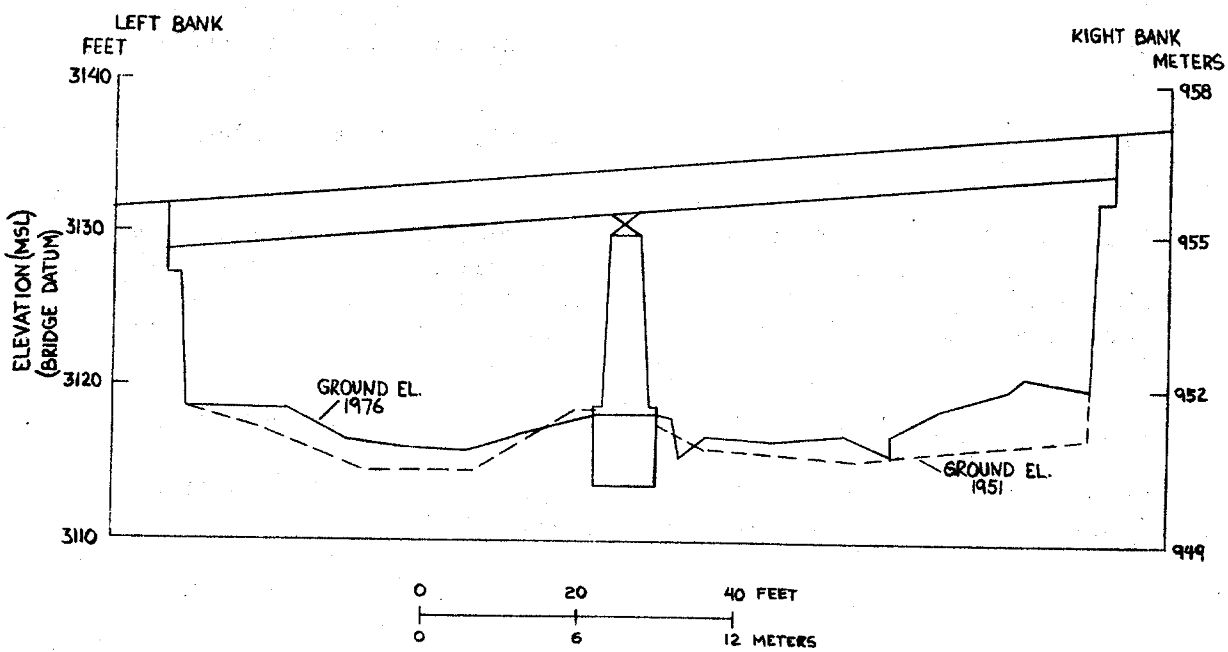


Figure 113. Cross sections of S.F. Kings River at SR-180 crossing, 1951 and 1976.

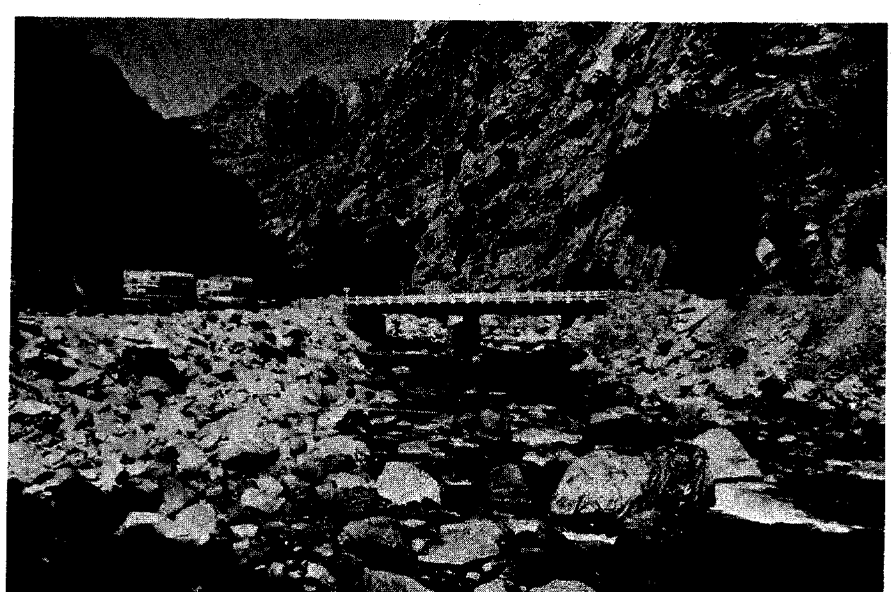


Figure 114. View downstream of S.F. Kings River at SR-180 bridge in 1976.

SITE 24. YUBA RIVER AT SR-20 NEAR SMARTVILLE, CALIF.

Description of site: Lat 39°13', long 121°20', location as shown in fig. 115. Bridge with open concrete spandrel arches and concrete-girder approach spans on concrete piles. Abutments are spillthrough, on spread footings. Stream regulated by Englebright Reservoir since 1941.

Drainage area, 1,193 mi^2 (3,090 km^2); valley slope, 0.0023; channel width, about 600 ft (183 m). Stream is perennial, regulated, semialluvial, gravel and cobble bed, in valley of moderate relief, no flood plain. Channel is meandering, locally braided.

Hydraulic problems and countermeasures:

Bridge maintenance reports starting in 1938 record the problem of scour around the base of the piers. The scour is related to an unstable channel bed caused by the many feet of channel filling during the years of hydraulic mining for gold in the Yuba River Basin. Periodic observations of the amount of channel change during the years 1848-1977 are given in table 5.

Table 5. Summary of channel changes, Yuba River near Smartville.

Year	Low water channel-bed elevation at bridge (In feet, above mean sea level)
1848	146
1900	227
1912	212
1913	bridge built
1942	198
1946	201
1948	193
1950	191
1977	185

The average rate of change in bed elevation has been about -0.6 ft (1.8 m) per year since 1900, and -0.4 ft (1.2 m) per year since the bridge was built. Channel bed degradation at the bridge may also be affected by construction of upstream reservoirs prior to 1969.

1913 Bridge built, total length 685 ft (20.9 m), length between abutments 560 ft (171 m); zero skew (fig. 116). Each river pier is supported by concrete footings on 50 timber piles, each about 30 ft (9 m) long, inside concrete caissons (fig. 117). The stream carries much floating debris and during floods transports boulders. For protection, the pier shaft was rounded in shape and faced with heavy steel armor plate on the upstream side.

1951 Flood of November 1950 (discharge 109,000 ft^3/s or 3,322 m^3/s) caused channel degradation and scour around the piers to progress to the extent that strengthening and repairing of the footings was needed. Pier footing caissons were exposed to the stream and suspended material had worn the caisson concrete sufficiently to expose reinforcing steel. Repairs consisted of solidifying the streambed material around piers 7, 8, and 9 (fig. 118) by pressure grouting. The grout wall was shaped to prevent lodging of debris, and streamlined as much as possible to prevent scour around the pier.

1965-66 Floods of December 23, 1955 (discharge 148,000 ft^3/s or 4,511 m^3/s) and December 22, 1964 (discharge 171,000 ft^3/s or 5,212 m^3/s) and continued channel degradation had exposed the upper parts of the concrete grout curtain (fig. 120) built in 1951. Rock riprap of 2-ton weight (fig. 119) was placed around the piers to prevent additional scour and to protect the piers and the grout curtain. Most of the riprap was placed at the upstream end of the piers.

1970 Flooding during January 1970 (discharge 94,100 ft^3/s (2,665 m^3/s) and continued channel degradation caused severe scour around pier 9 (fig. 121). Reports prepared after the flood noted only a small amount of rock placed in 1966 around pier 9 was still visible. None of the rock displaced is visible downstream from the bridge, however, and it is possible that the migration of the rock is as much vertical (settling in the channel bed) as horizontal. Repairs consisted of replacing the displaced rock riprap with 2½-ton riprap.

1977 Surveys of the channel indicate that degradation is continuing, but the pier footings and rock riprap appear in good condition.

Discussion: This stream with a degrading channel poses special problems in bridge design and maintenance because the ultimate extent of channel change, and the time required for stability, is difficult to determine. Concrete grout walls, built to extend the depth of footings, provided protection and support of piers because the wall is extended below the depth of potential scour or degradation. Heavy rock riprap placed around the piers, of sufficient size to remain in position, provided protection of the pier against scour but also occupied a part of the channel, thereby reducing the capacity of the waterway.

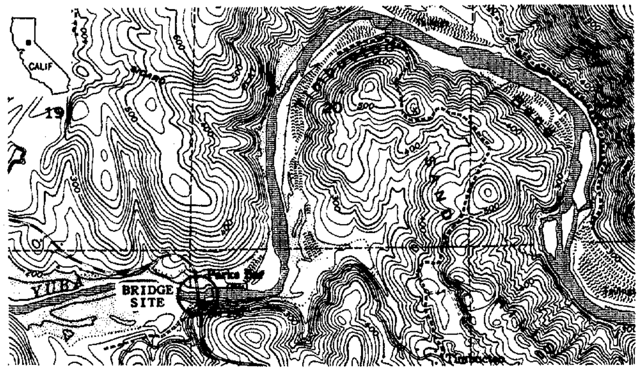


Figure 115. Location of Yuba River at SR-20 near Smartville. (Base from U.S. Geol. Survey Smartville, Calif., 7.5' quadrangle, contour interval 20 feet, 1951 revised 1973.)

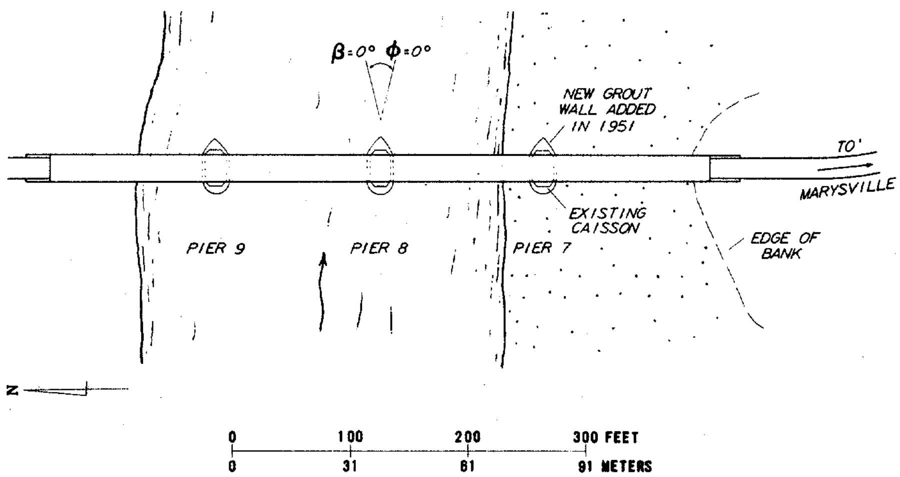


Figure 116. Plan of Yuba River at SR-20 near Smartville.

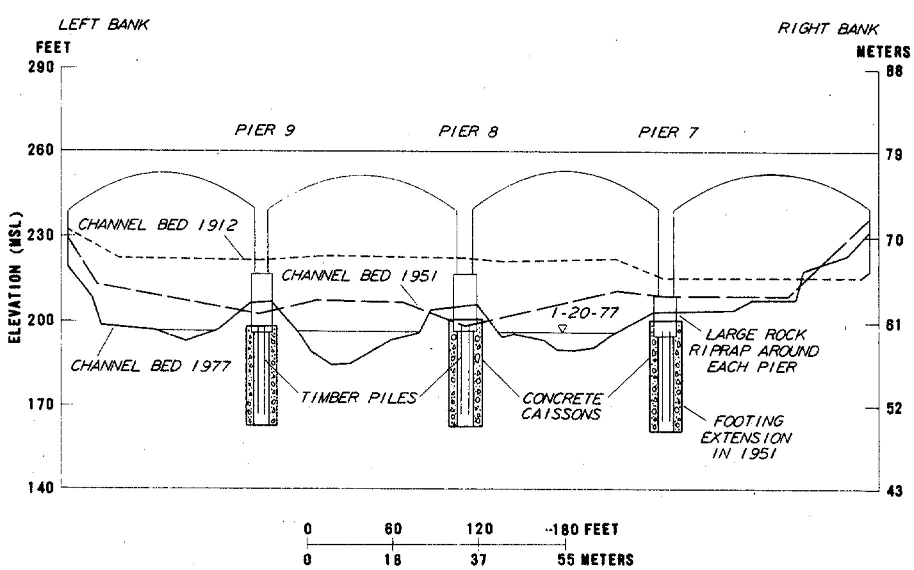


Figure 117. Cross sections of Yuba River at SR-20 in 1912, 1951, and 1977.

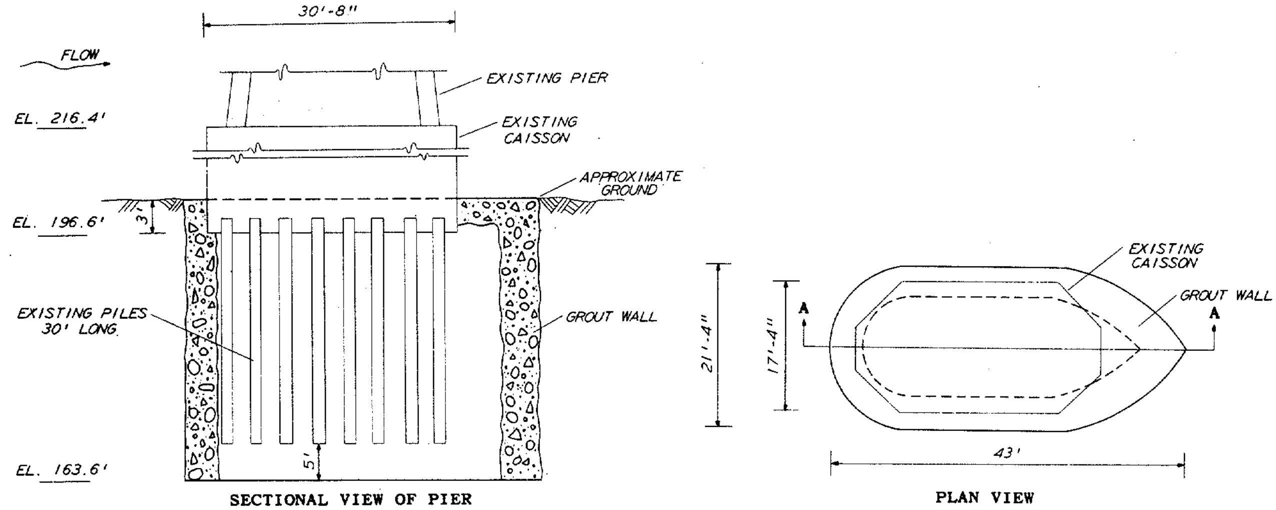


Figure 118. Detail of piers 7, 8, and 9 after extension in 1951.

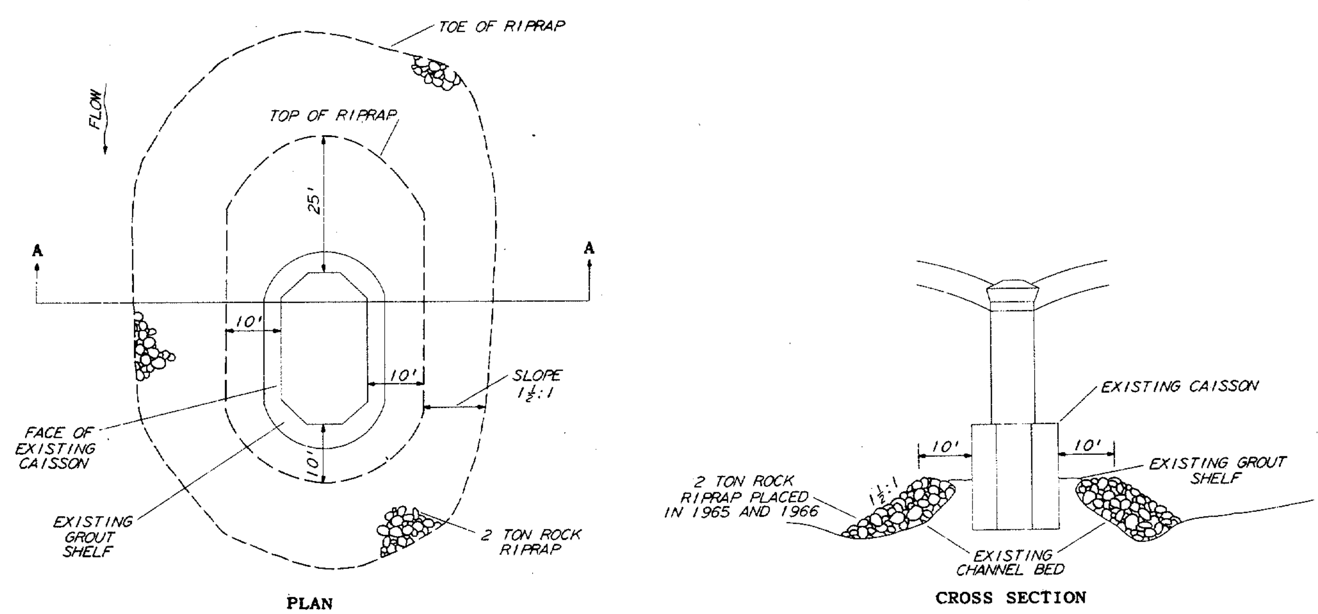


Figure 119. Details of repairs at piers in 1965 and 1966.

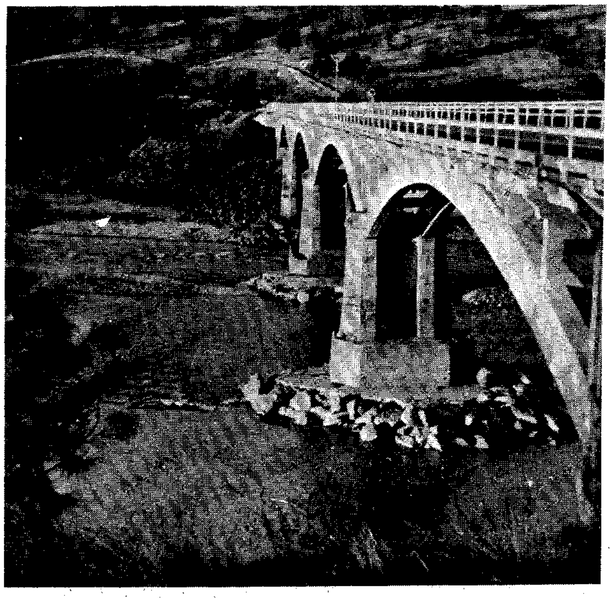


Figure 120. View from left bank of rock riprap around pier, about 1965. (From Calif. Dept. of Transportation.)

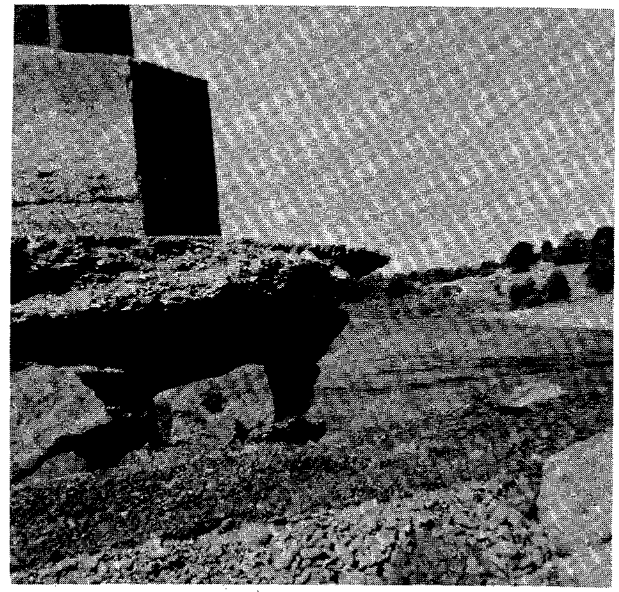


Figure 121. Scour around pier footing in 1970. (From Calif. Dept. of Transportation.)

Figure 122. Debris on upstream end of bridge near right bank on Dec. 13, 1937. (From Nevada Dept. of Highways.)

SITE 25. BROCKLISS SLOUGH AT SR-88 NEAR MINDEN, NEV.

Description of site: Lat 38°53;, long 119°47'. Timber bridge, constructed in 1935, destroyed during flood of December 1950. New concrete bridge built in 1952.

Valley slope, 0.0029; channel width, about 80 ft (24 m). Stream is perennial, alluvial, sand bed, in valley of low relief, wide flood plain. Channel is sinuous, irregular width variation, silt-sand banks.

Hydraulic problems and countermeasures:

1935 Timber bridge constructed, 75 ft (23 m) long, supported by timber pile bents spaced at 18 ft (5.1 m). Abutments were timber bulkheads supported by timber piles. Clearance of bridge stringers above the channel bed averaged 4.5 ft (1.4 m). No countermeasures.

1937 Flood of December 11, 1937, recurrence interval of 30 yr, deposited large amounts of debris against bridge. Right-bank abutment and half the approach roadway width were eroded (fig. 122).

1940 Bridge maintenance reports indicate channel aggraded with sand to about 4 ft (1.2 m) below stringers. Reports in 1944 indicate two center spans partly blocked by sandbar.

1950 Flood of December 3, 1950, caused a large amount of drift to collect at bridge due to inadequate clearance of bridge members above flood level.

1952 New concrete bridge constructed, 130 ft (40 m) long, supported on concrete piers with pile foundations. Abutments and streambanks adjacent to bridge were protected by concrete slope paving. The slope paving is 4 in (0.10 m) thick with wire mesh reinforcing with cutoff walls placed at each end and paving extended into a trench below the channel bottom.

1955 Flood of December 23, 1955, R.I. about 100 yr, caused no damage to bridge or concrete slope paving (fig. 123). This flood did cause lateral erosion downstream from the slope paving and damaged concrete irrigation headgates several hundred feet downstream from the bridge.

Discussion: The older bridge had inadequate clearance for debris and apparently was damaged as a result of the following factors: (1) force of water and debris against the bridge members, (2) accumulation of drift that diverted flow away from the bridge opening, causing erosion of streambank and approach embankment erosion, and (3) accumulation of drift that reduced flow velocities upstream from the bridge and caused aggradation, with consequent reduction in flow capacity.

The slope paving was an effective countermeasure to protect bridge abutments and streambanks because the foundation soil had not been eroded, adequate cutoff walls were present at toe and upstream and downstream ends of slope paving, and weepholes were provided to prevent hydrostatic pressure behind the slope paving.

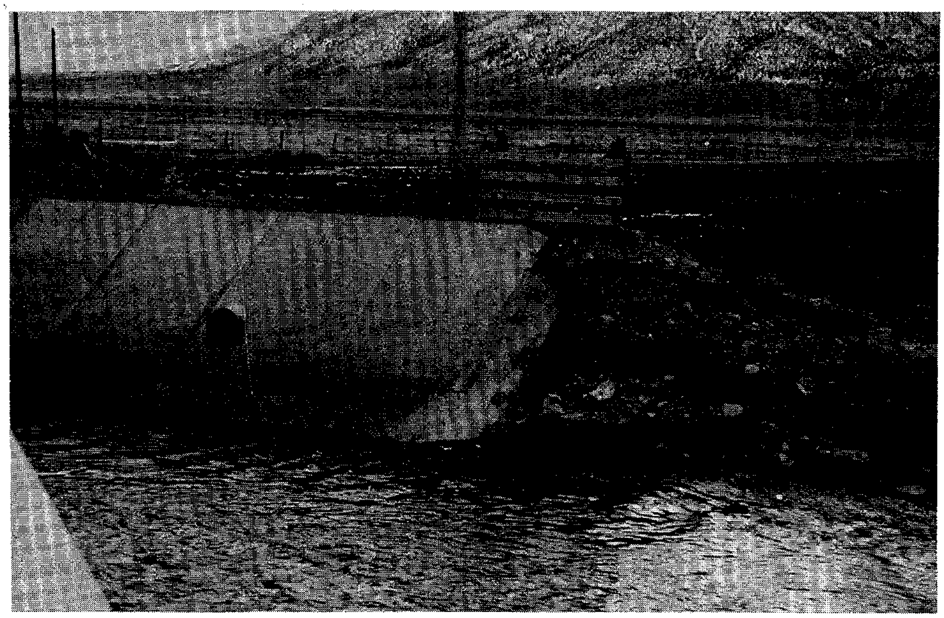


Figure 123. Slope paving left bank at downstream side of bridge on Dec. 27, 1955. Note lateral erosion of bank downstream from end of slope paving. (From Nevada Dept. of Highways.)

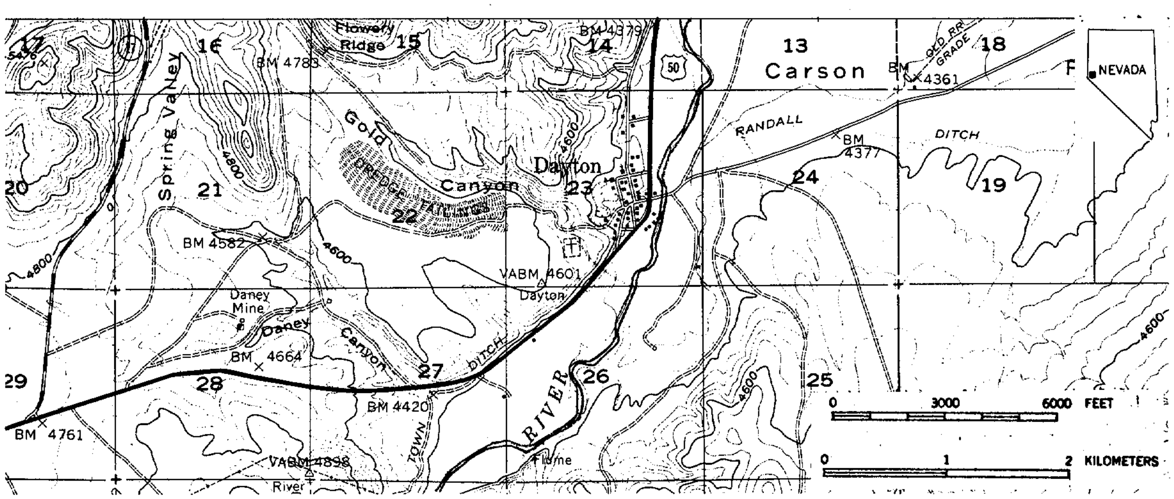


Figure 124. Location of Carson River crossing (circled) at Dayton. (Base from U.S. Geol. Survey Dayton, Nev., 15' quadrangle, contour interval 40 feet, 1956.)

SITE 26. CARSON RIVER AT DAYTON LANE AT DAYTON, NEV.

Description of site: Lat 39°14;, long 119°35', location as shown in fig. 124. Concrete bridge supported on two concrete piers was destroyed during flood of 1955. A new concrete bridge was built in 1956, as described below.

Drainage area, about 1,000 mi^2 (2,590 km^2); valley slope, 0.0033; channel width, 180 ft (55 m). Stream is perennial, alluvial, gravel bed, in valley of high relief, narrow flood plain. Channel is sinuous, locally braided, wider at bends, cut banks local, sand-gravel banks, tree cover at less than 50 percent of bankline.

Hydraulic problems and countermeasures:

1951 Concrete bridge constructed, 132 ft (40 m) long supported on two concrete piers and concrete abutments (fig. 125). The two end spans are 36 ft (11 m) long and center span is 40 ft (12 m) long. Piers and abutments are on spread footings. The lef-bank abutment is a stub-type with wingwalls, and the right-bank abutment is a spillthrough type protected by concrete slope paving.

1955 Flood of December 24, 1955, R.I. about 70 yr, eroded several feet from the left bank near the wingwalls. Most of the slope paving on the right bank was destroyed, and the slope paving was transported downstream from the bridge site. Both piers in the channel settled several feet, causing complete collapse of the bridge (fig. 126). Flows through the bridge were near supercritical, and shear stresses during the flood were sufficient to cause movement of bed and bank material.

1956 Concrete bridge constructed, 212 ft (65 m) in length, supported by two piers and concrete stub abutments with wingwalls. Center span is 87 ft (26 m) long. Piers are supported by steel H-piles. Sacked concrete was placed on both banks upstream and downstream from the wingwalls (fig. 127) to prevent bank scour.

1963 Flood of February 1, 1963, recurrence interval about 40 yr, caused no damage.

Discussion: The bridge in 1951 constricted floodflow such that shear stresses caused extensive bank and bed scour. Flows were near supercritical at the bridge opening. The slope paving on the right bank was almost entirely washed away during the 1955 flood. Concrete slope paving, when in the process of failure, does not have the self-healing properties of rock riprap. Instead of dropping into areas of erosion in the streambed or along the streambanks, the concrete slabs tended to be carried downstream. The use of sacked concrete to protect the streambanks was effective because the flow area at the new bridge was sufficiently large to prevent excessive flow velocities.

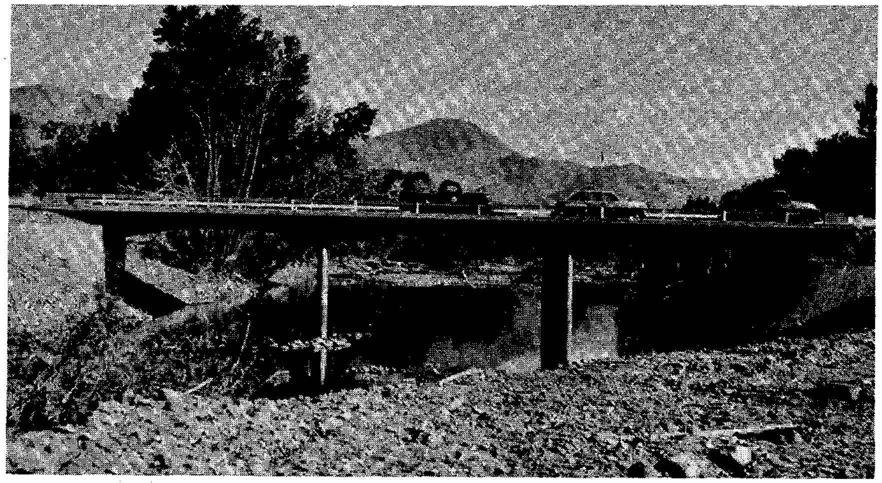


Figure 125. Abutment protection at bridge built in 1951. (From Nevada Dept. of Highways.)

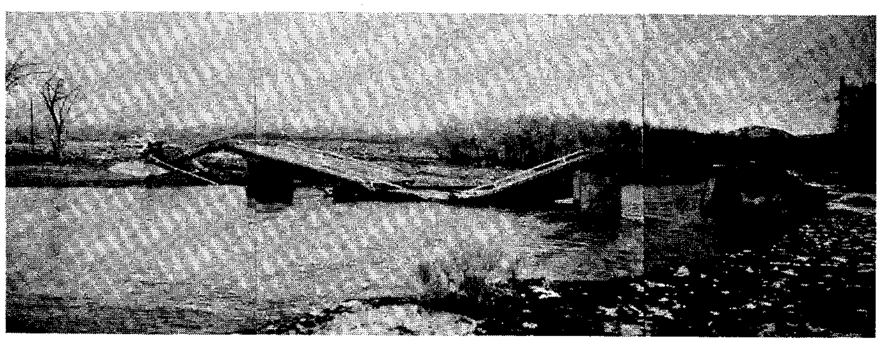


Figure 126. View of collapsed bridge, scoured piers and eroded abutments following flood of December 24, 1955. (From Nevada Dept. of Highways.)

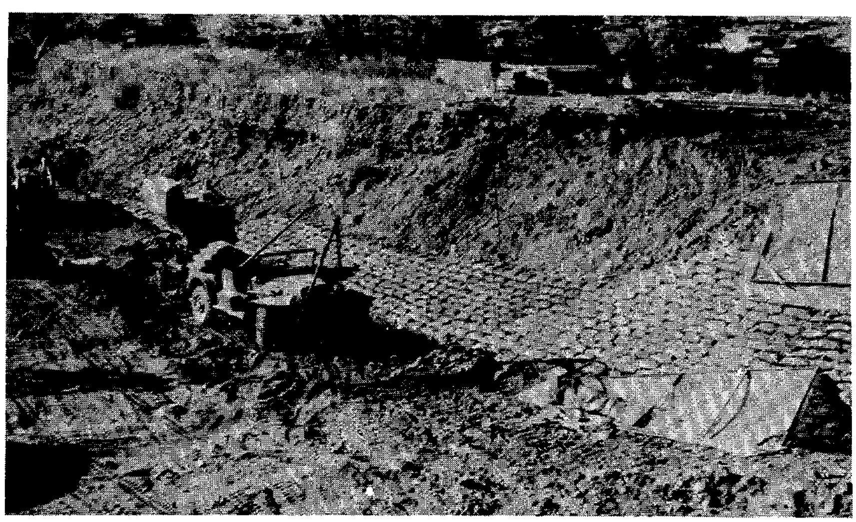


Figure 127. Installation of sacked concrete riprap on upstream leftbank side of bridge in 1956. (From Nevada Dept. of Highways.)

SITE 27. EAST FORK CARSON RIVER AT SR-56 NEAR GARDNERVILLE, NEV.

Description of site: Lat 38°56', long 119°45'. Concrete bridge was destroyed during 1955 flood. New concrete bridge was built in 1956 and placed at same location, as described below.

Drainage area, 280 mi^2 (725 km^2); valley slope, 0.0064; channel width, 135 ft (41 m). Stream is perennial, regulated, alluvial, gravel bed, on piedment slope. Channel is sinuous, generally braided, gravel banks.

Hydraulic problems and countermeasures:

1936 Concrete bridge, 103 ft (31.4 m) in length, two concrete piers, full-height concrete abutments with wingwalls, no other countermeasures. Bridge spans are 34, 35, and 34 ft (10.4, 10.7, and 10.4 m). Piers and abutments are founded on spread footings with bottom 5 ft (1.5 m) below channel bed.

1937 Flood of December 11, 1937, discharge 11,000 ft^3/s (311.5 m^3/s), R.I. 30 yr, washed over the roadway and around the right-bank abutment causing settlement of the bridge. Overbank flow on the left-bank flood plain forced the main channel flow to concentrate near the right bank. Following the flood, the channel was cleared of debris, and an earthfill dike placed along the upstream right bank to prevent erosion of the right-bank abutment.

1950 Flood of November 1950, discharge 10,000 ft^3/s (283.2 m^3/s), R.I. 30 yr, eroded the channel bed sufficiently to cause about 6 in (0.15 m) settlement of the right-bank pier.

1955 During flood of December 1955, discharge 17,000 ft^3/s (481.4 m^3/s), R.I. 100 yr, logs and debris piled up near right-bank abutment. Footings of both piers scoured causing the bridge to collapse (fig. 128). The earthfill dike placed on the right bank upstream from the bridge, as well as the debris lodged against the right side of the bridge, apparently prevented scour of the right-bank abutment such as occurred in 1937.

1956 Concrete bridge built, following collapse of previous bridge during 1955 flood. Bridge is 173 ft (52.7 m) long and supported by two concrete piers and vertical abutments with wingwalls, founded on steel H-piles. Concrete slope paving was installed on both banks upstream from bridge (fig. 129), and an earthfill dike protected with large boulders pushed from the streambed was constructed on the left bank downstream from the bridge.

1963 During flood of February 1963, discharge 12,000 ft^3/s (339.8 m^3/s), R.I. 50 yr, caused no damage to slope paving of bridge.

1964 Flood of December 1964, discharge 7,000 ft^3/s (198.2 m^3/s), R.I. 25 yr, caused large amounts of erosion of streambanks both upstream and downstream from bridge. Erosion of both abutments and the left-bank approach embankment also occurred. Failure of the upstream left-bank concrete slope paving probably occurred because erosion of the left streambank 600 ft (183 m) upstream from the bridge allowed flows to overtop the flood plain and approach the bridge along the approach fill at a large skew angle, causing overtopping of the slope paving near the bridge. The right bank slope paving upstream from the bridge was also damaged by erosion owing to shifting of the flow alinement approaching the bridge.

1965 Eroded parts of streambank were repaired by placement of dirt and rock along the channel.

1976 No significant floods have occurred since 1965.

Disucssion: Addition of another span to the bridge increased the size of waterway and helped prevent lodgment of debris at the bridge. Changes in flow alinement upstream from the bridge caused water to flow along the left-bank approach embankment. The overland flow reentered the main channel after overtopping and crossing the slope paving, which subsequently failed by erosion normal to its alinement. Placement of concrete slope paving and dikes along the channel bank was effective in preventing damage to the bridge as long as flows were confined to the channel alinement that was assumed in the design of the bridge and countermeasures.

Figure 128. Failure of piers founded on spread footings, during Dec. 1955 flood. (From Nevada Dept. of Highways.)

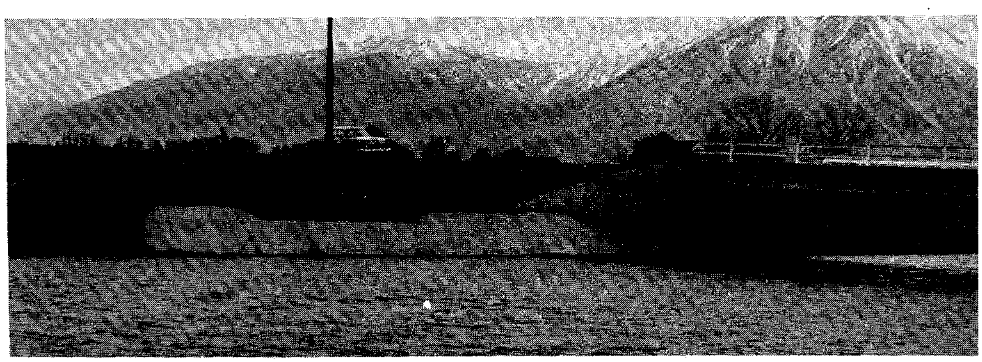


Figure 129. Slope paving on left bank at upstream side of bridge on May 8, 1957. (From Nevada Dept. of Highways.)

SITE 28. HUMBOLDT RIVER AT AIRPORT ROAD AT LOVELOCK, NEV.

Description of site: Lat 40°11', long 118°27'. Concrete bridge, 132 ft (40 m) long, three spans, two concrete piers supported by timber piles, spillthrough abutments. Channel bed and banks at bridge paved with concrete to prevent scour.

Drainage area, 16,000 mi^2 (41,440 km^2); valley slope, 0.0012; channel width, about 125 ft (38 m). Stream is perennial, regulated (but subject to high releases of water at times), alluvial, gravel bed, in valley of high relief, wide flood plain. Channel is meandering, wider at bends, point bars, cut banks general, erodible silt-sand banks.

Hydraulic problems and countermeasures:

1950	At time of bridge construction (fig. 130), both abutments were protected by concrete slope paving placed around the approach embankment. The bridge opening imposed some constriction on the waterway (fig. 131).
May 2, 1952	High flows, due to snowmelt and rain in Humboldt River basin and subsequent release from an upstream reservoir between May 2 and 16, 1952, caused water to overtop the concrete slope paving.
May 7 and 8, 1952	Sacked concrete installed to height of about 5 ft (1.5 m) on top of slope paving on both banks.
May 12, 1952	Water undercut sacked riprap on left bank upstream side of bridge. In addition, by May 16, the sacked concrete was washed away from the left bank downstream side of bridge causing failure of slope paving (fig. 132).
May 16, 1952	Rock riprap was dumped into scoured area on left bank downstream side of bridge (fig. 133).
February 1976	Channel appears stable.

Discussion: Erosion damage to the bridge abutments was a serious problem because flows that were contracted by the bridge created eddies along the streambanks downstream from the bridge. Concrete slope paving, used for protection of the bridge abutments, performed well until overtopped and subjected to reverse flows in the eddy downstream from the bridge. Damage to the slope paving on the downstream side of the left-bank abutment was apparently due to erosion of the supporting earthfill underneath and at top of the slope paving.

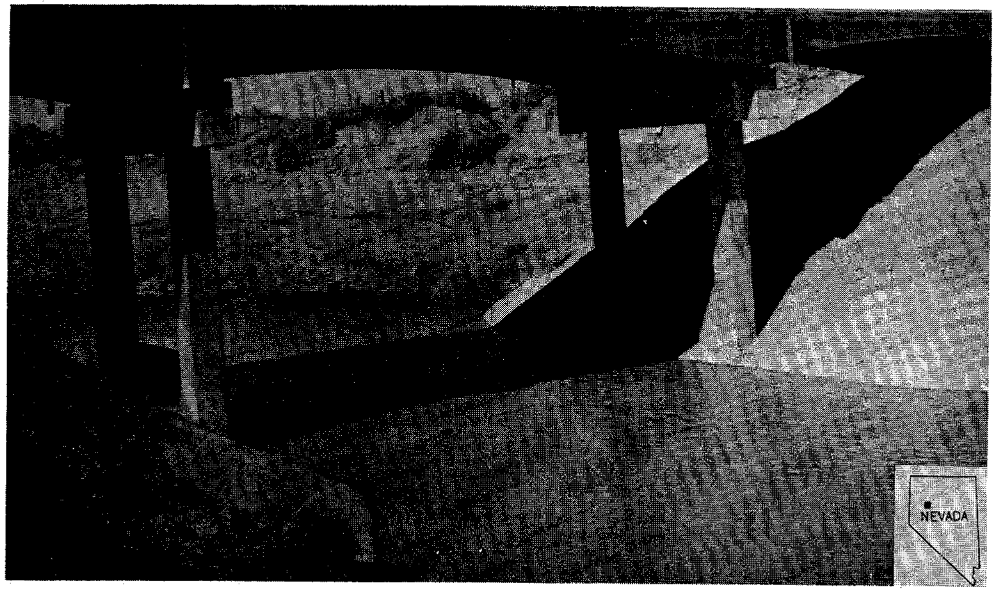


Figure 130. Pavement on abutment slopes on April 17, 1952. (From Nevada Dept. of Highways.)

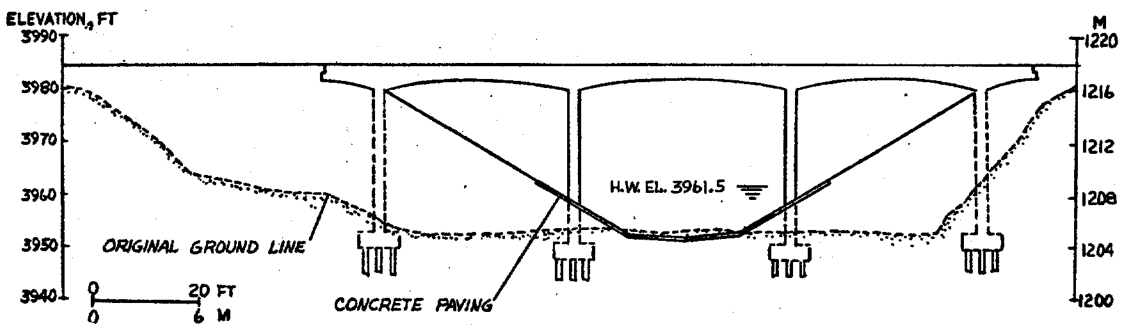


Figure 131. Cross section of Humboldt River bridge.

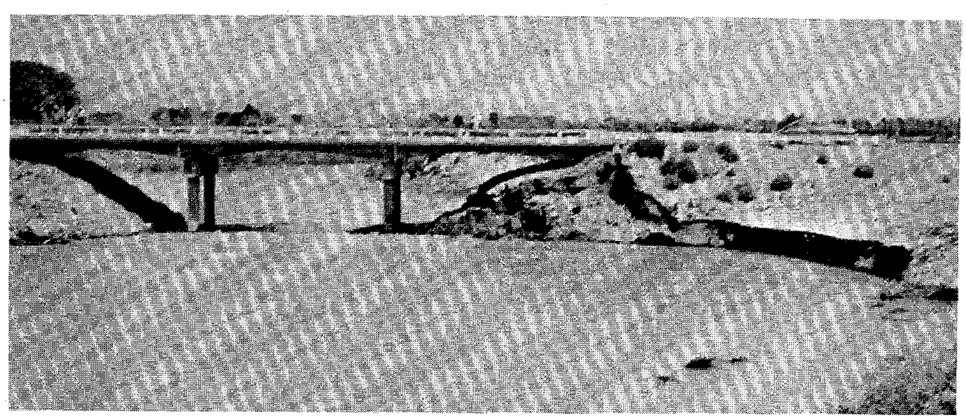


Figure 132. View upstream at bridge during flood of May 2-16, 1952. Note flow constriction caused by abutments and lateral erosion of left bank downstream from bridge. (From Nevada Dept. of Highways.)

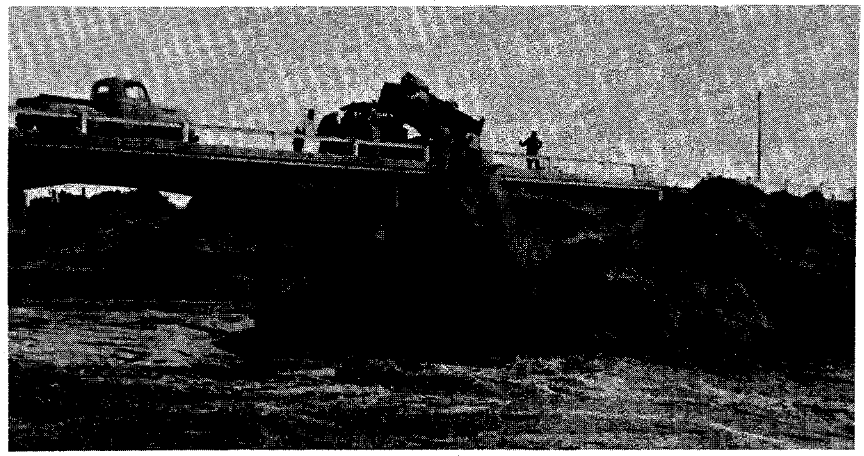


Figure 133. Erosion of sacked concrete from left bank on downstream side of bridge. Note fall in water surface beneath bridge and eddy near left bank causing lateral erosion of approach embankment. (From Nevada Dept. of Highways.)

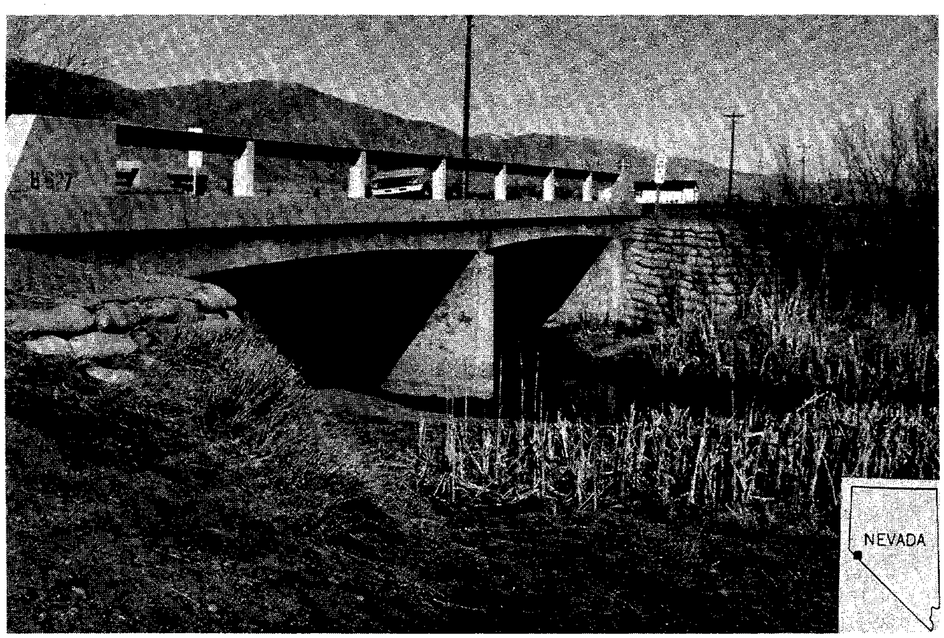


Figure 134. Sacked concrete revetment on upstream side of SR-88 bridge at Rocky Slough, February 1976. (From Nevada Dept. of Highways.)

SITE 29. ROCKY SLOUGH AT SR-88 NEAR MINDEN, NEV.

Descritpion of site: Lat 38°55', long 119°47'. Timber bridge built in 1934, 35 ft (11 m) long, and supported on timber-pile bent at midspan. Abutments are timber pile supported with bulkhead protection. Bridge replaced in 1957 with a 64 ft (20 m) long concrete bridge on concrete piers supported by pile foundation. Bridge crosses a slough that carries a small part of the overflow from East Fork Carson River.

Drainage area, about 280 mi^2 (725 km^2), most of which is non-contributing; valley slope, 0.0033; channel width, about 35 ft (11 m). Stream is perennial, alluvial, sand bed, on piedment slope. Channel is sinuous, equiwidth, incised, silt-sand banks.

Hydraulic problems and countermeasures:

1934 Bridge built, without countermeasures.

1937-50 Floods in December 1937, and November and December of 1950, washed out approaches to bridge at both ends.

1955 Flood of December 23, 1955, discharge about 1,000 ft^3/s (28 m^3/s), caused major damage to bridge. Bridge size was inadequate to handle flows of this magnitude.

1957 Concrete bridge, 64 ft (20 m) long was built and supported by a concrete pier placed at midspan on piles. Abutments are vertical (full-height). Both abutments and approach roadway are protected by sacked concrete revetment (fig. 134).

1963 Flood of February 1963, 900 ft^3/s (25 m^3/s), R.I. about 50 yr caused no damage to bridge or approach roadway.

1976 Maintenance report indicates the sacked concrete is in excellent condition, and has prevented any damage to the abutments or roadway approach.

Discussion: Use of sacked concrete riprap at abutments was apparently effective in preventing damage to bridge by a flood of 50-yr R.I.

SITE 30. CHEMUNG RIVER AT LR-08066 NEAR SAYRE, PA.

Description of site: Lat 41°59', long 76°33', location as shown in fig. 135. The principal structure at this site is a six-span, plate-girder bridge built in 1972-73. The bridge has a length of 700 ft (213 m) and is supported by five 3-ft (0.9-m) wide, round-nose, hammerhead piers which are alined with flow and are founded on piles. Abutments are spillthrough with 45° stub wingwalls and founded on piles. The structure is located between two streambends in the Chemung River. An approach embankment extends across the low wide left-bank flood plain and the bridge is skewed 22°. In 1973, a spur dike was constructed at the end of the approach embankment where overbank flow constriction during June 1972 flooding caused severe scour problems at the partially constructed left abutment.

Drainage area, 2,570 mi^2 (6,656 km^2); bankfull discharge, about 30,000 ft^3/s (849 m^3/s); valley slope, 0.00073; channel width, about 350 ft (107 m). Stream is perennial, alluvial, cobble bed, in valley of moderate relief, narrow flood plain. Channel is sinuous, locally braided, equiwidth, cut banks general, bank material silt-clay where banks intersect modern flood plain, fine to medium gravel where banks intersect low terrace. Tree cover at less than 50 percent of bankline. A small tributary to the river, Dry Brook, was relocated across the left flood plain upstream from the bridge because of approach embankment and spur-dike construction.

Hydraulic problems and countermeasures:

1972 Bridge construction was begun. During flooding in June 1972, severe scour occurred at the partially completed bridge. Maximum depth of scour was estimated to be 15 ft (4.6 m) in loose gravel around the pilings at the left abutment, although no structural damage occurred. Only the concrete abutment and stub wingwall structure was in place at time of flooding, and no compacted backfill or embankment material had yet been placed. Bank erosion and scour problems also developed at the right (western) bridge abutment and adjacent bridge pier, which were also under construction at time of flooding.

1973 The left approach embankment was completed and countermeasures were constructed as follows: (1) Spur dike constructed upstream from the left abutment according to recommended quarter ellipse design having a 2.5:1 ratio of major to minor axis. Its length is 237 ft (72.4 m) and high water elevation was designed to be 779.0 ft (237.4 m). The spur dike was constructed using compacted embankment fill material and was protected with rock facing (fig. 136). This riprap facing (D_{50} about 1.25 ft or 0.38 m) was built completely around the upstream nose and along the entire length of the spur dike; it extends through the abutement opening and around the downstream side of the approach embankment. (2) A thick sod cover protects all other exposed areas of the embankment and spur dike. (3) Rock riprap also was extensively used to protect the high right streambank. The revetment extends from a point 500 ft (150 m) upstream from the right abutment through the river bend to a point 200 ft (61 m) downstream from the bridge. The revetment was placed from the top of the streambank to the bottom of the channel. (4) A rock terrace was keyed into alluvium along the length of the approach embankment to prevent possible underscour.

SITE 31. CONESTOGA CREEK AT SR-23 NEAR CHURCHTOWN, PA.

Description of site: Lat 40°08', long 75°59'. Reinforced concrete T-beam bridge was built in 1925; length 60 ft (18 m), two 30-ft (9 m) spans, one 3-ft (0.9 m) wide sharp-nosed pier, vertical abutments, 45° upstream wingwalls. Both pier and abutments were founded on spread footings. Design clearance above waterway, 6.0 ft (1.8 m). This bridge was destroyed during Hurricane Agnes flood in 1972. A new bridge was completed in 1973 as a single span with vertical abutment foundation piles, and 45° wingwalls on upstream and downstream sides of bridge. The bridge, a single-span reinforced concrete box-beam is 60 ft (18 m) long with 6 ft (1.8 m) clearance above the waterway.

Drainage area, 26 mi^2 (68 km^2); valley slope, 0.0025; channel width, about 70 ft (21 m). Stream is perennial, alluvial, gravel bed, in valley of low relief, wide flood plain. Channel is sinuous, silt-clay banks. The bridge is at a gentle bend in the stream and has a 20° skew to flood flow. Some constriction caused by the approach embankment occurs during overbank flow.

Hydraulic problems and countermeasures:

1925 Two-span bridge was built, without countermeasures.

1972 Flood during Hurricane Agnes on June 22, 1972 caused scour and undermining of pier and abutments and consequent roadway surface tilt. The bridge sustained structural damage, due to settling, which necessitated replacement.

1973 New bridge completed. Countermeasures included design of bridge as a single-span structure with no piers, wingwalls added on both upstream and downstream sides of the bridge, and support of abutments on piles.

1975 Flooding during tropical storm Eloise September 24, 1975 caused a peak flow of 1,470 ft^3/s (42 m^3/s) (R.I. about 4 yr). No damage to the structure was documented.

1976 No further high flows have occurred to test the countermeasures.

Discussion: The absence of center support piers has eliminated the problem of local scour around piers and debris lodging at bridge. The addition of wingwalls on both sides of this bridge should further protect the abutment from scour. Use of pile-supported abutments for the bridge built in 1973 probably prevented damage to bridge during 1975 flood. Use of spread footings to support the older bridge proved satisfactory for a 47-year period (1925-72), but scour and undermining of the footings, with subsequent bridge failure, occurred during a flood with a recurrence interval greater than 100 years.

1975 Peak flow resulting from tropical storm Eloise in September 1975 was efficiently channeled through the constricted bridge opening. A large scour hole developed in the flood plain immediately upstream from nose of the spur dike, where its radius of curvature is sharpest and where overbank flow impingement was greatest (figs. 135 and 137). However, the scour hole in no way endangers the bridge or spur dike.

1977 At a field inspection in January 1977, performance of the bridge and countermeasure was judged to be effective.

<u>Discussion</u>: The spur dike worked effectively as a countermeasure to protect the approach embankment, abutment, and pier from scour caused by the overbank flow constriction. The scour hole formed at the toe of the spur dike is not considered detrimental to the safety of the bridge or spur dike. The rock revetment used along the outside of the bend upstream, and downstream from the bridge adequately protected the channel bank during the 1975 flood.

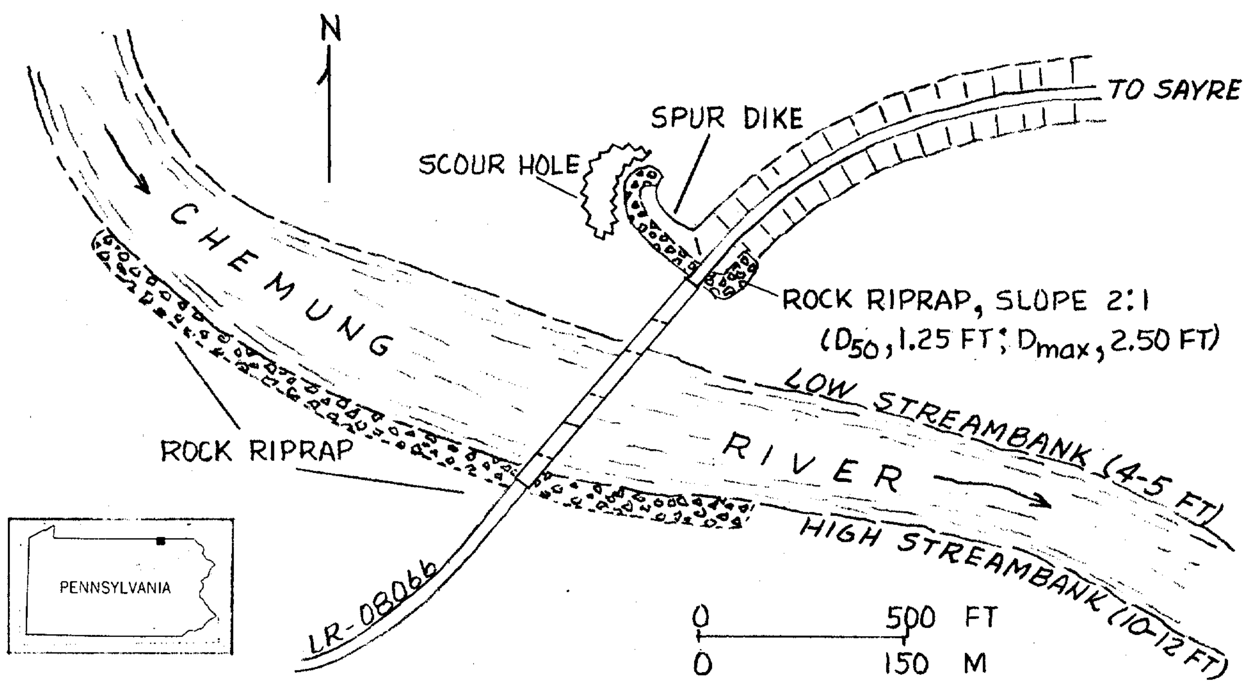


Figure 135. Plan sketch of Chemung River at LR-08066 crossing.

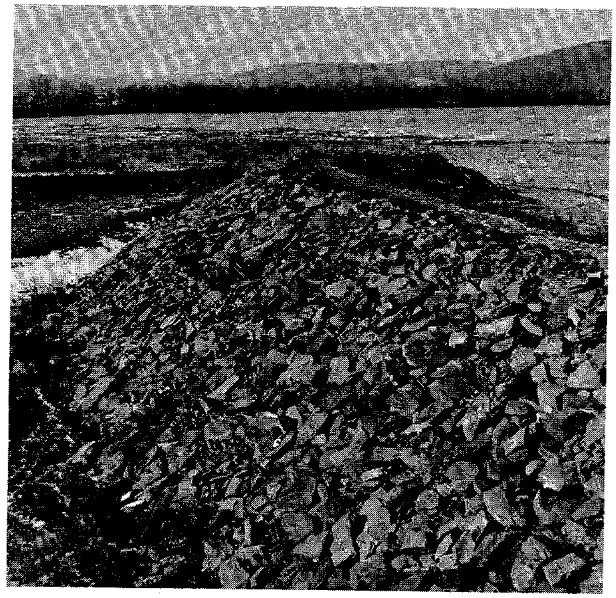


Figure 136. View upstream in 1977 of spur dike and rock riprap protection.

Figure 137. View in 1977 of scour hole at nose of left-bank spur dike.

SITE 32. CONESTOGA CREEK AT SR-10 NEAR MORGANTOWN, PA.

Description of site: Lat 40°08', long 75°54'. A reinforced concrete T-beam bridge with two 40-ft (12.2 m) spans, 11 ft (3.4 m) clearance above design floodstage and a 3 ft (0.9 m) wide sharp-nosed center pier was built in 1929. During the flooding of Hurricane Agnes in 1972, the left abutment scoured and collapsed. A new structure, built in 1973, is a single-span prestressed-concrete box-beam bridge with one clear span of 60 ft (18.3 m) and 10.6 ft (3.2 m) clearance above the waterway. Wingwalls were added at both abutments on the upstream and downstream side. Abutments are vertical and are founded on steel piles.

Drainage area, 17 mi^2 (44 km^2); bankfull discharge, 850 ft^3/s (24 m^3/s); valley slope, 0.0018; channel width, 80 ft (24 m). Stream is perennial, alluvial, sand bed, in valley of moderate relief, wide flood plain. Channel is sinuous, cut banks general, silt-clay banks. The normal flow is slightly constricted by the bridge waterway, and overbank flow is very constricted by the approach embankments.

Hydraulic problems and countermeasures:

1929 Two-span bridge built with upstream wingwalls set at 45° angle.

1972 Hurricane Agnes flood, discharge 7,770 ft^3/s (220 m^3/s), clogged the right span of the bridge with debris causing the channel to shift to the left and constricting the flow through the left span. The left abutment was scoured until it collapsed.

1973 New single-span bridge built with 45° upstream and downstream wingwalls (fig. 138).

1975 Flooding during tropical storm Eloise in September 1975, discharge 1,120 ft^3/s (32 m^3/s), caused no debris pileup; however, the natural waterway area was reduced by the new bridge thus constricting flows that exceed bankful stage. As a result, a scour problem has occurred at the abutments and the upstream wingwalls and deposition of material in midchannel (fig. 138) has occurred because of turbulence caused by the overbank flow returning to the main channel. No countermeasures have been used for the recently noted scour problem.

Discussion: Removal of the center pier reduced the chance for accumulation of debris at the bridge and consequent scour at the piers. The new bridge is shorter and more constrictive to flood flow, but the wingwalls should help prevent scour at the toe of abutments due to flow constriction. No large floods have occurred to test the adequacy of the new bridge design.

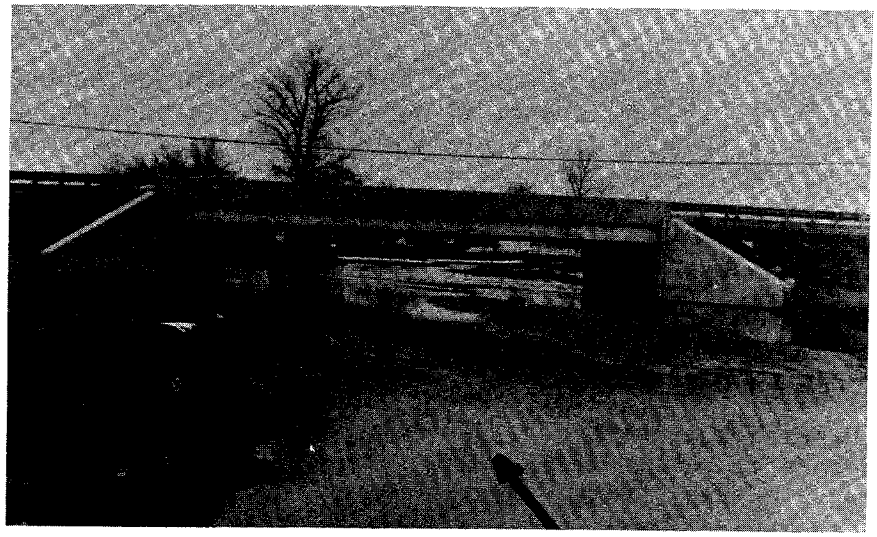


Figure 138. View in 1976 of full-height abutments. Mid-channel bar upstream from bridge is attributed to turbulence near abutments.

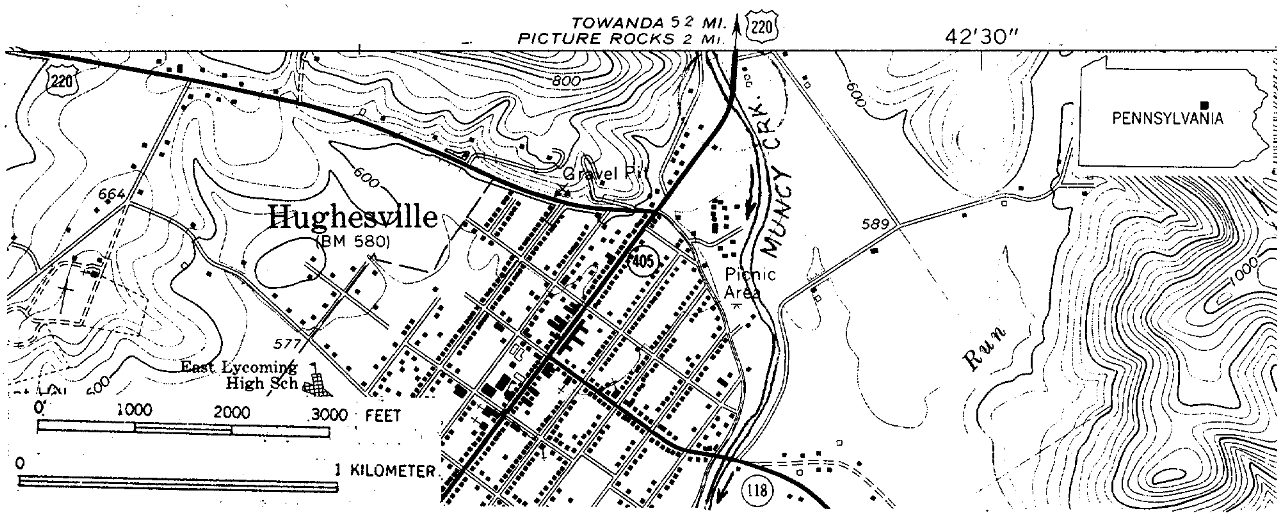


Figure 139. Location of US-220 crossing (circled) on Muncy Creek. Base from U.S. Geol. Survey Hughsville, Pa., 7.5' map, contour interval 20 ft, 1968.)

SITE 33. MUNCY CREEK AT US-220 AT HUGHESVILLE, PA.

Description of site: Lat 41°15', long 76°43', location as shown in fig. 139. The bridge is a three-span, prestressed-concrete, I-beam structure, 232 ft (71 m) long. The two piers are wall-type, 3 ft (0.9 m) wide with round nose. The abutments are spillthrough, founded on piles driven into the approach embankment. The skew of crossing is 30°. The piers are alined to floodflow, but the bridge crosses Muncy Creek at a bend where the channel alinement appears unstable.

Drainage area, 82 mi^2 (214 km^2); valley slope, 0.0041; channel width, 200 ft (61 m). Stream is perennial, alluvial, cobble bed, in valley of moderate relief, wide flood plain. Channel is sinuous, wider at bends, point bars, silt-sand banks, tree cover at 50-90 percent of bankline.

Hydraulic problems and countermeasures:

1972 The original bridge at this site was destroyed during flooding from Hurricane Agnes.

1973 A new bridge was constructed at the same site with the addition of extensive streambank protection. Bulldozers were used to clean and deepen the flood-damaged channel after 1972 flooding; streambed material was used to rebuild the streambanks. The outsides of stream bends in the bridge site area were then protected with selected limestone rock riprap with a D_{50} of about 2.0 ft (0.6 m).

1975 On September 26, 1975, a flood caused by tropical storm Eloise provided a severe test for the new bridge and the bank-protection measures. The stream thalweg shifted to the right during flooding, resulting in scour at the bridge pier closest to the outside of the stream bend (pier 1) (fig. 140). Much cobble bed material was deposited in the middle and inner bridge spans along the inside of the stream bend. The channel of Muncy Creek is still poorly alined with the bridge opening; streamflow is channeled directly into the scoured area around pier 1, resulting in severe eddy currents at the pier even during normal flows (fig. 141).

Discussion: Use of rock riprap with D_{50} of 2 ft (0.6 m) has successfully prevented bank erosion on the outside of the bend at the bridge. No countermeasures have been installed to solve the pier scour problem caused by poor channel alinement. Solutions used in similar situations include spurs of rock riprap and elevated flood plains to improve and maintain stable flow alinement through the center of the bridge. The bridge piers were placed with good alinement to floodflow but were skewed to the low-water channel, causing local scour problems.

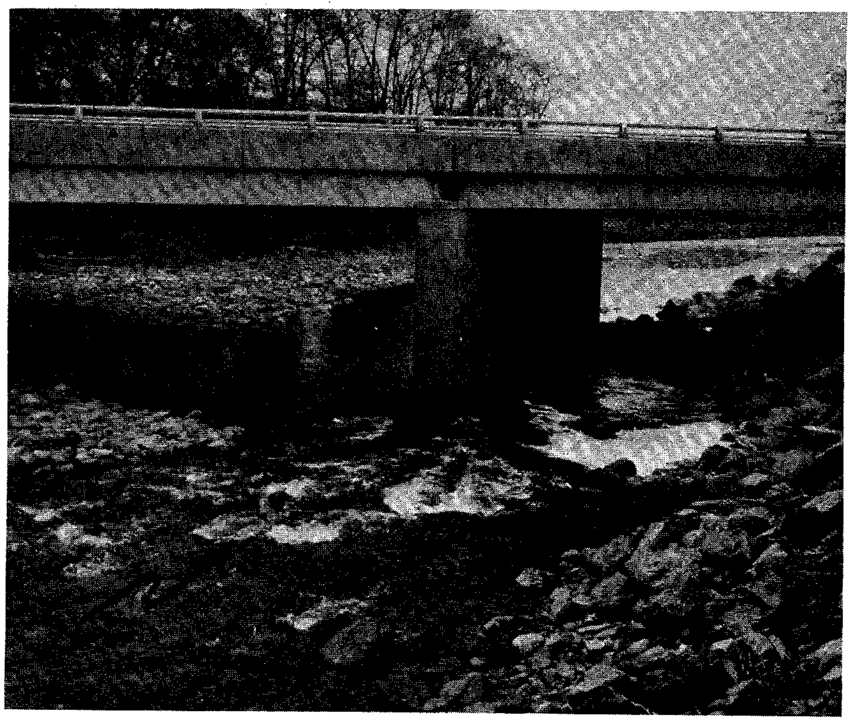


Figure 140. Flow alinement associated with scour at pier 1, as photographed in 1976.

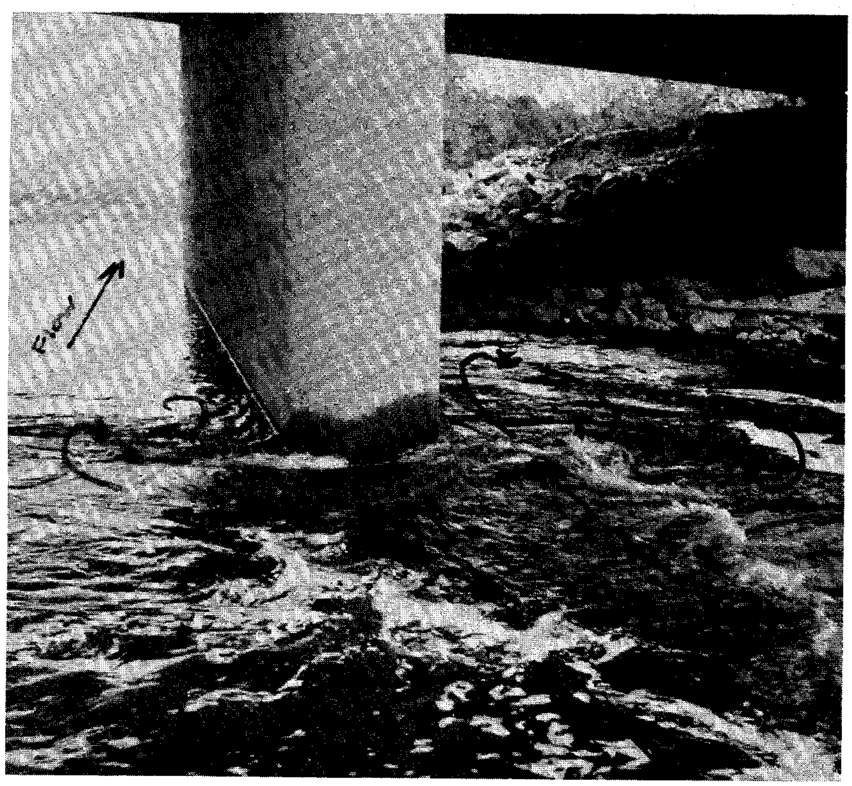


Figure 141. Eddy currents at pier 1, and lateral erosion of right bank, as photographed in 1976.

SITE 34. MUNCY CREEK AT LR-1073 NEAR MUNCY, PA.

Description of site: Lat 41°13', long 76°47', location near crossing in fig. 139. Two parallel, three-span, prestressed-concrete I-beam bridges built late in 1972. Each bridge is 210 ft (64 m) long. The bridges are supported by two wall-type piers each 3 ft (0.9 m) wide, with rounded nose, set on spread footings in alluvium. Abutments are spillthrough. The right abutment is set into bedrock along the streambank. The left abutment is founded on piles driven deeply into the approach embankment which is raised above the flood plain. The approach roadway is well alined with the bridge, although the embankment crosses the left-bank flood plain and would severely constrict overbank flow.

Drainage area, 182 mi^2 (471 km^2); bankfull discharge, about 8,000 ft^3/s (226 m^3/s); valley slope, 0.0035; channel width, about 100 ft (30 m). Stream is perennial, semi-alluvial, cobble bed, in valley of low relief, wide flood plain; channel is sinuous, locally anabranched, wider at bends, point bars, cut banks local, sand-gravel banks, tree cover at more than 90 percent of bankline.

Hydraulic problems and countermeasures:

1972 Bridge built. Because the left-bank approach embankment encroaches on the flood plain and constricts overbank flow, a spur dike was included in the crossing design. Countermeasures applied at the site include: (1) The excavation for piers construction was backfilled with rock riprap instead of original streambed material. (2) Construction of an elliptical spur dike whose minor axis length is 0.71 Ls (top length of spur). The top of the spur dike was 3.1 ft (0.94 m) below the bottom of the bridge beams. Riprap revetment on the spur dike was extended along the left bank of the channel downstream past the dual bridges. (3) Trees along the streambank were kept close to the toe of the spur dike embankment. A natural drainage channel with intermittent flow enters Muncy Creek between the tree line and the spur dike toe.

1975 During the early stages of flooding associated with tropical storm Eloise in September 1975, the spur dike efficiently guided overbank flow into the main channel and through the bridge with a minimum of turbulence. The spur dike was later damaged (fig. 142) when overtopped and breached during peak flow. A scour hole 5 to 6 ft (1.5-1.8 m) deep was eroded at the toe of the spur dike embankment. Much of the eroded material was depositied downstream along the left bank. The spur dike served its intended purpose; no scour or bank erosion damage occurred directly at the dual bridges.

1976 No repairs have been made to the spur dike.

Discussion: The spur dike served its purpose in protecting the bridge, but will require the addition of heavier riprap to prevent future flood damage. Rock riprap on top of the spur dike is needed to prevent damage by overtopping flow.

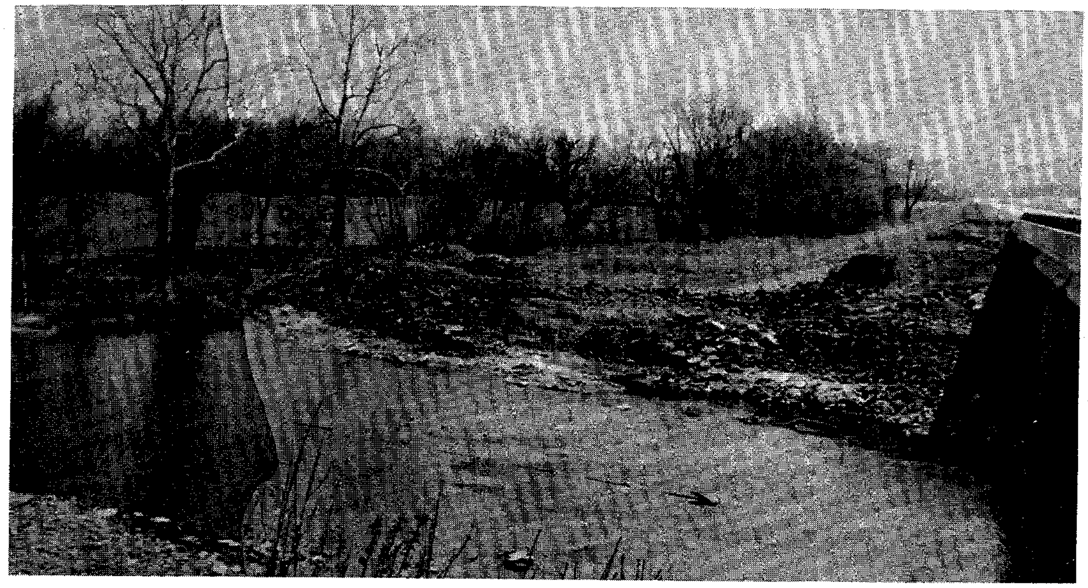


Figure 142. View of damaged spur dike along left bank, in 1976.

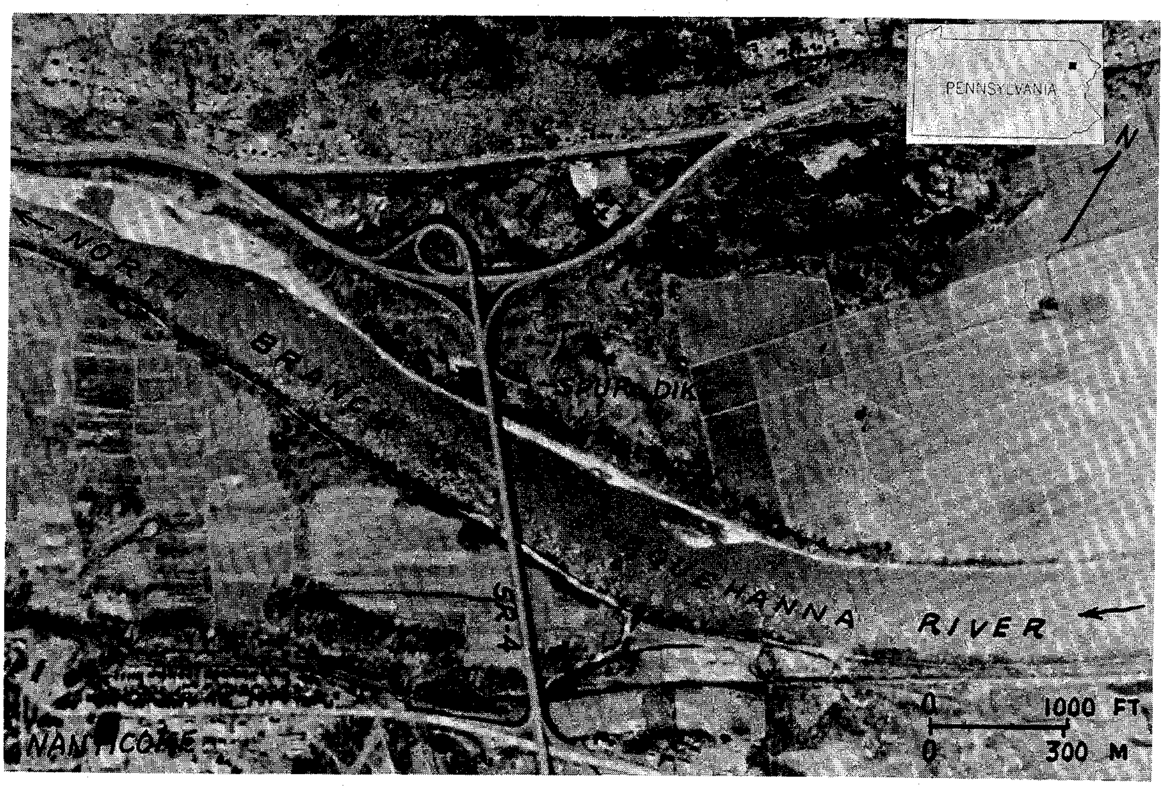


Figure 143. Aerial photograph of SR-4 crossing, North Branch Susquehanna River, January 28, 1969. (From U.S. Geological Survey.)

SITE 35. NORTH BRANCH SUSQUEHANNA RIVER AT SR-4 NEAR NANTICOKE, PA.

Description of site: Lat 41°13', long 75°39', location as shown in fig. 143. Continuous-deck steel-plate girder, bridge has 16 spans, is 2,740 ft (835 m) long and supported by 15 piers. The piers are wall-type with rounded nose, founded on shale bedrock and alined with the flow. Abutments are vertical with 45° wingwalls on the left bank and spillthrough type on the right bank. The bridge is skewed 39° as determined from 1969 aerial photographs. The approach embankment to the right bank abutment extends across much of the flood plain, and relief bridges provide openings for flows on the left-bank flood plain.

Drainage area, 10,035 mi^2 (25,990 km^2); bankfull discharge, about 115,000 ft^3/s (3,250 m^3/s); valley slope, 0.0027; channel width, about 800 ft (244 m). Stream is perennial, alluvial, cobble bed, in valley of moderate relief, narrow flood plain. Channel is sinuous, locally braided, equiwidth, silt-clay banks, tree cover at more than 90 percent of bankline.

Hydraulic problems and countermeasures:

1954 Bridge built, without countermeasures.

Pre-1964 Hydraulic problems at the site between 1954-64 are indicated by the addition of paved concrete revetment at the first pier adjacent to the right bank abutment where local scour and undercutting occurred. Scour near pier 1 was probably caused by turbulence induced by floodflows returning to the main channel from the right bank flood plain (fig. 143). Floods occurring prior to 1964 were in October 1955, March 1956, and April 1960.

To counter hydraulic problems due to the severe constriction of floodflow by the approach embankment, a rock spur dike was added to the right bank bridge abutment (figs. 1431 and 144) sometime before 1964. The spur dike was built completely of rock (diameter up 2 ft or 0.61 m) and extends about 330 ft (100 m) upstream from the right abutment as a continuation of the approach embankment. Major flooding in March 1964 caused no damage to the spur dike.

1972 Flood waters associated with tropical storm Agnes (June 24, 1972) were the highest on record. R.I. about 300 yr. Flows overtopped the spur dike but no damage was noted.

1975 Flooding on September 27, 1975, R.I. about 45 yr, caused no damage to the spur dike or bridge.

1977 Field inspection in January 1977 found no hydraulic problems although a minor scour hole has developed downstream from the right abutment at the spur dike terminus. Apparently the spur dike has required little maintenance; trees and brush have overgrown much of the spur dike and the surrounding flood plain and the structure appears very stable. The fact that no damage resulted from overtopping during 1972 flooding may be attributed to the amount and size of rock used in the spur dike; furthermore, the heavy overgrowth of trees and brush has helped to stabilize the rock and diminish overbank flow velocity.

Discussion: A spur dike made of rock has successfully channeled overbank flow through a constricted bridge opening with little damage to the bridge or associated flood plain. The dike was built before modern design procedures for spur dikes were available and is not elliptical in plan. As a result, flows do not satisfactorily follow the alinement of the spur dike (fig. 144). Concrete revetment was used in an attempt to protect the area around the bridge pier from scour, but construction of the spur dike has probably prevented further scour in the vicinity of the pier.

Figure 144. Upstream view of spur dike at SR-4 crossing, during flood of March 1964.

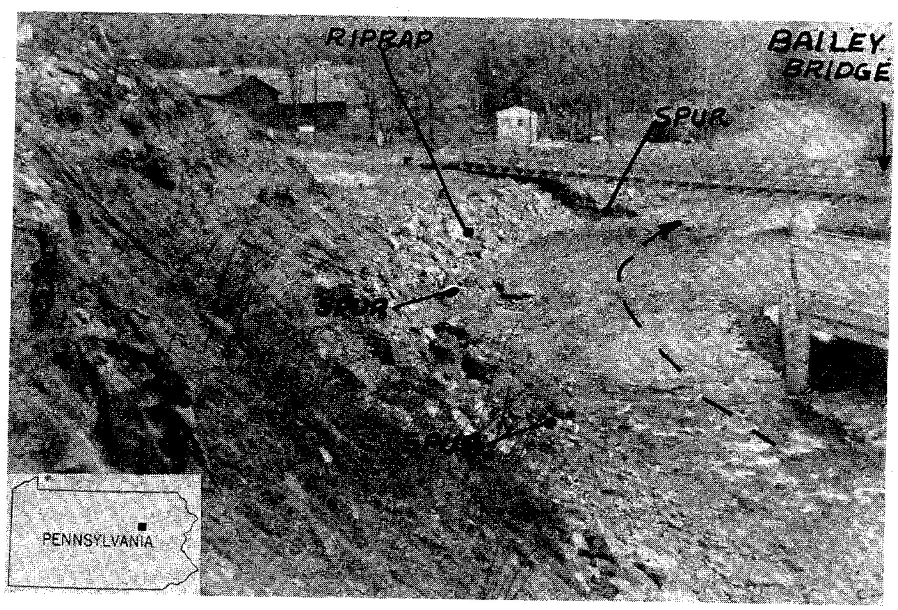


Figure 145. Upstream view of West Branch Fishing Creek in 1976, showing Bailey bridge, rock spurs, and riprapped right bank.

SITE 36. WEST BRANCH FISHING CREEK AT LR-16 NEAR ELK GROVE, PA.

Description of site: Lat 41°18', long 76°23'. From 1932 to 1972, bridge at site was timber-decked, steel I-beam, with single span of 58 ft (18 m). Skew of crossing 0° (zero degrees). From 1972 to 1975, bridge at site was concrete box-beam with two spans of 70 ft (21 m), which was erected 100 ft (30 m) upstream from older bridge destroyed during Hurricane Agnes. Bridge skew was 45°. Center pier with round nose was in good alinement with normal flow, but skewed to flood flow. After flood of September 1975, a temporary Bailey bridge has been used.

Drainage area, 32 mi^2 (83 km^2); bankfull discharge, 1,750 ft^3/s (50 m^3/s); valley slope, 0.0166; channel width, 75 ft (23 m). Stream is perennial but flashy, alluvial, cobble bed, in valley of high relief, narrow flood plain. Channel is sinuous, locally braided, cut banks local, gravel banks, tree cover at more than 90 percent of bankline.

Hydraulic problems and countermeasures:

1932 A timber-decked I-beam bridge was built with abutments founded on alluvium and no countermeasures to protect structure.

1972 During flooding caused by Hurricane Agnes, the left bank of the creek was severely eroded and the bridge approach embankment washed out as the channel shifted toward the left bank.

1973 A new concrete box-beam bridge, with two spans and a greater waterway capacity, was built at a slight bend in the stream with a skew of 45°. Abutment and pier footings were founded at a depth of 6 ft (1.8 m) in gravel alluvium. Sandstone riprap was placed at both abutments and along the upstream banks of the stream.

1975 During tropical storm Eloise, in September 1975, flooding caused erosion of the left channel bank at the new bridge site. The sandstone riprap was not adequate to protect the loose gravel banks along the outside of the stream bend. As in 1972, the channel again shifted left and the approach embankment and bridge abutment were eroded until failure occurred. After 1975, a Bailey bridge was installed with rock riprap (limestone: D_{50} = 2.5 ft) and rock spurs as countermeasures (fig. 145). No flow hazards have occurred since 1975 to test the effectiveness of these countermeasures.

Discussion: Piers and abutments placed 6 ft (1.8 m) deep in coarse gravel alluvium were damaged by scour during an extreme flood in 1972. Failure of riprap bank revetment in 1975 is attributed to erosion of the upstream end and undermining of the toe.

SITE 37. FISHING CREEK AT LR 19026 AT LIGHT STREET, PA.

Description of site: Lat 41°02', long 76°26'. Three-span concrete box-beam bridge, built in 1974, is 232 ft (70.7 m) long and supported by two round-nosed piers 4 ft (1.2 m) in width. The piers and the spillthough abutments are founded on piles that extend through alluvium to sandstone bedrock. The skew of crossing is 7° and the piers are alined to flow. The approach roadways have good alinement with the structure, although the enbankments constrict the flood plain. The bridge is located at the same site as a 1938 bridge, at which scour at the left abutment caused failure during the June 1972 flood.

Drainage area, 287 mi^2 (743 km^2); bankfull discharge, 7,500 ft^3/s (212 m^3/s); valley slope, 0.0013; channel width, about 125 ft (38 m). Stream is perennial, alluvial, gravel bed, in valley of moderate relief, wide flood plain. Channel is sinuous, locally braided, cut banks general, gravel banks. Apparent instability of channel is attributed, at least in part, to removal of some small dams upstream from the bridge and to meander cutoffs, both natural and artificial.

Hydraulic problems and countermeasures:

1938 Original bridge built.

1968 A small dam 2.3 mi (3.7 km) downstream was totally removed and 5,000 ft (1,524 m) of stream channel was relocated when Interstate 80 was constructed through the area. Earlier in the 1960's, a small public-utilities dam upstream 3.3 mi (5.3 km) also was removed.

Pre-1972 A railroad tie retaining wall (fig. 146) was installed to protect a residence upstream on the left bank, but ended about 100 ft (30 m) upstream from the bridge.

1972 Flooding caused by tropical storm Aganes in June 1972, with a discharge of 31,600 ft^3/s (894 m^3/s) scoured the left-bank abutment until it failed. After the Agnes flood, the U.S. Army Corps of Engineers cleared debris from the banks and channel and further attempted to restore and stabilize the deep channel of Fishing Creek by adding riprap to the stream bends.

1974 A new bridge was built. Countermeasures for the bridge include: (1) The abutment foundations from the original bridge were left in place to provide a solid toe for the riprap along the stream channel and around the abutments of the new bridge. (2) The new bridge 67 ft (20 m) longer than the old bridge. (3) The abutment slopes were seeded and mulched in an effort to provide stability.

1975 Tropical storm Eloise caused a flood discharge of 29,300 ft^3/s (830 m^3/s), and was the first test for the new bridge. Overbank flow along the right flood plain was constricted by the approach embankment and abutment. Areas protected with riprap at the abutments remained intact, but the unprotected channel banks upstream from the bridge and approach embankments were eroded (fig. 146).

1976 Additional limestone riprap was tied in with the existing sandstone riprap upstream from the right abutment. A rock-lined channel (fig. 147) was built along the base of the approach embankment to allow drainage from the right overflow area to reenter the stream channel without causing erosion as in 1975. The extension of the limestone riprap in 1975 to protect the approach embankment and streambank near the bridge appears to be an adequate countermeasure. No flow hazard has occurred to date, however, to test the effectiveness of the additional riprap protection. The abutment slopes which were seeded and mulched in 1974 now have a thick sod cover.

Discussion: Removal of small dams from a stream, a large flood event, manmade channel changes, and natural meander cutoffs produced an unstable channel. Channel degradation and bank-stability problems are evidence of the unstable channel. Armoring the channel banks and abutments may prevent problems of lateral erosion in the future, but degradation of the main channel may continue causing scour problems at the bridge piers and toe of abutments.

Constriction of overbank flow, lack of spur dikes, turbulence caused by overbank flow when reentering the main channel and an unstable channel are probably the primary factors in causing bank erosion and general channel scour upstream from the bridge and at the abutments. Increasing the length of the bridge probably did not reduce the amount of bridge contraction sufficiently to reduce the potential for scour at the bridge abutments.

Figure 146. Erosion of left bank between retaining wall and bridge, as photographed in 1976.

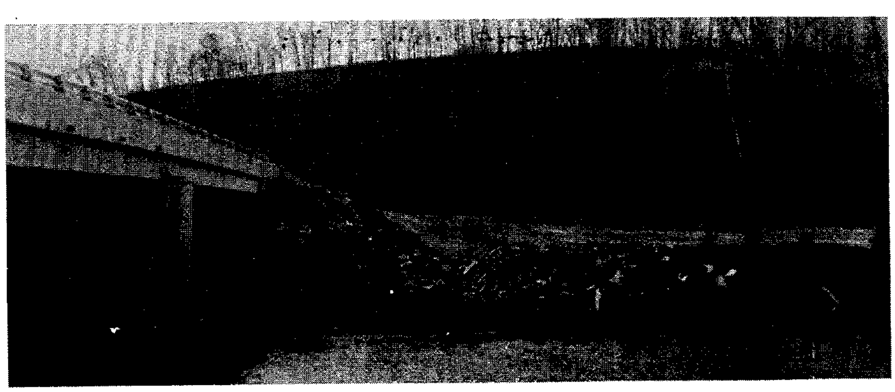


Figure 147. Riprap on right bank as photographed in 1976. Ss, original sandstone riprap; Ls, limestone riprap placed after 1975 flood.

SITE 38. SALINE RIVER AT US-70 NEAR DIERKS, ARK.

Description of site: Lat 34°05', long 94°05', at location shown in fig. 148. Steel I-beam bridge 461 ft (141 m) long with concrete-slab roadway supported by seven concrete piers. Piers consist of a pair of rectangular columns with web, placed on spread footings. Base of pier footings is set about 6 to 10 ft (2 to 3 m) below streambed on gravel and boulders. Abutments are spillthrough type. Approach embankments on both banks constrict overbank floodflow.

Drainage area, 124 mi^2 (321 km^2); valley slope, 0.002. Stream is perennial, alluvial, gravel bed, in valley of moderate relief, wide flood plain. Channel is sinuous, locally anabranched.

Hydraulic problems and countermeasures:

1951 Bridge built with pier footings about 6 ft (2 m) below streambed and rock riprap on both abutments (fig. 149).

1961 Flood of May 1961, discharge 52,100 ft^3/s (15,880 m^3/s), R.I. 50 yr, caused no damage to bridge. Flood high-water marks indicate most overbank flow occurs on the left-bank flood plain. Road overflow occurred on both approach embankments.

1968 Flood in May, discharge 59,200 ft^3/s (18,044 m^3/s), R.I. 75 yr, scoured under pier 6 (fig. 150) causing the pier to settle 1 ft (0.3 m) (fig. 151) at the downstream end. Settlement of the pier was uneven, causing the pier to tilt slightly in the downstream direction. Road overflow occurred at both approach embankments.

Repairs following the flood consisted of extending the spread footing under pier 6 deeper, filling the scour hole (fig. 150), and adding to the pier cap to level the bridge deck (fig. 152). Large rock riprap, up to about 2 ft (0.6 m) in diameter was placed at the left abutment slope and on the flood plain between piers 5 and 7.

1969 Flood of January 1969, discharge of 28,600 ft^3/s (8,717 m^3/s), R.I. 12 yr, caused no damage to bridge or countermeasures. The riprap, although performing satisfactorily, reduces the waterway on the flood plain about 10 percent.

Discussion: Undermining of the bridge pier on the left-bank flood plain during the 1968 flood was caused by cross currents of flow when overbank flow re-entered the main channel at the bridge. The bridge piers, about 26 ft (8 m) long, were skewed to the direction of overbank flow and contributed to the inefficiency of the overflow opening and to the resultant scour. Placement of the rock riprap (following the 1968 flood) on the left-bank flood plain under the bridge to prevent scour of the pier footings reduces some of the space formerly part of the waterway. Placement of a flow-control structure, such as a spur dike, on the upstream side of the left-bank approach embankment would help prevent the formation of cross currents near the abutment and reduce the possibility of pier or abutment scour.

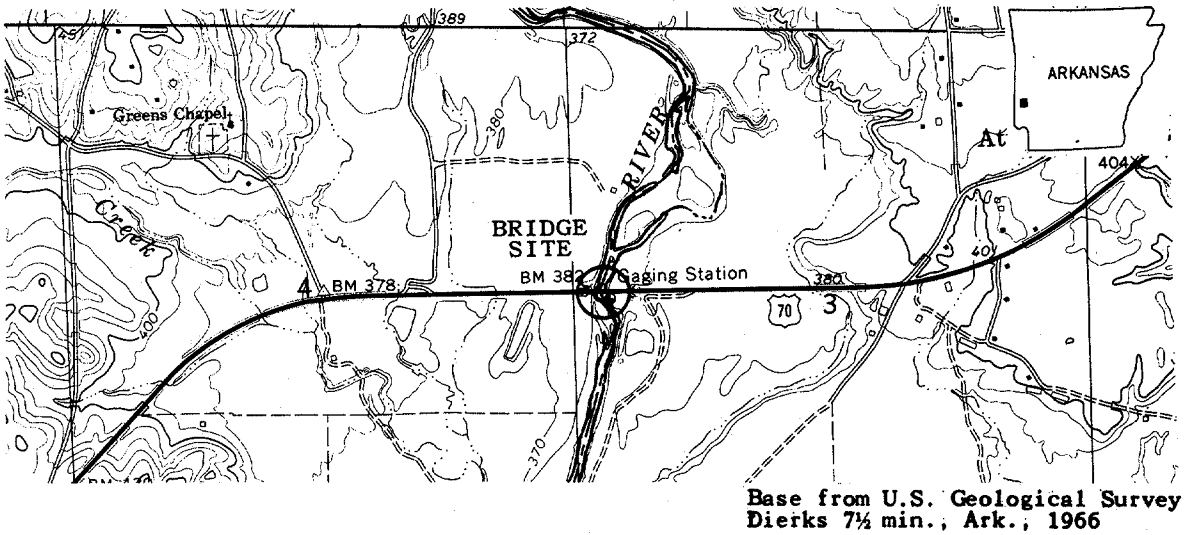


Figure 148. Location of Saline River at US-70 near Dierks, Arkansas.

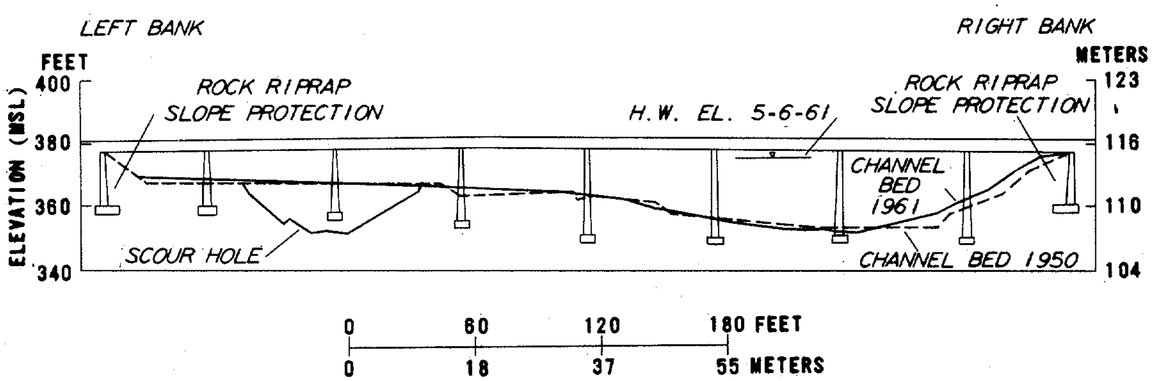


Figure 149. Cross sections of Saline River bridge.

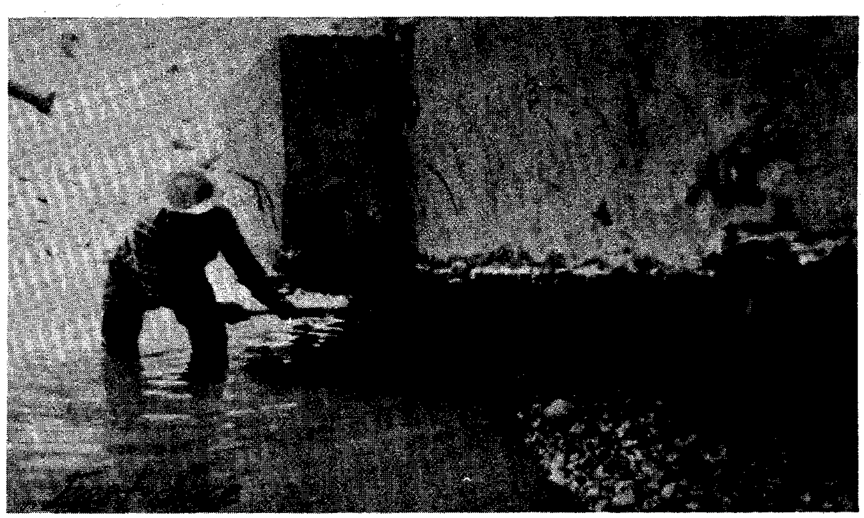


Figure 150. Footing of pier on US-70 bridge over Saline River near Dierks, Arkansas, following flood of May 13, 1968. (From Arkansas Dept. of Highways.)

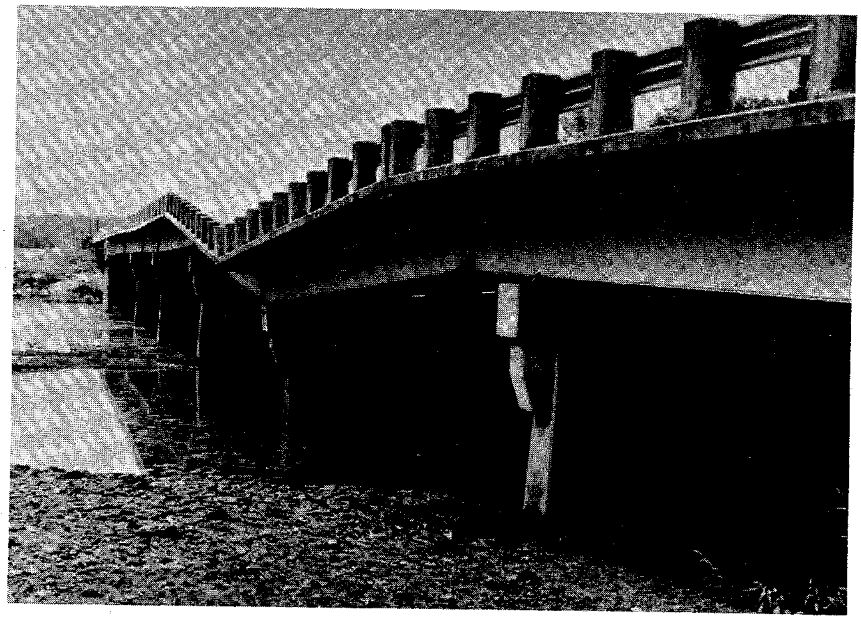


Figure 151. Bridge on US-70 over Saline River near Dierks, Arkansas, following flood of May 13, 1968. (From Arkansas Dept. of Highways.)

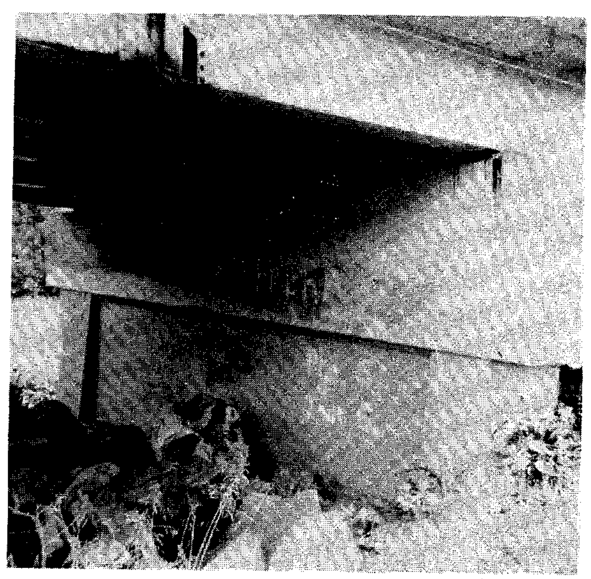


Figure 152. Cap on pier 6 has variable thickness to compensate for uneven settlement of pier. At lower left, riprap placed on flood plain. (From Arkansas Dept. of Highways.)

SITE 39. EAST FORK TRINITY RIVER AT ATCHISON, TOPEKA, AND SANTA FE RAILWAY BRIDGE NEAR LAVON, TEX.

Description of site: Lat 33°02', long 96°29', location as shown in fig. 153. The bridge crosses the E. F. Trinity River immediately downstream from the spillway for Lavon Dam on the relocated channel. Slope of the new channel is 0.001 ft/ft. Bridge is 589 ft (180 m) long, constructed of steel-plate girders, with 127 ft (39 m) timber-pile approach trestle and supported by eight concrete wall-type pointed nose piers. Piers are supported by pilings driven down to hard shale layer. Abutments are spillthrough type, protected with concrete slope paving. Design release for floods with 20-year recurrence interval from the reservoir is 2,000 ft^3/s (57 m^3/s).

Hydraulic problems and countermeasures:

1951 Bridge built. Concrete slope pavement at abutments was protected at toe by cutoff walls extending 3 ft (1 m) below the streambed.

1966 Degradation of channel at site was first observed by the railroad company during the period 1957-66. Peak flows of 39,000 ft^3/s (1,104 m^3/s) in 1957, 5,700 ft^3/s (161 m^3/s) in 1958, and 6,000 ft^3/s (170 m^3/s) in 1966 occurred.

1972 Surveys made in 1953, 1965, 1967, and 1972 indicate that degradation beneath the bridge was in the range of 5 to 8 ft (1.5 to 2.4 m) for the period 1953-65. Damages that have occurred as a result of flows released from the reservoir show that the slope protection is inadequate.

1974 Studies of spillway releases for design floods with a R.I. of about 100 yr by the U. S. Corps of Engineers, indicate that the spillway channel in the vicinity of the bridge abutments would become relatively stable at elevation 435 ft (133 m) and that the flow velocity would be about 8 ft/s (2.4 m/s). Countermeasures proposed to protect the bridge abutments for the new channel conditions include: Extension of the concrete slope pavement downward 5 ft (1.5 m) at both abutments, construction of a new 3-ft (1-m) high concrete cutoff wall at the lower streambed elevation, and placement of a rock riprap blanket to protect the toe of the pavement.

Discussion: At this site, relocation of the channel apparently contributed to instability. Streambed degradation continued for about 15 years before the channel approached stability. The cutoff wall built at the toe of the existing concrete slope paving was insufficient to prevent undermining.

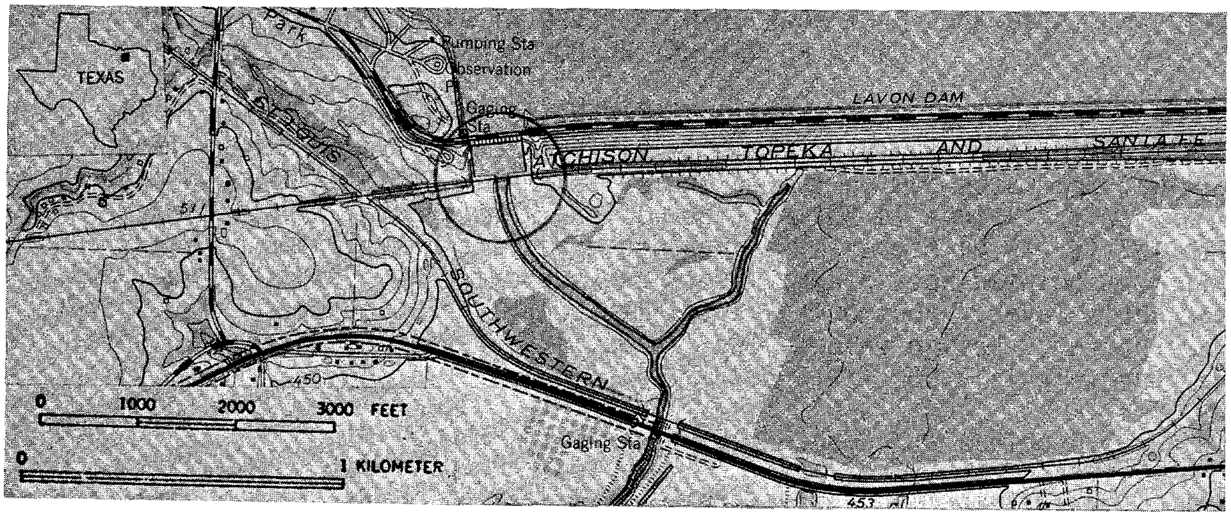


Figure 153. Location of bridge (circled) on Atchison, Topeka, and Santa Fe Railroad, downstream from Lavon Dam. (Base from U.S. Geol. Survey Lavon 7.5' quadrangle, Texas, contour interval 10 ft, 1963.)

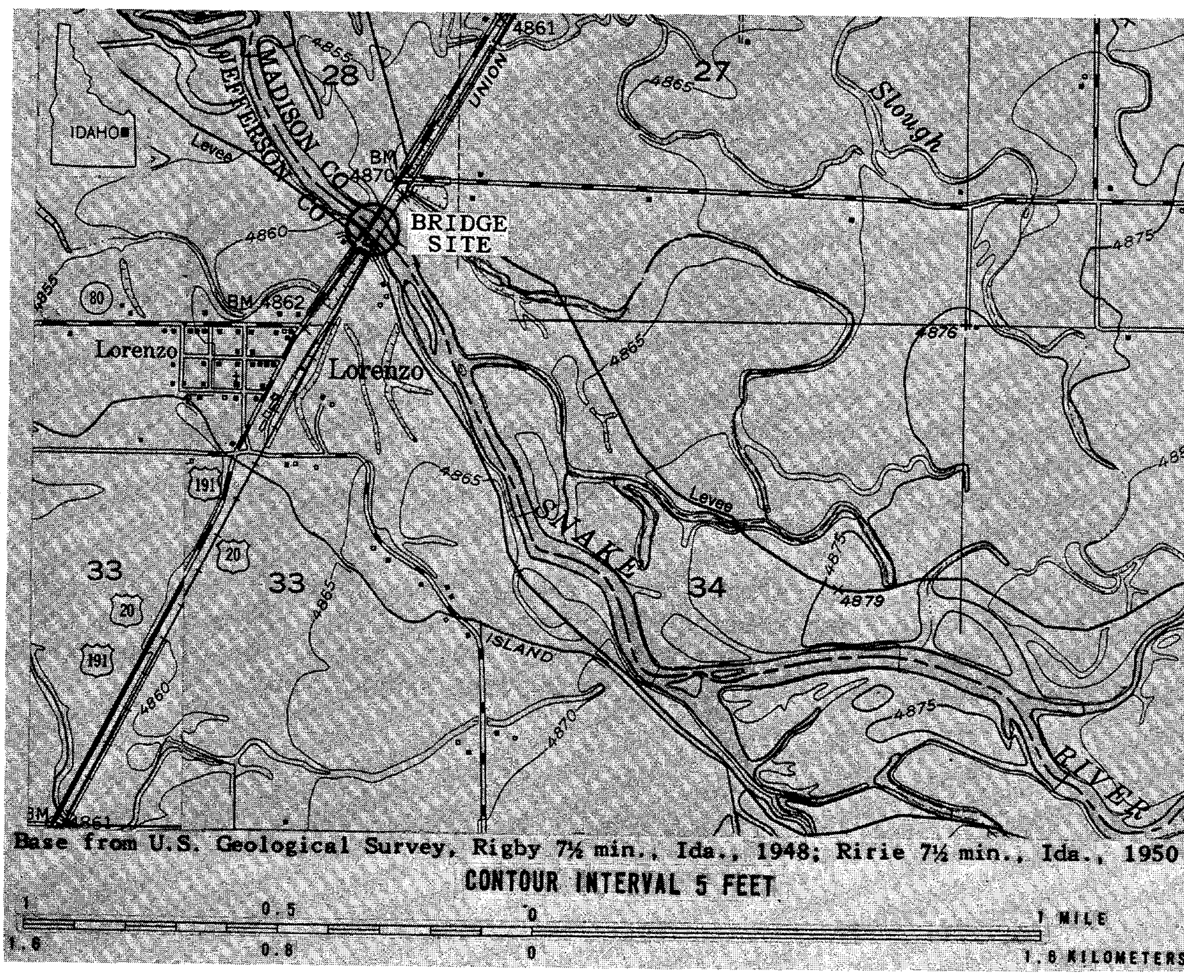


Figure 154. Location of Snake River at US-20 and US-191 near Heise, Idaho.

SITE 40. SNAKE RIVER AT US-20 AND US-191 NEAR HEISE, IDAHO

Description of site: Lat 43°42', long 111°54', location as shown in fig. 154. The downstream bridge (fig. 155), built about 1973, is 642 ft (196 m) long and supported by five reinforced-concrete piers. The piers are rectangular with round nose and founded on H-beam piles. Skew of the bridge if 11°. The pier curtain wall was placed to the then-existing streambed. The abutments are spillthrough, founded on H-piles. The existing upstream bridge is 696 ft (212 m) long and is supported by four piers of the webbed, round-column type (fig. 40.2). Piers for the older bridge are not skewed to flow alinement (fig. 40.2).

Valley slope, 0.003; channel width, about 200 ft (61 m). Stream is perennial, alluvial, gravel bed, on piedmont slope. Channel is locally anabranched and locally braided.

Hydraulic problems and countermeasures:

1973 A new southbound bridge was built with three more piers than the older upstream bridge (fig. 155). Piers 2 and 4 are located downstream from the older bridge piers, and because of their location relative to the existing bridge piers, and changes in pier skew angles (fig. 155), considerable turbulence of flow occurs near these piers.

1977 Floodflows of about 22,000 ft^3/s (623 m^3/s) occurring each year since construction of the bridge has apparently caused the present scour condition around one of the new bridge piers (fig. 156). No countermeasures have been used at this time.

Discussion: The pier scour problem in this case may have been avoided by locating piers for the new bridge sufficiently away from the existing piers to avoid turbulence of flow originating at the upstream bridge. Evaluation of figs. 155 and 156 indicates that the height of the concrete curtain wall for pier 2 is probably less than the curtain walls for other piers on the new bridge. This would indicate the higher channel bed shown on fig. 155 scoured during the period 1971-76. Bridge designers should consider the possibility of future lateral movement of the stream channel and subsequent exposure of pier footings to flow. Local scour problems at pier 2 of the new bridge may be aggravated if debris lodges on the exposed piling or pier nose and creates additional turbulence.

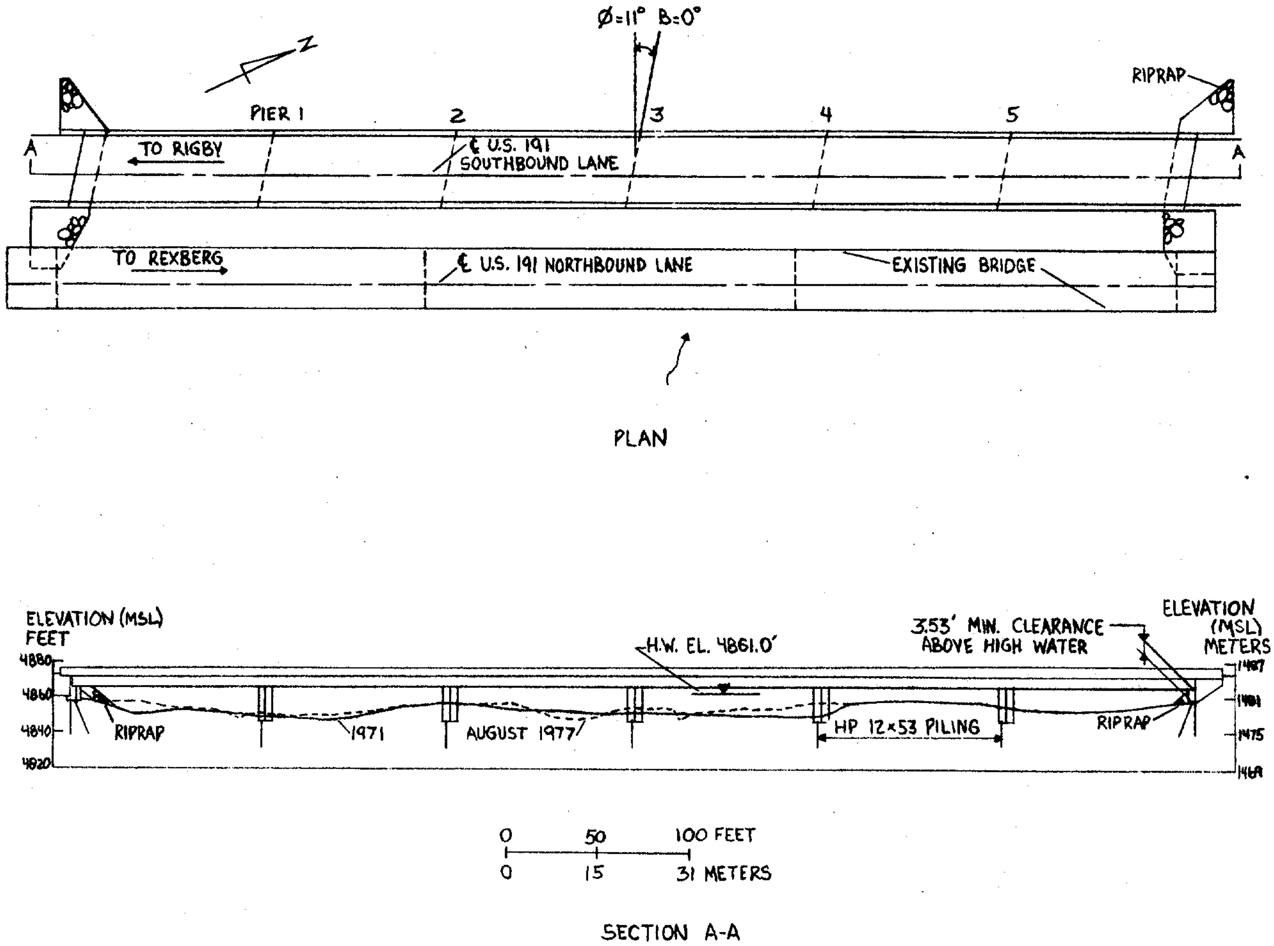


Figure 155. Plan and cross section at bridge site.

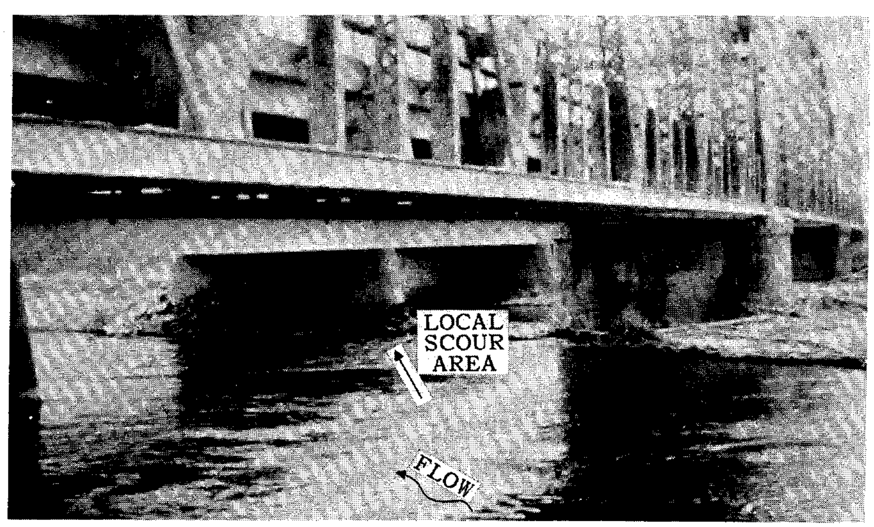


Figure 156. Scour at pier 2 of downstream bridge. (From Idaho Dept. of Transportation.)

SITE 41. TOMBIGBEE RIVER RELIEF BRIDGE 2 AT US-45 NEAR ABERDEEN, MISS.

Description of site: Lat 33°49', long 88°30', location as shown in fig. 157. Bridge consists of four steel-beam spans at 25-ft (7.6-m) centers, supported on timber-pile bents, which are approximately alined with flood flow. The bridge is located on the Tombigbee River flood plain and total bridge length is 100 ft (30 m). Abutments are spillthrough. During overbank flooding, flows are highly constricted at the bridge. There is another 125-ft (38-m) relief bridge 3,000 ft (914 m) to the west and the Nichols Creek bridge is 5,000 ft (1524 m) to the east.

Drainage area, 2,169 mi^2 (5,618 km^2); valley slope, 0.00034; channel width, about 150 ft (46 m). Stream is perennial, alluvial, sand bed, in valley of low relief, wide flood plain. Channel is meandering, equiwidth, probably incised, silt-sand banks, tree cover at more than 90 percent of bankline.

Flows at the relief bridge occur during period of overbank flow on the Tombigbee River. At times during flooding on the Tombigbee River, however, flows at the relief bridge are affected by backwater (fig. 158) from the main channel downstream from the site. Consequently, maximum discharge, flow velocities, and associated scour at the relief bridge channel may occur at times other than maximum stage. Maximum measured values of discharge and velocity for a given stage at this site exceed the values computed using natural channel-bed slope. This indicates that most flows occurring at the site are not of uniform depth.

Hydraulic problems and countermeasures:

1934 Bridge built. The spillthrough abutments had a slope of 1.5:1 and were armored with sacked concrete.

1955 Flood of March 1955 caused overbank flooding for 4 days at the bridge. A discharge measurement made near the crest shows velocities exceeding 9 ft/s (2.7 m/s). No scour problem was reported following the flood. Cross sections of the channel surveyed at the bridge between 1954 and 1971 show only minor (2 to 3 ft or 0.6 to 0.9 m) changes.

1973 Flood of March 1973, discharge 5060 ft^3/s (143.3 m^3/s), R.I. 65 yr, overtopped bridge floor. A discharge measurement was made on March 18 near the flood crest. Measurements of velocity and soundings show a small vortex or eddy current at the right abutment and scour near the left abutment (fig. 159). A measurement obtained on March 21 after the crest shows additional scour (fig. 159) and a 30-ft (9.1-m) wide vortex or eddy current near the right-bank abutment. After the flood, the Highway Department found the downstream part of the left-bank abutment had been undermined and a section of road about 8 ft (2.4 m) wide extending to the road centerline, had collapsed. Depth of scour extended approximately to the elevation of the flood plain. Soundings of the main channel at the downstream side of the bridge indicate the depth of scour was greater than at the upstream side, and piling at bents 2, 3, and 4 had only about 2 ft (0.6 m) of penetration left.

Countermeasures during and after the flood include: (1) Dumping old car bodies over side of bridge to prevent additional scour (fig. 160). (2) Placement of fill, compacted by bulldozer, in scour hole to level of former flood plain. (3) Replacement of the left-bank abutment fill on a 2:1 slope and use of sacked concrete riprap for protection (fig. 161). (4) Construction of two spur dikes 50 ft (15 m) in length upstream from the bridge abutments, which were faced with rock riprap.

1975 Flood of March 1975, discharge 4360 ft^3/s (123.5 m^3/s), R.I. 18 yr, caused overbank flooding and flow velocities through the bridge opening exceeding 8 ft/s (2.4 m/s). Soundings at the upstream side of the bridge obtained on March 15 showed no scour. Following the flood, the Highway Department found minor scour at bents 2 and 4 (fig. 159), probably because fill material placed near the piling in 1973 was not compacted completely.

Construction of the spur dikes at both abutments improved the distribution of flow and effectiveness of the waterway to handle the March 1975 flood (fig. 162). Flow velocities across the channel were more uniform (showed less variation between stations), and stages for the March 1975 flood were 3.2 ft (1 m) less than levels recorded during the 1973 flood even though the discharges were similar. The effect of possible backwater caused by the main channel of the Tombigbee River downstream from the relief bridge is unknown. The discharge measurement made on March 15, 1975, showed a very uniform distribution of flow across the channel. Following the flood, the minor scour holes near bents 2 and 4 were filled and the entire area under the bridge, extending 10 ft (3 m) upstream and 10 ft (3 m) downstream, was covered with rock riprap in the 50 to 150 lb (0.02 to 0.07) size range.

Discussion: The relief bridge was subjected to a significant amount of flow contraction. General scour, associated with contraction of flood-plain flow, extended about 15 ft (5 m) or more below the original channel bed. In this case, maximum velocity of flow and discharge through a bridge opening does not occur at peak stage if backwater, caused by high stages in the river downstream from the site, occurred. In areas of very flat relief, such as areas with slopes less than 0.0002 ft/ft, bridge design for maximum values of velocity and scour potential should take into account the possibility of varied flow conditions in which the friction slope (energy gradient) is greater than the natural channel slope. Lateral erosion of an approach embankment at the downstream side may result if flows are highly contracted, or eddy currents are present near the abutment on the upstream side.

Construction of spur dikes at both abutments improved the uniformity of flow distribution across the channel at the bridge so the entire waterway was more effectively used; reduced the possibility of eddy current formation near the upstream side of abutments caused by flood-plain flow returning to the main channel; and reduced the chance for lateral erosion, by eddy currents, of the approach embankment downstream from the bridge.

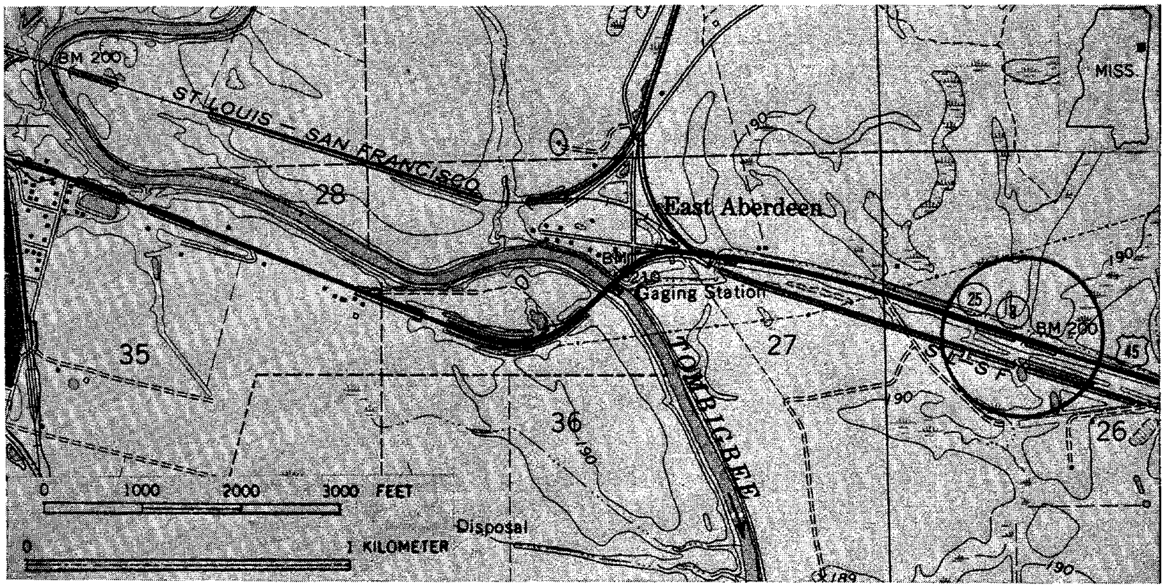


Figure 157. Location of Tombigbee River relief bridge 2 (circled). (Base from U.S. Geol. Survey Aberdeen, Miss., 7.5' quadrangle, contour interval 10 feet, 1966.)

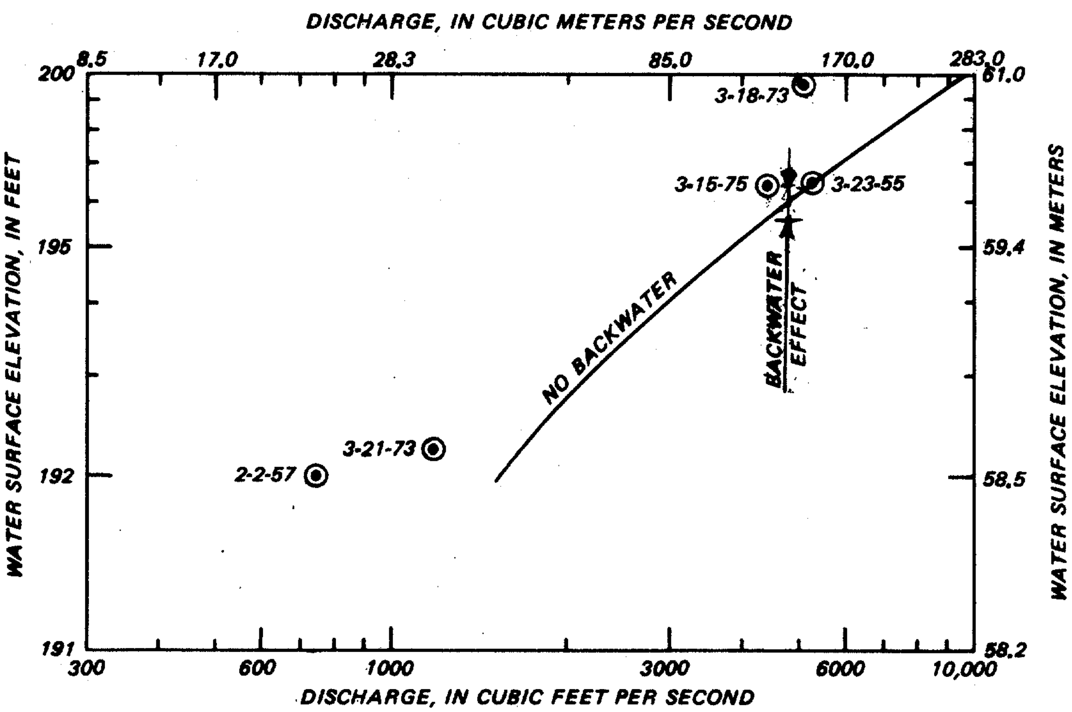


Figure 158. Effect of backwater on the relation of water-surface elevation to discharge.

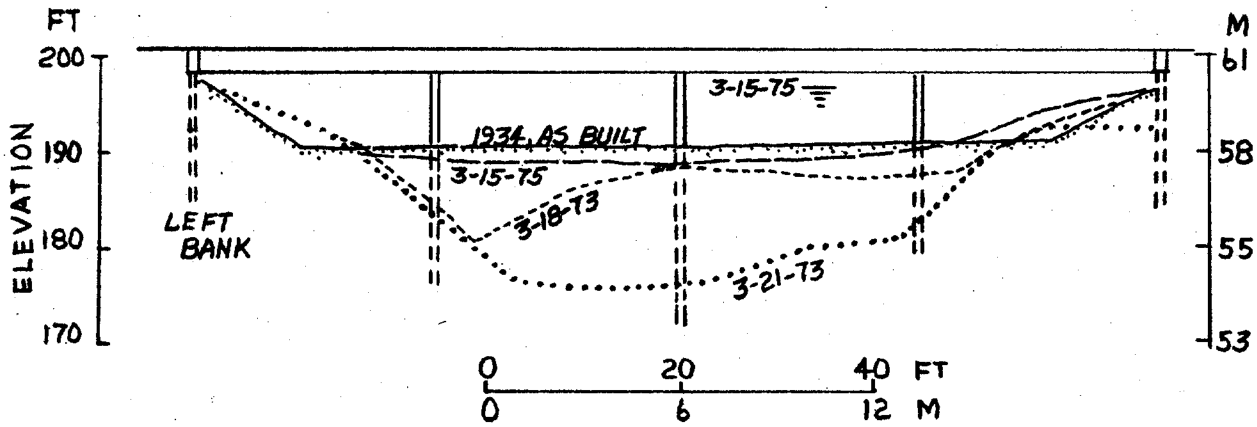


Figure 159. Changes in cross section of channel, 1934-75, Tombigbee River.

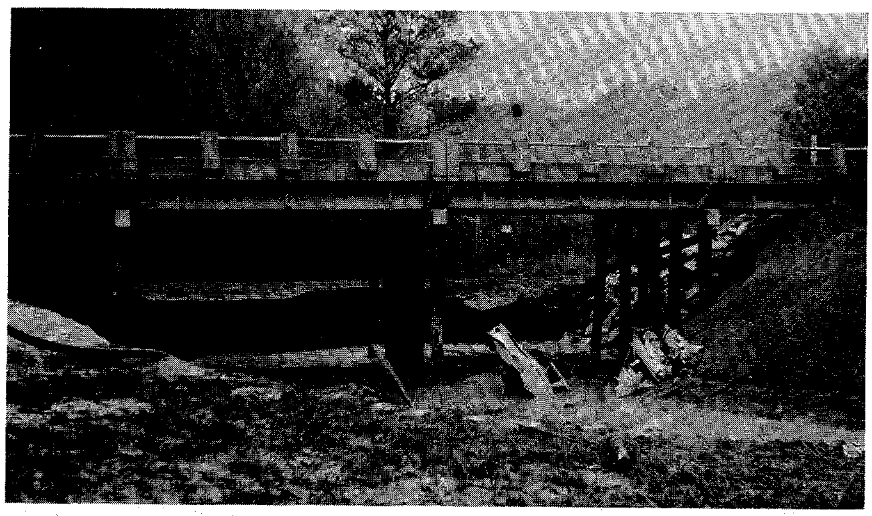


Figure 160. Old car bodies in local scour area to prevent additional scour during March 1973 flood.

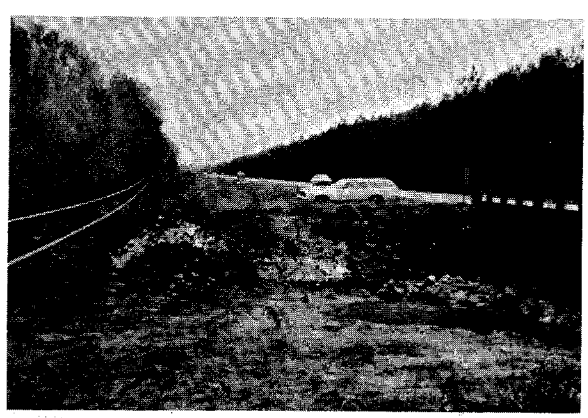


Figure 161. Protection of left-bank spur dike and channel bed with sacked concrete and rock riprap, 1975.

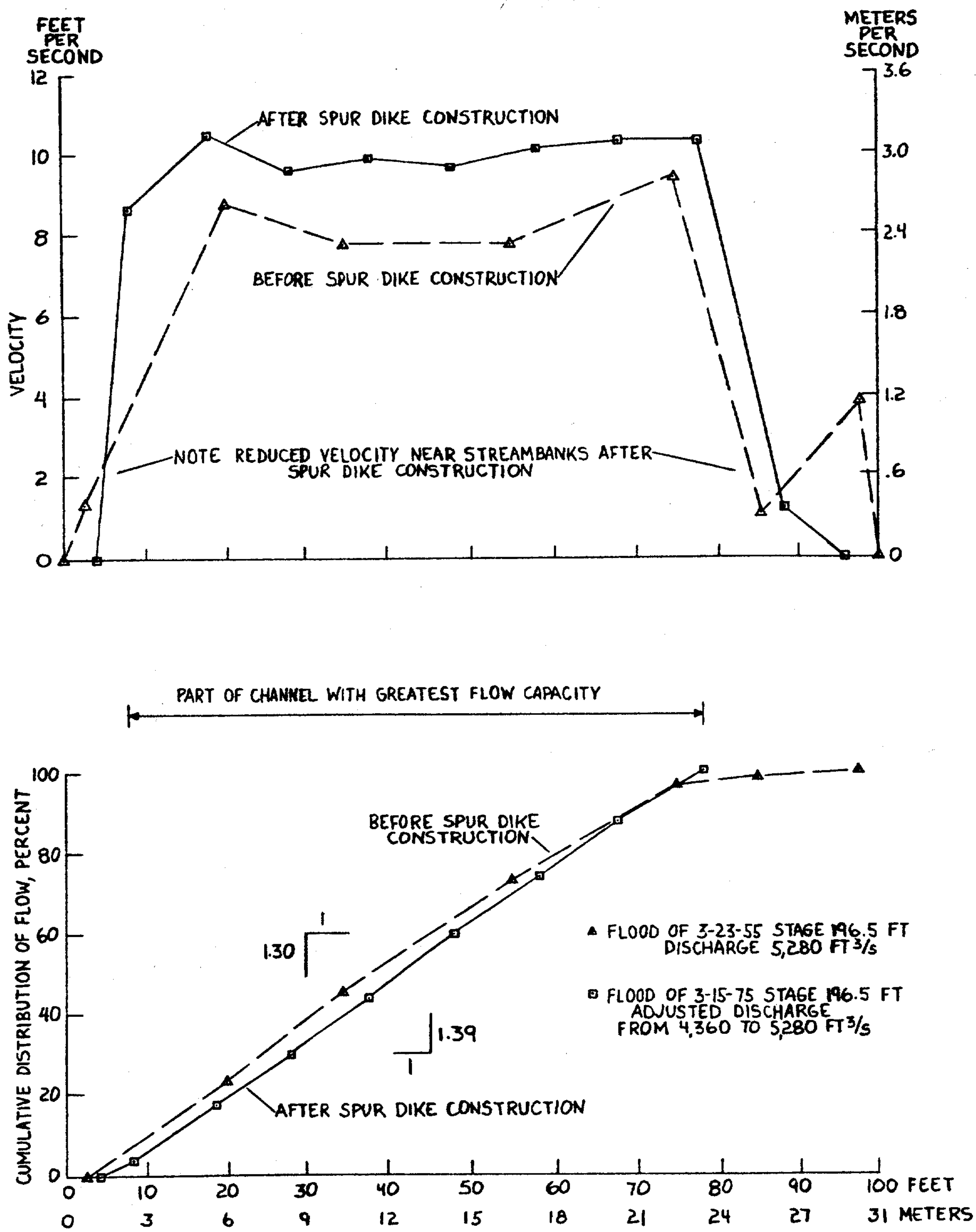


Figure 162. Top, reduction in flow velocity after construction of spur dike. Bottom, improvement in distribution of flow in bridge waterway.

SITE 42. FISHING CREEK AT SR-487 AT ORANGEVILLE, PA.

Description of site: Lat 41°05', long 76°24', location as shown in fig. 163. The three-span concrete I-beam bridge was built in 1973. It is 240 ft (73.1 m) long and has two 3-ft (0.9-m) wide round-nosed piers. The piers and vertical abutments are founded on piles that extend through alluvium to bedrock. The bridge has a 49° skew to the channel. The right approach embankment extends about 700 ft (210 m) across the wide flood plain, severely constricting overbank flow at the right-bank bridge abutment. The left-bank approach embankment parallels the channel at the base of Knob Mountain.

Drainage area, 233 mi^2 (603 km^2); bankfull discharge, 6,100 ft^3/s (173 m^3/s); valley slope, 0.0025; channel width, about 170 ft (52 m). Stream is perennial, alluvial, gravel bed, in valley of moderate relief, narrow flood plain. Channel is sinuous, locally braided, random width variation, cut banks local, gravel banks. Channel stability has been adversely affected by removal of small dams downstream from the crossing site and by meander cutoffs, both natural and artificial.

Hydraulic problems and countermeasures:

Pre-1972 The original bridge, 120 ft (37 m) long, was built (date unavailable) prior to 1972. During the early 1960's, several small dams were removed downstream from bridge site.

1972 Tropical storm Agnes caused a flood discharge of 28,200 ft^3/s (799 m^3/s) at this site. The original bridge was destroyed when the left abutment and pier scoured and settled away from the superstructure.

1973 A new bridge was built and extensive amounts of rock riprap were placed along the entire length of the right bank approach embankment (fig. 164) and at the bridge abutments.

1975 Flooding associated with tropical storm Eloise peaked at 26,100 ft^3/s (739 m^3/s) in September 1975. Some water overtopped the right-bank approach embankment about 600 ft (180 m) from the bridge opening. No erosion occurred at the abutment or along the approach embankments. General scour near the middle of the channel and local scour at pier 1 was attributed to debris pileup at the bridge in that area. Overbank flow reenters the main channel at the area of scour and may also have been responsible for the scour. The streambed around pier 1 was restored with rock riprap, 50 percent of which had a minimum volume of 13 ft^3 (0.4 m^3).

Discussion: Removal of small dams from the stream, a large flood, and manmade channel changes caused meander cutoffs which resulted in an unstable channel. Channel degradation and bank stability problems associated with the unstable channel were observed. Armoring the channel banks and abutments prevented problems of lateral erosion, but continued degradation of the channel caused scour at the bridge piers and toe of the abutments. Constriction of overbank flow, associated with turbulence of overbank flow returning to the main channel and an unstable channel bed, are probably the primary factors causing lateral bank erosion and general channel scour at this site.

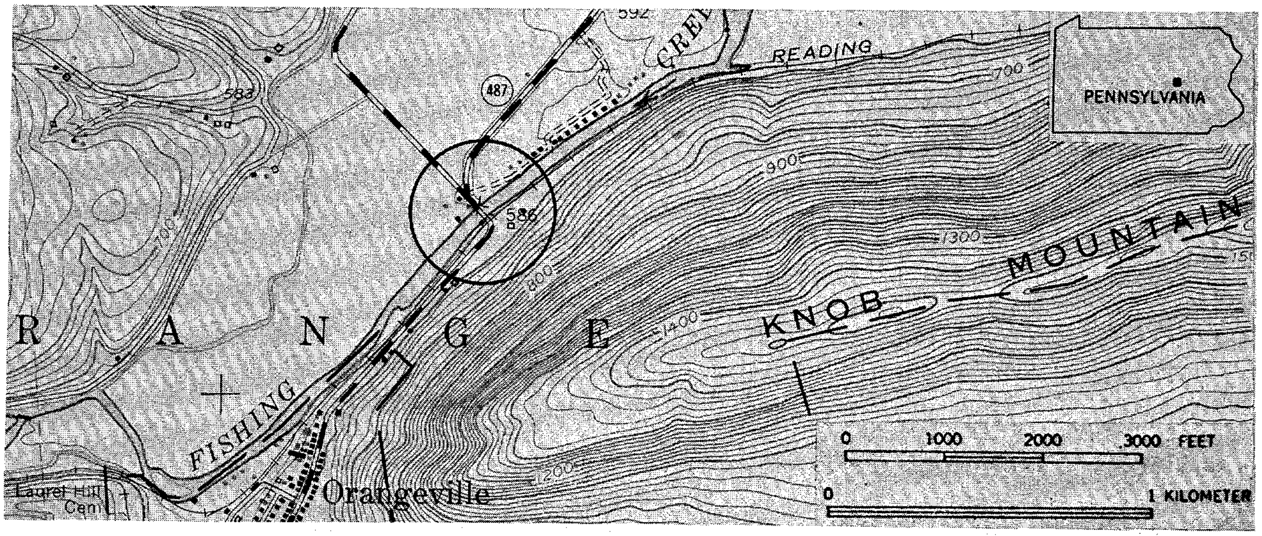


Figure 163. Location of SR-487 crossing, Fishing Creek. (Base from U.S. Geol. Survey Bloomsburg, Pa., 7.5' quadrangle, contour interval 20 feet, 1953 photorevised 1969.)

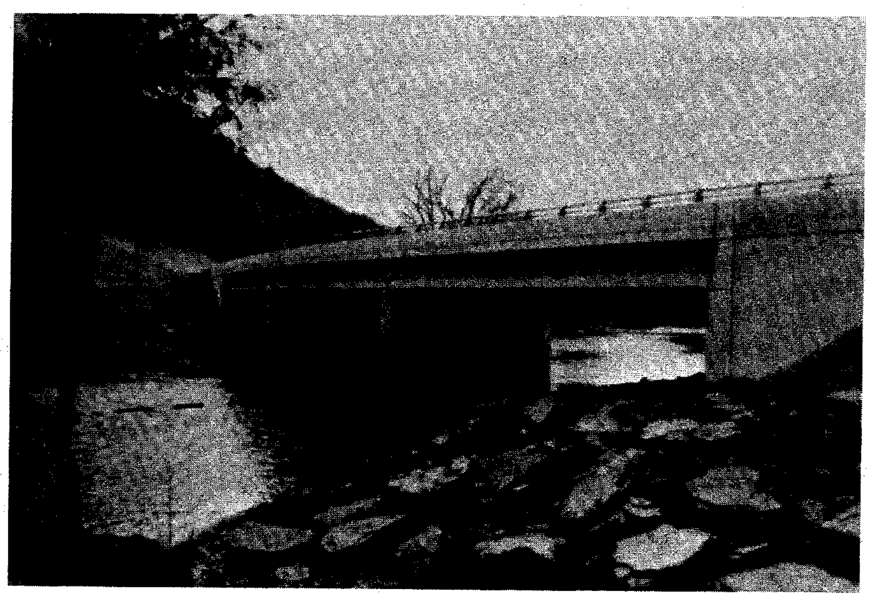


Figure 164. View from right bank upstream of bridge. Drainage channel and riprap protection in foreground. Photographed in 1976.

SITE 43. LYCOMING CREEK AT US-15 NEAR WILLIAMSPORT, PA.

Description of site: Lat 41°16', long 77°03', location as shown in fig. 165. The four-span concrete deck and I-beam bridge was built in 1955. The bridge is 248 ft (75.6 m) long and is supported by three 5 ft (1.5 m) wide round-nosed piers founded on piles. The full-height (vertical) abutments have 45° wingwalls and are founded on pilings. The crossing is located at a bend in the creek; skew of crossing is 30° and the bridge piers are alined with the flow.

Drainage area, 265 mi^2 (686 km^2); bankfull discharge, about 11,000 ft^3/s (311 m^3/s); valley slope, 0.0023; channel width, 150 ft (45 m). Stream is perennial, alluvial, cobble bed, in valley of moderate relief, narrow flood plain. Channel is sinuous, locally braided, sand-gravel banks.

Hydraulic problems and countermeasures:

1955 Bridge built, without countermeasures.

1972 Flow associated with tropical storm Agnes, discharge 34,800 ft^3/s (985 m^3/s), caused channel changes but no structural damage to the bridge. The channel shifted laterally to the right and general scour occurred throughout the bridge span adjacent to the right bank, with local scour at the right bank abutment of up to 13 feet (4.0 m). The approach embankment was breached almost completely across the roadway by eddy currents at the right bank abutment. All of the abutment pile supports were exposed. Deposition occurred under the three other spans toward the inside of the stream bend, markedly reducing the bridge capacity. These spans were left dry when flow subsided, and flow was then channeled under the right span with higher than normal velocities, a condition which presented a direct hazard to the bridge.

Post-flood repairs at the site included the addition of measures to realine and maintain stable flow conditions through the center of the bridge opening: (1) The steep right bank was extensively protected with a rock riprap (D_{50} = 1.75 ft , 0.53 m) throughout the length of the stream bend (2) Three rock spurs (deflectors) (figs. 165 and 166) constructed with selected limestone riprap (D_{50} = 2.25 ft., 0.69 m) were placed where flow most seriously encroached on the right bank. These spurs were spaced 75 feet (23 m) apart and were built about 6.6 ft (2.0 m) up from the streambed. The upstream spur extends obliquely 100 ft (30 m) into the channel, skewed 30° from the downstream bank; the middle spur extends 50 ft (15 m) and is skewed 60°; the downstream spur extends normally from the bank about 30 ft (9.1 m). (3) Other repairs and use of countermeasures included backfilling the scoured bed under the right span to its original level with material depositied in the other spans and (4) construction of an elevated flood plain (berm) was built of rock (D_{50} = 1.0 ft, 0.3 m) placed along the right bank from the downstream spur through the bridge waterway.

1976 Flood waters from tropical storm Eloise, discharge of 23,800 ft^3/s (673 m^3/s), were effectively channeled through the bridge opening with no scour or bank erosion. As of May 1976, the measures constructed at this site to improve and maintain good flow alinement through the bridge were judged to be very effective. The countermeasures, while preventing any recurrence of 1972 hydraulic problems, sustained very little damage; in fact, deposition occurred behind the upstream spurs (figs. 165 and 166), which further indicates their design effectiveness.

<u>Discussion</u>: This bridge was built near a stream bend and in easily eroded bed materials, and subsequently damaged by scour along the outside stream bank due to increased velocity. Extensive riprap was placed on the channel bank, and deflectors (spurs) and a rock berm constructed to maintain the channel near the center of the bridge. There countermeasures effectively protected the site from damage during a flood with a 25-year recurrence interval.

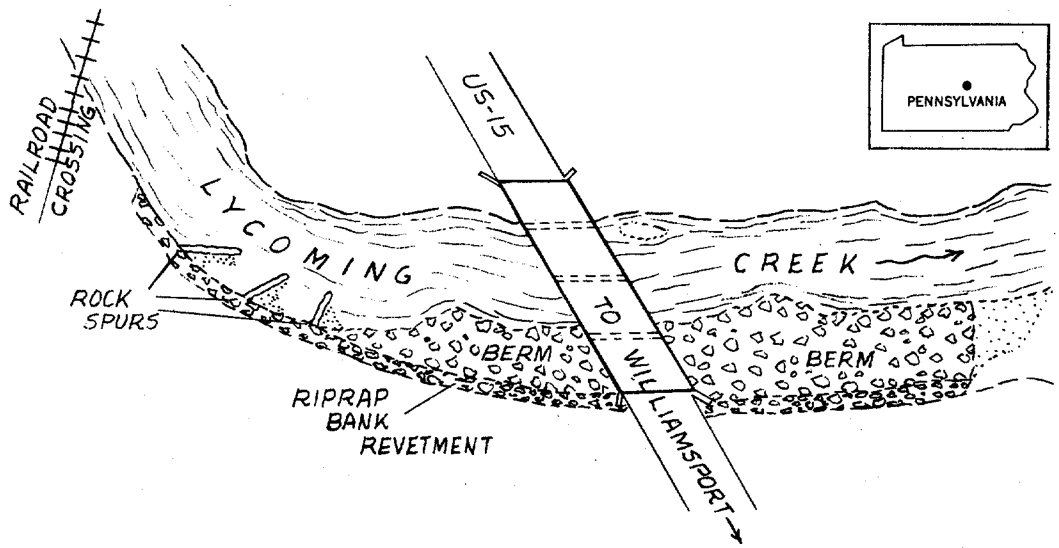


Figure 165. Plan sketch of countermeasures at US-15 crossing, Lycoming Creek.

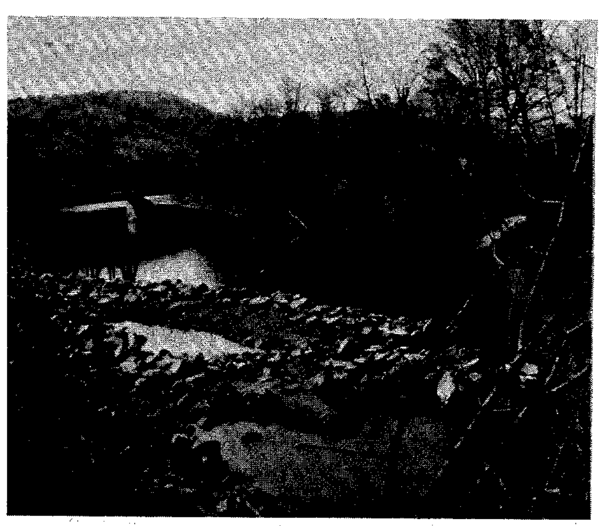


Figure 166. View upstream in 1976 toward spurs on right bank. Note accumulation of sediment.

SITE 44. NORTH BRANCH SUSQUEHANNA RIVER AT SR-93 AT BERWICK, PA.

Description of site: Lat 41°03', long 76°14', location as shown in fig. 167. The steel-truss bridge is 1,520 ft (463 m) long and supported by five pointed-nose piers 10-ft (3.0-m) wide. Four of the five masonry piers are founded on fissile, steeply dipping strata of shale beneath the alluvial channel bed. Pier 4 is supported by timber piles. The stone masonry abutments are vertical with no wingwalls.

Drainage area, 10,300 mi^2 (26,700 km^2); bankfull discharge, about 225,000 ft^3/s (6,367 m^3/s; valley slope, 0.00062; channel width, about 1,500 ft (457 m). Stream is perennial, alluvial, sand-gravel bed, in valley of moderate relief, little or no flood plain. Channel is sinuous, locally braided, random width variation, probably incised, cut banks rare, tree cover at more than 90 percent of bankline. Where Nescopeck Creek enters the Susquehanna, just downstream from the bridge, a delta has formed (fig. 167) that causes flow through the bridge to be skewed to piers.

Hydraulic problems and countermeasures:

1905 Bridge built.

1972 Flooding associated with tropical storm Agnes in June 1972, peaked at 355,000 ft^3/s (10,000 m^3/s. Severe local scour occurred at piers 2, 3, and 4, with a maximum scour depth of 4.5 ft (1.4 m) at the nose of pier 2.

1974 Concrete-filled fabric bags were used to repair and counter the local scour created at piers 2 and 3 by the 1972 flood. Divers removed debris from the scour holes and placed a "Fabriform" (fabric) bag under and around the footings of each pier. An expansion type intrusive grout was then injected into the bag to specific pressures, and by use of pipes, the remaining voids under each footing were filled. A rock cover 3 ft (0.9 m) thick was placed above the grout filling as further protection against future scour.

1975 Flow associated with tropical storm Eloise in September 1975, peaked at about 258,000 ft^3/s (7,300 m^3/s. A large gravel delta has formed in the Susequehanna River channel at the mouth of Nescopeck Creek which enters the river just downstream from the bridge on the left bank (figs. 167 and 168). Recent channel changes in Nescopeck Creek, such as channel relocation and artificial meander cutoffs, have been made upstream from the confluence. Abnormally large gravel deposits in the Susquehanna River channel have caused the flow at piers 2, 3, and 4 to become skewed as much as 30° where previously the flow was parallel to the piers.

1977 An underwater inspection, which had to be delayed until very low river stage because of high water velocities even at normal stage, was made at all piers and both abutments on September 8. According to this report, some of the concrete-fabric bags installed at piers 2 and 3 in 1974 had been displaced and the previously undermined areas at the pier footings had been partially re-exposed.

Discussion: The underlying shale bedrock at piers 2 and 3 was apparently fractured and susceptible to erosion by floodflows. Piers placed on shale bedrock should be designed to counter the possibility of shale erosion at the base of the footings. Abnormally large amounts of sediment transported by the tributary stream caused the formation of new or larger deltas at the confluence with the main stream. This delta tended to constrict flows in the main channel, cause localized turbulence and high flow velocities, and forced the realinement of the main channel. Subsequent scouring conditions in the main channel may affect the alinement of flow and the stability of bridge piers located upstream from the tributary. Owing to strong turbulence, the concrete-fabric bags were not entirely effective as a countermeasure. A possible solution to the problem, not yet put into effect, is to use heavier bags that extend farther outward from the pier face.

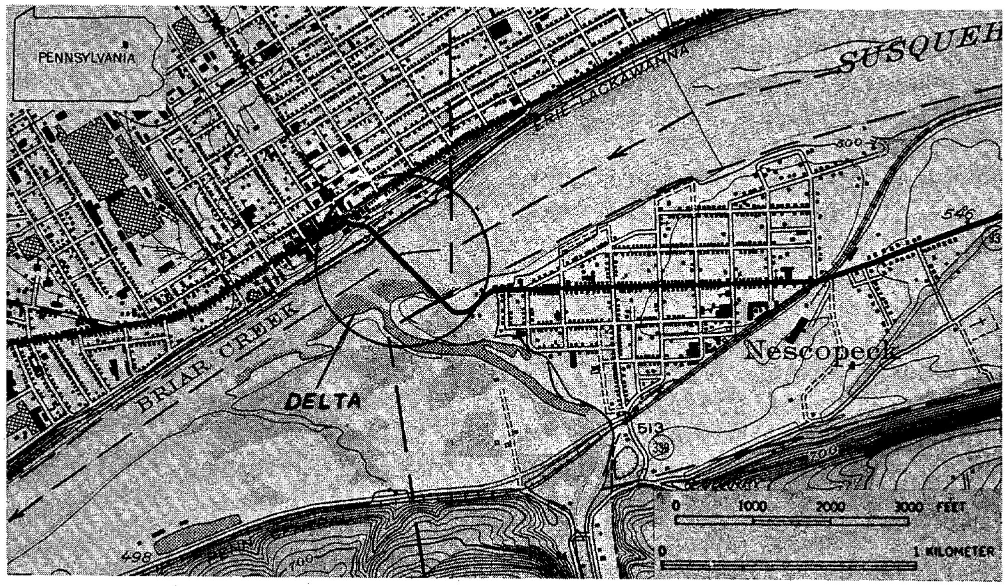


Figure 167. Location of North Branch Susquehanna River at SR-93 (circled). (Base from U.S. Geol. Survey Berwick, Pa., 7.5' quadrangle, contour interval 20 feet, 1955 photorevised 1969.)

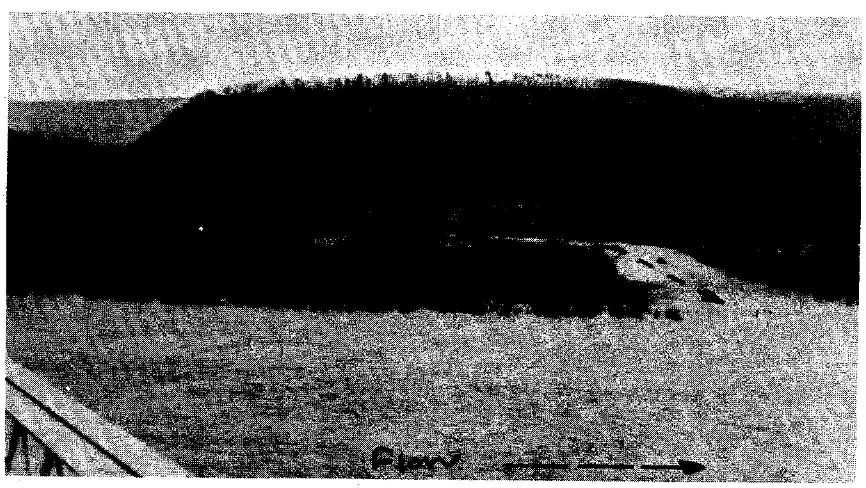


Figure 168. View of left bank of Susquehanna River showing confluence of Nescopeck Creek, photographed in 1976.

SITE 45. COOL CREEK AT I-55 NEAR KENTWOOD, LA.

Description of site: Lat 30°57', long 90°32', location as shown in fig. 169. Concrete girder bridge, 100 ft (30 m) long, supported by 5 square concrete columns per bent and founded on piles. Abutments are spillthrough type. The channel was relocated at time of bridge construction (fig. 170).

Valley slope, 0.0032. Stream is perennial but flashy, alluvial, sand bed, in valley of low relief. Natural channel is meandering, sand banks, tree cover at more than 90 percent of bankline.

Hydraulic problems and countermeasures:

1965 Bridge built. At time of construction, the channel was relocated and widened (fig. 170), and channel slope increased from about 0.0013 ft/ft for the old channel to about 0.0028 ft/ft for the new. The bridge abutments were protected with sacked concrete (on clay fill) on the right bank and concrete slope paving on the left bank (fig. 171).

1974 The 1974 flood almost overtopped the highway. The sacked concrete revetment subsided and the lower part of the bank slid into the creek. The concrete slope paving also failed due to undermining (figs. 172 and 173). Damage to the abutment protection was probably caused by a combination of factors: (1) High flow velocities and associated scour and lateral erosion through the constriction during the 1974 flood. (2) Turbulence as overbank flow reentered the main channel near the upstream side of the abutments and eddy currents as flows expanded downstream from the bridge. (3) Channel degradation, attributed to the channel relocation and the increase in slope, caused undermining of the abutment and bank slope protection.

1976 Repair of the abutment protection is planned, but no work has been done.

Discussion: This site illustrates the need for channel bank and abutment protection extended below the streambed and protected with cutoff walls to prevent undermining. Protection of the revetment from undermining is especially important because of the occurrence of channel degradation associated with the relocated stream. Progressive damage to the revetment protection occurred because the toe of the concrete slope protection and sacked concrete failed, allowing the upper revetment material to slide down into the eroded area.

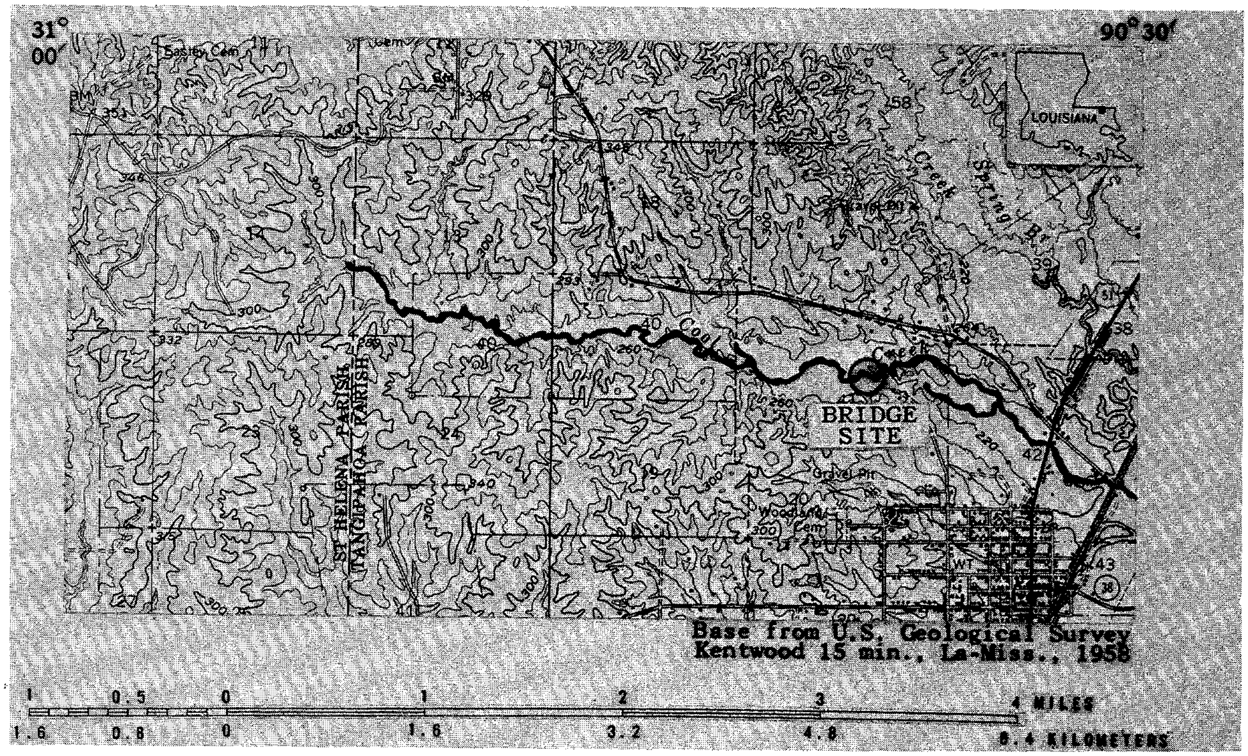


Figure 169. Location of Cool Creek at I-55 near Kentwood, Louisiana.

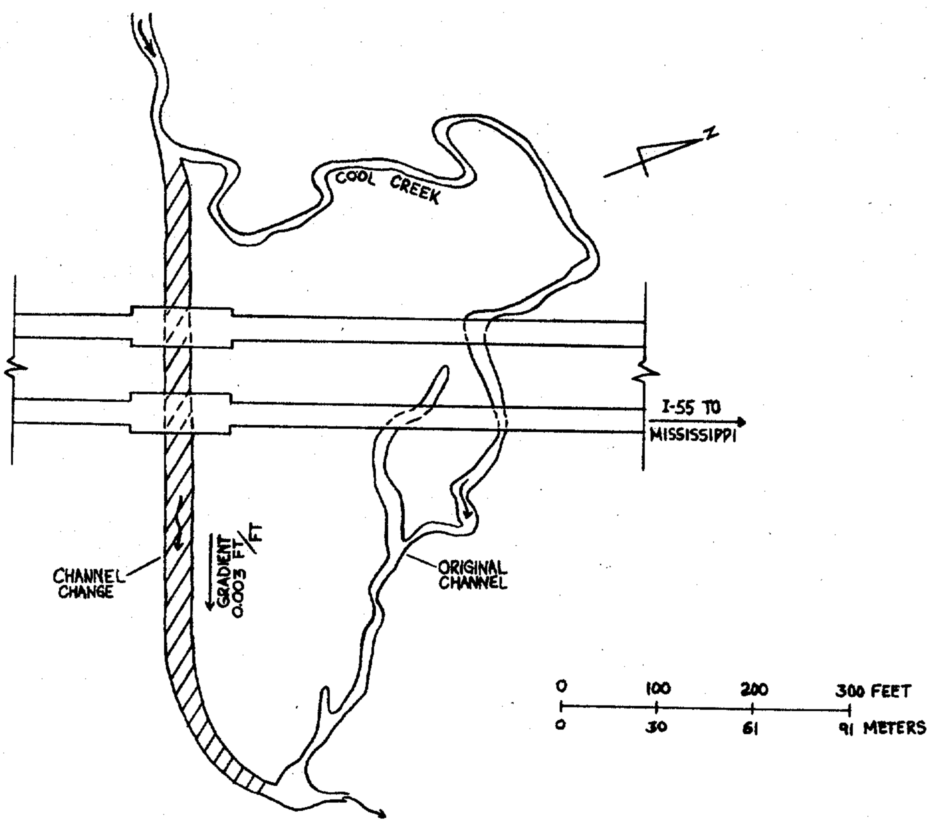


Figure 170. Channel alignment changes made at time of bridge construction.

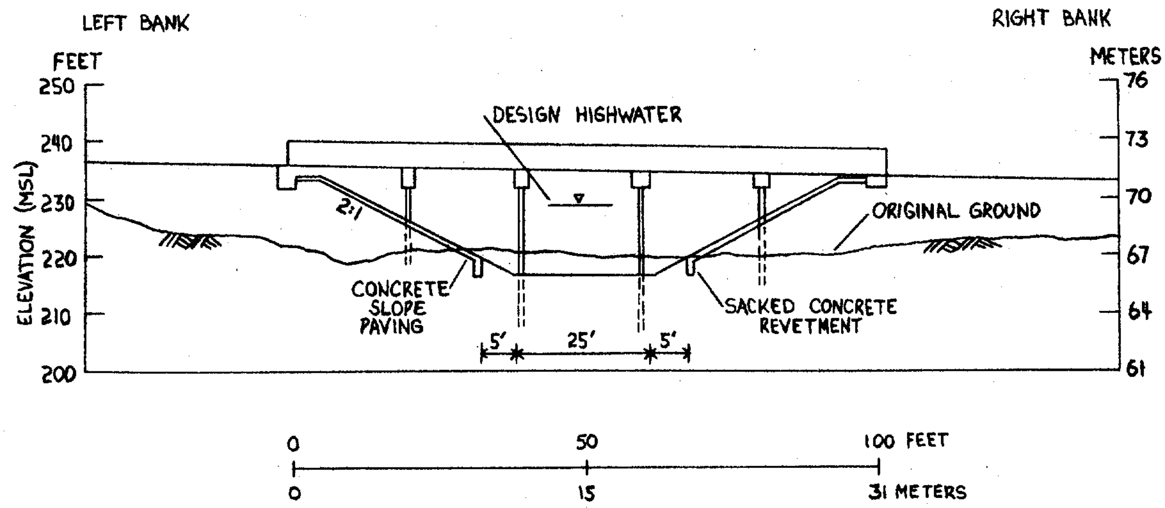


Figure 171. Cross section of channel at bridge site.

Figure 172. Aerial view of bridge site in 1975 showing failure of abutment protection and location of former creek channel. (From Louisiana Dept. of Highways.)

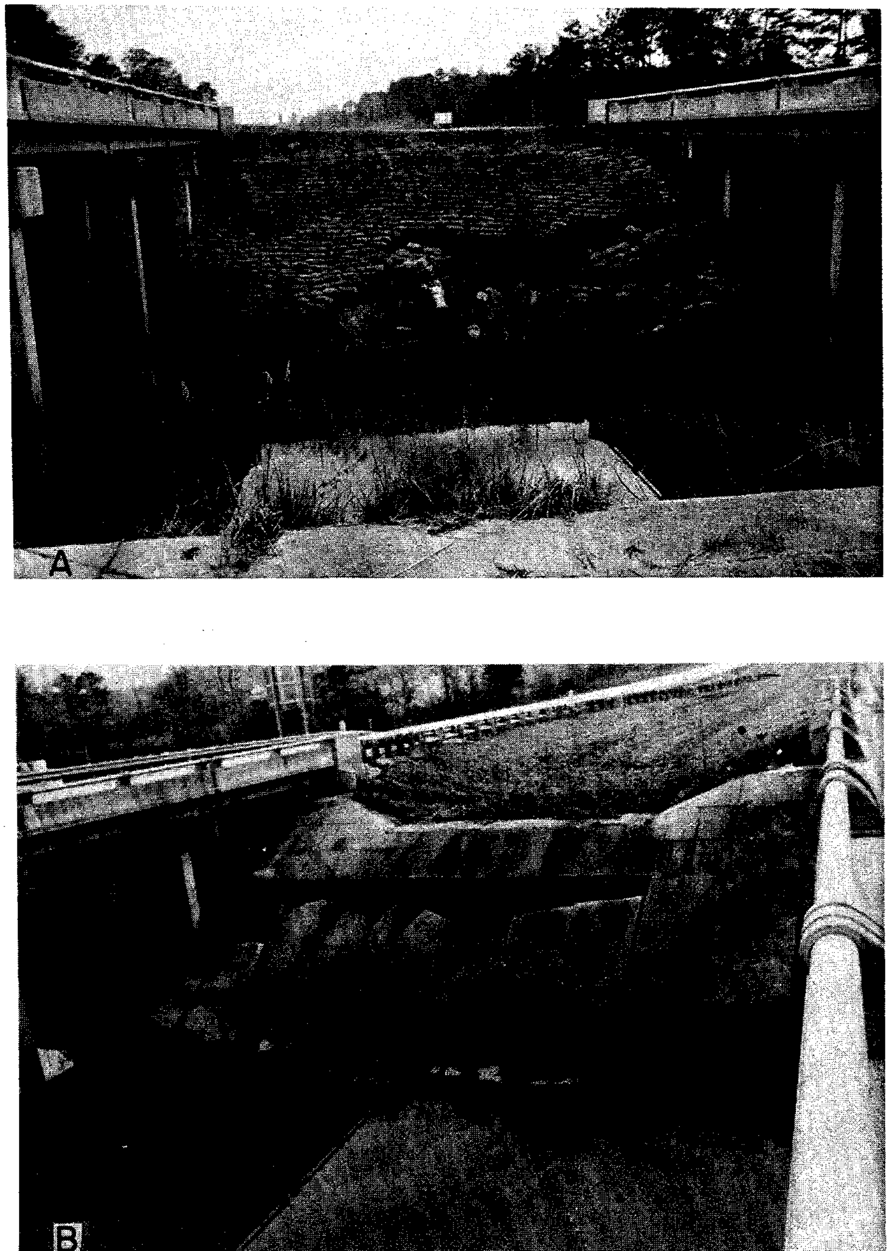


Figure 173. A, sacked concrete revetment and B, concrete pavement damaged in 1974 flood, as photographed in 1976.

SITE 46. FOURCHE LA FAVE RIVER AT SR-28 NEAR BLUFFTON, ARK.

Description of site: Lat 34°55', long 93°35', location as shown on fig. 174. Bridge is 504 ft (154 m) in length; seven webbed concrete piers consisting of a pair of rectangular columns and founded on spread footings, and two pile bents; one pier and one pile bent located on fill-slope of left abutment; spillthough abutments on right bank. Crossing is at bend of river and on flood plain having distinct overflow channels. Skewness of bridge crossing and of piers to flow is 21°.

Drainage area, 491 mi^2 (1,272 km^2); bankfull discharge, 14,400 ft^3/s (408 m^3/s); channel width, 1,400 ft (427 m); stream is perennial with semi-alluvial, gravel bed, in high relief valley, wide flood plain. Channel is sinuous, locally anabranched, random width variation, not incised, cut banks rare, silt-sand banks with some gravel. Flood plain has distinct channels that are attributed to scour by overbank flow (fig. 175).

Hydraulic problems and countermeasures:

1967 Bridge built.

1971 During flood of December 1971 discharge (75,000 ft^3/s or 2,124 m^3/s) serious bank scour occurred on the left bank about 35 ft (11 m) upstream from the bridge (fig. 174). This scour is attributed to concentration of overflow on the left bank, which is partly return flow from the flood plain and partly channel flow deflected at the upstream bend (fig. 175). Local scour at bent 10 (fig. 174) was about 3 ft (1 m) deep, and at pier 9, 6 ft (1.8 m) deep. Water on the left-bank flood plain returned to the main channel upstream from the bridge and caused scour of the unprotected bank. Then, after passing under the bridge, flows return to the flood plain and cause scour of the unprotected downstream bank. The downstream left bank eroded (fig. 174), and a hole 60 ft (18 m) in diameter and 6 ft (1.8 m) deep developed about 180 ft (55 m) downstream from the bridge and 20 ft (6 m) shoreward from the top of the bank. Scouring at the downstream location is associated with return of flood flow to the left-bank flood plain. Clearing of vegetation on the left bank prior to the flood probably contributed to bank erosion. Effect of erosion on bridge was potential only.

The distribution of flow at the bridge site during the December 1971 flood indicates flow from the left-bank flood plain occupied 150 ft (49 m) of the 504-ft (154-m) bridge (fig. 176). Because of poor flow conditions at the juncture of the left-bank overflow and main channel flow near the left-bank abutment, about 70 ft (21 m) of the bridge opening was not used to transfer flows downstream.

1973 A spur dike, revetted with rock riprap and about 175 ft (53 m) in length, was installed on the left bank upstream from the bridge abutment. The dike was extended downstream 255 ft (78 m) from the bridge to prevent erosion of the bank when flows leave the main channel. A rock riprap spillway (berm) was built upstream from the bridge next to the spur dike (fig. 174) to prevent lateral erosion of the bank.

1974 Flood, discharge of about 60,000 ft^3/s (1,700 m^3/s), occurred in June. Spur dike and dike extension performed satisfactorily, but apparently caused some erosion of right bank downstream from bridge.

Discussion: This stream severely constricted (by 53 percent) during the 1971 flood that caused bank erosion problems both upstream and downstream from the bridge. Removal of natural vegetation along the channel banks may have contributed to the bank erosion. The left-bank side of the main channel in the vicinity of the bridge was subject to erosion primarily because overbank flows occupied a large natural channel on the left-bank flood plain which terminated at the left bank approach embankment. Flow patterns near the bridge were then affected by the curvature of the main channel upstream from the bridge and the embankment, which forced the left-bank overflow toward the main channel. The lack of a spur dike to aline overbank flows with flows in the main channel caused a net reduction in effective bridge length of 70 ft (21 m) during the December 1971 flood.

Bank erosion also occurred downstream from the bridge constriction. During the December 1971 flood, approximately 47 percent (35,300 ft^3/s or 1,000 m^3/s) of the total flow was forced to leave the flood plain and pass through the bridge constriction. Part of the remaining 53 percent of flow (39,700 ft^3/s or 1,124 m^3/s) exceeded the capacity of the main channel. As a result, flows formerly on the flood plain and more temporarily constricted at the bridge could not be contained in the main channel and subsequently reentered the flood plain downstream from

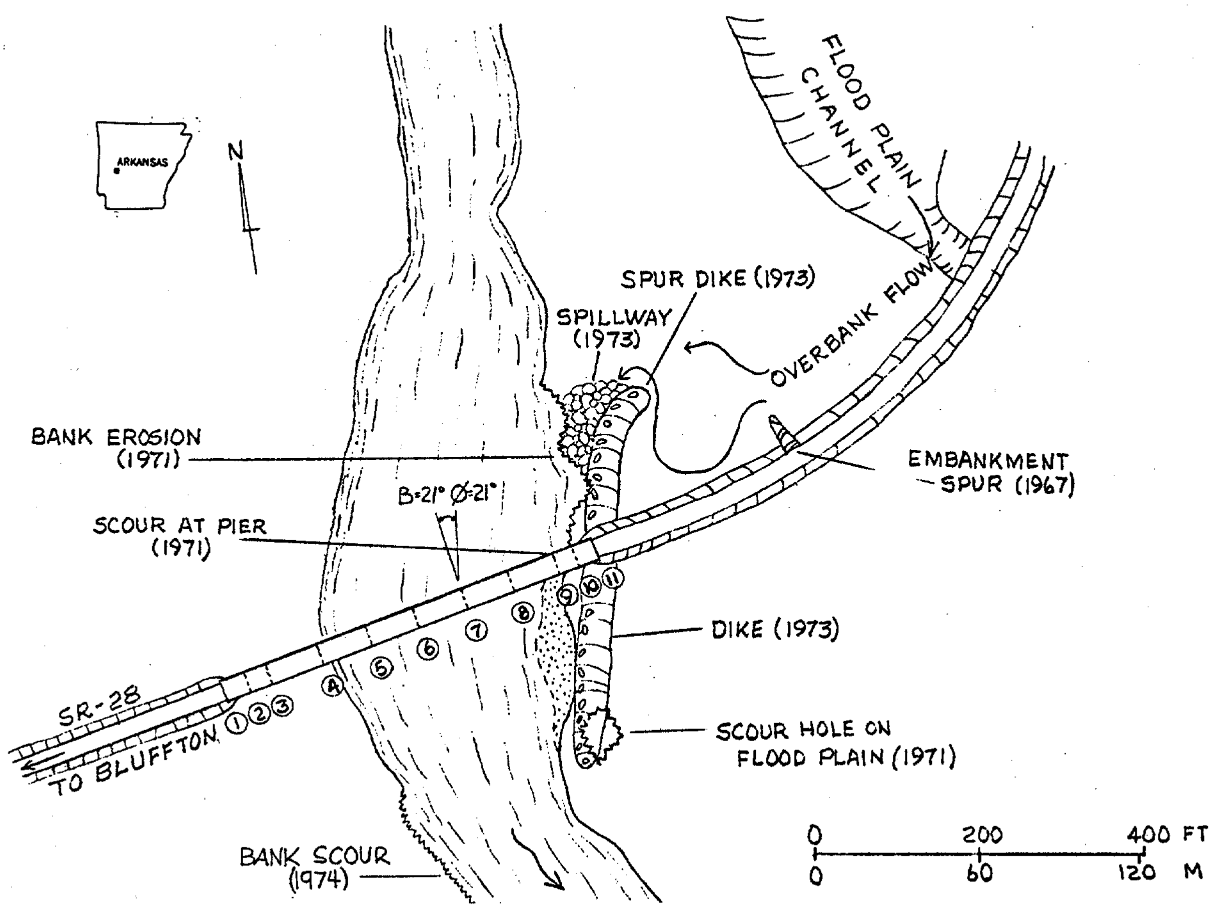


Figure 174. Plan of Fourche La Fave River at SR-28. (Based on aerial photograph dated 12-27-72.)

the bridge on both banks of the stream. Local areas of bank erosion at the points of reentry then occurred. A dike constructed in 1973 as an extension of the spur dike on the left bank prevented the problem of bank erosion on the left bank during the 1974 flood. During the 1974 flood, bank erosion on the right bank downstream from the bridge represents the effect of flow alinement changes caused by the left-bank spur dike extension.

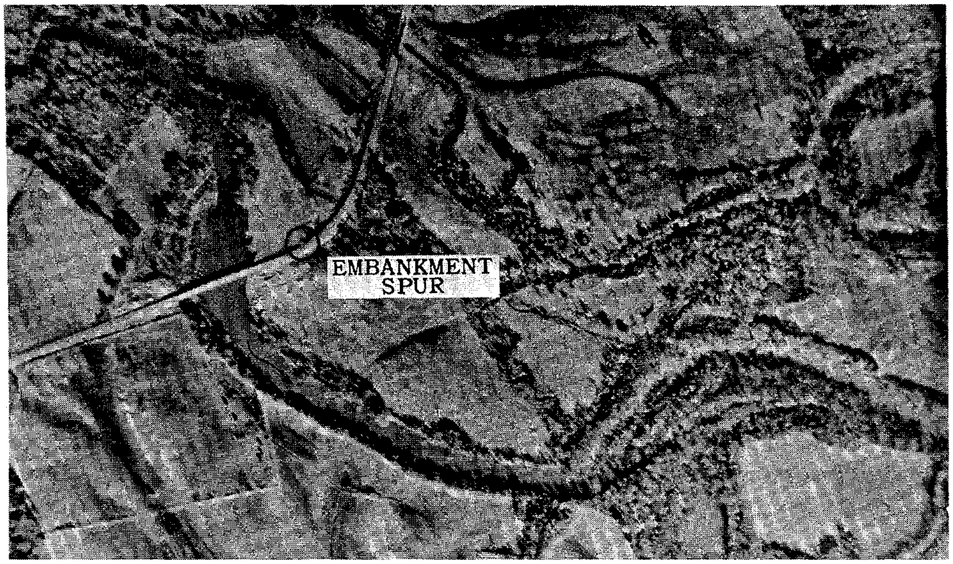


Figure 175. Aerial photograph of Fourche La Fave River at SR-28, taken in 1972 prior to installation of countermeasures. (From Arkansas Dept. of Highways.)

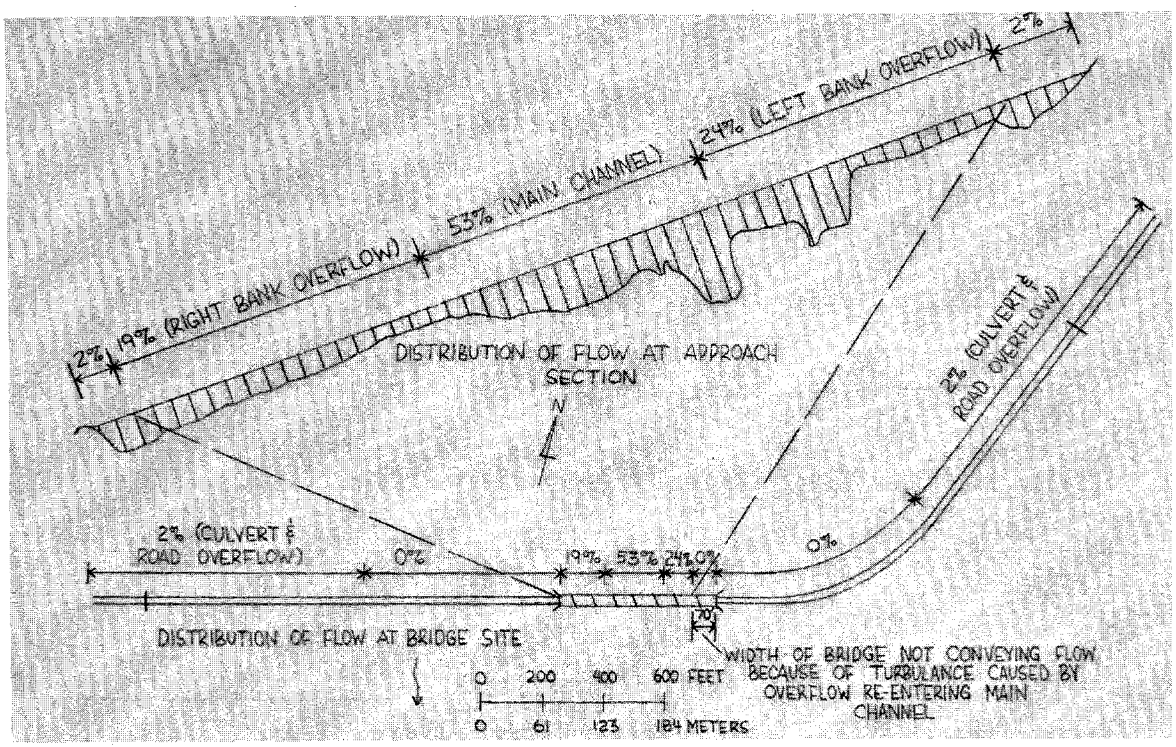


Figure 176. Approach cross section and flow distribution for flood of December 10, 1971, Fourche La Fave River at SR-28. (From J. N. Sullavan, written communication, 1972.)

SITE 47. BULL MOUNTAIN CREEK AT SR-25 NEAR SMITHVILLE, MISS.

Description of site: Lat 34°05', long 88°24', location as shown in fig. 177. Steel girder bridge with concrete decking, length 1,050 ft (320 m), supported by concrete-covered steel-pile bents, three spaced at 30 ft (9 m), one at 210 ft (64 m), and 25 at 30 ft (9 m). Center spans are supported by concrete piers on pile supported footings. Abutments are spillthrough type. Bridge skew to flow is zero degrees.

Drainage area, 335 mi^2 (868 km^2); valley slope, 0.0008; channel width, about 80 ft (25 m). Stream is perennial, alluvial, sand bed, in valley of low relief, wide flood plain. Channel is meandering, locally anabranched, tree cover at greater than 90 percent of bankline.

Hydraulic problems and countermeasures:

1952 Bridge constructed, abutments protected with concrete slope paving (figs. 177 and 178). A short earthen spur dike about 15 ft (5 m) in length was constructed at the left-bank abutment.

1955 Flood of March 1955 destroyed the spur dike and scoured holes under bents 2-5.

1957 Flood of February 1957 scoured a hole about 120 ft (37 m) long and 8 ft (2.4 m) deep at bents 2-5 (fig. 178). Discharge measurements indicate flow approached the bridge at angles up to 20°.

1964 At some time prior to 1964, the 15-ft (5-m) spur dike was rebuilt. A cross-section measurement made during March 1964 showed the scour hole observed in 1957 had filled and flow alinement was normal near the abutment.

1973 During flood of March 1973, a water discharge of about 36,000 ft^3/s (1020 m^3/s) flowed under the bridge, and 8,500 ft^3/s (241 m^3/s) flowed over the roadway on the left-bank flood plain. Presence of the spur dike caused flows to approach the main channel opening normally, maximum point velocities were about 5.5 ft/s (1.7 m/s), and the scour holes near bents 2-5 were no longer evident.

Discussion: Local scour around bridge pile bents and at the upstream toe of the abutment occurred after the spur dike was destroyed. Construction of a new spur dike provided protection of the abutment from turbulent flows during a flood with 40-year recurrence interval and flow depths of about 6 ft (1.8 m).

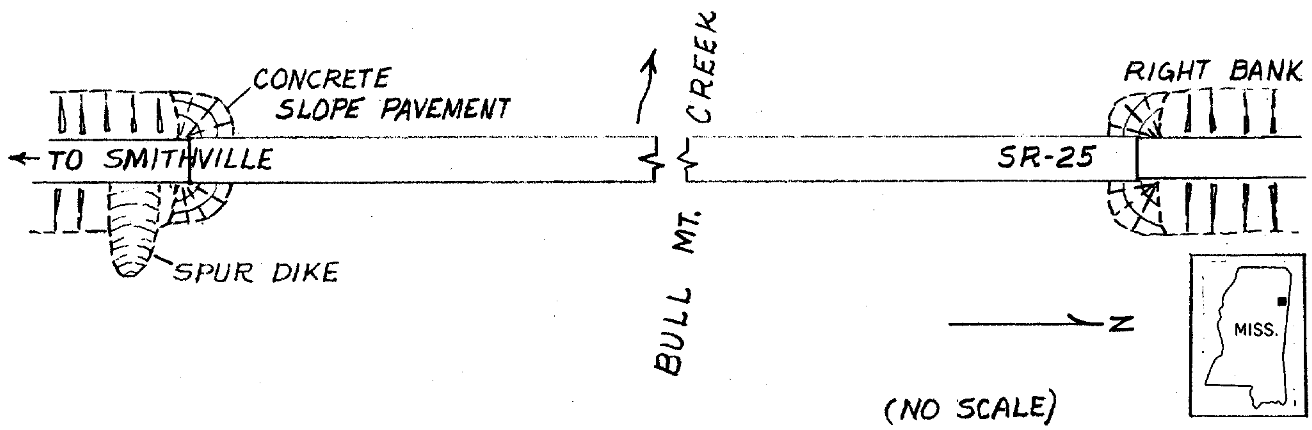


Figure 177. Plan sketch of SR-25 crossing at Bull Mountain Creek.

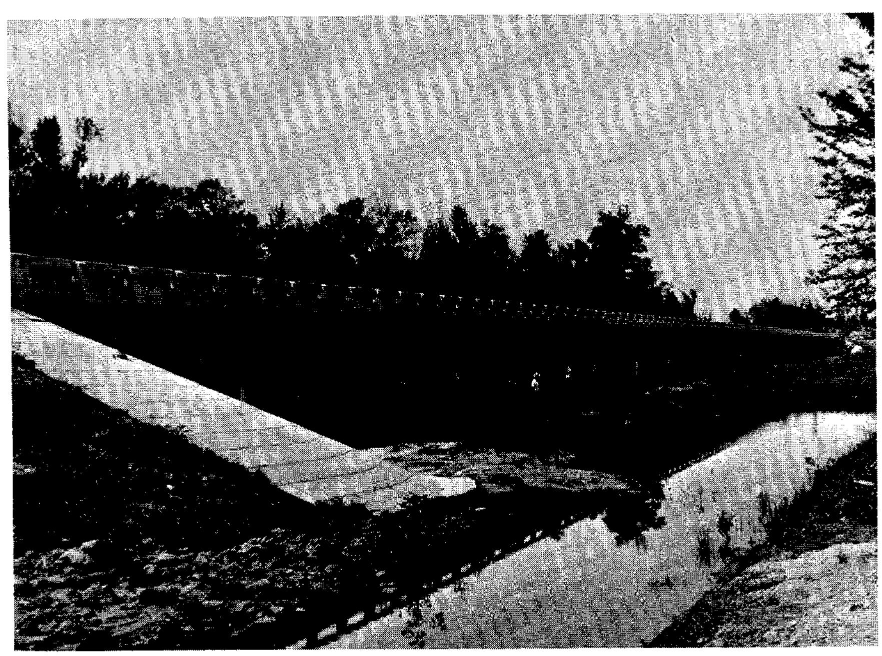


Figure 178. View of upstream side of left bank abutment and scour hole, as photographed in 1957. (From Miss. Dept. of Highways.)

SITE 48. BEAVER CREEK AT I-55 NEAR TANGIPAHOA, LA.

Description of site: Lat 30°53', long 90°32', location as shown in fig. 179. Dual concrete T-beam bridges, 162 ft (49 m) long, supported by 3 bents having five rectangular columns per bent. Centerlines of adjacent bridges are 88 ft (27 m) apart. The channel was relocated (fig. 179) to facilitate bridge construction. The spillthrough abutments are protected with concrete slope paving.

Drainage area, 23 mi^2 (60 km^2); valley slope, about 0.0024. Stream is perennial but flashy, alluvial, sand bed, in valley of low relief, wide flood plain. Channel is meandering, silt-sand banks.

Hydraulic problems and countermeasures:

1965 Bridge built. Slope of relocated channel was about 0.0059 ft/ft compared with the original channel slope of about 0.0034 ft/ft. Countermeasures included clearing and grading of the channel, and construction of concrete slope paving to protect the abutments.

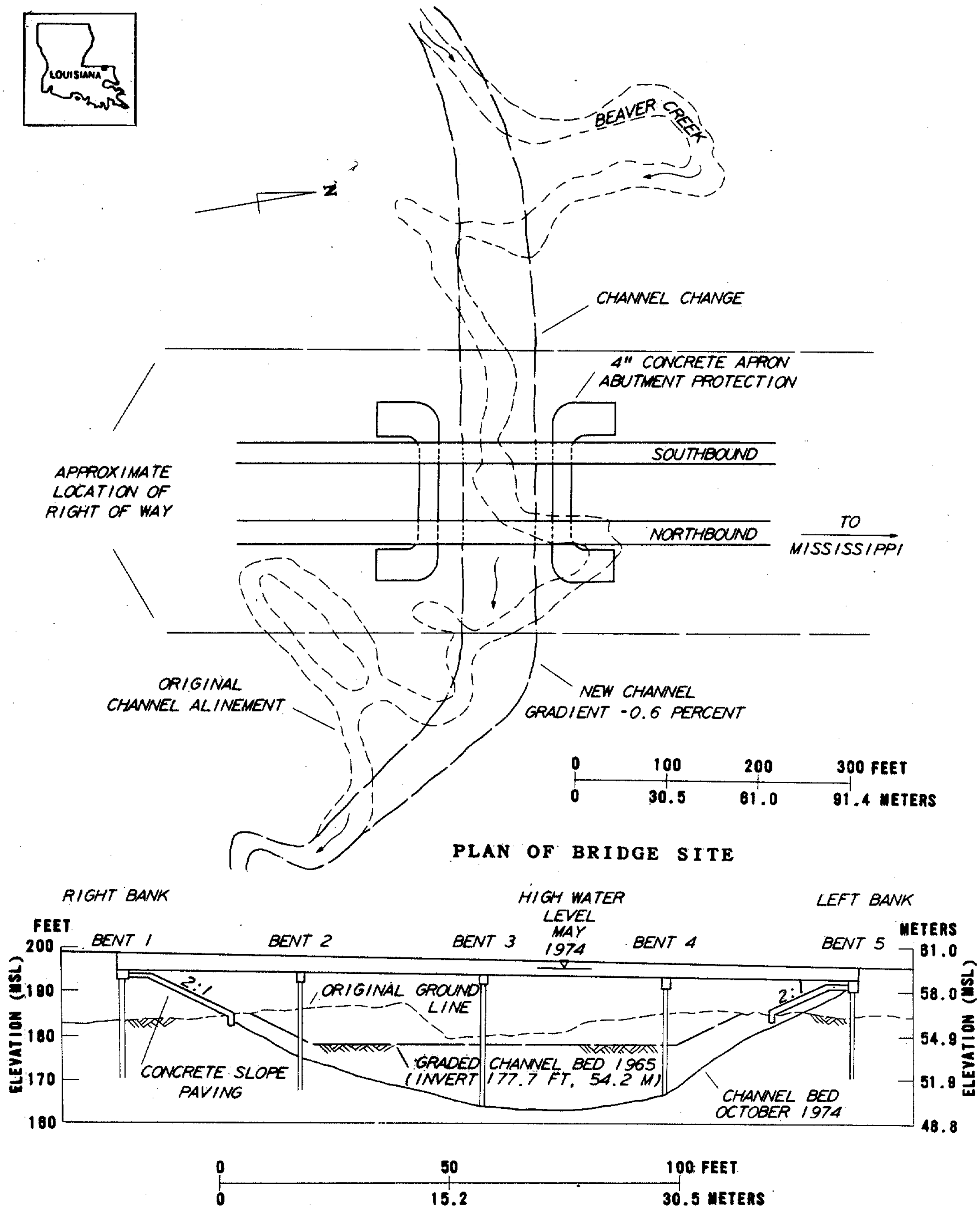
1974 Flood of May 22, 1974 overtopped the roadway. The estimated peak discharge was about 21,400 ft^3/s (606 m^3/s), more than 1.5 times the discharge of the 100-yr R. I. flood. Considerable damage to both abutments and the concrete slope protection occurred (fig. 180) during the flood. The channel bed scoured about 14 feet (4.3 m) near the downstream side of the southbound lane (fig. 181). Inspection of the aerial photograph (fig. 180) and 1974 cross section surveys of the channel indicate that most of the channel bed scour in the vicinity of the bridges was probably caused by contraction of the flow during the 1974 flood. The largest amount of scour occurred around the upstream bridge, which provided the initial and greatest constriction of the flow.

Some of the scour observed after the flood may have been caused by channel degradation, typically associated with channel relocation work. Lateral erosion on the right bank below the downstream bridge (fig. 180) was probably caused by eddy currents as flows expanded following contraction through the bridge openings. Expansion of flow downstream from the bridge and resultant sediment deposition is indicated in the aerial photograph.

1975 Countermeasures applied at the crossing to prevent further scour of the abutments included placement of toe walls, and the use of broken limestone rock riprap along the channel bank (fig. 182).

1977 Some of the limestone rock riprap on the channel banks has been removed by high water (fig. 183).

Discussion: The constriction associated with the dual bridges caused the greatest amount of scour at the upstream bridge. Scour depth of about 14 ft (4.3 m) were caused by a combination of the bridge constriction and the degrading channel. Lateral erosion of the channel at the downstream bridge abutments and downstream from the contraction were probably caused by eddy currents as flows expanded downstream from the bridge. There is evidence that the high velocities and channel degradation occurring during the 1974 flood are associated with the relocated and graded channel. The concrete slope pavement acted as rigid revetment, and when undermined or overtopped, were subject to progressive failure because there was no support for the remaining revetment protection. Toe walls might have prevented undermining.



CROSS SECTION OF CHANNEL AT DOWNSTREAM SIDE OF SOUTHBOUND LANE

Figure 179. Plan and cross section of Beaver Creek at I-55 bridge.

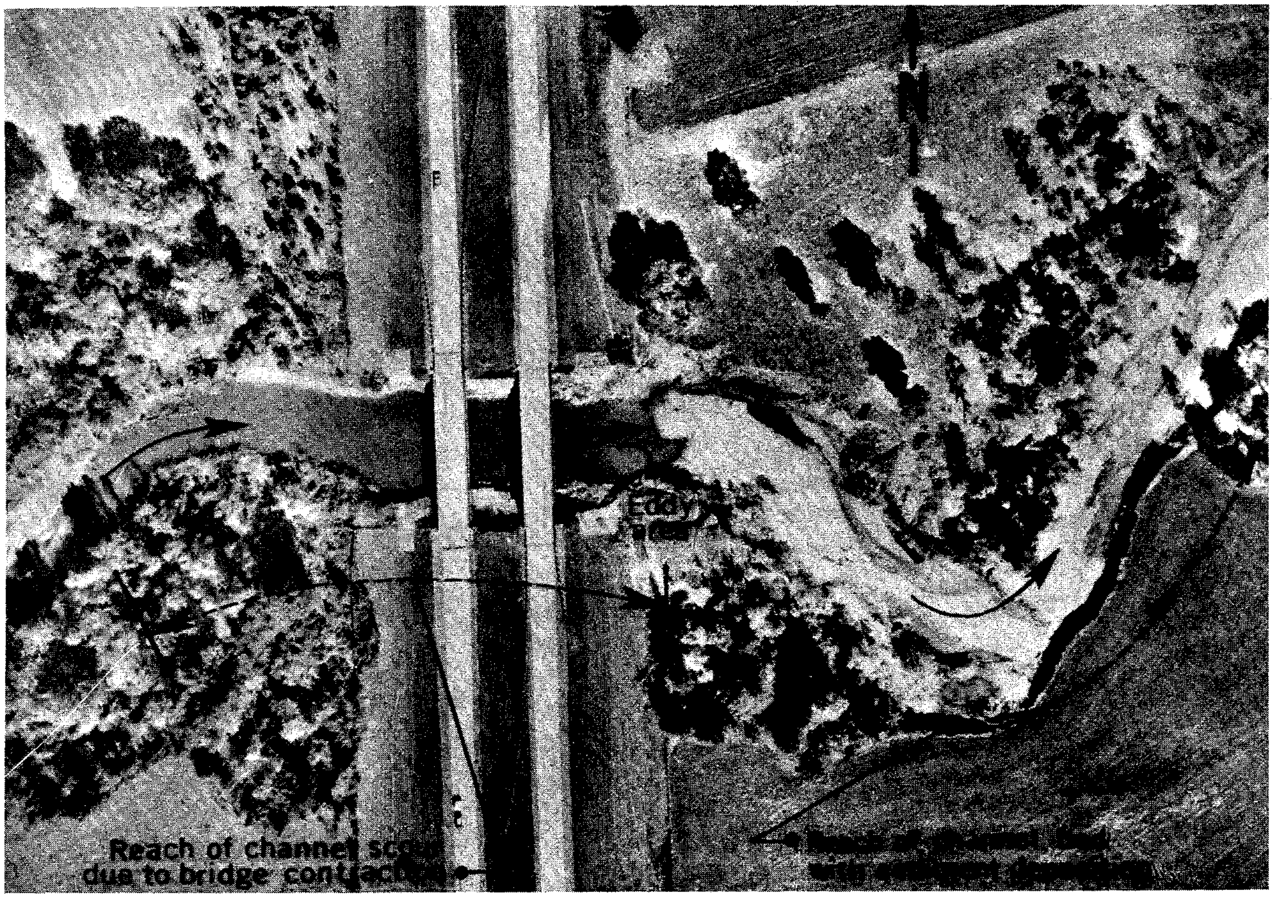


Figure 180. Aerial photograph showing channel bed scour in vicinity of the bridges. (From Louisiana Dept. of Transportation.)

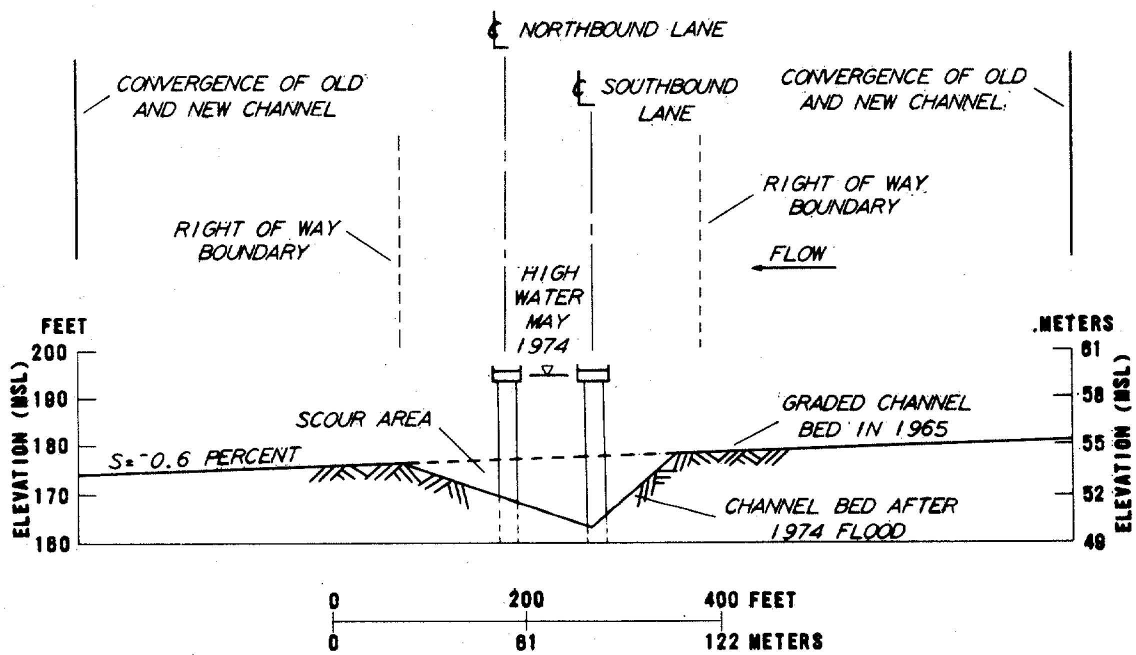


Figure 181. Profile of channel bed before and after 1974 flood.

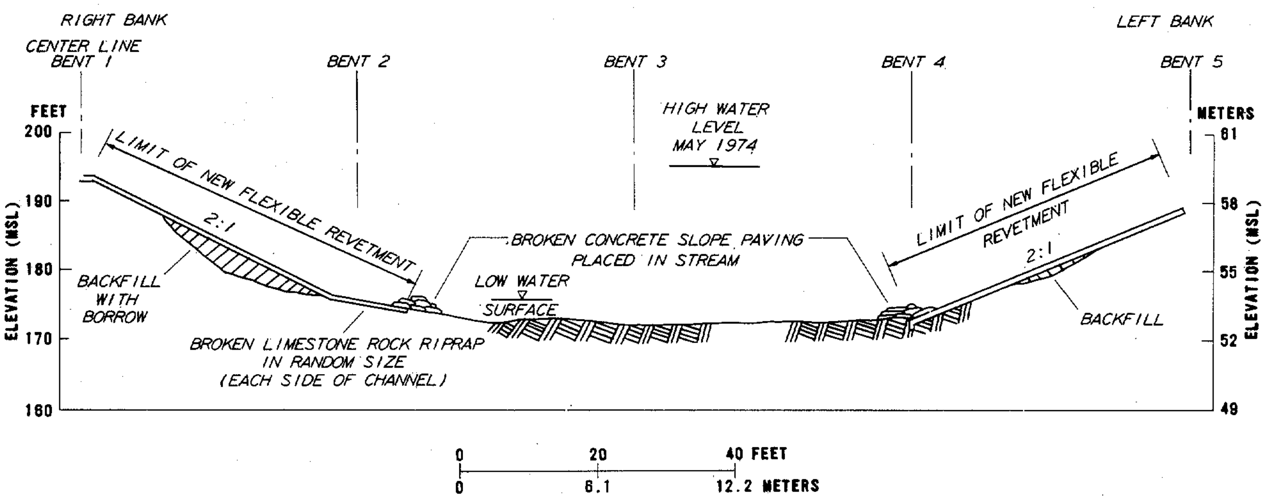


Figure 182. Details of channel protection constructed after 1974 flood.

Figure 183. View of left bank abutment and rock riprap in March, 1977.

SITE 49. EAST PEARL RIVER AT I-10 NEAR BAY ST. LOUIS, MISS.

Description of site: Lat 30°18', long 89°38', at location shown in fig. 184. Bridge consists of 66 spans of 60-ft (18-m) length, six spans of 90-ft (27-m) length, prestressed-concrete girders, and one 482 ft (147 m) bascule and steel girder span for an overall length of 4,982 ft (1519 m) (fig. 185). Abutments are spillthrough type. Bents are precast-concrete with concrete-pile supports. Bridge piers supporting the main span are protected by a timber fender system. Channel and bridge designed for navigation.

Drainage area, 8,700 mi^2 (22,533 km^2); valley slope, about 0.0001; channel width, 250 ft (76 m). Stream is perennial, alluvial, sand bed, in valley of low relief, wide flood plain, low natural levees. Channel is meandering, locally anabranched, equiwidth, cut banks rare, silt-clay banks; tree cover at more than 90 percent of bankline. As the Pearl River approaches the sea, it flows through swampland and divides into three meandering channels (connected by minor anabranches), of which the East Pearl River is one.

Hydraulic problems and countermeasures:

1964 The East Pearl River channel was cleared and dredged for navigation. The deepest point of the riverbed near the bridge, located 100 ft (30 m) upstream from Interstate 10, was about 43 ft (13 m) below mean sea level in 1961 (fig. 186). During construction of the navigation channel, the riverbanks and channel were cleared downstream from the proposed Interstate 10 crossing. Upstream from the proposed highway, islands were removed and the channel was straightened.

1966 Bridge built with long approach trestles and bascule spans to provide clearance for navigation and reduce the amount of flow contraction. A spur dike was built at the right bank abutment to prevent local scour and lateral erosion (fig. 187). A discharge measurement made during flooding in March 1961 indicated that about 60 percent of the total Pearl River flow normally passes down the East Pearl River (main) channel and adjacent flood plain. Long approach trestles leading to the main channel bridge were built (fig. 185) to prevent a large degree of flow contraction at the site. The length of the approach trestles was based on the proportion of flow on the flood plain normally included with the East Pearl River channel.

The East Pearl River channel is used for navigation, and large pier fenders, 40 ft (12 m) wide and 225 ft (69 m) long were constructed to protect the two main concrete piers (fig. 188) which are supported on piles. The piers and fenders, however, tend to constrict flows in the main channel and create turbulence in the vicinity of the bridge.

1972-75 Floods in 1973, 1974, and 1975 caused about 80-ft (24-m) shift in the channel alinement and extensive general scour of the channel at the bridge. The flood of April 1974 was equal to about a 50-year flood. The scoured area was about 800 ft (244 m) long, 120 ft (37 m) wide, and 20 ft (6.1 m) deep. During the 1974 flood, pile foundations of the pier guards were undermined (fig. 189) and the bridge foundation endangered. Because little amounts of sediment are carried by the stream, it was considered unlikely the scour hole would refill naturally in the near future.

Changes in hydraulic properties of the East Pearl River main channel for conditions before and after bridge construction indicate that the bridge piers effectively constricted the waterway, (table 6) even though flows on the flood plain were not significantly affected by the bridge. The channel remained stable between 1964 and 1973 when flows were average. Extensive high water, however, occurred during the period 1973-75, and caused much of the observed channel change at the bridge contraction. Average monthly flow for January 1974 and 1975 was about 45,000 ft^3/s (1,274 m^3/s), four times the average monthly flow for the period 1938-67. On the basis of observed channel changes at this site, (table 6) 21 ft (6.4 m) of general scour between the bridge piers occurred as a result of the bridge pier and fender construction which initially occupied 31 percent of the waterway area. By the end of the 1974-75 flood period, the bridge piers and fenders occupied about 23 percent of the waterway.

1975 Countermeasures designed and built to prevent further general scour at the bridge included: (1) Placement of a gravel fill near bent 40 (fig. 189) to increase the bed elevation and coverage of piles. (2) Addition of new longer steel piles within the fender and pier system (fig. 190) to provide additional bridge support. (3) Placement of a rock riprap blanket 2-ft (0.6-m) thick around bents 40 and 41 (fig. 189). A stream velocity of 8 ft/sec (2.4 m/s) was used in design of the riprap protection. The riprap has a maximum weight of about 600 lbs (273 kg) and a median weight of about 150 lbs (68 kg).

1977 No floods have occurred to test the countermeasures since bridge repairs were made in 1975.

Discussion: General scour occurred at the bridge, which does not greatly constrict overbank flows. Piers occupied 31 percent of the original waterway. Efforts to enlarge and improve the channel by dredging and straightening did not result in a stable channel. The timing and magnitude of channel changes that occurred as a result of a bridge constriction are associated with flow events at the site. In this case, no channel scour was observed between 1961 and 1972, even though channel improvements were made in 1964, and the bridge built in 1966. Flooding between 1973-75, however, with one crest equivalent to a 50-year flood, caused considerable channel change. General scour up to 20 ft (6.1 m) in depth and extending 200 ft (61 m) upstream and 400 ft (122 m) downstream from the bridge occurred at an average flow velocity of about 4 ft/s (1.2 m/s). This scour is attributed to constriction of flow by piers.

Table 6. Changes in hydraulic properties of East Pearl River main channel between 1961 and 1975

Date of survey	Water-surface elevation (feet)	Elevation of channel bed at lowest point (feet)	Width of channel [2](feet)	Contraction ratio, (M)	Discharge [3](ft³/s)	Gross area [4](ft²)	Area piers (ft²)	Ratio pier area to gross area	Average velocity [5](ft/s)
2-26-61	9.3	-43	306	1	59,400	12,500			4.8
				Bridge built, 1966					
11- 3-69	[1]9.3	-38.3	309	--	--	11,279	3,241	0.29	--
2-25-72	[1]9.3	-40	325	--	--	12,100	3,704	.31	--
4-20-74	9.3	-58	335	1	57,000	14,900	3,712	.25	5.1
6-18-74	[1]9.3	-59	337	--	--	14,700	3,785	.26	--
7-30-74	[1]9.3	-60	338	--	--	14,800	3,801	.26	--
1-24-75	[1]9.3	-61	338	--	--	14,200	3,310	.23	--

[1] Assigned value for comparative purposes.
[2] Conversion factor, feet to meters, is 0.305.
[3] Conversion factor, ft^3/s to m^3/s, is 0.0283.
[4] Conversion factor, ft^2 to m^2, is 0.093.
[5] Conversion factor, ft/s to m/s, is 0.305.

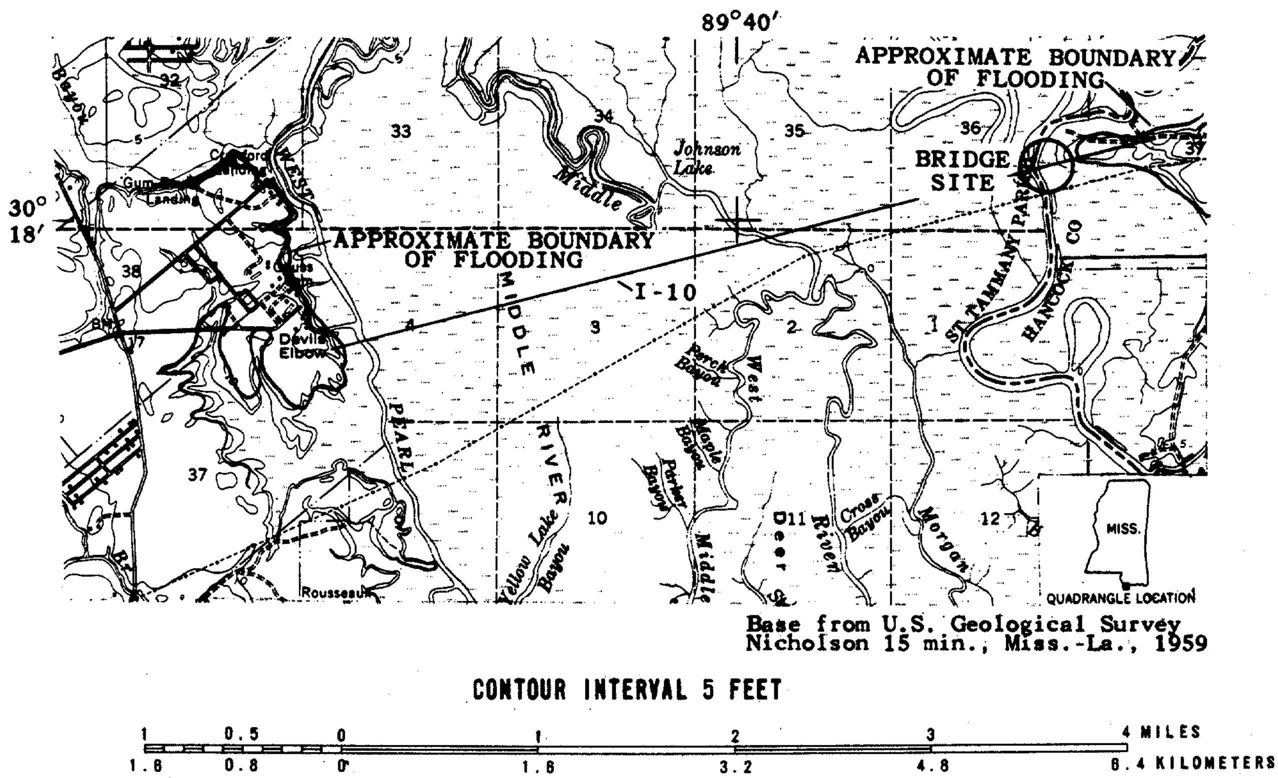


Figure 184. Location of East Pearl River at I-10 crossing near Bay St. Louis, Mississippi.

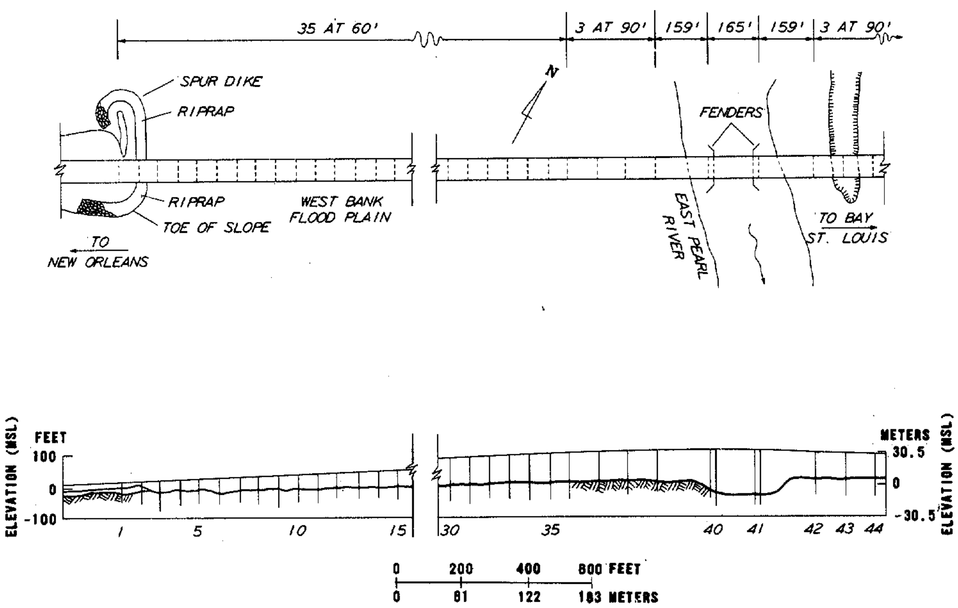


Figure 185. Plan and cross section of I-10 bridge across Pearl River.

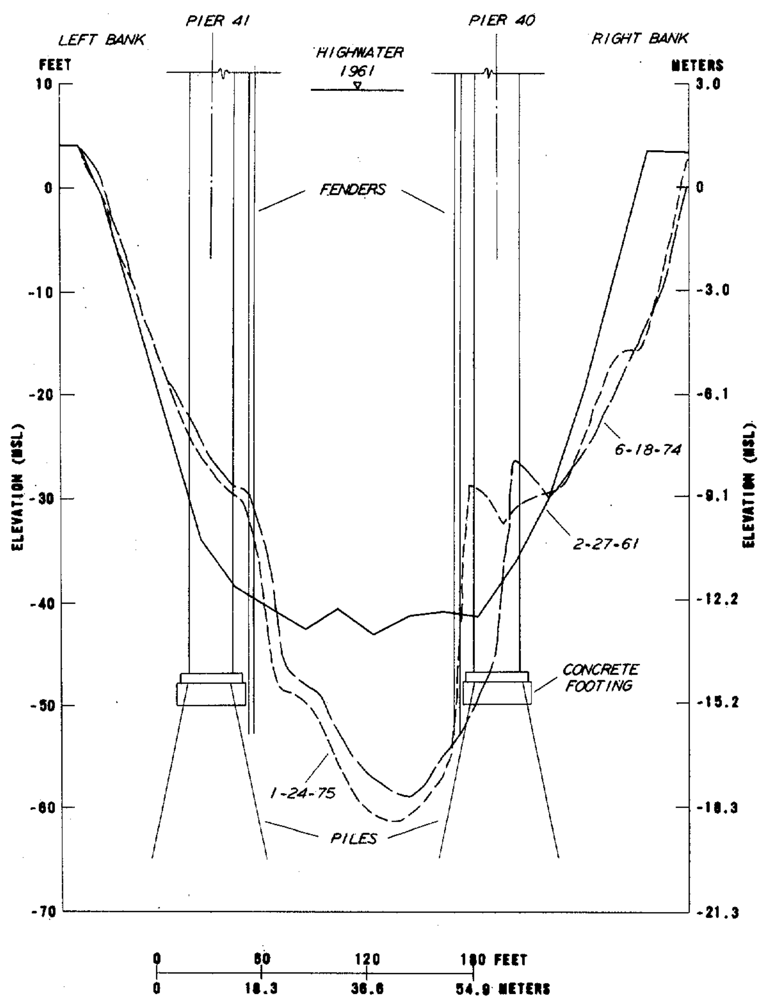


Figure 186. Changes in channel cross section between 1961 and 1975, 38 feet (11.6 meters) downstream from centerline of bridge.

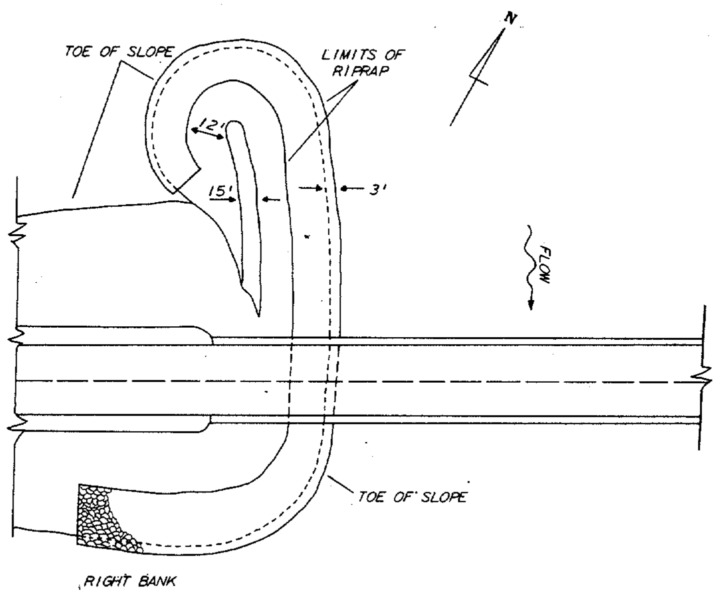


Figure 187. Details of spur dike.

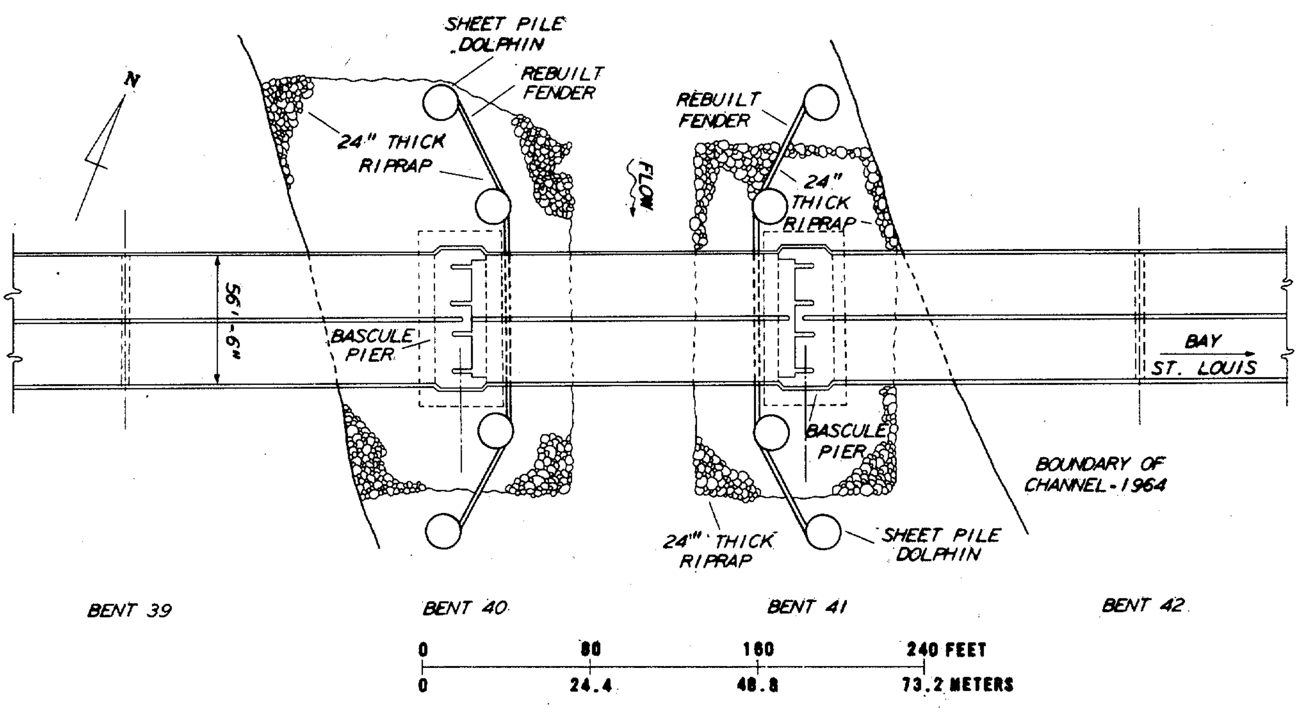


Figure 188. Plan of main channel and bridge footing repairs.

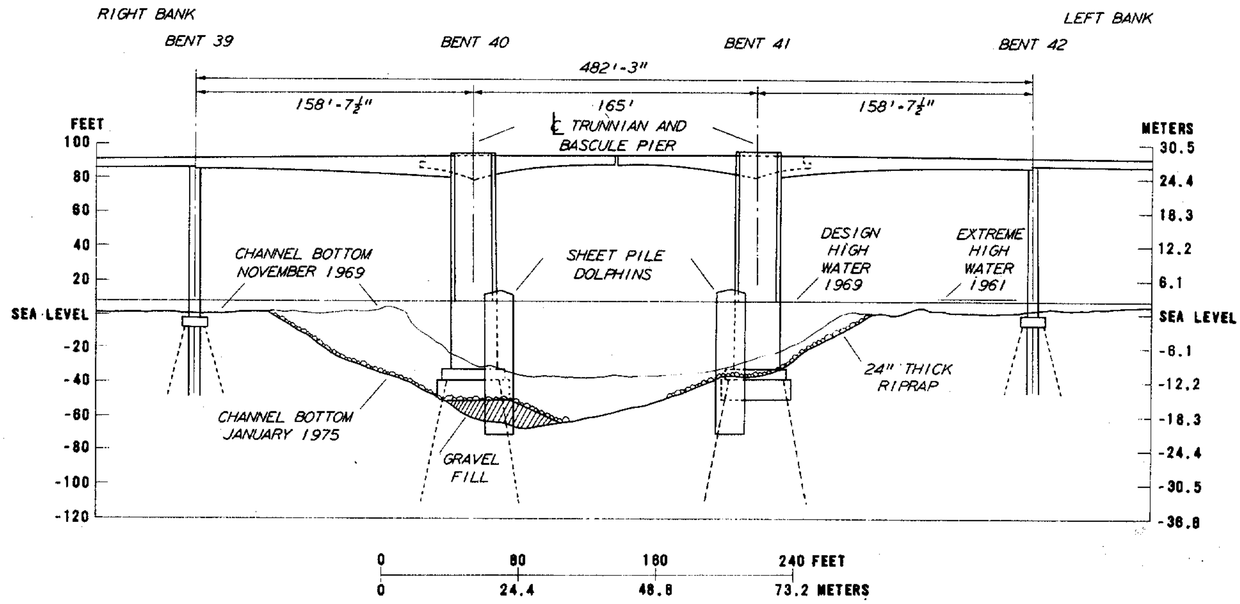


Figure 189. Cross section of main channel showing bridge footing repairs.

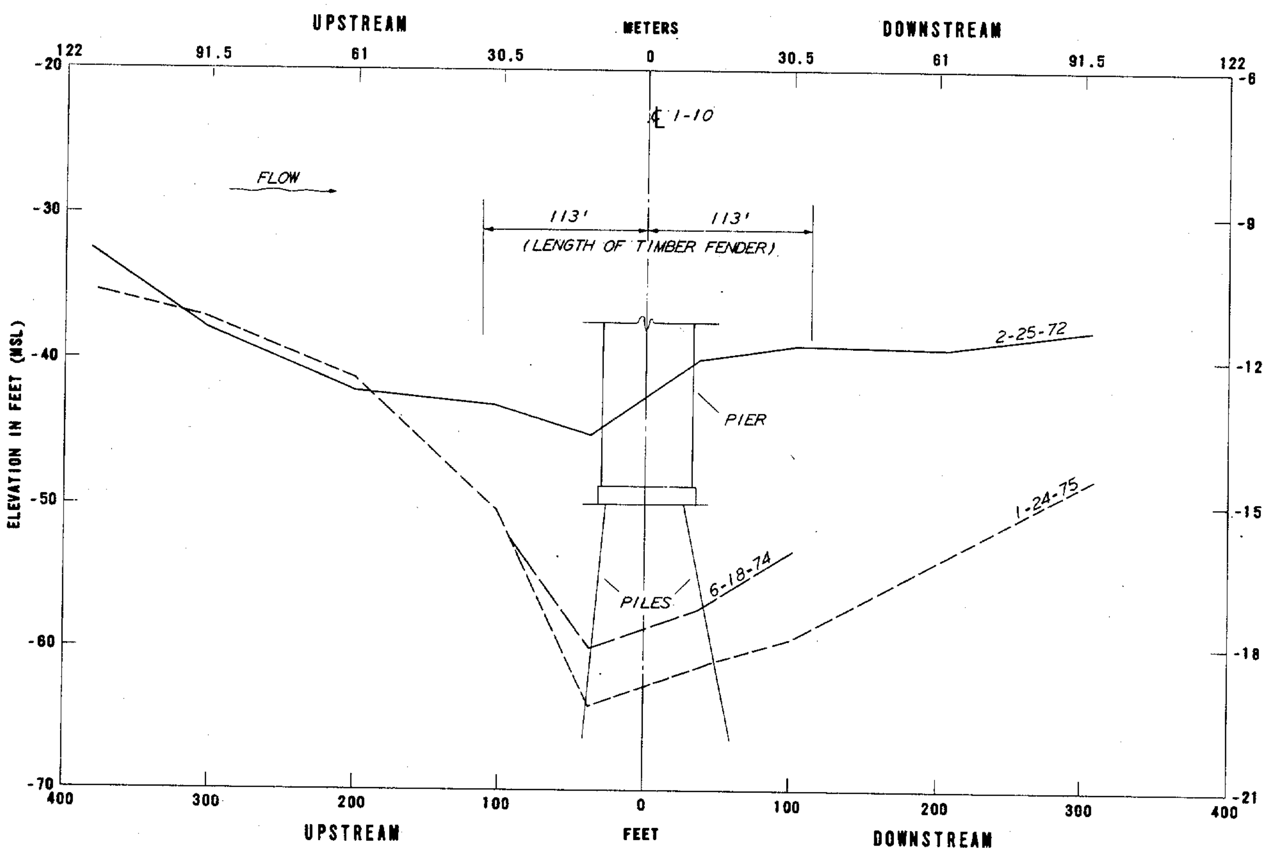


Figure 190. Changes in channel bed profile at bridge between 1972 and 1975.

SITE 50. NICHOLS CREEK NEAR CONFLUENCE WITH TOMBIGBEE RIVER AT US-45 NEAR ABERDEEN, MISS.

Description of site: Lat 33°49', long 88°29', location as shown in fig. 191. A 200-ft (61-m) bridge was built at this site in 1934, consisting of eight 25-ft (8-m) spans supported on timber-pile bents. In 1949, eight 25-ft (8-m) spans supported on timber-pile bents were added to the left-bank end of the bridge. Abutments are spillthrough, protected by rock riprap in the 50- to 100-lb (0.02- to 0.05-t) range.

The bridge is on Nichols Creek where it traverses the flood plain and transmits flood flow of the Tombigbee River. In a sense, the Nichols Creek bridge acts as a relief bridge for floods on the Tombigbee, which has a drainage area of 2,169 mi^2 (5,617 km^2) at the site. Valley slope along Nichols Creek at the bridge is 0.0029. Nichols Creek is perennial, alluvial, silt-sand bed. Channel is meandering, with well-developed natural levees, silt-sand banks. Hydraulic problems at the bridge have occurred during floods on the Tombigbee.

Hydraulic problems and countermeasures:

1934 Original bridge constructed, rock riprap revetment at abutments.

1949 Bridge extended 200 ft (61 m) on the left bank (fig. 192) and the new abutment protected with rock riprap. The bridge was extended after access to another relief bridge had been cutoff by constructing State Route 25.

1955 Flooding in March 1955 (106,000 ft^3/s or 3483 m^3/s) destroyed four spans near the left bank (fig. 193) and several scour holes developed near the left-bank abutment at both the highway and railroad bridges (fig. 192). Damage to the bridge, and problems of scour and lateral erosion are associated with the following factors: (1) Overbank flow approaching the Nichols Creek bridge was highly contracted by the bridge opening (fig. 193). (2) The two opposing directions of flow approaching the bridge opening tended to constrict the effective area of flow. The ineffectiveness of the opening is indicated by the fact that the first span on the left bank was preserved (fig. 193) even though four adjacent spans were destroyed. (3) Existing borrow pits and ditches along the road and flood plain tended to concentrate flows along the approach embankment. (4) The capacity of the railroad bridge was insufficient for the flood and excess water continued toward the southeast between the railroad and highway embankments (fig. 193). Eddy currents set up at the downstream side of the highway subsequently caused lateral erosion of the approach embankment. Lateral erosion at the downstream side of the approach embankment is primarily associated with eddy currents set up by flows contained between the highway and railroad embankments.

1956 Bridge rebuilt with steel H-piling rather than the timber piling previously used. The scour holes near the bridge were filled and covered with a layer of rock riprap in the 50- to 100-lb (0.02- to

0.05-t) size range. A spur dike was constructed upstream from the left-bank abutment (figs. 192 and 194) and protected with similar riprap.

1973 In March 1973, an even larger flood (123,000 ft^3/s, or 3483 m^3/s) occurred, and overbank flooding persisted for 7 days. There was no scour problem at the bridge, riprap placed under the bridge in 1956 was not disturbed, and there was no visible scour just downstream from the riprapped area. Although the spur dike was overtopped by about one foot, only minor scour occurred along the inside top and near the toe of the dike.

Discussion: A major cause of inefficiency in conveyance of flow at this relief bridge (as compared with normal waterway openings) is the effect of opposing jets of flow moving adjacent to the embankment, which must turn at right angles in order to pass through the bridge opening. Spur dikes effectively alined overbank flow with the bridge opening and retained their structural integrity--even though overtopped--because they were protected with riprap and built with gentle side slopes. Construction of borrow pits and drainage ditches on the flood plain upstream from the approach embankment caused a change in flow patterns approaching the bridge opening and endangered the bridge. Excessive flows that did not pass through the railroad bridge moved instead downstream between the highway and railroad embankments, with resultant eddy currents and velocity that eroded the downstream side of the highway embankment.

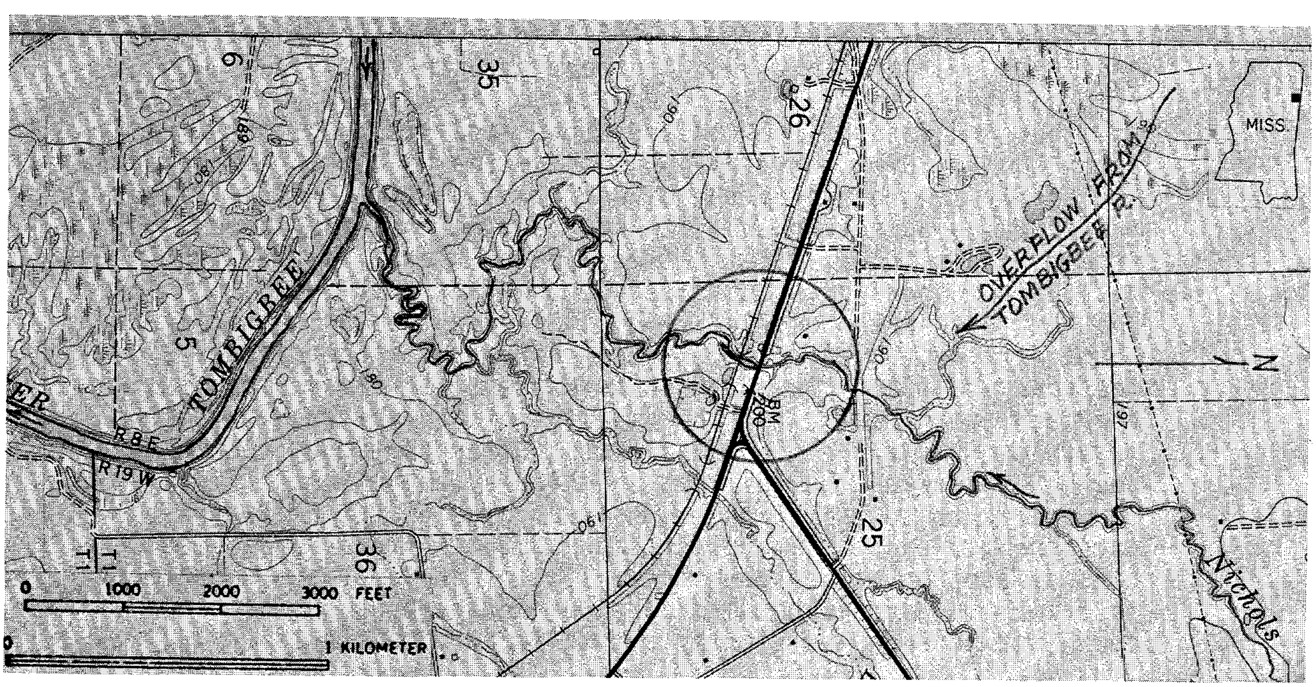


Figure 191. Location of US-45 crossing, Nichols Creek. (Base from U.S. Geol. Survey Amory SW, Miss., 7.5' map, contour interval 10 feet, 1966.)

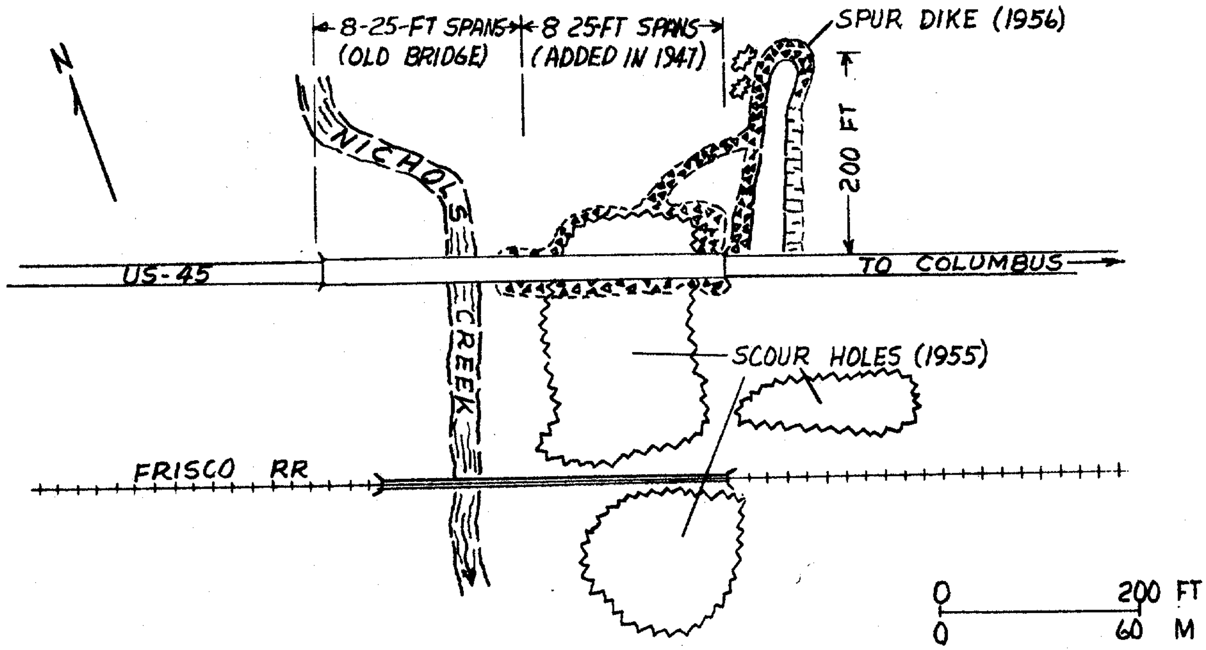


Figure 192. Plan sketch of US-45 crossing, Nichols Creek.

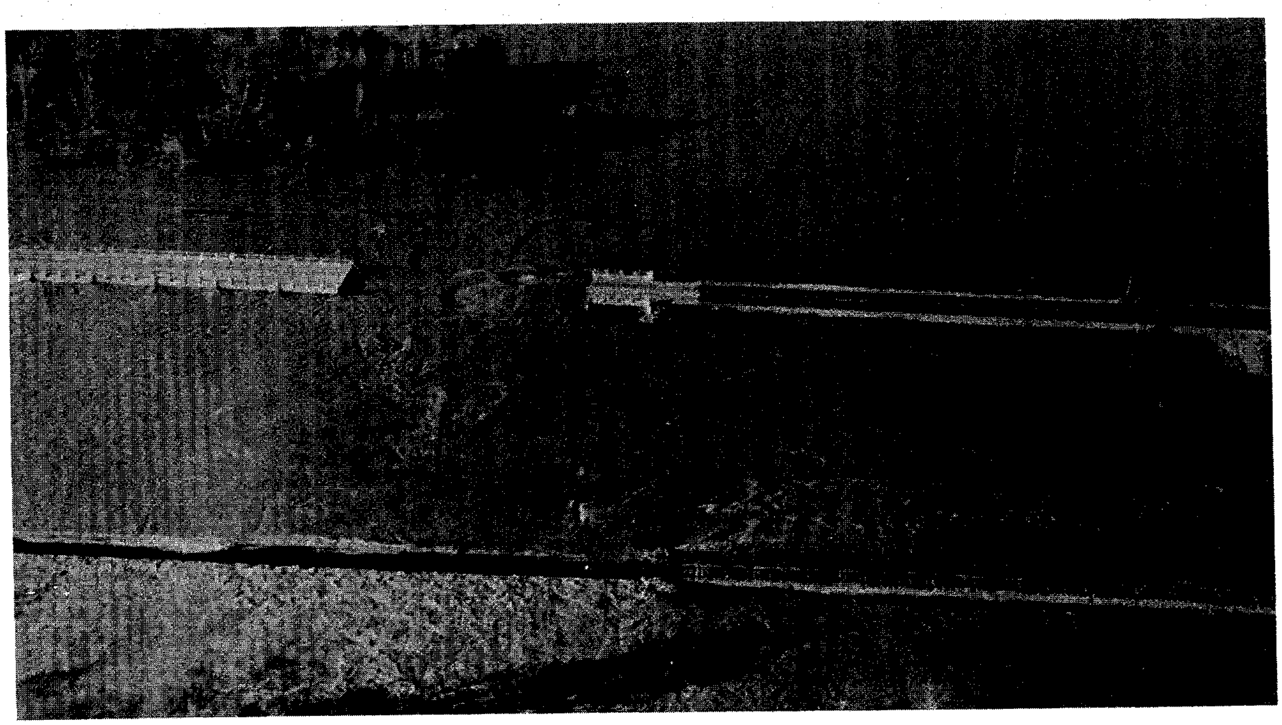


Figure 193. Aerial view of Nichols Creek bridge showing alinement of flow during March 1955 flood.

Figure 194. Rock riprap at spur dike built in 1956. Note size of vegetation, which was undamaged during the 1973 flood. (photographed in 1975.)

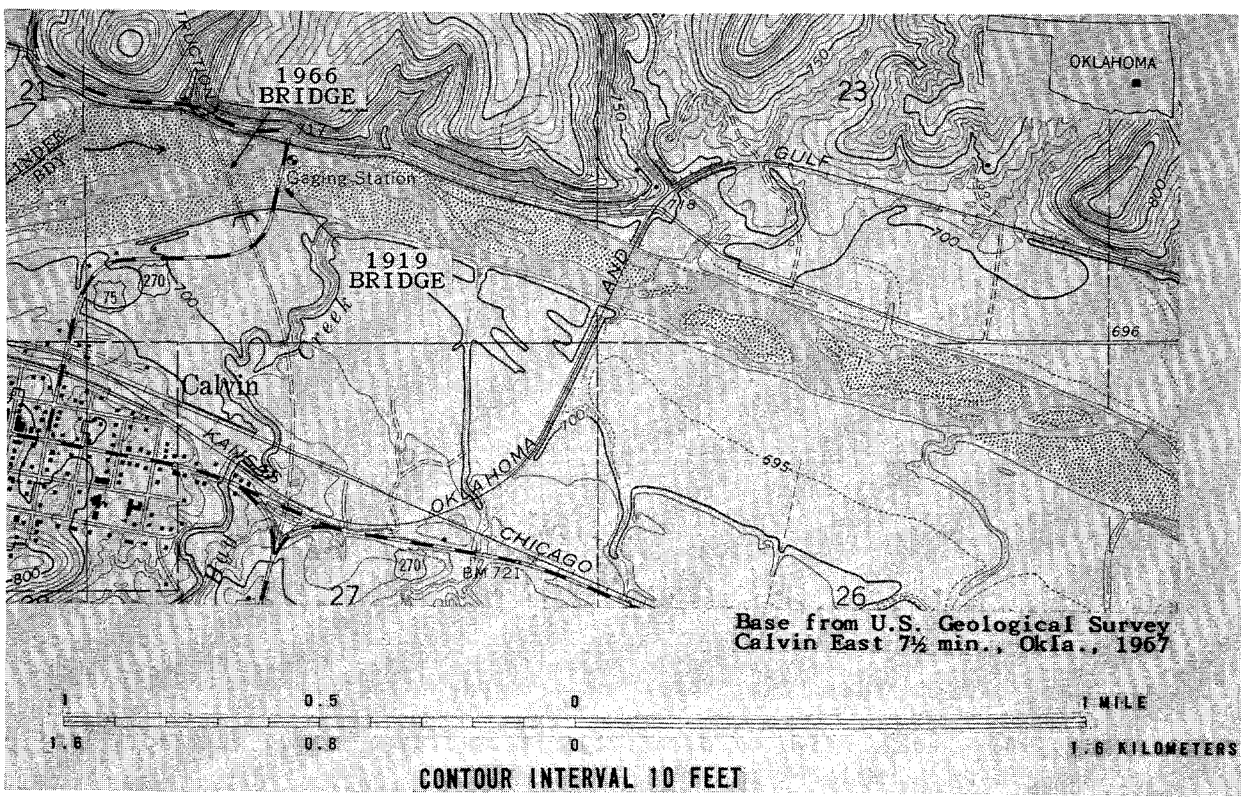


Figure 195. Location of Canadian River at US-75 and 270 near Calvin Oklahoma.

SITE 51. CANADIAN RIVER AT US-75 AND US-270 NEAR CALVIN, OKLA.

Description of site: Lat 34°59', long 96°14', location as shown on fig. 195. Bridge was a prestressed-concrete, steel-beam type built in 1966 and lost in 1976. Channel banks and abutments were protected by riprap.

Drainage area, 27,952 mi^2 (72,400 km^2); valley slope, 0.00087; channel width, about 800 ft (244 m). Stream is perennial, alluvial, sand bed, in valley of moderate relief, narrow flood plain. Channel is straight, generally braided.

Hydraulic problems and countermeasures:

1919 Steel-truss bridge built (fig. 196), 760 ft (232 m) in length, three dual-column webbed piers, vertical abutments founded on spread footings. Bridge and piers were alined to flow.

1966 New low-profile bridge, prestressed-concrete beams, built about 800 ft (244 m) upstream from original bridge (fig. 195). Bridge was 1,225 ft (373 m) in length, 12 dual circular-column piers (fig. 197). Nine piers were founded on piles, and three on spread footings. Abutments were spillthrough, founded on piles. Bridge and piers skewed about 19°.

1976 On May 21, 1976, spans 2, 5, 6, and 9, and pier 5, (fig. 198) of the new bridge collapsed. No specific flood was associated with the failure. The bed material is composed of sand, and channel scour occurs during flooding. Annual floods occurring at the bridge site between 1966 and 1976, based on flow records from the gaging station (gage 07231500) at the former US-75 truss bridge are as follows:

Water year	Date	Water surface elevation (ft) (mean sea level datum)	Discharge ft^3/s
1966	4-27-66	689.6	6,060
1967	4-13-67	695.5	66,000
1968	5-14-68	693.7	45,700
1969	5-07-69	692.8	33,800
1970	10-12-69	694.0	53,100
1971	10-08-70	699.2	130,000
1972	12-15-71	690.2	16,800
1973	4-22-73	693.8	51,900
1974	11-24-73	694.9	67,000
1975	5-24-75	692.4	38,100
1976	4-20-76	691.4	27,000

The largest flood of record for the period 1935-1976 occurred on May 11, 1950; discharge 174,000 ft^3/s (4,928 m^3/s). The 1970 flood was the seventh largest for the period of record. A discharge measurement made at the truss bridge during the October

1970 flood indicates that about 13 ft (4 m) of scour (based on the channel bed elevation surveyed in 1916) occurred. Because the gross waterway area at the new bridge is less than at the old bridge (table 7), scour depths were probably equal to, or larger than, those observed at the truss bridge site during the October 10, 1970 measurement.

Bridges are typically designed using the channel geometry surveyed during low-flow conditions. During periods of flooding, however, the channel size increases, and flow depth and velocities generally also increase. The hydraulic data in table 7 gives a comparison of measured values of depths and velocities at the gage (old bridge site), with estimated values for the new bridge site based on channel surveys made during periods of no flow. The maximum measured depth of scour of about 13 ft (4 m) is associated with a discharge of 94,100 ft^3/s (2,665 m^3/s). The peak discharge during the Oct. 8, 1970 flood was 130,000 ft^3/s (3,682 m^3/s) or 1.4 times larger than the measured discharge. With depths of scour of about 13 ft (4 m), measured on October 8, 1970, even greater depths of scour below the streambed would be possible during the flood peak. Excavation of the channel bed around the pier 9 footing after the bridge failure revealed logs lodged about 20 ft (6 m) below the stream bed (fig. 199).

1977 New bridge built (fig. 200) about 37 ft (11 m) longer than the 1966 bridge with new piers founded on caissons drilled at least 15 ft in the shale (fig. 201) and new abutments founded on piles.

Discussion: A discharge measurement made during the flood of October 8, 1970 on the Canadian River recorded about 13 ft (4 m) of scour in a sand bed channel. Average flow velocities were about 10 ft/s (3 m/s) for a discharge of 94,100 ft^3/s (2,665 m^3/s). Greater depths of flow and maximum depths of scour probably occur at higher discharges. Excavation of the sand bed channel around one of the pier footings at the new bridge site revealed logs lodged on the steel piling about 20 ft (6 m) below the normal streambed surface. This case history demonstrates the need to design footings in sand bed channels for the possibility of scour caused by lodging of debris on the footing.

Failure of the bridge built in 1966 is probably related to scour of the channel bed during the 1970 flood, which led to differential settlement of some of the bridge pier foundations. Although failure of the bridge did not occur until 6 years after the flood, other floods - in the 50,000 ft^3/s (1,416 m^3/s) to 70,000 ft^3/s (1,982 m^3/s) range - probably caused additional scour of sufficient depth to continue the differential settlement of pier foundations as indicated by the 10-ft (3-m) depth of scour observed during an April 1976 flood measurement (table 7) at the downstream truss bridge site. A discussion of the bridge structural conditions prior to and after failure is presented in a report by Modeski and Masters, Consulting Engineers, dated March 1977.

Table 7. Hydraulics of Canadian River at US-75 and 270 bridge for floods occurring in 1970 and 1976

Location and channel conditions	Date of channel survey	Water surface elevation [5](ft)	Discharge [6](ft³/s)	Gross area [7](ft²)	Mean velocity (ft/s)	Mean depth (ft)	Maximum measured velocity (ft/s)	Maximum measured depth (ft)	Maximum depth of scour [1] (ft)
Bridge built in 1919; channel as measured at flood flow	10-08-70	696.7	94,100	9,630	9.8	15.1	14.5	22.1	12.8
	4-20-76	690.1	16,000	3,620	4.4	5.9	8.2	13.0	10.2
Bridge built in 1966; channel as measured at no flow	[3]1966	[4]697.3	[2]94,100	8,570	11.0	9.5	--	--	--
	[3]1976	[4]597.3	[2]94,100	8,040	11.7	8.9	--	--	--
	[3]1976	[4]690.7	[2]16,000	2,860	5.6	3.4	--	--	--

1 Based on the average channel bed elevation shown on the 1919 bridge plans.
2 Arbitrary value assigned for comparative purposes.
3 From bridge design plans.
4 Adjusted for fall in water surface between gage and bridge.
5 Feet multiplied by 0.305 equals meters
6 ft^3/s multiplied by 0.0283 equals m^3/s
7 ft^2 multiplied by 0.093 equals m^2
8 ft/s multiplied by 0.305 equals m/s

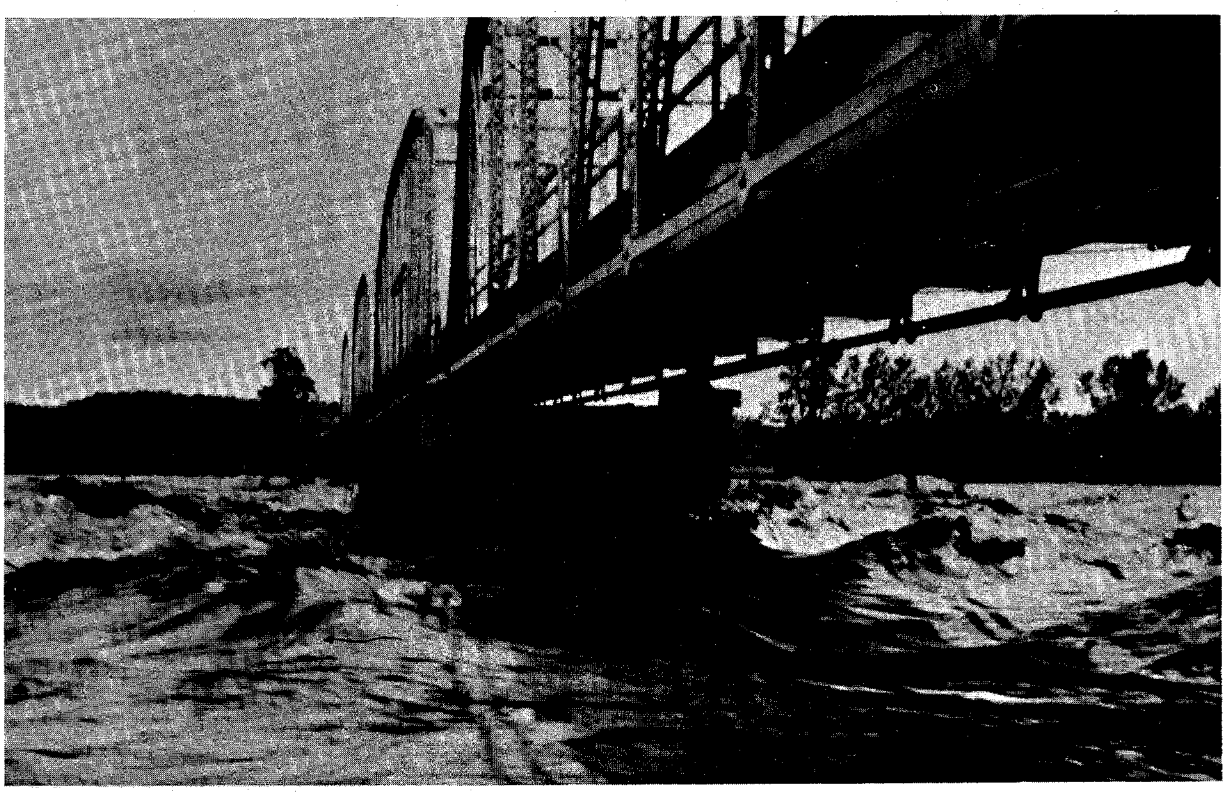


Figure 196. Flood of May 5, 1941 at US-75 truss bridge, built in 1919. Note pileup of water at upstream face of pier.

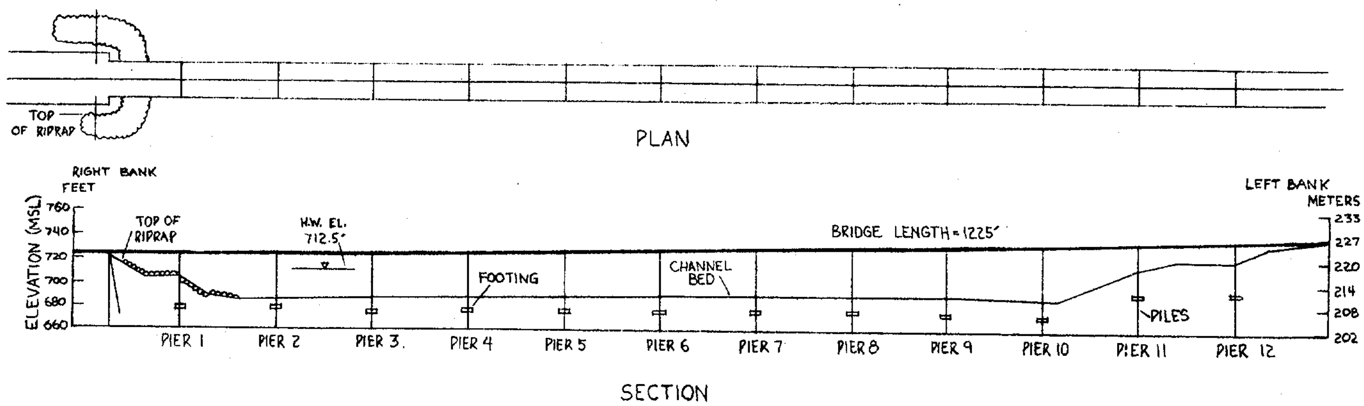


Figure 197. Plan and section of bridge built in 1966.

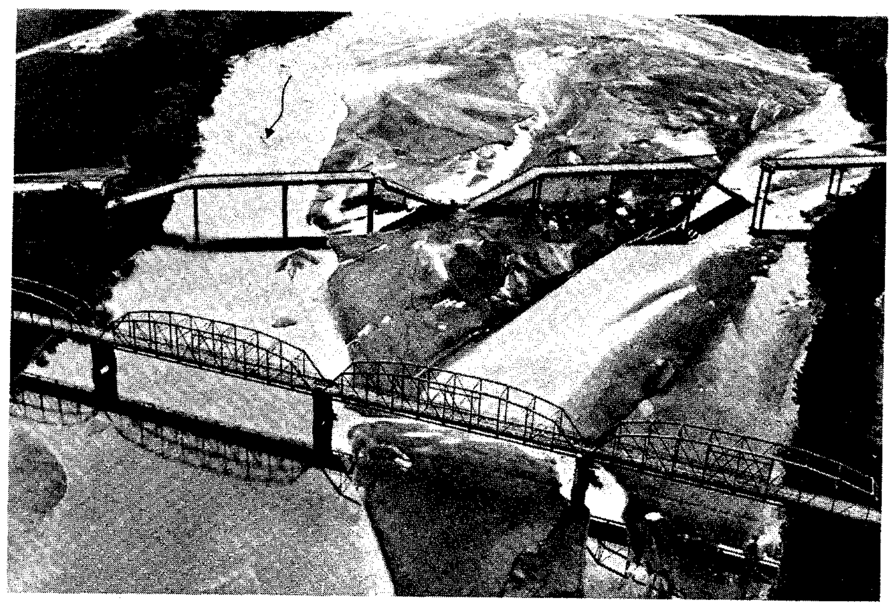


Figure 198. Failure in May, 1976 of US-75 and 270 concrete-beam bridge built in 1966. Older truss bridge in foreground. (Photograph from the Oklahoma Journal.)

Figure 199. Wood debris lodged under pier 9 during a flood event prior to 1976. (From Federal Highway Administration.)

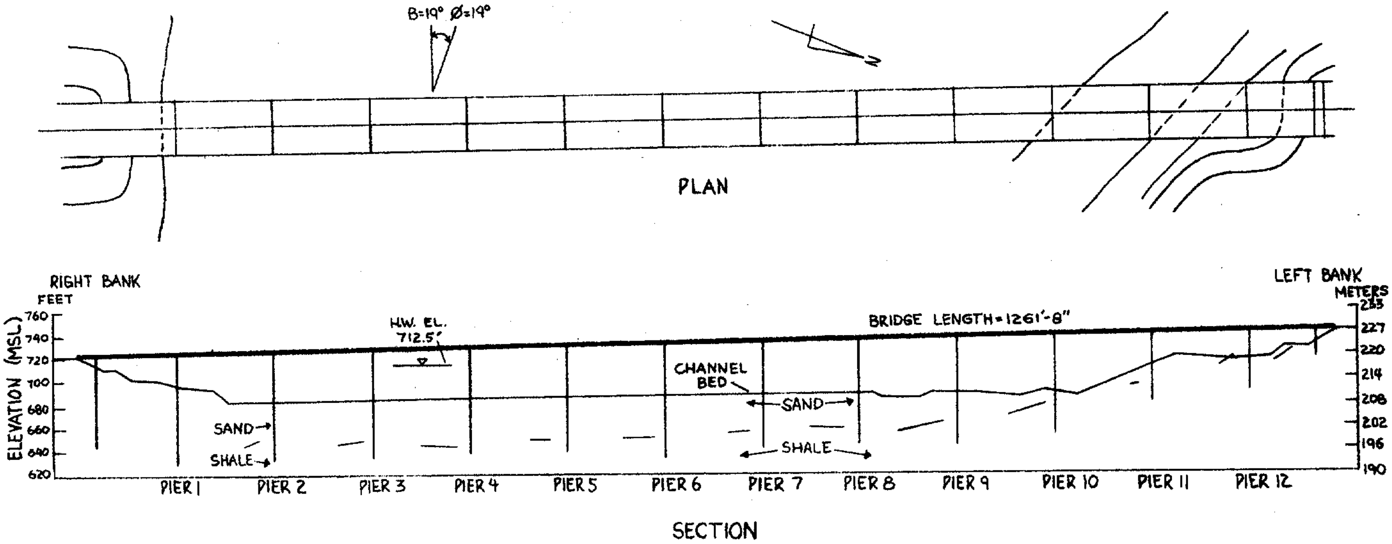


Figure 200. Plan and section of bridge built in 1977.

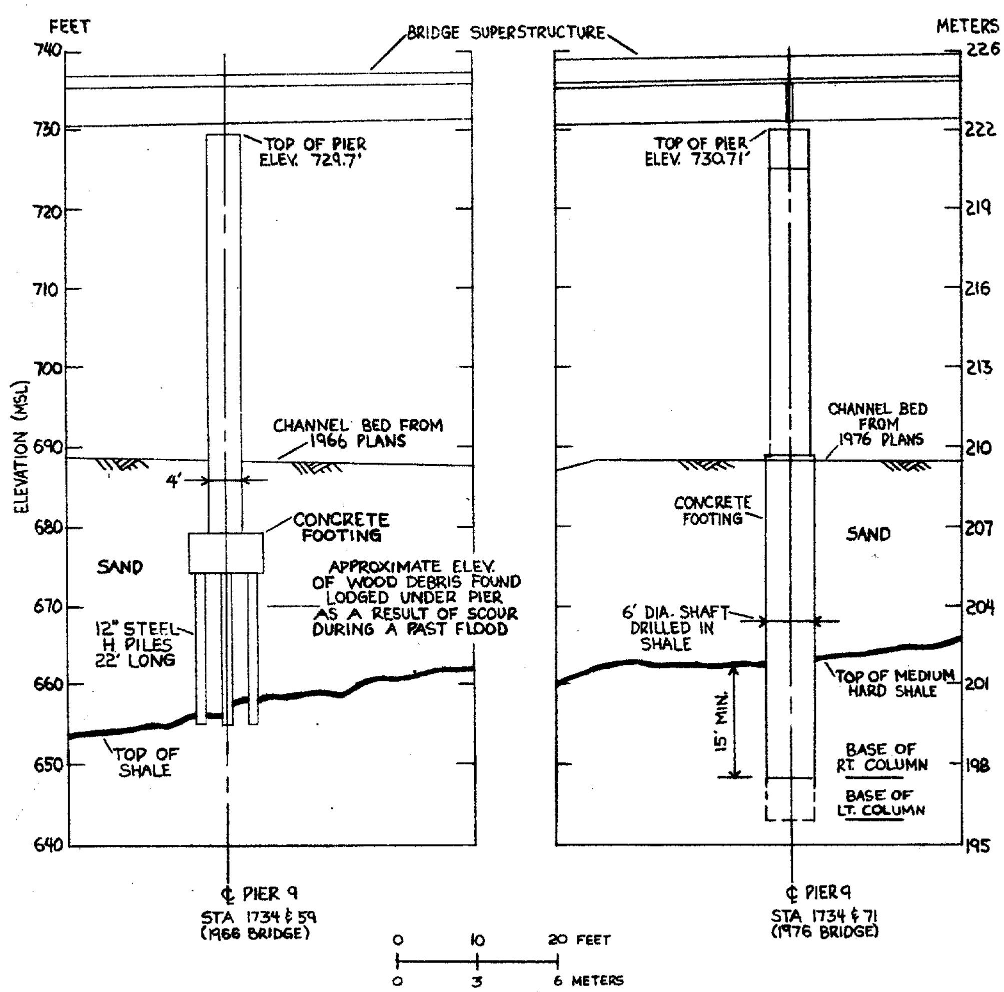


Figure 201. Comparison of pier 9 footing design for bridges built in 1966 and in 1976.

SITE 52. PIGEON ROOST CREEK AT SR-305 NEAR LEWISBURG, MISS.

Description of site: Lat 34°50', long 89°49', location as shown in fig. 202. The bridge has a concrete deck on prestressed-concrete beams, which are supported by steel piles encased in concrete. Length of bridge is about 352 ft (107 m): three 80-ft (24-m) spans and one 110-ft (34-m) span. Abutments are spillthrough. The bridge crosses the stream at a right angle.

Drainage area, 228 mi^2 (591 km^2); bankfull discharge, 34,500 ft^3/s (976 m^3/s); valley slope, 0.0009; width of artificial channel, 225 ft (69 m). Stream is perennial, alluvial, sand bed, in valley of low relief, wide flood plain. Channel is straight, was first straightened about 1920 and again straightened and widened in 1968. Silt-clay banks, tree cover entirely removed from bankline during 1968 channelization.

Hydraulic problems and countermeasures:

1950 Bridge built. Countermeasures included provision of a 60-ft (18-m) span across the main channel and use of concrete slope paving to protect the abutments. Original channel, following improvements made about 1920, was about 100 ft (30 m) wide.

1968 U.S. Soil Conservation Service rechanneled creek (fig.202). The bottom width of the channel was enlarged from 50 ft (15 m) to 205 ft (62 m), and the depth was increased by several feet. Countermeasures for the bridge include: (1) Use of two new double-pile bents driven at least 30 ft (9.1 m) below channel bed to reinforce existing bridge piling. (2) Construction of nose piles to deflect drift. (3) Use of timber sheathing to encase the double pile bents to prevent lodging of debris at pile bents. (4) Protection of spillthrough abutments with rock riprap for 50 ft (15 m) upstream and downstream from bridge. The channel enlargement placed several bents in the main thread of flow, thus subjecting the bents to increased flow velocities and heavy drift.

1969 During flood of April 10, 1969 structural failure occurred by a settlement of 9 in (0.23 m) at bend 8, and 6 in (0.15 m) at bent 11. Scour about 20 ft (6.1 m) deep was observed at bent 8. Bridge damage was attributed to reduced pile penetration associated with scour caused by high velocities and drift. The bridge was repaired to original condition.

1970 Site inspection indicated the channel was shifting away from the 60 ft (18 m) main span. Aggradation of the original channel was occurring, and the low-water channel had shifted toward the shorter spans.

1973 Flood of April 19, 1973 caused complete failure of bridge and changed the main channel location. After the flood, a new bridge was built with several measures to protect the bridge from damage by lateral erosion and lodging of drift at the bents: (1) Individual steel piles for a bent were encased in concrete sheathing to protect against lodging of drift. The concrete encasement was extended below the ground surface at least 1.5 ft (0.5 m). (2) The upstream pile in a bent was placed with a batter of 3/4 in per ft of length (0.02 in per 0.30 m) to act as a drift deflector. (3) Broken concrete riprap was placed on the channel banks to prevent lateral erosion. (4) The bridge spans were increased in length from 24 ft (7.3 m) to a minimum of 80 ft (24 m) to reduce the chance for debris lodging.

1977 No hydraulic problems at the new bridge have been reported.

Discussion: Enlargement of the channel by widening and deepening did not result in a stable channel size and location. Because the stream carried large amounts of debris, bents spaced at 24 ft (7.3 m) were inadequate to pass the debris without lodgement. The amount of scour, caused by debris lodged at the bridge and by high flow velocities, exceeded 20 ft (6.1 m) below the normal streambed. Encasement of steel piles by concrete sheathing to act as a debris deflector were used to prevent the collection of debris at bridge bents or between piles. Even though the stream gradient was less than 0.1 percent, local scour problems were caused by contraction of flow and by lodgment of debris at the bridge, which decreased the size of the effective waterway.

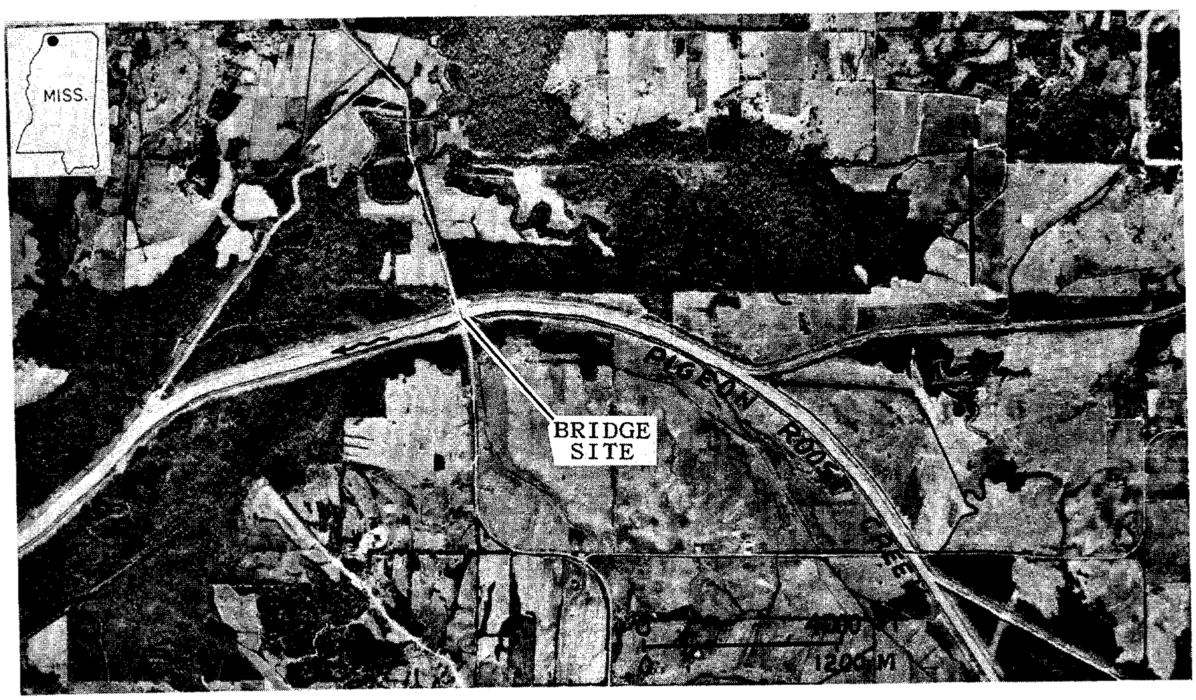


Figure 202. Aerial photograph of Pigeon Roost Creek at SR-305. (From Mississippi Dept. of Highways, 1974.)

SITE 53. BOWIE CREEK AT US-84 NEAR COLLINS, MISS.

Description of site: Lat 31°83', long 89°45', location as shown in fig. 203. Concrete deck bridge, 240 ft (73 m) in length, supported on eleven timber-pile bents spaced at 20 feet. Abutments are vertical with no wingwalls. At time of bridge construction in 1937, the channel was straightened for about 900 ft (274 m) upstream and downstream of crossing. Skewness of the bridge is 30°.

Drainage area, 47 mi^2 (148 km^2); valley slope, 0.0016; channel width, 30 ft (9 m). Stream is perennial, alluvial, silt bed, in valley of low relief, wide flood plain. Channel is sinuous, equiwidth, cut banks rare, silt-clay banks, dense tree cover at more than 90 percent of bankline (fig. 204).

Hydraulic problems and countermeasures:

1937 Channel realined and cleared at time of bridge construction. Rock riprap was placed at ends of abutments to prevent scour. The flood plain is very wide compared to the size of bridge opening and the stream is highly contracted during floodflow.

1952 A cross section of channel obtained January 6, 1952 by the Highway Department indicated scouring near the right bank abutment (fig. 205).

1961 A cross section surveyed in March 1961 by the Geological Survey showed further scouring of the channel near the right bank (fig. 205).

1974 On April 13, 1974, a record flood of 15,000 ft^3/s (425 m^3/s) caused general scour at the bridge, removed two pilings at bent 3 and undermined three piles at bent 10 (fig. 205). After the flood, the scour hole was filled and the washed-out piling replaced. The amount of initial bridge contraction and subsequent increase in waterway size between 1937 and 1974 are given in table 8:

Table 8. Changes in waterway size at Bowie Creek bridge between 1937 and 1974

Date of cross-section survey	Waterway available below elevation 350 ft (10.7 m), discharge 15,000 ft^3/s (425 m^3/s)			
	Contraction ratio (M)	Area[1] (ft^2)	Velocity[2] (ft/s)	Froude number (F)
1937	0.45	1,800	8.3	9.56
1952	--	2,466	6.1	.31
1961	--	2,543	5.9	.30
1974	--	5,587	2.7	.10

[1] Conversion factor, ft^2 to m^2, is 0.093
[2] Conversion factor, ft/s to m/s, is 0.305

Discussion: The amount of initial bridge contraction at this site (M = 0.45) was sufficient to cause scour problems during a period of more than 37 years after bridge construction. Changes in waterway size at the bridge are dependent on the size and timing of large flow events. In this case, a 41 percent increase in channel size occurred during the period 1937-61. Between 1961 and 1974, however, the waterway area increased 120 percent, largely associated with the flood of April 1974 (recurrence interval at least 100 years). Spur dikes placed near both abutments may have prevented some of the scour problems associated with the return of overbank flow to the main channel.

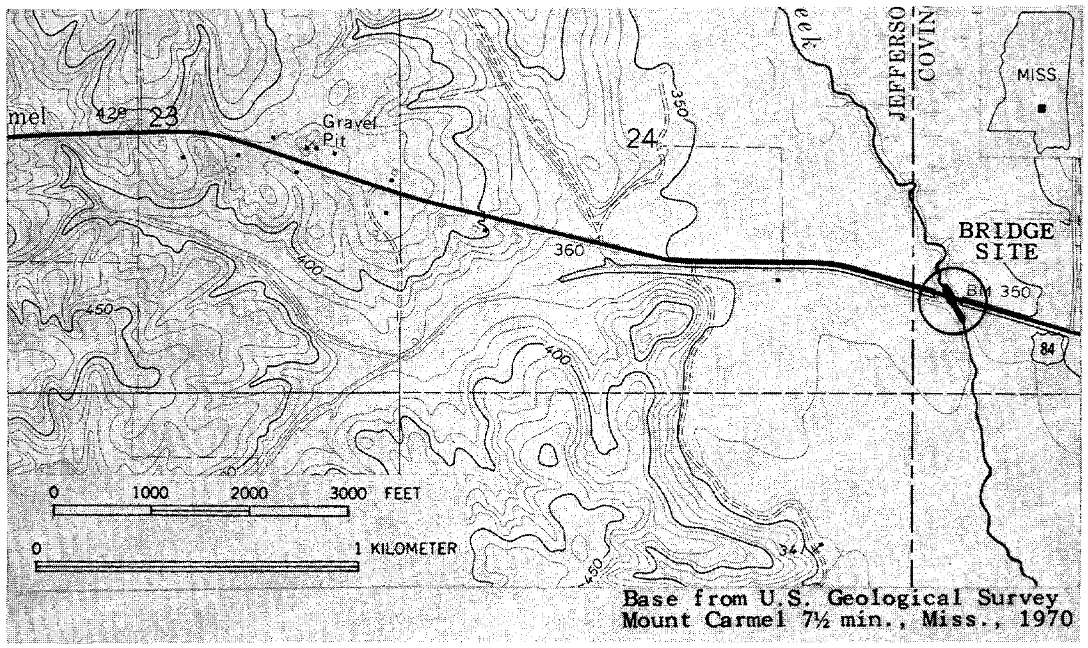


Figure 203. Location of Bowie Creek at US-84 near Collins, Mississippi.

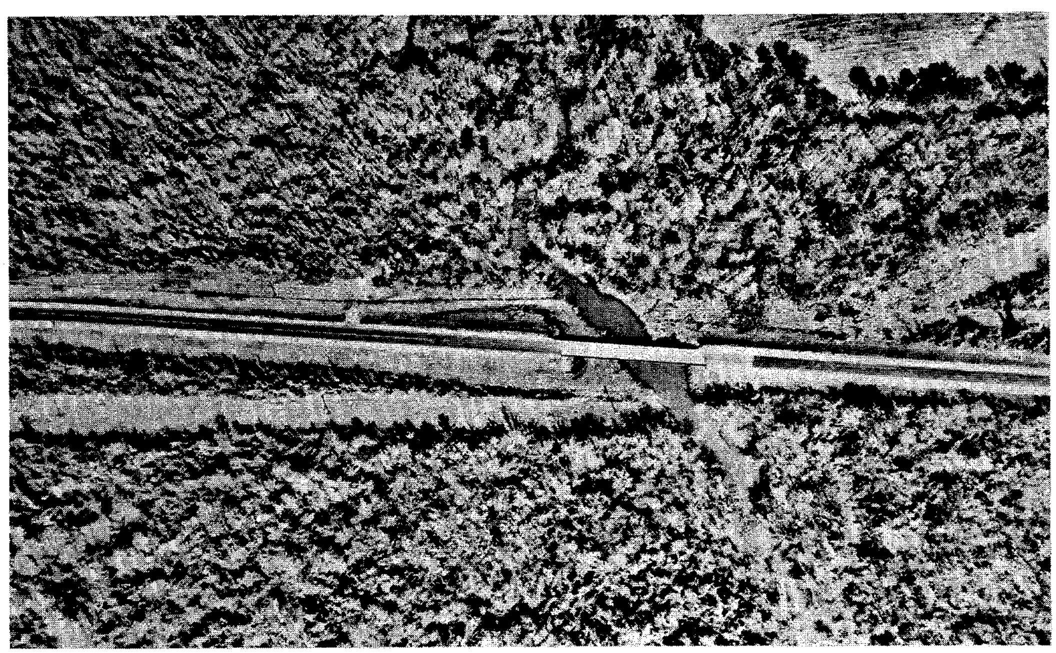


Figure 204. Aerial photograph of Bowie Creek at US-84 crossing. (From Mississippi Dept. of Highways, 1973.)

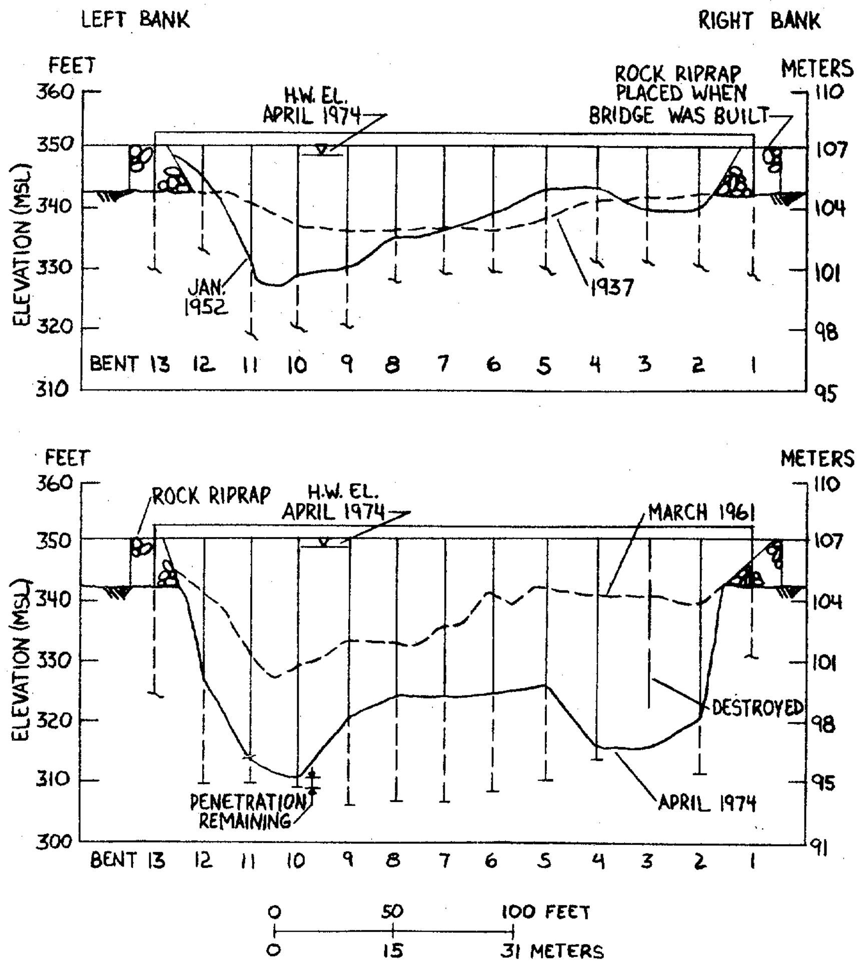


Figure 205. Changes in channel at Bowie Creek bridge between 1937 and 1974.

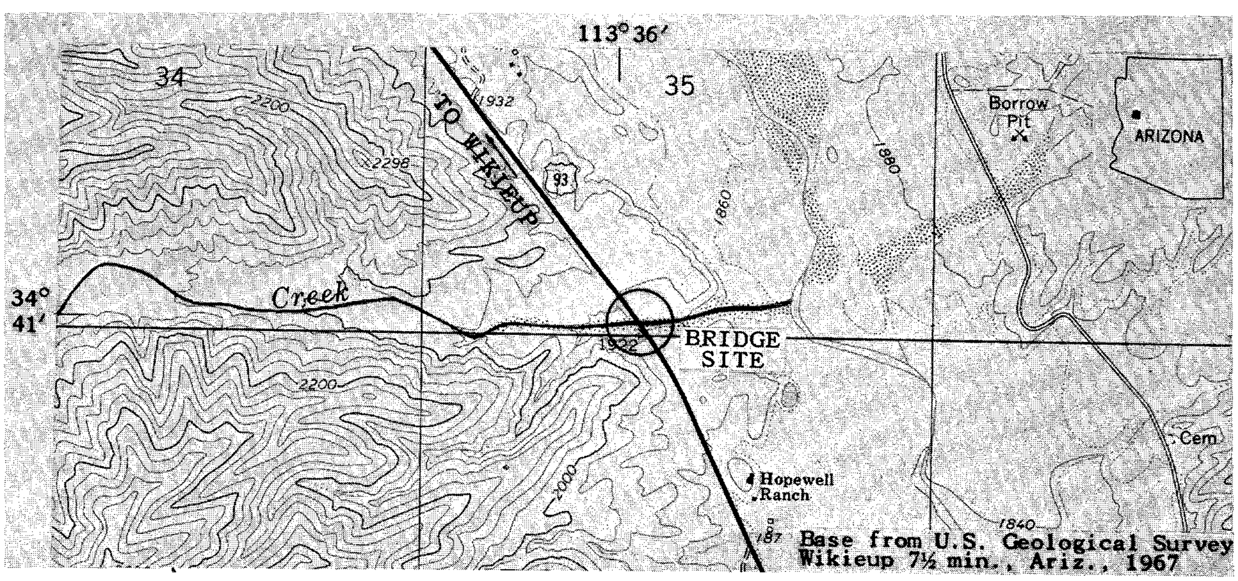


Figure 206. Location of Bronco Creek at US-93 near Wikieup, Arizona.

SITE 54. BRONCO CREEK AT US-93 NEAR WIKIEUP, ARIZ.

Description of site: Lat 34°41', long 113°36', location as shown in fig. 206. Bridge is prestressed-concrete, 190 ft (58 m) in length, skewed 30° to flow (figs. 207 and 208), supported by spillthough abutments and two round-nose concrete wall-type piers, alined with flow. Piers are founded on piles in alluvial bed material.

Drainage area, 19 mi^2 (49 km^2); valley slope, 0.031; channel width, about 400 ft (122 m). Stream is ephemeral, sand-gravel bed, in valley of moderate relief, no flood plain. Channel is straight, generally braided, erodible silt-sand-gravel banks at crossing.

Hydraulic problems and countermeasures:

1961 Bridge built with approach embankment causing some constriction of overbank flow (fig. 208). Measures to protect the bridge and approach embankment during flood flow included the use of steel piling driven to about 26 ft (8 m) below the streambed (fig. 207) and use of rail bank revetment (figs. 207 and 209) to prevent lateral erosion.

1971 Flood of Aug. 19, 1971 caused channel bed scour (fig. 207) and extensive damage to the bank revetment (locally called "railbank") and to the bridge abutments (fig. 210). The flood overtopped the bridge and at intervals during the peak, large waves moved downstream, hit the bridge and splashed across the deck and embankment. The magnitude of this flood (73,500 ft^3/s or 2,080 m^3/s is among the largest recorded in the U.S. for a drainage basin of this size (19 mi^2 or 49 km^2). Damage to the bridge and approach embankment included lifting of about 100 ft (30 m) of pavement and abrasion damage to the upstream end of the piers (fig. 211). After the flood, the railbank protection was replaced where needed and extended upstream an additional 218 ft (66 m) on the right bank (figs. 207 and 212).

Discussion: The railbank-type channel and abutment protection effectively prevented structural damage to the bridge during a flood event of extreme magnitude. Abrasion damage to the upstream nose of piers was significant at this site and is potentially significant on other streams that carry large amounts of suspended material during floods.

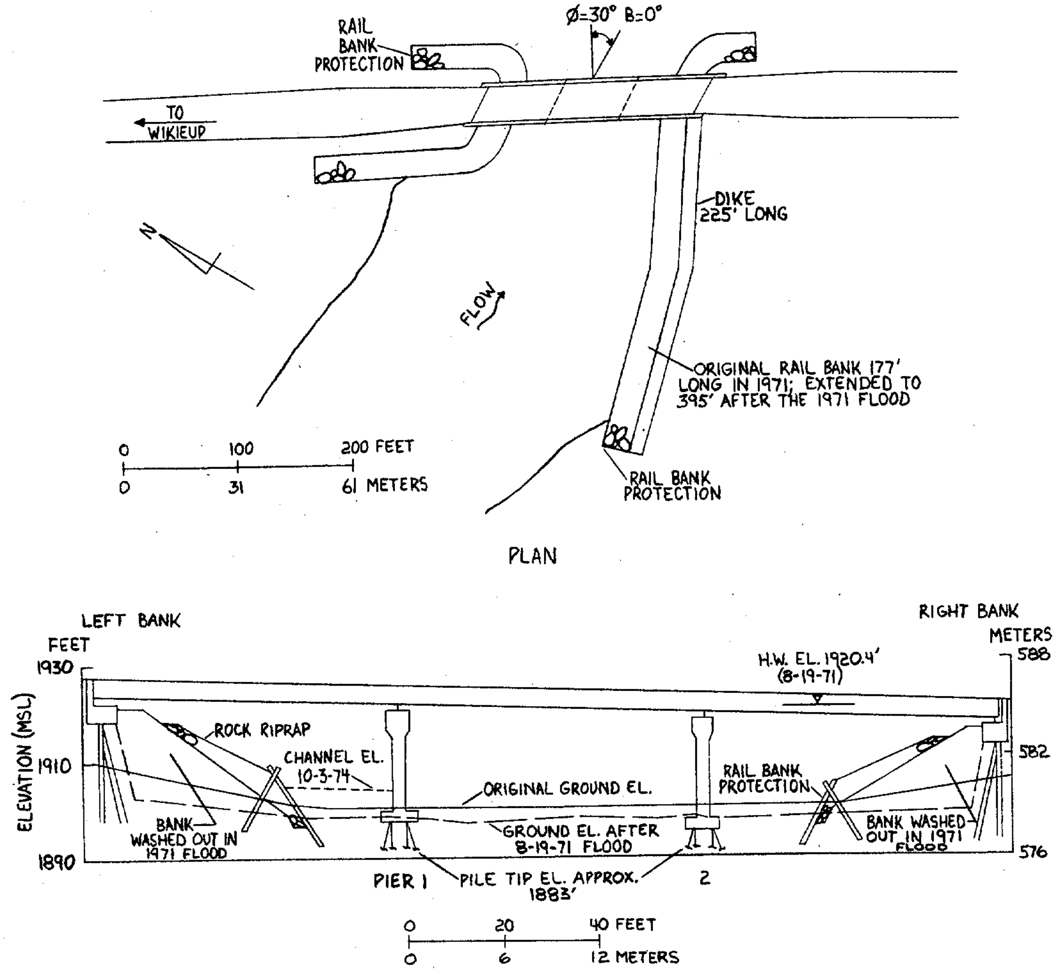


Figure 207. Details of Bronco Creek bridge.

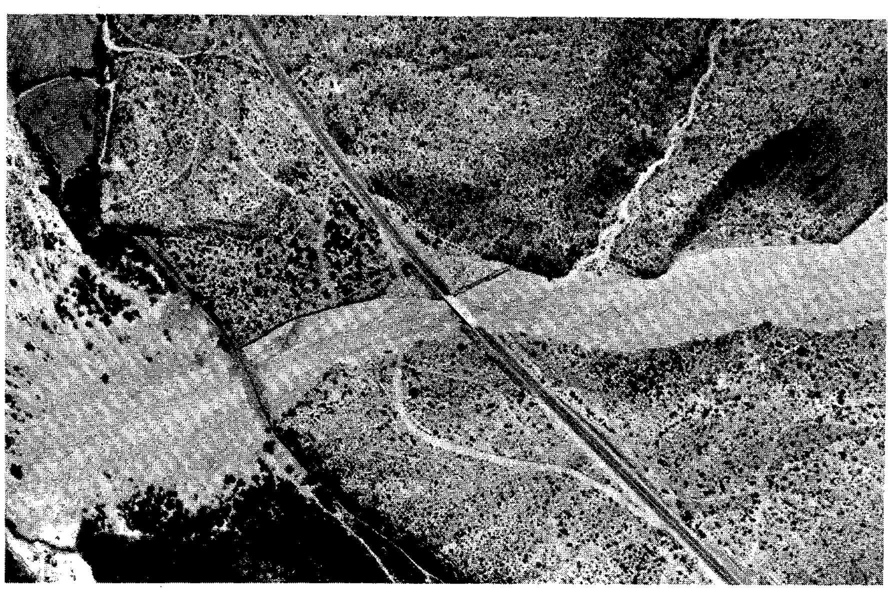


Figure 208. Aerial photograph of Bronco Creek showing channel constriction at bridge and skewness of bridge. (From Arizona Dept. of Highways.)

Figure 209. Damaged railbank protection on right bank upstream from bridge after August 1971 flood. (From Arizona Dept. of Highways.)

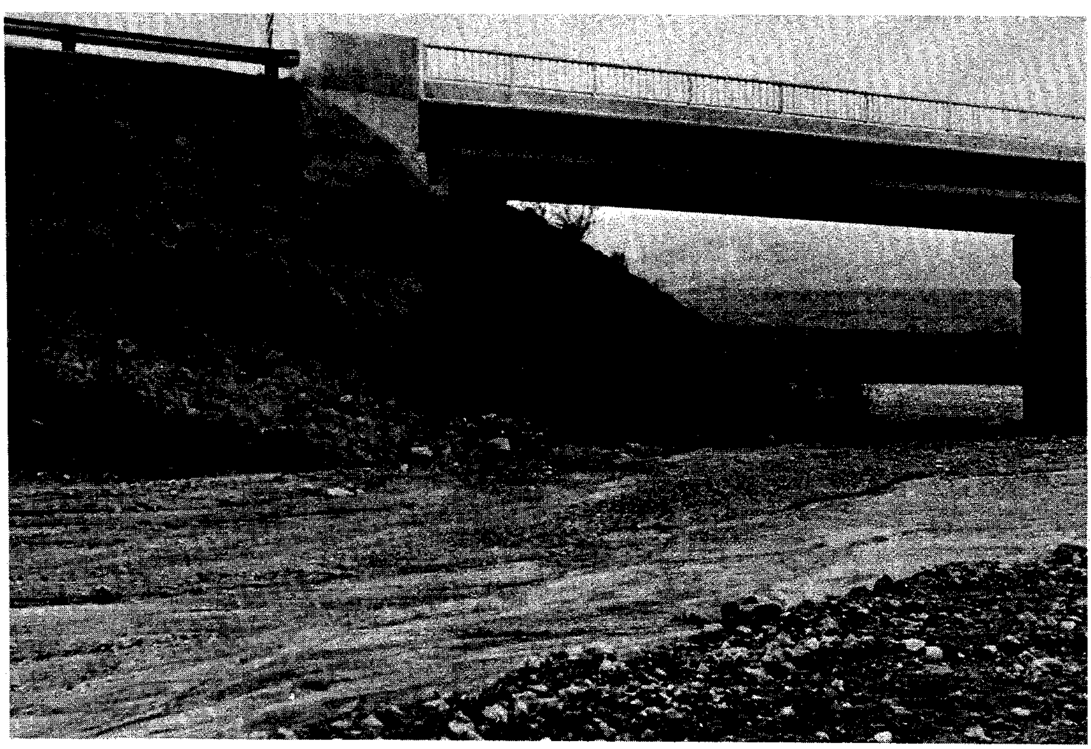


Figure 210. View of eroded left bank abutment after the 1971 flood. (From Arizona Dept. of Highways.)

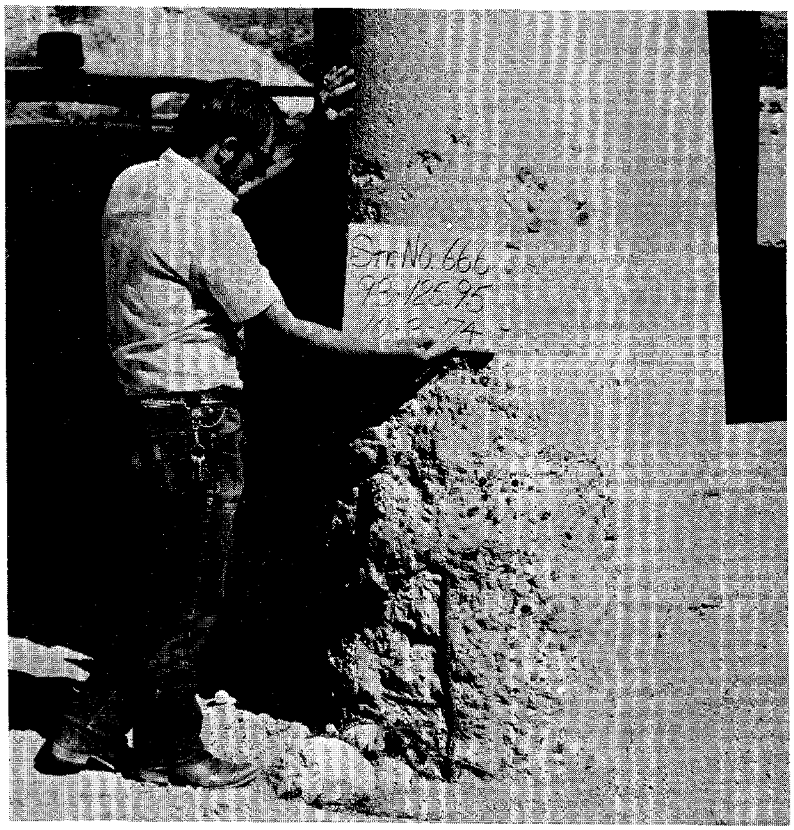


Figure 211. Abrasion damage on upstream side of pier, in 1974. (From Arizona Dept. of Highways.)

Figure 212. Downstream view of bridge and railbank protection after repairs following 1971 flood. (From Arizona Dept. of Highways.)

SITE 55. LEE MOORE WASH AT SR-89 NEAR SAHUARITA, ARIZ.

Description of site: Lat 32°01', long 110°57', location as shown in fig. 213. Prestressed-concrete box-beam bridge, 128 ft (39 m) long, two spans of 64-ft (20-m) length. Bridge is supported by a wall-type pier with rounded nose, founded on piling. The bridge and piers are alined with flow. Abutments are spillthrough type.

Drainage area, 132 mi^2 (342 km^2). Natural channel drains a piedmont slope and enters the flood plain of Santa Cruz River just upstream from the crossing (fig. 213). Channel at crossing site is artificially straightened, has ephemeral flow, sand bed, erodible silt-sand banks.

Hydraulic problems and countermeasures:

1938-70 A concrete bridge with three 16-ft (4.9-m) spans, located about 150 ft (46 m) downstream from the Southern Pacific Railroad, was built (fig. 214). A concrete floor and apron was added to the channel at the bridge in 1969. During the period 1938-70, land levelling and diking for agricultural purposes channelized the flow in Lee Moore Wash and prevented the spread of flow on the alluvial fan upstream from the bridge site.

1971 Channel degradation and lateral erosion downstream from the bridge caused partial failure of the concrete apron (fig. 214) and the bank bank revetment by undermining. A complicating hydraulic factor at this site is overbank flow from the Santa Cruz River. By 1969, the channel bed was about 4 ft (1 m) lower than in 1938 (fig. 215). The observed channel changes in Lee Moore Wash are caused by the decreased opportunity for overbank storage upstream from the bridge and increased flows associated with overflow on the right-bank flood plain from the Santa Cruz River.

1972 The channel degradation and lateral erosion problem at the bridge site had continued such that extensive remedial measures were needed to protect the bridge. According to studies by the Arizona Highway Department, hydraulic problems associated with the bridge were as follows: (1) About 10 ft (3 m) of scour had occurred at the downstream end of the apron. (2) Lateral erosion, attributed to overflow from the Santa Cruz River, was affecting the downstream side of the approach embankment for a distance of 1500 ft (457 m). (3) Flow in Lee Moore Wash, together with overflow from the Santa Cruz River, was eroding the right bank of the channel downstream from the bridge (fig. 216). (4) The concrete apron and abutment protection were being undermined and damaged. Before plans were completed for new countermeasures, flooding on Oct. 6, 1972 destroyed the bridge (fig. 217).

1973 A new bridge was built with several modifications to prevent damage from increased runoff from the watershed and overbank flow from the Santa Cruz River. The bridge length was increased from 55 ft (17 m) to 128 ft (39 m) and only one pier was placed in the waterway (fig. 218). The pile foundation of the pier was designed in anticipation of continued channel degradation. Railbank rock-and-wire protection was provided around the abutments and along the downstream side of the approach embankment (fig. 219). Steel sheet-piling was used for bank protection and direction of flow in the vicinity of the railroad bridge.

1974 Flood of July 7 (discharge 9,150 ft^3/s or 259 m^3/s) on Lee Moore Wash caused no damage at the highway bridge where the two channels join just upstream from the railroad bridge. However, the curved steel sheet-pile revetment placed to guide the flow from the south channel (fig. 216), was damaged and the right bank upstream from the railroad bridge was eroded by flow from the south channel.

1975 Site inspection indicated that the railbank protection is functioning effectively and the channel has stabilized.

1977 Flood of Oct. 10, discharge about 8,000 ft^3/s (227 m^3/s), caused some damage to the sheet-pile revetment near the railroad bridge, but the railbank protection was undamaged (fig. 220).

<u>Discussion</u>: The channel at this site enlarged by lateral erosion and degradation because of increased flows attributed to: (1) channelization upstream from the bridge site, which prevented spreading of flow and storage on the flood plain, (2) channelization caused by the railroad embankment which forced overflow from the Santa Cruz River to merge with water in Lee Moore Wash at the bridge site, and (3) increased right-bank overflow from the Santa Cruz River. The use of a concrete apron to prevent scour around the bridge piers was not effective because channel degradation undermined the cutoff wall at the toe of the apron. The railbank revetment was effective in preventing lateral erosion of channel banks, even though the channel is degrading.

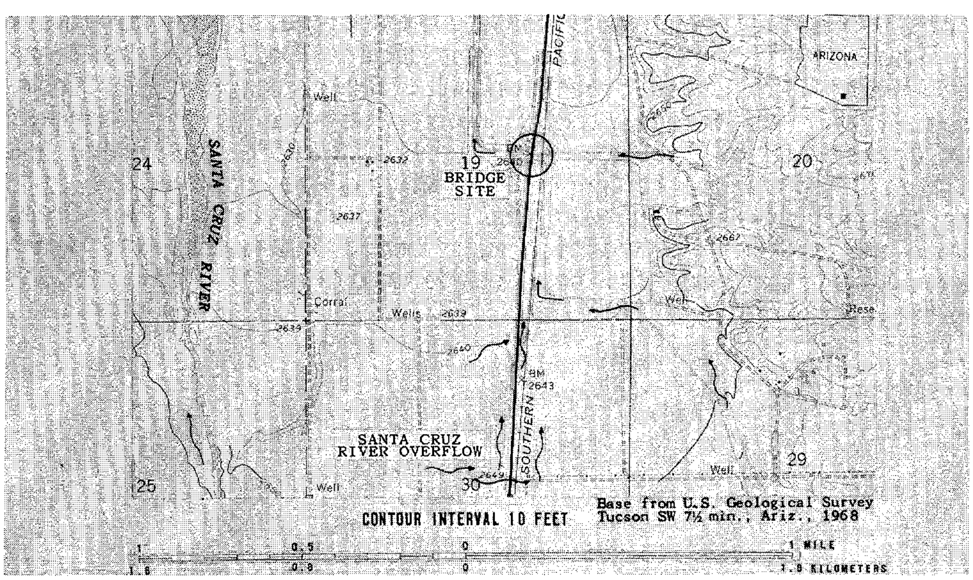


Figure 213. Location of Lee Moore wash at SR-89 near Sahuarita, Arizona.

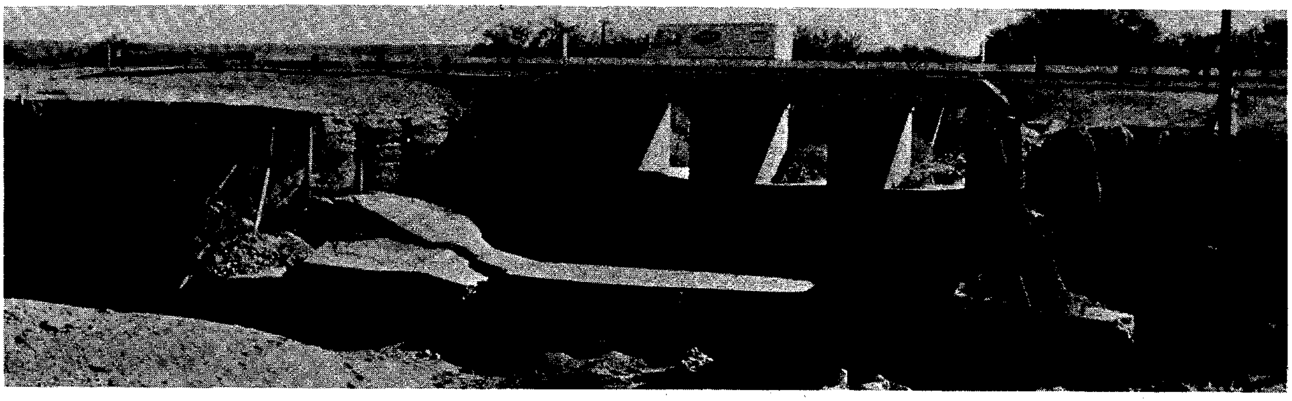


Figure 214. Channel degradation and concrete apron damage at downstream side of bridge in 1971. The concrete apron was added in 1969. (From Arizona Dept. of Highways.)

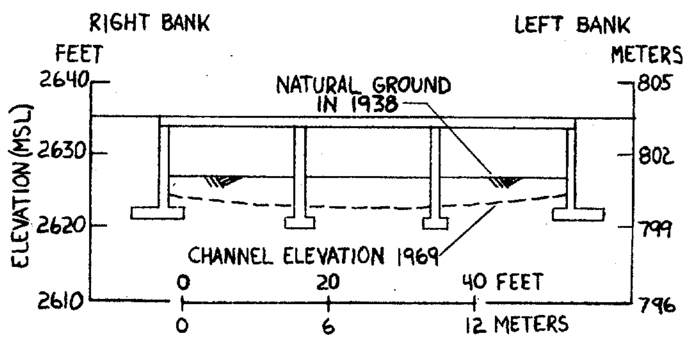


Figure 215. Changes in channel bed elevation between 1938 and 1969.

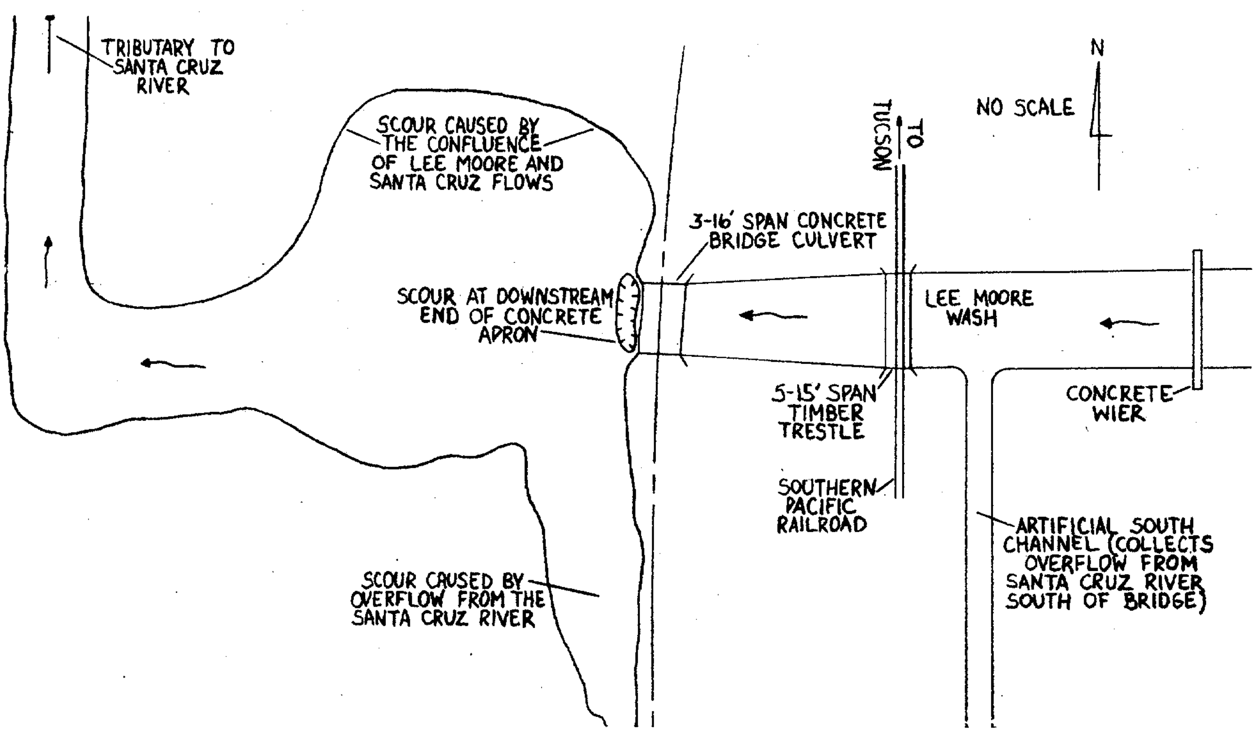


Figure 216. Location of various hydraulic problems at the bridge site in 1972.

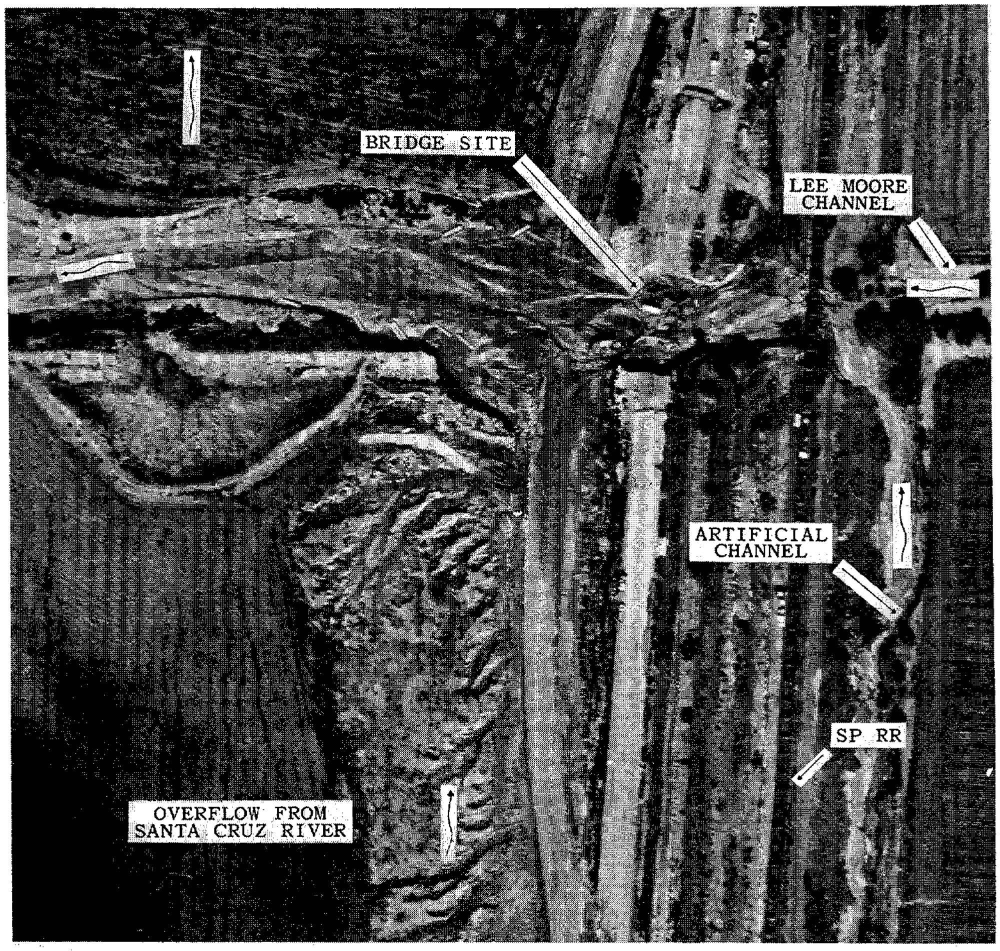


Figure 217. Aerial view in 1972 of bridge destroyed in October 6, 1972 flood. Note the larger size of the artificial channel compared with the Lee Moore wash channel upstream from the railroad. (From Arizona Dept. of Highways.)

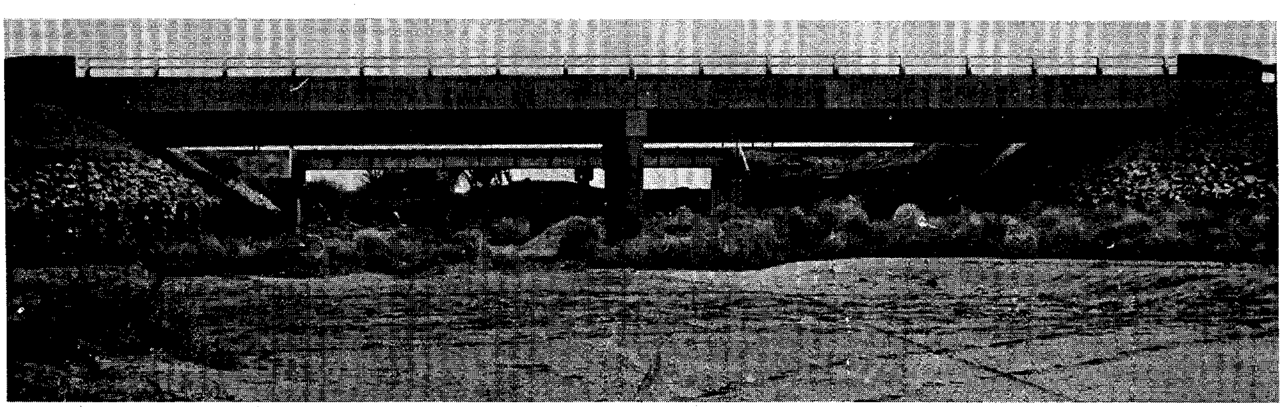


Figure 218. View upstream at new bridge constructed in 1973. (From Arizona Dept. of Highways.)

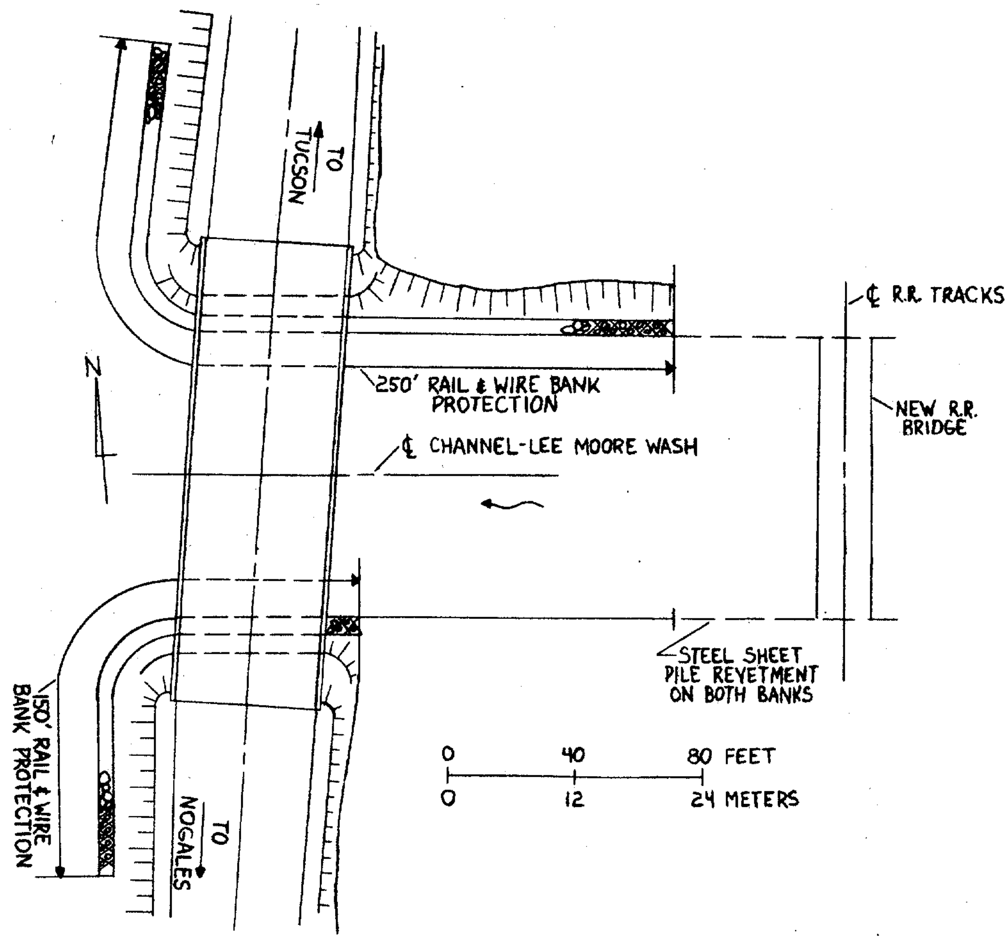


Figure 219. Plan of bridge and countermeasures constructed in 1973.

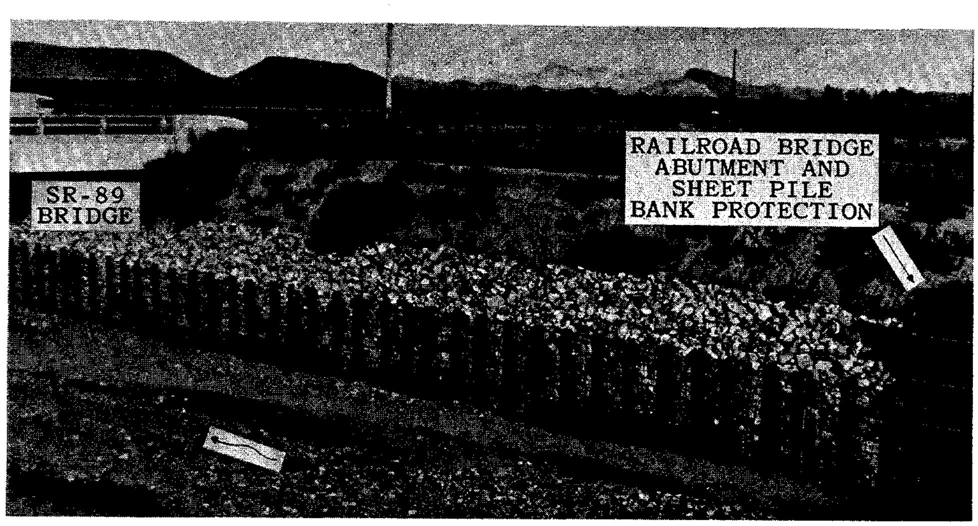


Figure 220. "Railbank" revetment at right bank, installed in 1973, as photographed in November 1977, after floods of 1974 and 1977.

SITE 56. BIG SANDY WASH AT I-40 NEAR KINGMAN, ARIZ.

Description of site: Lat 35°09', long 113°38', location as shown in fig. 221. Steel girder bridge, 286 ft (87 m) long, supported by wall-type piers with rounded noses and founded on concrete piles. Bridge skew is 30 degrees, piers are alined with flow. Abutments are spillthrough.

Drainage area, 90 mi^2 (233 km^2); valley slope, 0.008; channel width, about 500 ft (152 m). Stream is ephemeral, alluvial, sand-gravel bed, in valley of moderate relief, narrow flood plain. Channel is sinuous, generally braided, arid-region channel or "wash".

Hydraulic problems and countermeasures:

1967 Dual bridges built with steel rail, wire, and rock revetment (railbank protection) around both abutments. Fill material for the approach embankments was obtained from a borrow pit on the right bank flood plain upstream from the bridge (fig. 221).

1969 Changes in the flow pattern and excavation for the borrow pit caused changes in the flow alinement from the left side of the channel towards the right bank. During floods, the main flow approaches the approach embankment west of the bridge opening, then travels along the embankment and through the bridge. The concentration of flow on the right side of the channel has caused erosion of a 350-ft (107-m) dike separating the borrow pit from the channel (fig. 222) and, at the bridge, undermining of the railbank revetment on the east abutment (fig. 223). Countermeasures included excavation of the channel for a distance of 1500 ft (457 m) upstream from the upstream bridges and construction of a new dike upstream from the old dike.

1975 Scour of the main channel at the bridge was undermining the railbank protection at both abutments. About 3 ft (1 m) of channel scour occurred in the period 1967-75 at the bridge.

1976 New dual bridges were built at location of older bridges to improve traffic and load conditions. The existing right-bank railbank protection was replaced or modified (fig. 224) to prevent further undermining by the stream, and the concentration of flood flows on the right bank flood plain.

Discussion: At the time of bridge construction, the possibility of lateral channel shift was not anticipated when the borrow pit was located. Damage to the approach embankment by lateral erosion and scour occurred when flows on the flood plain moved laterally along the upstream side of the embankment before entering the main channel. The railbank (rock-and-wire) revetment, although undermined, provided protection from flood flows. The wire fabric was not damaged and the steel-rail support posts remained intact.

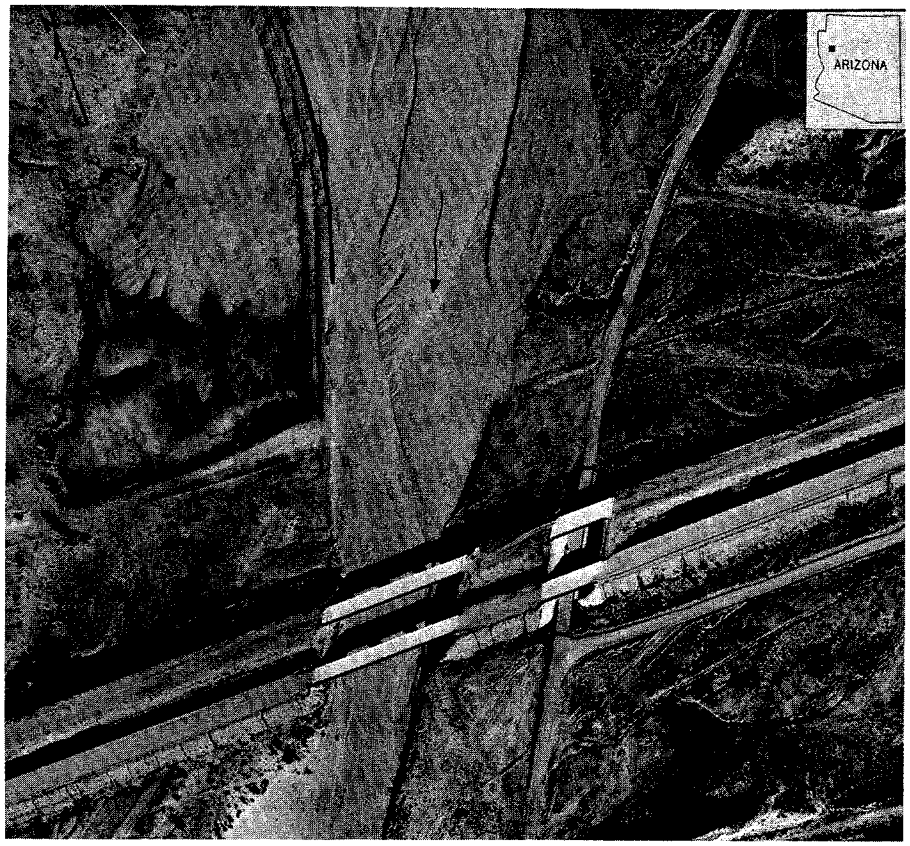


Figure 221. Aerial view of Big Sandy Wash at I-40 showing location of borrow pit on right bank upstream from bridge. Note lateral erosion of levee separating borrow pit from the channel and deposition of sediment at the upstream end of the borrow pit. (From Arizona Dept. of Highways, 1971.)

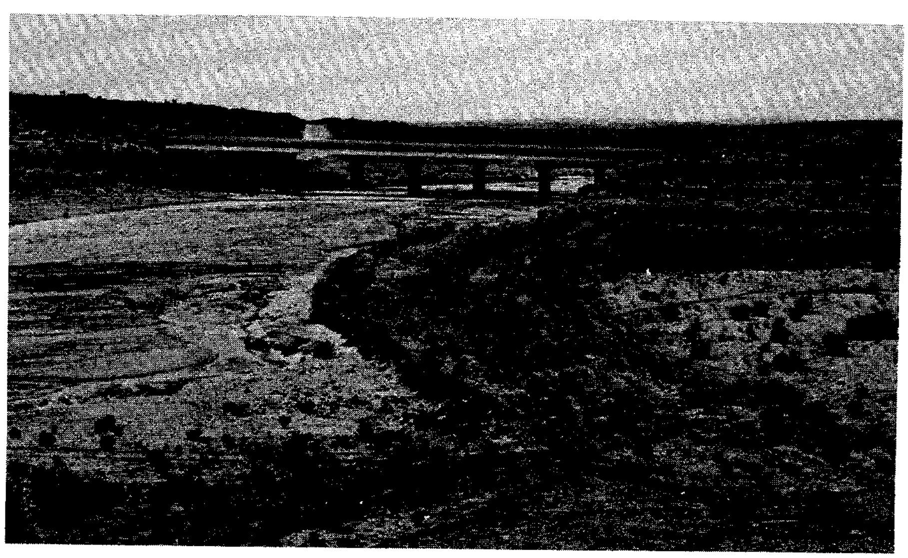


Figure 222. View of borrow pit on right-bank flood plain and damaged rock riprap protection of levee separating channel from borrow pit. (From Arizona Dept. of Highways.)

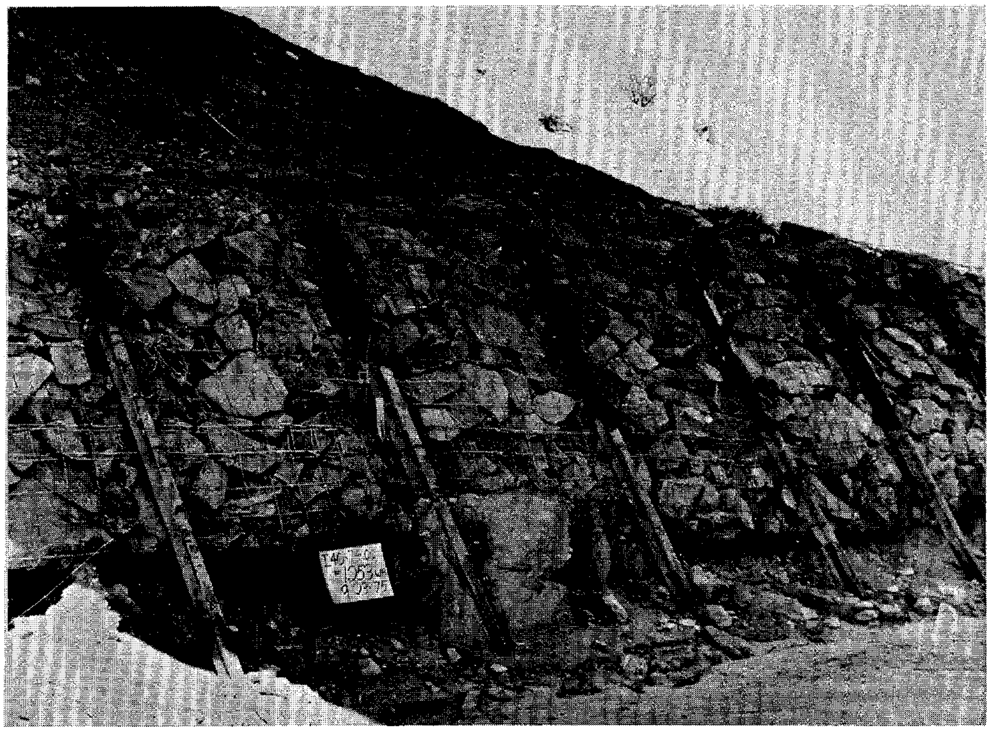


Figure 223. Lateral erosion of railbank (rock-and-wire) protection near upstream left-bank abutment in 1975. (From Arizona Dept. of Highways.)

Figure 224. View of downstream left-bank abutment and railbank (rock-and-wire) protection after repairs in 1976. (From Arizona Dept. of Highways.)

SITE 57. BRAWLEY WASH AT SR-86 NEAR THREE POINTS, ARIZ.

Description of site: Lat 32°05', long 111°20', location as shown in fig. 225. Precast-concrete bridge, 268 ft (82 m) in length, supported by wall-type concrete piers with rounded nose and founded on spread footings. Bridge skew is 20 degrees but piers are alined with flow. Abutments are spillthrough.

Drainage area, 776 mi^2 (2,009 km^2); bankfull discharge, about 7,000 ft^3/s (198 m^3/s); channel width, about 1,800 ft (549 m). Stream is ephemeral, alluvial, sand-gravel bed, in valley of moderate relief. Channel is sinuous, locally braided, locally anabranced, bordered by desert shrubs.

Hydraulic problems and countermeasures:

1964 A timber trestle bridge 210 ft (64 m) in length, supported by wood posts on concrete pads, was built across wash. The channel was enlarged and abutments protected with railbank (rock-and-wire) revetment.

1970 During the flood of September 4, 1970, water occupied the two overflow channels (fig. 225) and overtopped the road. The peak discharge was about 13,800 ft^3/s (391 m^3/s).

1975 Continued channel aggradation over the years caused a reduction in the flow capacity of the bridge. Because the wide flood plain and the overflow channels would have required three bridges, the left bank overflow channel was blocked off and two bridges were built. The new main-channel bridge was built 58 ft (18 m) longer than the former bridge and a spur dike was used to prevent scour of the left bank abutment.

1976 According to a bridge inspection report, the bridge, protective measures and channel appear in good condition. During the summer of 1976, a flow of 4,000 ft^3/s (113 m^3/s) occurred but this was insufficient to test the effectiveness of the new countermeasures.

Discussion: The original bridge design at this site did not allow for the possibility of an aggrading channel and consequent reduction in flow capacity of the bridge opening.

Figure 225. Aerial photograph in 1973 showing location of main channel and left-bank overflow prior to construction of new bridge. (From Arizona Dept. of Highways.)

SITE 58. OSBORNE WASH AT SR-172 NEAR PARKER, ARIZ.

Description of site: Lat 34°11', long 114°15'. Concrete-slab bridge, 269 ft (82 m) in length, supported by seven steel pipe-pile bents spaced at 35 ft (11 m). Bridge and pile bents are alined with flow. Abutments are spillthrough type, supported by steel pipe-piles.

Drainage area, 132 mi^2 (342 km^2); valley slope, 0.0067; channel width; about 1,000 ft (305 m). Stream is ephemeral, alluvial, sand bed, in valley of moderate relief, narrow flood plain. Channel is sinuous, generally braided, desert "wash".

Hydraulic problems and countermeasures:

1971 At the time of bridge construction, the stream was confined by blocking the left bank channel and the right bank channel was cleaned and excavated to handle the combined flows. Overbank flow was controlled at the bridge by use of two elliptical spur dikes 150 ft (46 m) in length, protected with "railbank" revetment (wire-mesh baskets filled with rock and supported by steel railroad rails).

Table 9. Comparison of channel hydraulics at bridge for design discharge and 1976 flood

Flow	Water Surface Elev. (ft)	Discharge	Estimated Recurrence Interval (yrs)	Net Area of Bridge Opening [3](ft^2)	Average Velocity [4](ft/s)
Design Q	377.3	14,000	50	2,217	6.3
Flood of 1976	374.6	7,710	35	[1]992	7.8
Flood of 1976	374.6	7,710	35	[2]1,117	6.9

[1]Includes effects of debris lodged at bridge piers.
[2]Assumes no debris lodged at bridge piers.
[3]ft^2 multiplied by 0.0923 equals m^2.
[4]ft/s multiplied by 0.305 equals m/s.

1976 A tropical disturbance on September 24 and 25 caused a flood flow of 7,710 ft^3/s (218 m^3/s) with a R.I. of 35 yr at the bridge site. Surveyed flood marks at the bridge site indicated about 1.5 ft (0.5 m) of fall between the bridge approach and contracted (downstream side of bridge) sections. During the 1976 flood, about 66 percent of the flow occupied the left bank flood plain with a maximum depth of flow of about 4 ft (1 m). No damage to the left bank spur dike was observed after the flood, indicating the spur and railbank protection performed as expected.

Average flow velocities during the 1976 flood were 7.8 ft/s (2 m/s), greater than anticipated for the design flood with a 50-yr R.I.(Table 9). The higher flow velocities during the 1976 flood were caused by deposition of debris on the pile bents, which reduced the size of the normal bridge openings by about 11 percent, and by aggradation of the channel bed. Although the debris does not consist of large individual pieces (fig. 226), the gross accumulation at each pier is significant in reducing the bridge waterway.

Discussion: Field surveys of the 1976 flood indicate the spur dikes were effective in controlling overbank flow and reducing scour at the bridge abutments. The 1976 flood (R.I. of 35 yr) caused flow velocities at the contracted (bridge) section slightly greater than anticipated for the design flood (50-yr R.I.). The higher velocities, which did not cause any hydraulic problems at the bridge, are attributed to lodgment of debris on the pile bents, and aggradation of the channel bed since bridge construction. About 11 percent of the waterway was occupied by debris lodged at the bridge.

The large accumulation of small debris at the pile bents indicates that this bridge, even though built in a desert area, should have been designed with pile bents spaced to minimize the number of bents in the waterway and exposure to lodgment of debris. During floods of larger magnitude than that of September 1976, greater amounts of debris, and higher water surface levels, should be expected at the bridge site, which may cause flow velocities much greater than anticipated in the bridge design.

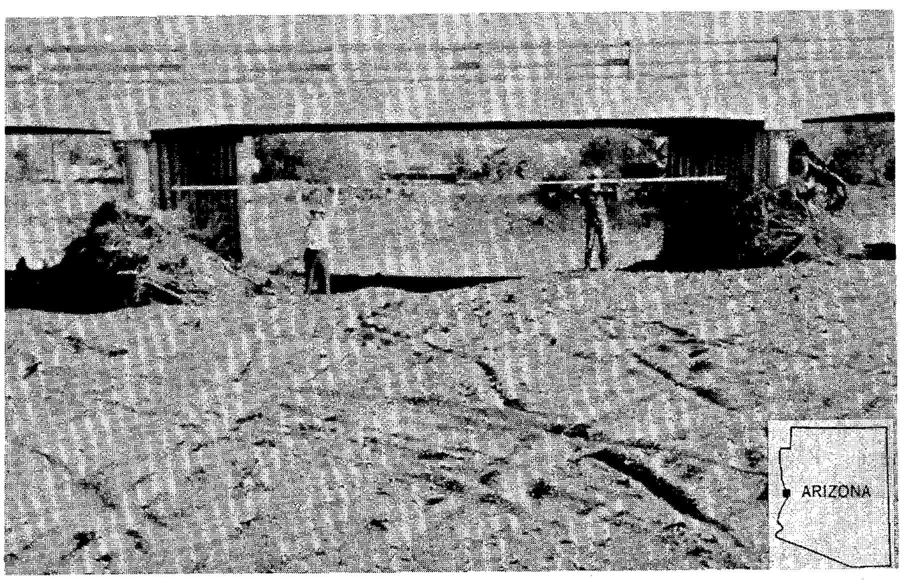


Figure 226. Upstream view of pile bents at piers 2 and 3 showing debris at bridge. Level rods are held at approximate water-surface elevation of 1976 flood. (From Arizona Dept. of Highways.)

SITE 60. BLACK CANYON WASH AT US-93 NEAR WICKENBURG, ARIZ.

Description of site: Lat 34°19', long 113°08', location as shown in fig. 227. Reinforced-concrete bridge with seven spans and total length of 201 ft (61 m). Bridge is supported by rectangular piers with rounded nose. Piers are integral with steel piling driven about 25 ft (8 m) below the streambed. Bridge skew is 30 degrees, piers are alined with the flow. Abutments are spillthrough type.

Drainage area, 28 mi^2 (72 km^2); channel slope, 0.029; channel width, about 750 ft (229 m). Stream is ephemeral, alluvial, sand bed, a wide, braided arid-region channel or "wash".

Hydraulic problems and countermeasures:

1959 Bridge built, with several countermeasures: (1) Bank revetment, locally called "railbank" (fig. 228), consisting of rock-filled wire-mesh fabric supported by steel rails, was placed at both banks at the bridge. (2) The piers were enclosed with concrete webs to prevent lodgment of debris (fig. 229). (3) The ends of the piers were protected with half-round steel pipe, welded in place, with end about 2 ft below the streambed (fig. 229).

1964 Flood from thunderstorms on Sept. 14, 1964 caused water to overtop the roadway at the right bank end of the bridge. The right-bank approach roadway washed out, closing the highway temporarily. Based on a study of the channel by the U.S. Geological Survey, water went over the top of the rock-and-wire railbank abutment protection on the right bank and was about 2 ft (0.6 m) deep over the roadway. The railbank protection washed out at the upstream corner of the abutment (fig. 230) but not at the downstream corner. Flows near the channel left bank were confined by the railbank protection. The discharge of this flood was about 26,400 ft^3/s (748 m^3/s) through the bridge opening. Depth of scour during the flood is unknown, but the channel bed was about 3 ft (1 m) lower at the bridge after the flood.

1975 Repairs at the bridge made after the 1964 flood included extension of the right-bank railbank revetment to a point 200 ft (61 m) upstream and orientation of the revetment more nearly parallel to the flow. In addition, a check dam of wire, railroad rail, and rock riprap was placed across the channel at the downstream side of the bridge. Part of the check dam was washed out between 1969 and 1975. The maximum water level since construction of the check dam was at a height of about 2 ft (0.6 m) on the right-bank revetment.

Discussion: The railbank (rock-and-wire mattress) probably prevented extensive damage to the right-bank bridge abutment during a flood with R.I. greater than 100 yr. Depth of flow, assuming no channel bed scour, was at least 10 ft (3 m). No abrasion damage to the concrete-encased steel-piling piers was reported, even though the channel bed is known to have scoured at least 3 ft (1 m). Flow velocities in the approach channel upstream from the bridge were about 5 ft/s (1.5 m/s).

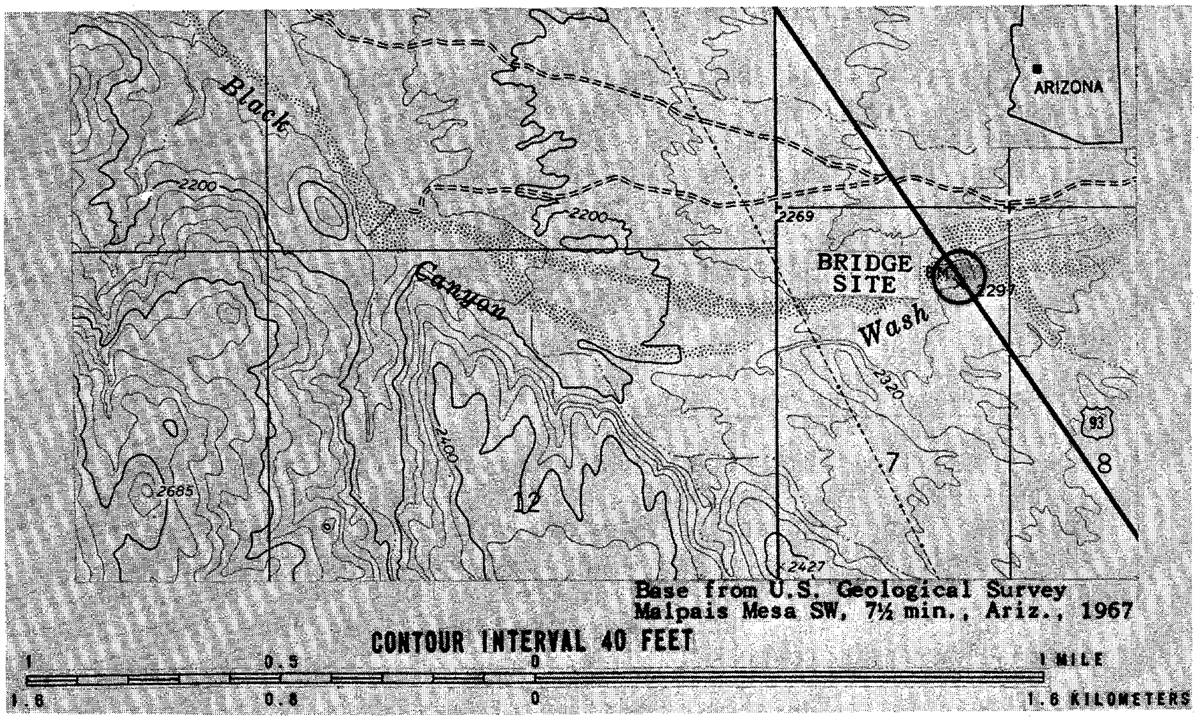


Figure 227. Location of Black Canyon wash at US-93 near Wickenburg, Arizona.

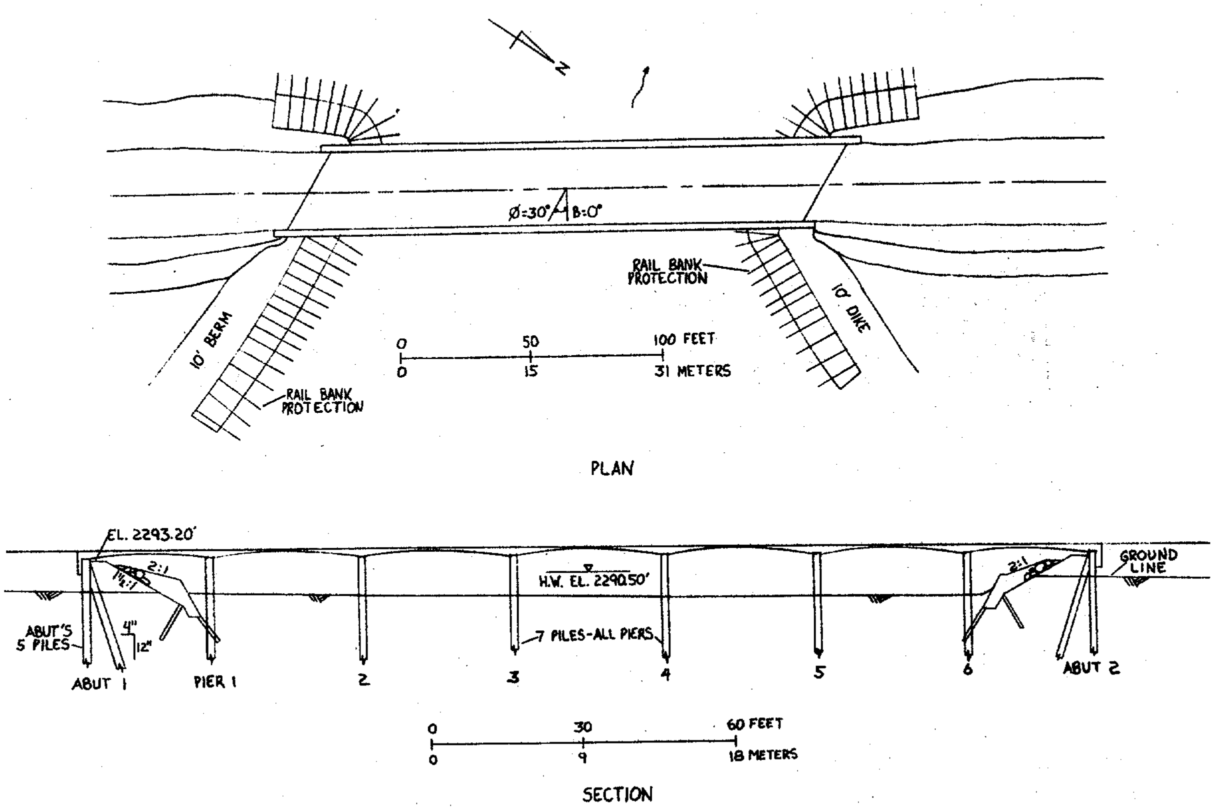
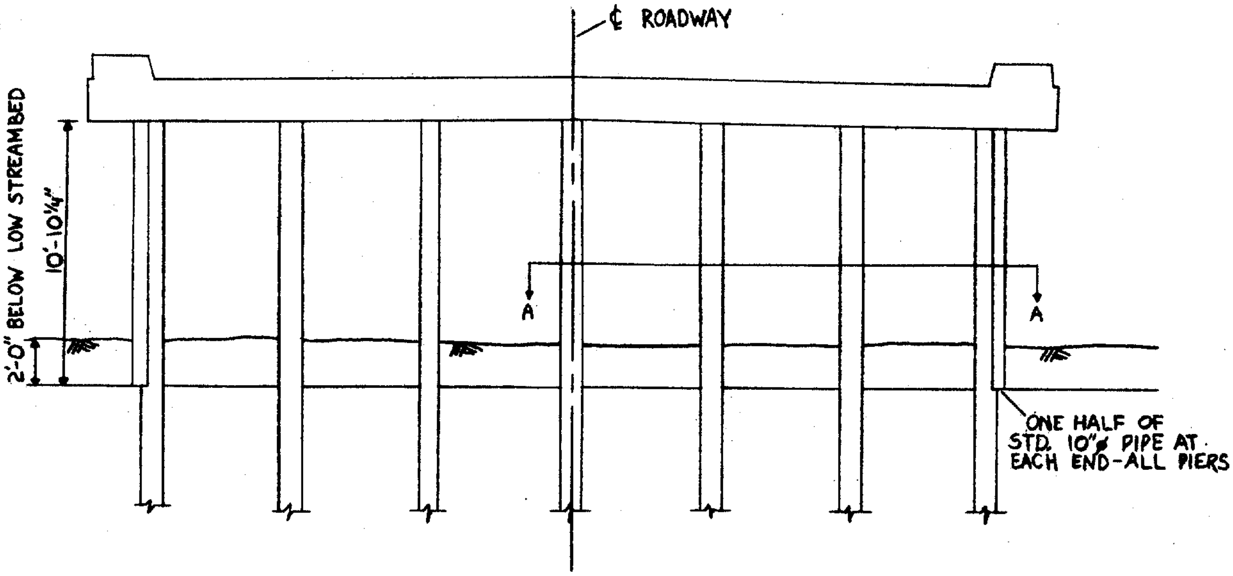


Figure 228. Plan and cross section of bridge site.



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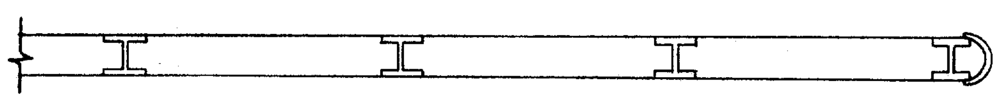


SECTION AA

Figure 229. Details of bridge construction.

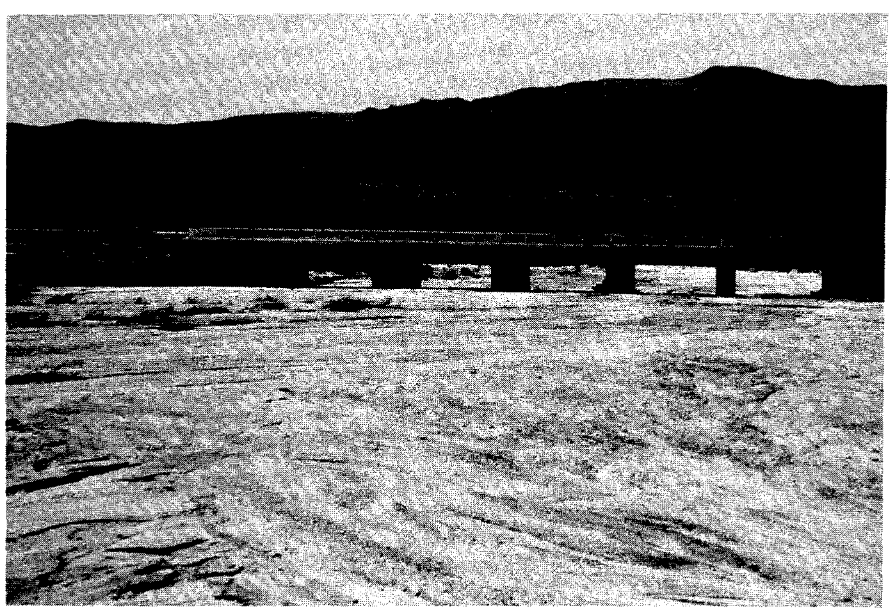


Figure 230. View downstream of bridge and damaged right-bank abutment in September 1964. (From Arizona Dept. of Highways.)

SITE 61. SYCAMORE CREEK AT SR-87 NEAR SUNFLOWER, ARIZ.

Description of site: Lat 33°47', long 111°30', location shown in fig. 231. Reinforced-concrete bridge, 246 ft (75 m) in length, three spans supported by concrete wall-type piers with rounded noses. Bridge skew is 15 degrees, and piers are alined with flow. The bridge replaces an older structure destroyed by the flood of September 1970. Piers are on spread footings. Abutments are spill-through type.

Drainage area, about 52 mi^2 (135 km^2); valley slope, 0.0014; channel width, about 200 ft (61 m). Stream is perennial but flashy, alluvial, bed material ranges from sand to boulders, in valley of moderate relief, narrow flood plain. Channel is sinuous, locally braided (fig. 232).

Hydraulic problems and countermeasures:

1970 Flood of September 5, discharge 16,100 ft^3/s (456 m^3/s), washed out original bridge. The bridge was about 140 ft (43 m) in length, supported by wall-type piers founded on spread footings. Base of the footings was about 10 ft (3 m) below the streambed. Bridge damage during the 1970 flood is attributed to inadequate length and capacity of the bridge opening, to scour that undermined one of the pier footings (fig. 233), and to location of the bridge crossing at a bend in the stream.

1973 New bridge constructed with several countermeasures to prevent damage by floods: (1) The bridge length and waterway opening were increased from 140 ft (43 m) to 246 ft (75 m) (fig. 233). (2) The base of the spread footings were placed more than 15 ft (5 m) below the streambed. (3) Bank revetment, locally termed "railbank", consisting of rock riprap enclosed in wire-fabric and supported by steel rails, was placed around the left-bank abutment. The steel-rail support posts were placed in holes drilled 3 ft (1 m) in granite bedrock to prevent lateral movement and undermining.

1976 Inspection of the site indicates that the railbank protection is in good condition. The maximum peak discharge since construction of the new bridge, as measured at a gage 5 mi (8 km) upstream, is 6,000 ft^3/s (170 m^3/s), R.I. about 8 yr.

Discussion: Failure of the original bridge is probably associated with a waterway opening that was inadequate for a flood with R.I. of about 25 yr, and with the location of the bridge at a bend of the stream. These factors probably caused the flow to concentrate near the outside of the channel bend and the bed was scoured by high flow velocity. Scour depths to about 10 ft (3 m) in the gravel-and-cobble streambed apparently occurred, as inferred from foundation depth of the pier that failed. Railbank revetment installed in 1973 performed effectively during a flood with R.I. of about 8 yr.

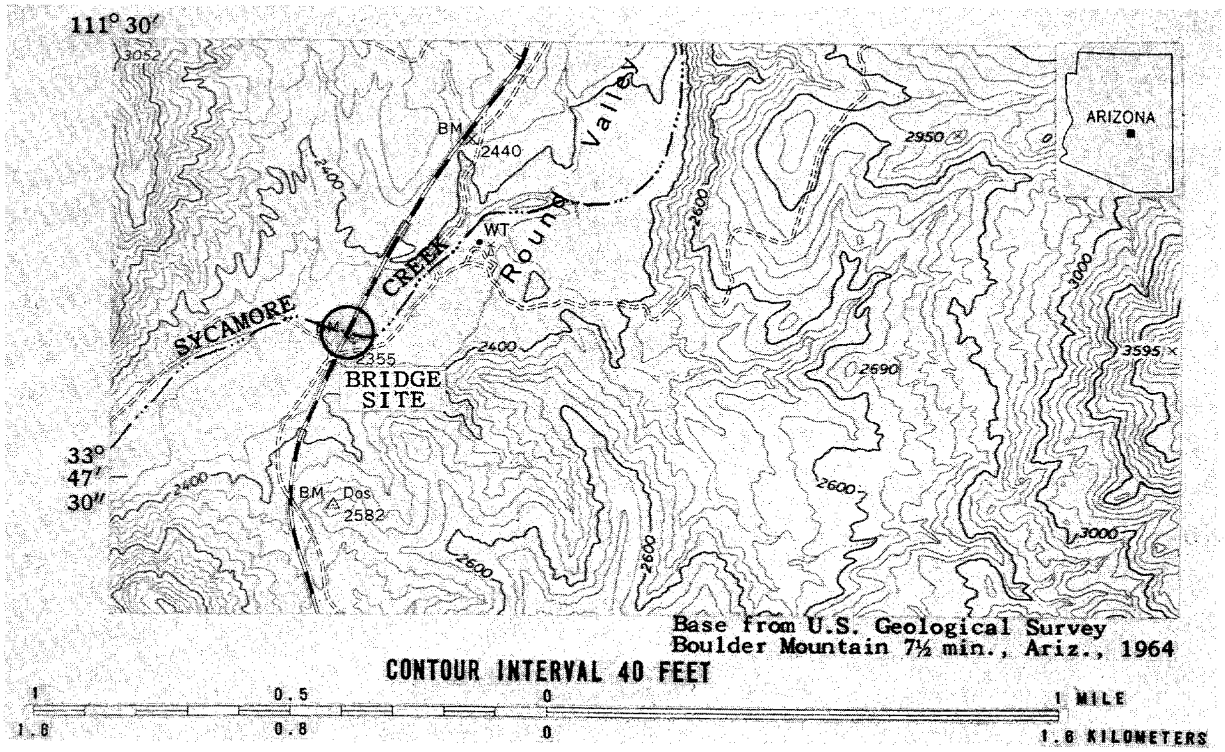


Figure 231. Location of Sycamore Creek at SR-87 near Sunflower, Arizona.

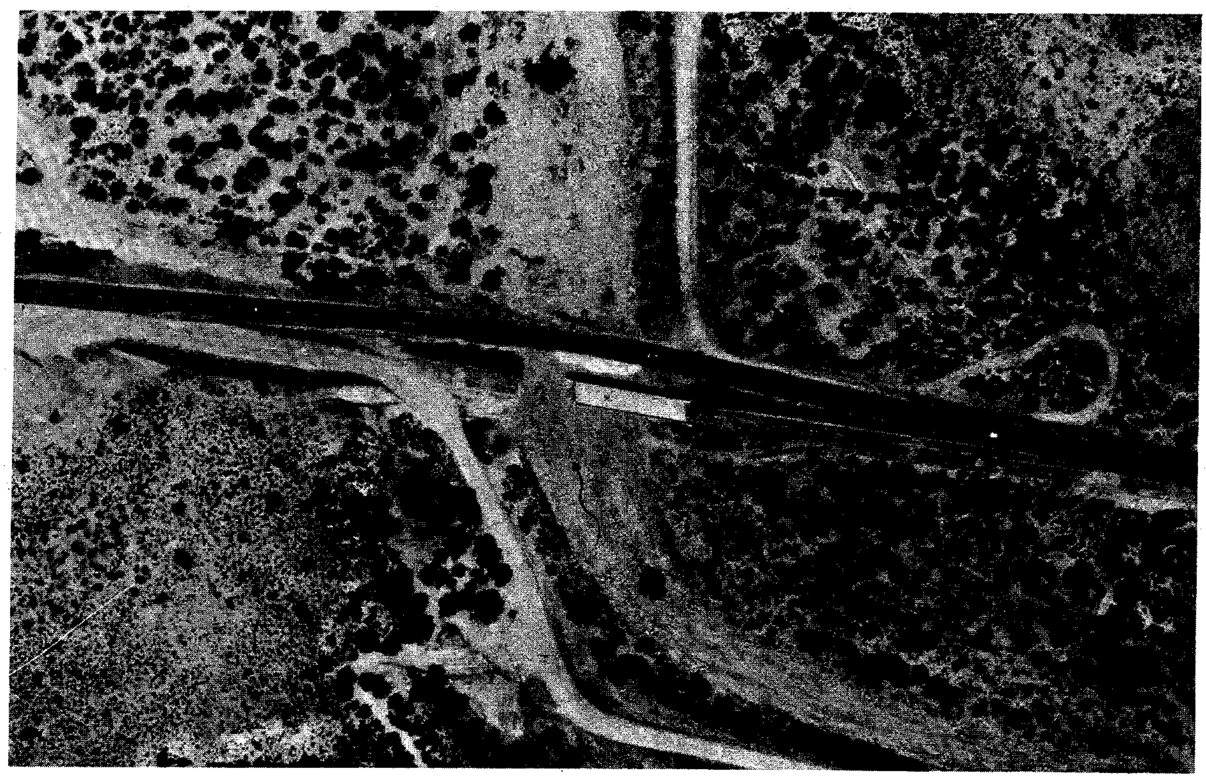


Figure 232. Aerial view in 1971 of bridge damaged by flood of September 1970. (From Arizona Dept. of Highways.)

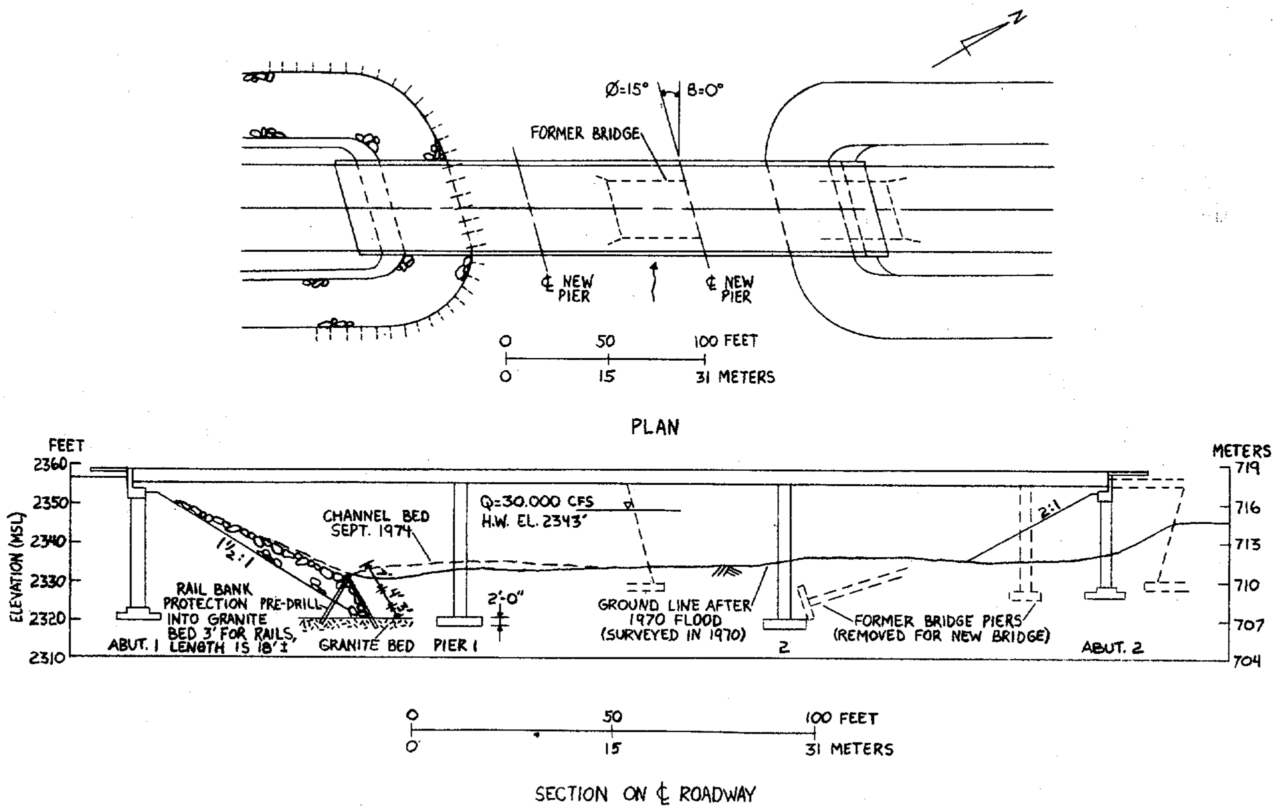


Figure 233. Plan and cross section of bridge built in 1973.

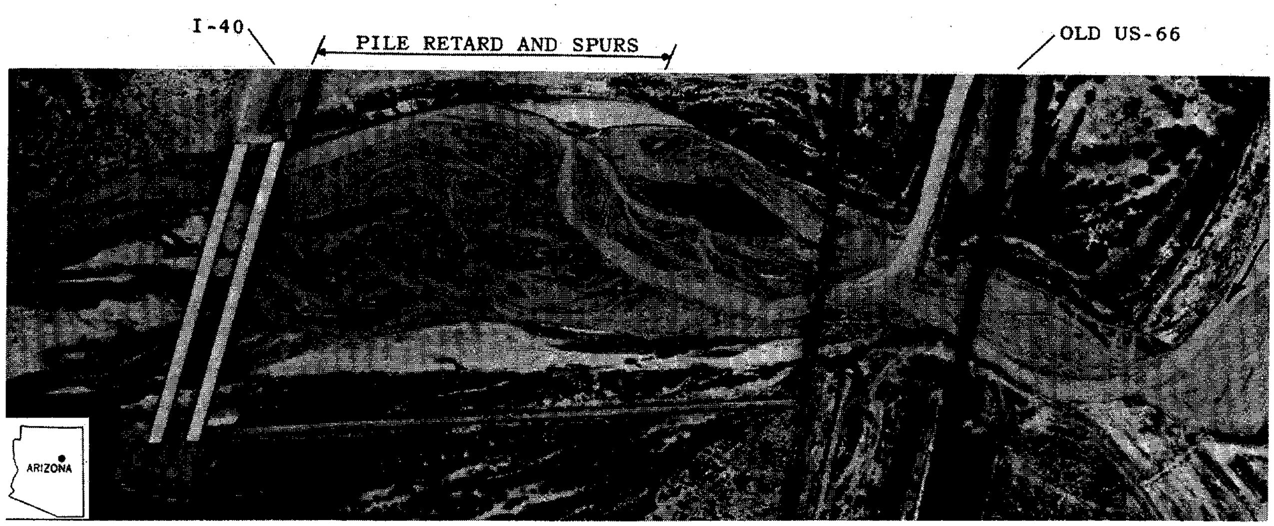


Figure 234. Aerial view in 1975 showing channel alinement and retard built to prevent lateral erosion of right bank. (From Arizona Dept. of Highways.)

SITE 63. LITTLE COLORADO RIVER AT I-40 NEAR WINSLOW, ARIZ.

Description of site: Lat 35°01', long 110°39', location as shown in fig. 234. Prestressed-concrete bridge, 1,004 ft (306 m) in length, supported by concrete piers founded on steel piles. Bridge is skewed 20 degrees but piers are alined with flow. Abutments are spillthrough type.

Drainage area, 16,100 mi^2 (41,700 km^2); valley slope, 0.0011; channel width, about 1,000 ft (300 m). Stream is perennial but flashy, alluvial, sand-gravel bed, in valley of moderate relief, wide flood plain. Channel is sinuous, generally braided, wandering thalweg.

Hydraulic problems and countermeasures:

1975 Bridge built, design flood of 50-year R. I., discharge 77,100 ft^3/s (2,183 m^3/s). Several countermeasures were built to prevent lateral erosion of the banks and bridge abutments, and shift of the channel: (1) A single timber-and-wire-mesh retard of fence type was built to protect the left bank approach channel. Base of rock fill about 2 to 3 ft (0.6 to 0.9 m) below ground surface. (2) A double timber-and-wire-mesh retard of fence type was constructed to protect the downstream right-bank bridge abutment (fig. 234). Base of rock fill was about 2 to 3 ft (0.6 to 0.9 m) below ground surface. (3) A steel sheet-pile retaining wall, held in place with piles and tie cables, was built to protect the bridge abutments on both banks. (4) A pile retard, supplemented with spurs (jetties), was built along the right bank upstream from I-40 to prevent lateral erosion and channel alinement changes. Base of the rock fill was about 2 to 3 ft (0.6 to 0.9 m) below ground surface.

1977 Surveys indicate minor changes in the channel cross section at the upstream bridge between 1975 and 1977. No significant floods have occurred since construction of the bridge and countermeasures.

Discussion: Retards and spurs were used to maintain an existing approach channel alinement to the bridge because the stream is unstable and braided. Both banks were protected from lateral erosion because the thalweg may shift from bank to bank during periods of high water.

SITE 68. MULBERRY RIVER AT I-40 NEAR MULBERRY, ARK.

Description of site: Lat 35°32', long 94°02', location as shown in fig. 235. Duel steel-girder bridge with concrete decking; length 486 ft (148 m); supported by five square-nose concrete wall-type piers set on spread footings in shale. Abutments are spillthrough. Bridge, abutments and piers are skewed 25 degrees to flood flow.

Drainage area, 398 mi^2 (1031 km^2); bankfull discharge, 4000 ft^3/s (1219 m^3/s); valley slope, 0.0016; channel width, 180 ft (55 m). Stream is perennial, semi-alluvial, gravel bed, valley of moderate relief, wide flood plain. Channel is sinuous, locally braided, locally anabranched, random width variation, cut banks rare, sand-gravel banks. Flood flow tends to follow bypass channels on the floodplain and adjust to the main channel.

Hydraulic problems and countermeasures:

1969 Bridge built. Two curved spur dikes, having the shape of a quarter ellipse in plan view and about 175 ft (53 m) long, were installed at time of bridge construction; both the spur dikes and spillthrough abutments were protected with riprap. The dike on the right bank extended into and almost across an overflow channel from an upstream bend (fig. 235).

1971 During the December flood, R. I. about 50 yr, the upstream end of the right-bank spur dike was eroded, most of the riprap on the dike removed by erosion, and a large scour hole, with a depth of 10-12 ft (3-4 m) was formed at the end of the dike. In addition, sediment was deposited beneath the bridge near the right abutment. Local scour occurred at piers 2, 3, and 4.

Analysis of the distribution of flow entering the bridge waterway indicated that rebuilding of the spur dike to its original configuration would result in a recurrence of the problem; but repair and riprapping the dike using its existing length (as shortened by erosion) would allow it to function sufficiently to prevent serious damage to the bridge. The scour hole at the dike was filled.

1973 During the November flood, slightly larger than the 1971 flood and approximately 1.1 times the magnitude of the 50-yr R. I. flood, the dike was undamaged but the scour hole formed again (fig. 236) and sediment again accumulated beneath the bridge at the right abutment. Flood flow, diverted around the right spur dike, was skewed at 20 to 45 degrees to piers 2, 3, and 4; local scour occurred at these piers, as did some general scour across the waterway.

1976 Analysis of floodflow distribution across the flood plain indicated that an overflow relief bridge near the right bank could be used to prevent the high degree of flow concentration at the right bank abutment.

Discussion: Location of the bridge at a bend of the stream caused flow concentration and local scour at the bridge, despite the presence of spur dikes. The use of relief bridges that allow for passage of natural overbank flow during floods of design magnitude would be desirable. In this case, the right-bank flood plain and overflow channels conveyed about 19 percent of the total flow during a flood with a R. I. of about 50 yr.

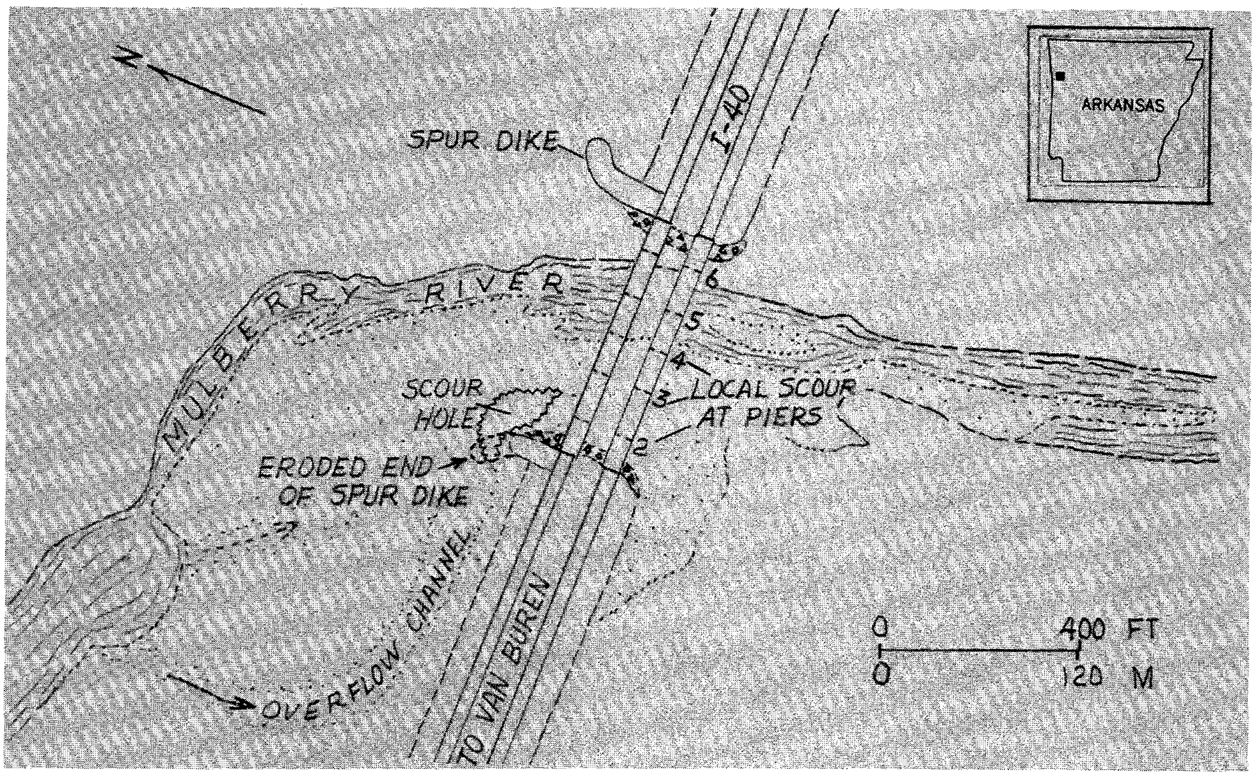


Figure 235. Plan sketch of I-40 crossing, Mulberry River, from aerial photograph taken in 1975.

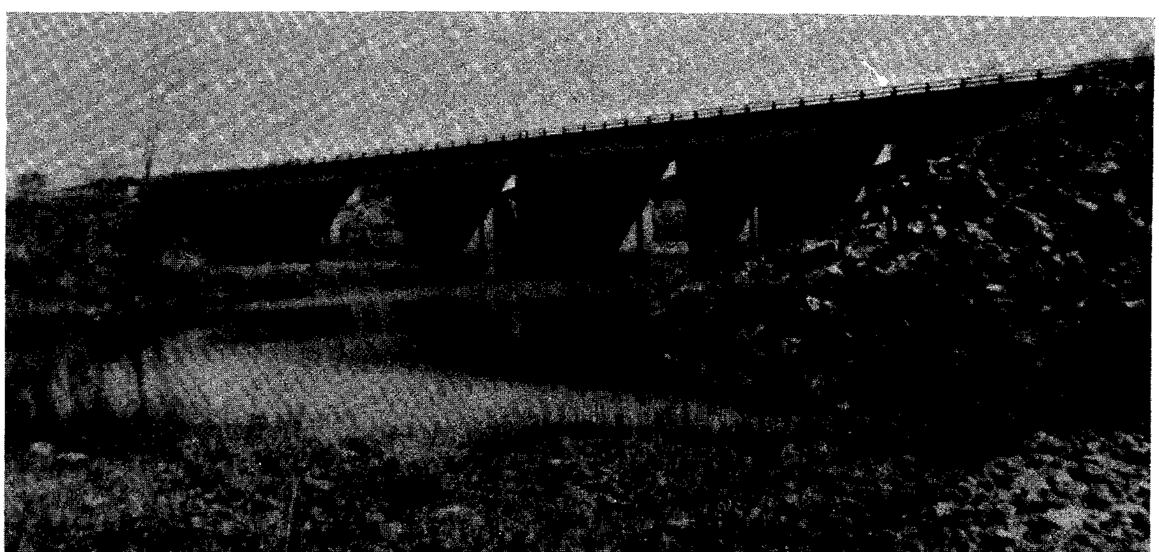


Figure 236. View in 1976 of spur dike and scour hole upstream from right abutment. Note orientation of bridge piers to right bank overflow channel.

SITE 69. POLK BAYOU AT SR-69 AT BATESVILLE, ARK.

Description of site: Lat $47^{\circ}30'$, long $91^{\circ}39'$, location as shown in fig. 237. Bridge is steel girder, length 332 ft (101 m), supported by two webbed concrete piers in channel and three bents outside the channel. Piling beneath piers and bents driven to bedrock. Abutments are spillthrough, set back from channel.

Bankfull discharge, 8,900 ft^3/s (252 m^3/s); valley slope, 0.0022; channel width, about 120 ft (37 m). Stream is perennial, semi-alluvial, gravel bed, in valley of moderate relief, narrow flood plain. Channel is sinuous, equiwidth, not incised, cut banks general, sand-gravel banks.

Hydraulic problems and countermeasures:

1970 Bridge built with 2 ft (0.6 m) clearance above design flood level. Abutments protected with dumped rock riprap of approximately 0.5 ft (0.15 m) maximum diameter, on a granular filter blanket. Riprap was extended to 1.5 ft (0.5 m) below ground line (fig. 238). The spread footings were keyed a minimum of 1.5 ft (0.5 m) in bedrock.

1975 During flood in March (R. I. approximately 50 yr), part of the bridge structure was submerged. General scour occurred on the flood plain (fig. 238) and under the bridge, exposing several pile bents to a maximum depth of about 5.0 ft (1.5 m) (fig. 239). Some riprap was also eroded.

1977 No further significant events have occurred.

Discussion: Inadequate vertical clearance of the bridge waterway caused increased velocities at the constriction when the bridge was submerged. Scour damage around the pier footings both in the main channel and on the flood plain then resulted. The actual flood level for the 1975 flood (R. I. about 50 yr) was about 3.3 ft (1 m) higher than the estimated water level for the design (50-yr R. I.) flood. If bridge clearance above the design flood water level is to be kept at a minimum, hydraulic studies of the channel to determine the relation between flood stage and discharge should be made. A standard 2-ft (0.6 m) freeboard is ample for some bridges and inadequate for others. The key factors to consider for increasing freeboard are potential for debris, low channel slope, high channel roughness, and constrictions downstream from the bridge.

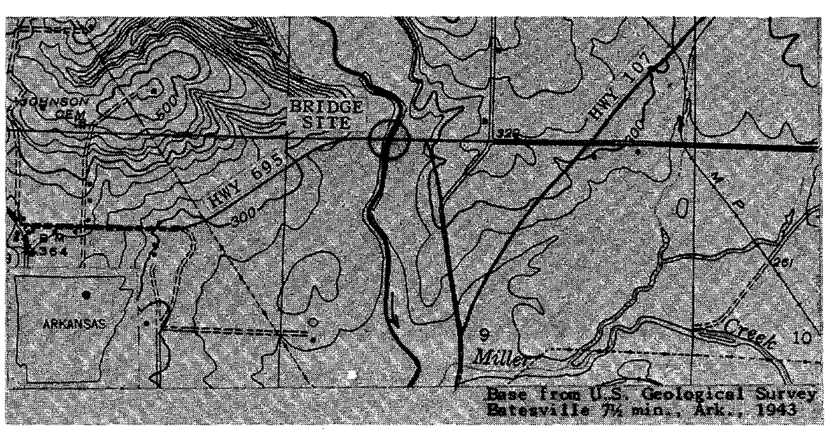


Figure 237. Location of Polk Bayou at SR-69 at Batesville, Arkansas.

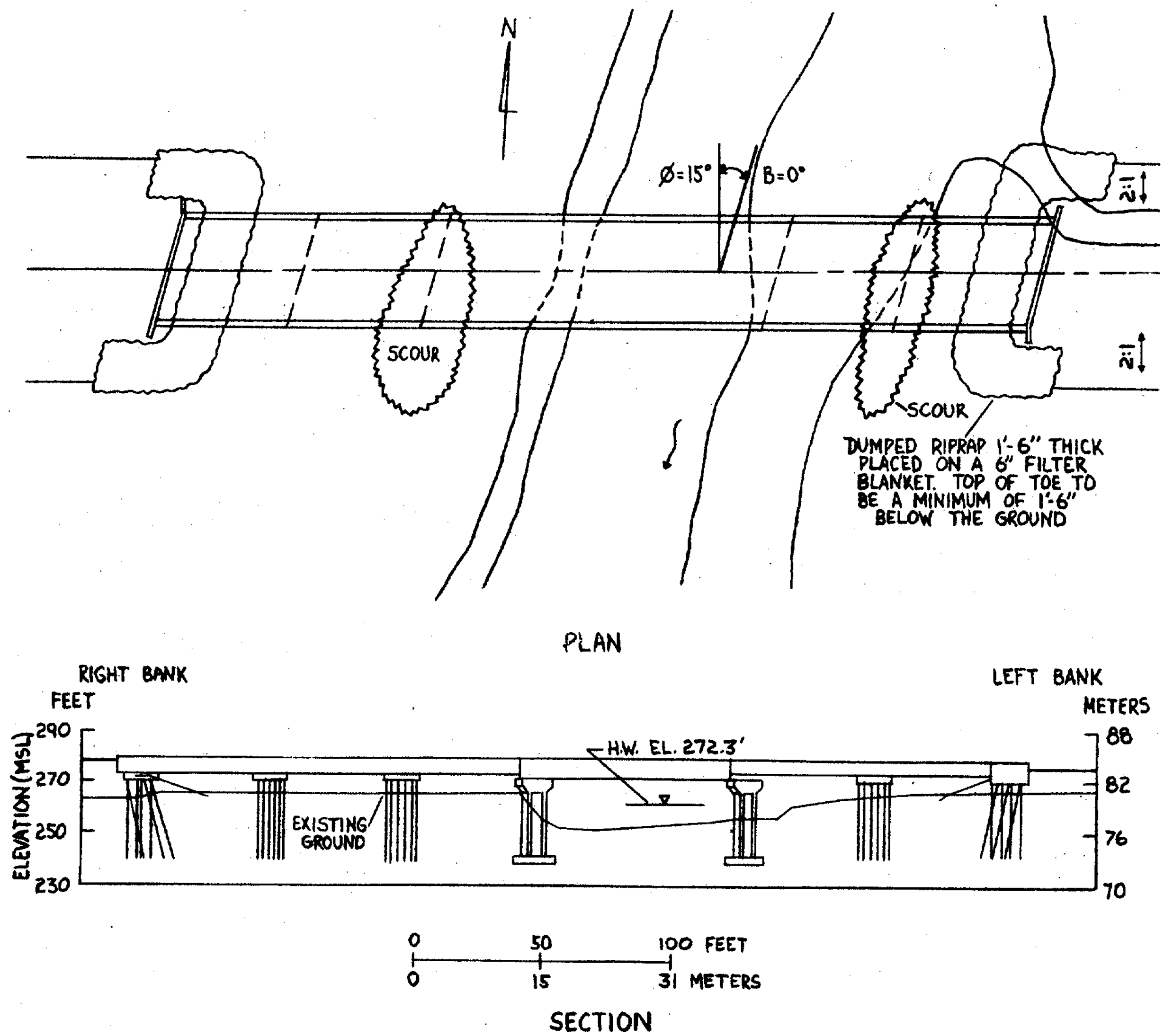


Figure 238. Details of crossing, Polk Bayou at SR-69.

Figure 239. View upstream beneath bridge in 1976, showing scoured area on floodplain and exposed pile bents (darker, lower part of piles).

SITE 70. BIG RUNNING WATER CREEK AT SR-228 NEAR CLOVER BEND, ARK.

Description of site: Lat 35°59', long 91°05', location as shown in fig. 240. Concrete box-girder bridge, length 125 ft (38 m), supported by four pile bents, each consisting of three pre-cast octagonal concrete piles. Abutments are spillthrough, supported on piles. Piles are driven into fine gray water-bearing sand (fig. 241).

Drainage area 49 mi^2 (127 km^2); bankfull discharge, 1,700 ft^3/s (48 m^3/s); valley slope, 0.0047; channel width, 43 ft (13 m). Stream is perennial, alluvial, bed mostly sand with minor amounts of silt and clay, low relief valley, wide flood plain. Channel is sinuous, cut banks rare because of dense forest cover, silt-sand banks.

Hydraulic problems and countermeasures:

1962 Bridge built, without countermeasures.

1975 Flood in March reached an elevation that was 1.7 ft (0.5 m) below the concrete girders and drift accumulated at the bridge. General scour reached a maximum depth of 28 ft (8.5 m) below the original streambed, exposing all but 5 ft (1.5 m) of bent 5 (fig. 241) allowing it to settle and causing the bridge to be closed to traffic. The damaged bridge was jacked up, new piles driven, and the bent rebuilt. The scour hole was filled and scour has not recurred during subsequent floods.

Discussion: The depth of scour during the 1975 flood was about 22 ft (6.7 m) below the normal bed elevation of the main channel. Scour is attributed to accumulation of drift and consequent constriction of waterway opening, and to lack of cohesion of the sandy alluvium beneath the channel. Except at the bridge, the channel banks are densely forested and channel is stable.

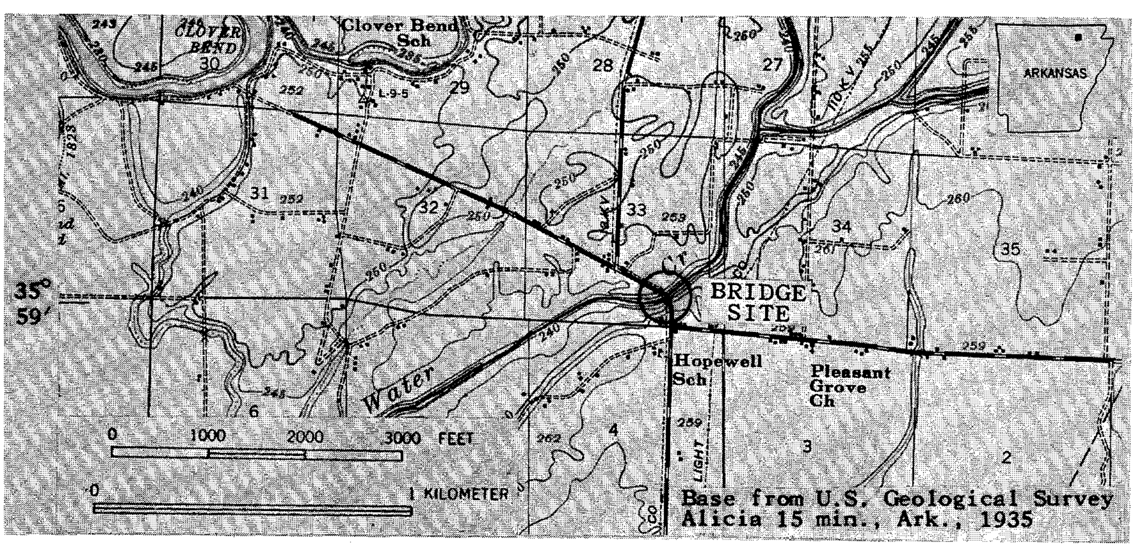


Figure 240. Location of Big Running Water Creek at SR-228 near Clover Bend, Arkansas.

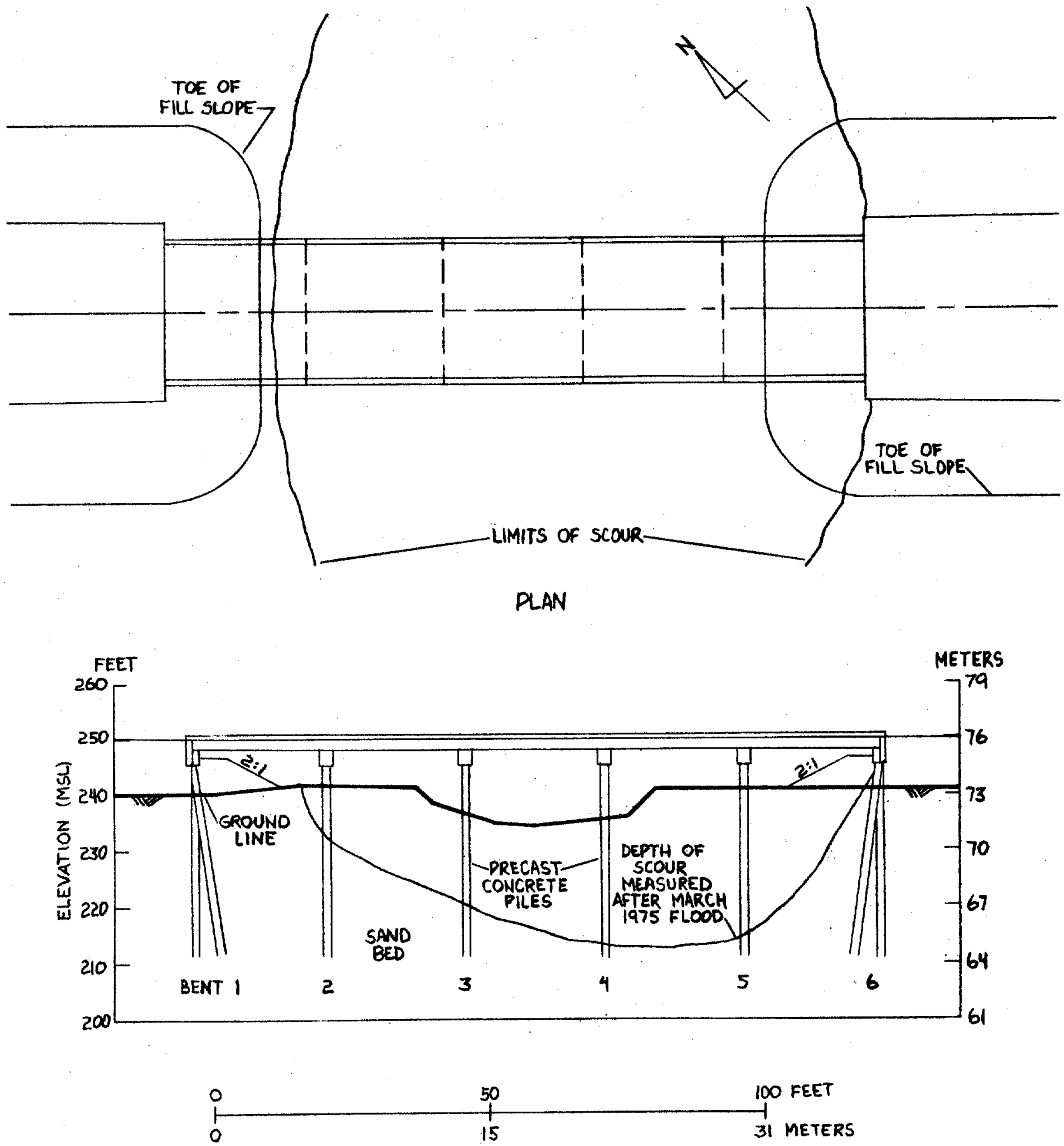


Figure 241. Plan of bridge and channel cross section.

SITE 71. SAN BENITO RIVER AT US-101 NEAR CHITTENDEN, CALIF.

Description of site: Lat 36°53', long 121°34', location as shown in fig. 242. Steel plate-girder bridge, length 722 ft (221 m), built in 1950. Bridge supported on concrete wall-type piers with pointed nose and pile foundations. This bridge is adjacent to an older bridge built in 1932, 710 ft (216 m) in length, consisting of 3 spans with steel trusses and 9 concrete-girder approach spans. The steel-truss spans were replaced with steel girders in 1958 to improve clearance above highwater. Abutments are spillthrough type. Flow is normal to bridge during periods of high flow.

Drainage area, about 680 mi^2 (1,760 km^2); bankfull discharge, 9,500 ft^3/s (269 m^3/s); valley slope, 0.002, channel width, about 700 ft (213 m). Stream is ephemeral, regulated, sand bed, in valley of moderate relief, narrow flood plain. Channel is sinuous, generally braided.

Hydraulic problems and countermeasures:

1932 Bridge built with the main-channel steel truss spans providing an inadequate clearance above the waterway. Hydraulic problems at this bridge are a result of debris lodging on bridge and a stream that is aggrading.

1938 Flood submerged bottom truss of bridge to a depth of 3.2 ft (1.0 m).

1941 Flood submerged truss to depth of 2.5 ft (0.8 m). Channel aggradation is caused by deposition of suspended material on the bridge members plus the change in flow conditions at the junction of Pajaro and San Benito Rivers just downstream from bridge (fig. 242).

1950 New steel plate-girder bridge adjacent to older bridge built with clearance of 3.5 ft (1.1 m) above 100-year flood.

1955 December flood reached an elevation of 137.8 ft (R. I. 10 yr) and caused submergence of bottom truss of old bridge about 5 ft (1.5 m). Silt and much debris collected on bridge, also causing an increased deposition of suspended material in channel. The silt and debris was removed following the flood.

1958 Main channel bed aggraded to about elevation 127 ft (38 m) or 4.7 ft (1.3 m) below bottom of the truss. The existing steel-truss spans for the older bridge were removed, the piers raised and plate-girder spans with less depth installed. Extent of aggradation in the sand bed channel at the bridge site is given in table 10:

Table 10. Aggradation of San Benito River at US 101 near Chittenden between 1931 and 1958

Year	Area of waterway below elev. 139.1 ft (ft^2)	Mean bed elevation (ft)
1931	9820	122.4
1941	5250	130.3
1958	4450	132.3

1964 The river channel was noted as not aggrading since 1958 when the 1932 bridge was raised. Raising the 1932 bridge to provide clearance for drift apparently reduced the rate of suspended material deposition.

1977 Channel appears stable. Flooding in 1970 and 1973 has not caused notable increases in channel aggradation at the site.

Discussion: Channel aggradation at the crossing was probably caused by backwater from the confluence of the San Benito and Pajaro Rivers and lodgment of debris on the bridge, which caused subsequent deposition of suspended material. Raising the bridge to provide adequate clearance above floods has reduced the problem of channel aggradation at the bridge site. Present rates of channel aggradation are considered insignificant. The original bridge, constructed without adequate clearance above the waterway, caused immediate maintenance problems, such as debris removal and long-range problems such as changes in the channel alinement and stream bed elevation as a result of sediment deposition.

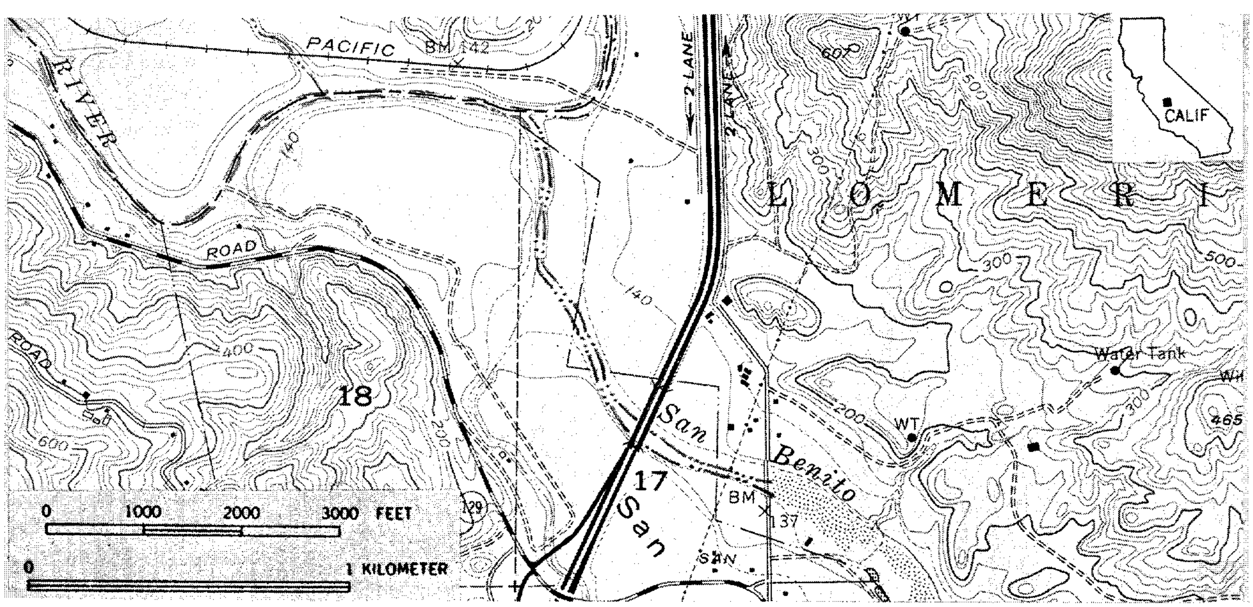


Figure 242. Location of San Benito River at US-101. (Base from U.S. Geol. Survey Chittenden, Calif., 7.5' map, contour interval 20 feet, 1955 photorevised 1973.)

SITE 73. SALINAS RIVER AT US-101 AT SOLEDAD, CALIF.

Description of site: Lat 36°25', long 121°19', location as shown in fig. 243. Reinforced-concrete continuous-girder bridge, 1,530 ft (446 m) in length, supported on wall-type piers with square nose and pile foundations. An adjacent southbound bridge, of the same length, was built in 1960. At low flow, the bridge skew is nearly zero degrees, but at high flow, the bridge and pier skew are about 30 degrees.

Drainage area, 3,563 mi^2 (9,225 km^2); valley slope, 0.0012; channel width, 1,200 ft (366 m). Stream is perennial, regulated, alluvial, sand bed, in valley of moderate relief, narrow flood plain. Channel is sinuous, generally braided, wandering thalweg, cut banks local, silt-sand banks.

Hydraulic problems and countermeasures:

1914 Steel-truss bridge built, at a site just upstream from that of a later bridge, constructed in 1936. To stabilize the left bank of the channel and prevent lateral erosion near the bridge abutments, several lines of steel-rail tetrahedrons were constructed in 1935 upstream and downstream from the bridge. These performed very successfully, resulting in outbuilding of the left streambank by deposition of sand. By 1936, almost all tetrahydrons were buried, only a few being visible on the downstream side of the bridge built in 1914. Additional lines of tetrahedrons were constructed along and parallel to the new bank in 1936 for bank protection until natural vegetal growth could be established. The success of the tetrahedrons in stabilizing the left bank at the 1914 bridge site since 1935 is illustrated by vegetal growth along the left bank (fig. 243).

1937 New bridge and approach built with length sufficient to provide safety from probable lateral bank erosion. The bridge was built with 5 ft (1.5 m) clearance above the January 1914 high water (elevation 178 ft or 54 m). Piers were not alined with respect to flow because the location and direction of flow is variable. The streambed scours during high flows and the estimate of scour depth around the bridge piers was 10 ft (3 m). The channel bed apparently fills to normal level after floods. Groundwater levels in 1936 were at elevation 161.5 ft (49.2 m), which is near the general water level of the surrounding area. Presence of the high groundwater level contributes to the unstability of the sandy bed material in the channel during periods of flow. Foundation piles were placed with tips about 50 ft (15 m) below the streambed.

1960 Adjacent bridge for southbound traffic built. The new bridge was built with the same length as the older structure (1,530 ft or 466 m) on wall-type piers with pointed nose. The main channel had moved toward the right bank, but there was little change in the lowest streambed elevation (thalweg). Foundation piles for the bridge were placed with tips about 50 ft (15 m) below the channel bed.

1969 Floods in January and February had the highest flows since 1938, and is probably the highest since 1914. According to a measurement made from the bridge on Feb. 28, 1969, the discharge was 82,500 ft^3/s (2,336 m^3/s) and the maximum scour depth was 20 ft (6 m). The flood caused no damage to the bridge.

Discussion: Tetrahedrons have successfully stabilized the banks of this sand channel. Streams with sand bed and high ground-water levels are subject to unstable channels and scour. The bridge piers, with pile tips about 50 ft (15 m) below the channel bed, were not damaged by scour during the 1969 flood. During the flood of February 1969, scour depths of 20 ft (6 m) below the streambed were measured.

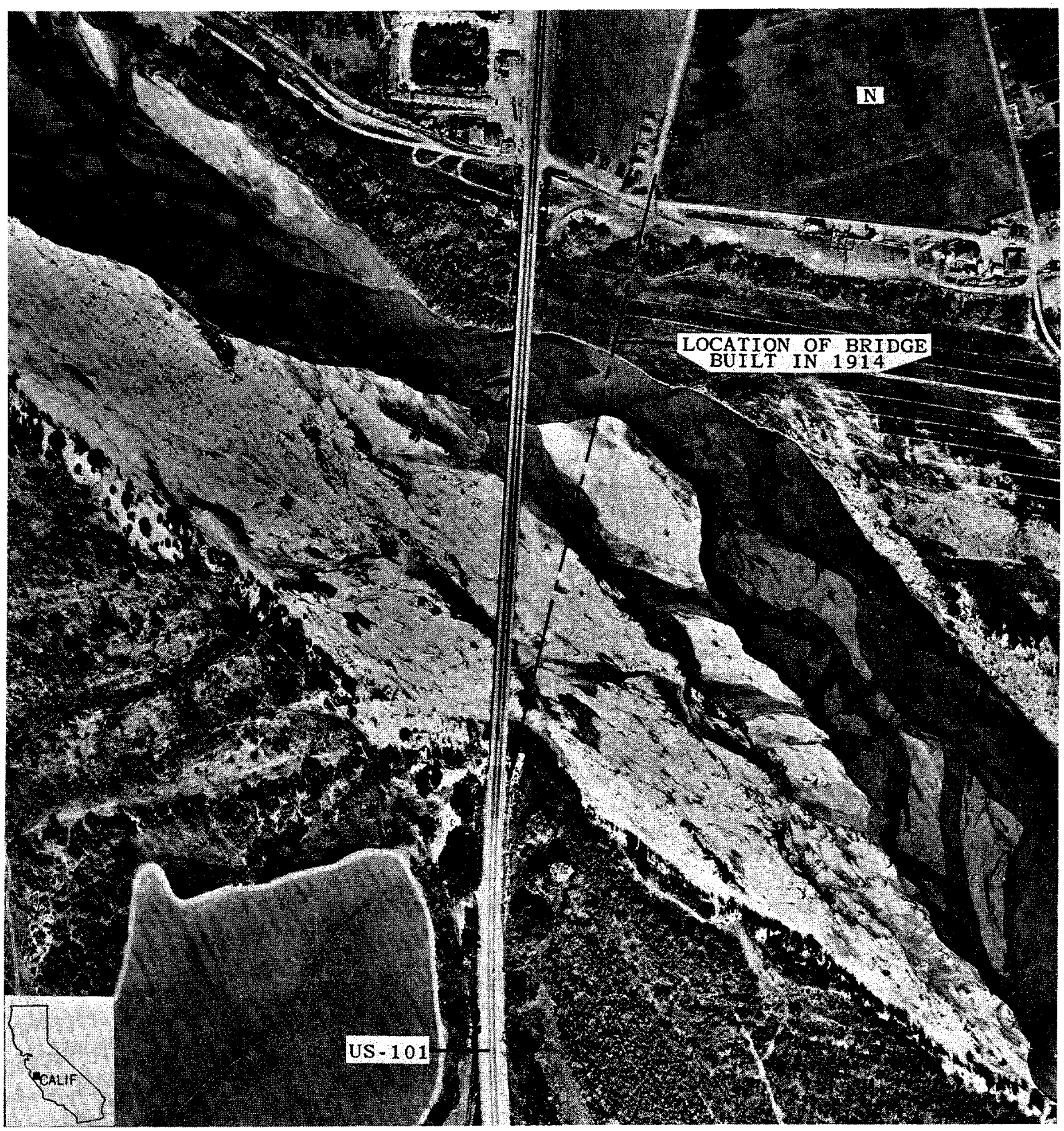


Figure 243. Aerial photograph of Salinas River at US-101, in 1953. (From Calif. Dept. of Transportation.)

SITE 74. RAMSHORN CREEK AT FOREST SERVICE ROAD 39N26 NEAR TRINITY CENTER, CALIF.

Description of site: Lat 41°09', long 122°39', location as shown in fig. 244. Single-span prestressed-concrete bridge 90 ft (27 m) in length, supported by concrete abutments on spread footings. Bridge is skewed 15 degrees to flow. The abutments are spillthrough.

Drainage area, 13 mi^2 (34 km^2); valley slope, 0.0133; channel width, about 300 ft (91 m). Stream is perennial, alluvial, gravel-cobble-boulder bed, in valley of high relief, no flood plain. Channel is sinuous, locally braided, sand-gravel banks.

Hydraulic problems and countermeasures:

1975 Bridge built with several measures to prevent channel bank erosion (fig. 244), with due regard to channel alinement changes that occurred during a flood in January 1974. (1) Flows were alined with the bridge opening by use of a left-bank dike and right-bank gabion wall. (2) The channel size at the bridge opening was increased and the dike was protected with heavy rock riprap brought from the channel bed. (3) No pier was placed in the channel. (4) The rock riprap and gabion-wall footings were placed about 3 ft (1 m) below the streambed to prevent undermining.

1977 The completed installation (fig. 245) has not been subject to flood flow.

Discussion: The alluvial channel at this bridge site is very unstable, and debris is supplied by logging operations in the watershed upstream. Measures to protect the bridge included the placement of dikes for flow alinement, construction of a bridge with no piers in the waterway, and extensive protection of the revetment toe to prevent damage from undermining. Although these countermeasures have not been tested by flood, they will probably perform well.

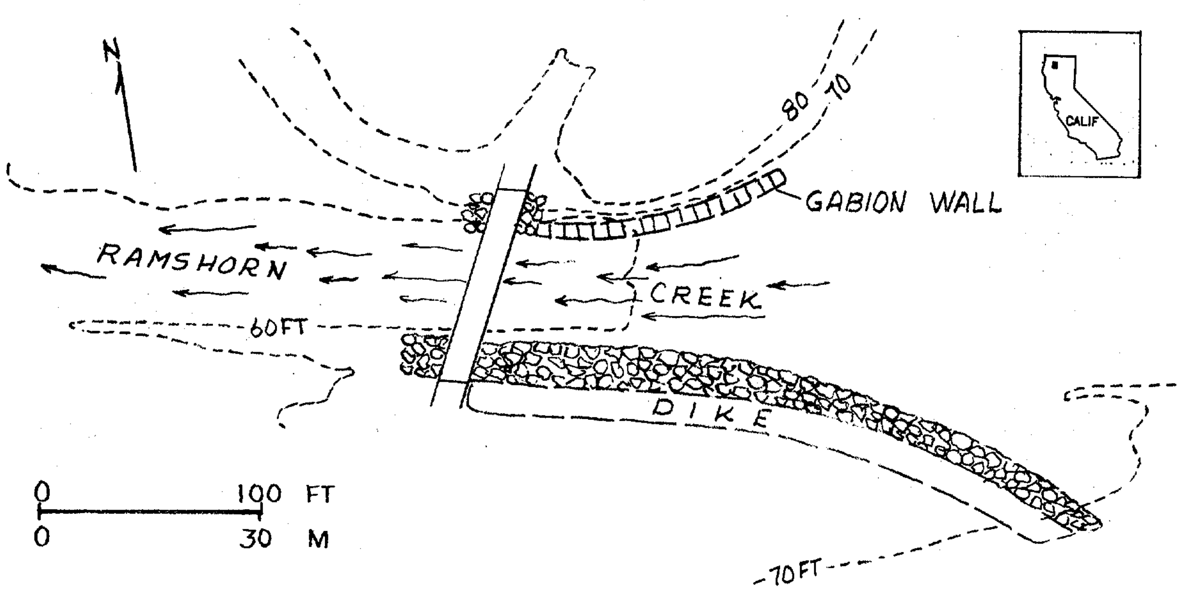


Figure 244. Plan snowing countermeasures at Ramshorn Creek.

Figure 245. View downstream towards bridge showing completed countermeasures, 1977.

SITE 75. STONY CREEK AT SR-32 NEAR ORLAND, CALIF.

Description of site: Lat 39°45', long 122°06'. Reinforced-concrete box-girder bridge, 1,492 ft (454 m) long with 18 spans of 76 ft (23 m) and 2 spans of 62 ft (19 m) length. Bridge is supported by wall-type concrete piers with rounded nose founded on steel pilings. Bridge skew is 46 degrees, piers are alined with flow. Abutments are spillthrough.

Drainage area, 770 mi^2 (1,994 km^2); bankfull discharge, 70,000 ft^3/s (1,981 m^3/s); channel width about 1,000 ft (305 m). Stream is ephemeral, regulated, alluvial, gravel bed (D_{50}, 7.5 mm), on a broad alluvial fan. Channel is sinuous, generally braided, has distinct natural levees.

Hydraulic problems and countermeasures:

1975 New bridge constructed upstream from original bridge. The channel was realined, and several countermeasures installed to stabilize the new channel location. Although the stream is regulated, flooding occurs during significant storms and the stream carries a high load of floating debris. Between 1961 and 1975, the channel alinement shifted toward the left bank and 9 ft (3 m) of scour occurred at the old bridge site. Existing steel jacks located on the left bank were utilized. Additional bank protection included construction of fence-type retards along the approach embankment upstream from the bridge (fig. 246) and along the channel banks downstream from the bridge (fig. 247). Grouted rock riprap was placed at the abutments.

Discussion: Countermeasures at channel banks and abutments were used because of observed channel instability problems at the former bridge site. Depths of scour at the old bridge site up to 9 ft (3 m) were observed during changes in the channel alinement between 1961 and 1975. Because the stream alinement and alluvial channel bed were very unstable, bank stabilization measures included the placement of retard footings and anchors 14 ft (4 m) deep to prevent undermining.

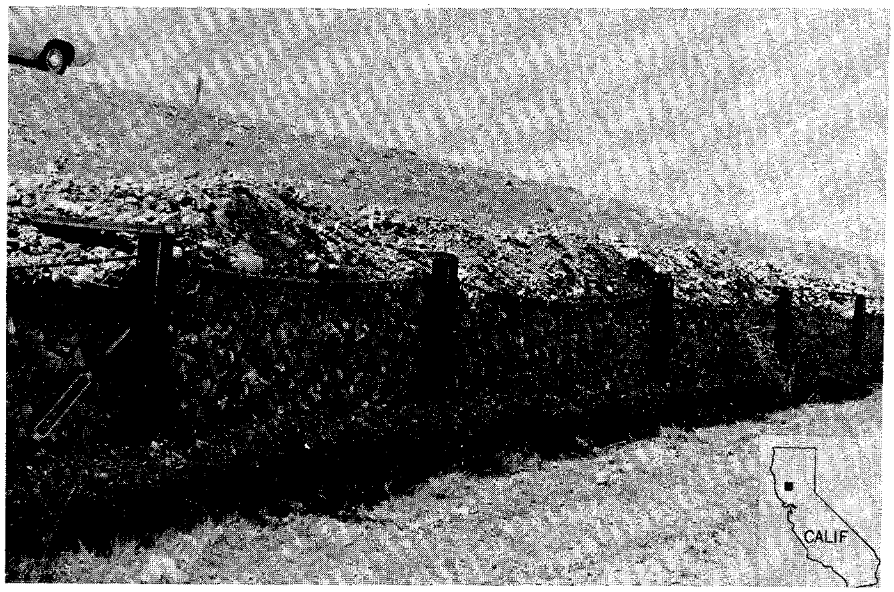


Figure 246. View of double-post rock-and-wire retard constructed along upstream side of right-bank approach embankment. Photographed in 1977.

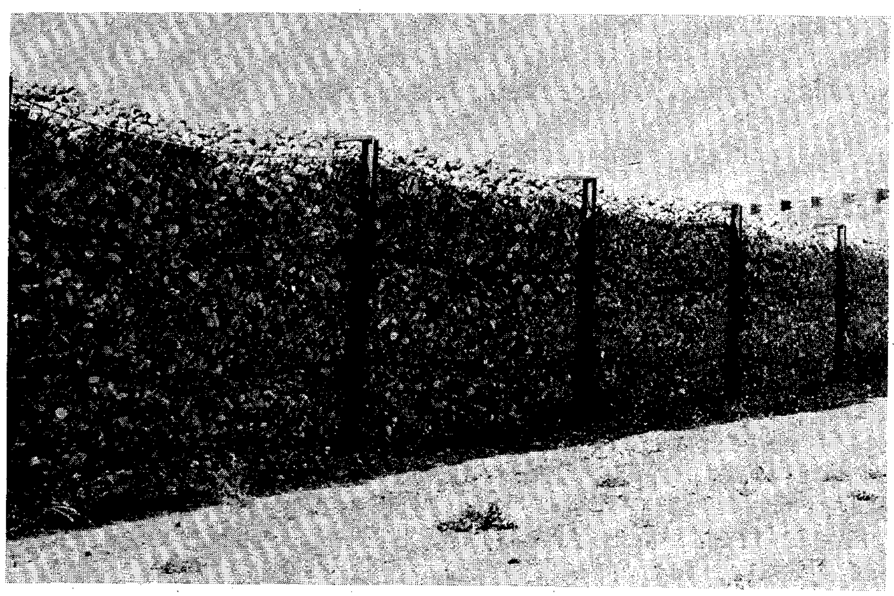


Figure 247. View of double-post rock-and-wire retard constructed along right bank of channel downstream from bridge. Note guardrail posts in background for scale. Photographed in 1977.

SITE 85. SAND CREEK AT QUEBEC STREET BRIDGE AND I-270 AT DENVER, COLO.

Description of site: Lat 39°47', long 104°54', location as shown in fig. 248. Concrete bridge about 230 ft (70 m) in length, supported by wall-type piers founded on piling. Bridge is skewed about 27 degrees, (fig. 248) but piers are alined to flow. Abutments are spillthrough type.

Valley slope, 0.0031; channel width, about 350 ft (107 m). Stream is perennial but flashy, sand bed, in valley of low relief, narrow flood plain. Channel is sinuous, generally braided.

Hydraulic problems and countermeasures:

The following description of hydraulic problems and countermeasures on Sand Creek in the vicinity of the Quebec Street Bridge is based on notes furnished by the Colorado Department of Transportation.

1940 Removal of sand and gravel from the Sand Creek channel since 1940 caused about 9 ft (3 m) of degradation between 1940 and 1970 (fig. 249) at the Vasquez Boulevard bridge. About half of the degradation occurred between 1967 and 1973.

1965 Quebec Street Bridge built, steel piling used for pier foundations.

1967-68 Interstate 270 was built along the Sand Creek channel. The embankment caused some encroachment on the Sand Creek flood plain and channel, and the channel was a borrow source for the interstate construction.

1969 Channel degradation caused exposure of the footing pilings at the Quebec Street Bridge. To prevent further exposure of the piling, a check dam was constructed downstream from the bridge.

1973 In May the first significant flood after construction of the check dam caused failure of the dam. Changes in the channel bed profile between 1972 and 1973 (fig. 250) show degradation in the downstream part of the reach.

1974 Several measures to prevent continued channel bed changes of Sand Creek and lateral erosion of the I-270 embankment in the vicinity of the Quebec Street bridge were constructed. (Protective measures in the reaches of Sand Creek upstream from Quebec Street bridge and downstream from Vasquez Boulevard bridge were also constructed, but are not described here.) (1) Timber-pile and wire-enclosed rock riprap revetment was placed along the I-270 embankment (fig. 251) to prevent lateral erosion. (2) To prevent undermining of the bank revetment, gabion slope mattresses were built (fig. 251). The mattresses were selected because deep excavation would not be needed, they were less susceptible to failure from shifting, and could be repaired, or added to, if more channel degradation occurred. (3) Steel sheet-pile check dams were built downstream from the Vasquez Boulevard bridge and Quebec Street bridge. To prevent further exposure of the bridge foundation, the steel piling were driven to shale bedrocks. A cross section of the channel and elevation of the steel sheet-pile check dam is shown in fig. 252. A gabion check dam was constructed upstream from Vasquez Boulevard bridge (fig. 253) to control the streambed elevation in the intermediate reach of channel. The dam was designed as a floating dam with gabions because bedrock was too deep to economically build a check dam with a cutoff wall.

1977 The steel sheet-pile check dam constructed downstream from the Quebec Street bridge (fig. 254) has stabilized the channel and is performing as intended.

Discussion: Construction of the bridge across Sand Creek, which was degrading, required consideration of the possible long-term effects of channel changes on the bridge foundations and embankment. The duel effects of flood flow and a degrading channel posed a design situation difficult to evaluate. As the ultimate amount of channel change on a degrading stream is difficult to estimate, special measures to control the amount of degradation in a limited reach (length) of channel were used. Check dams to prevent channel degradation have apparently been effective in protecting pier foundations.

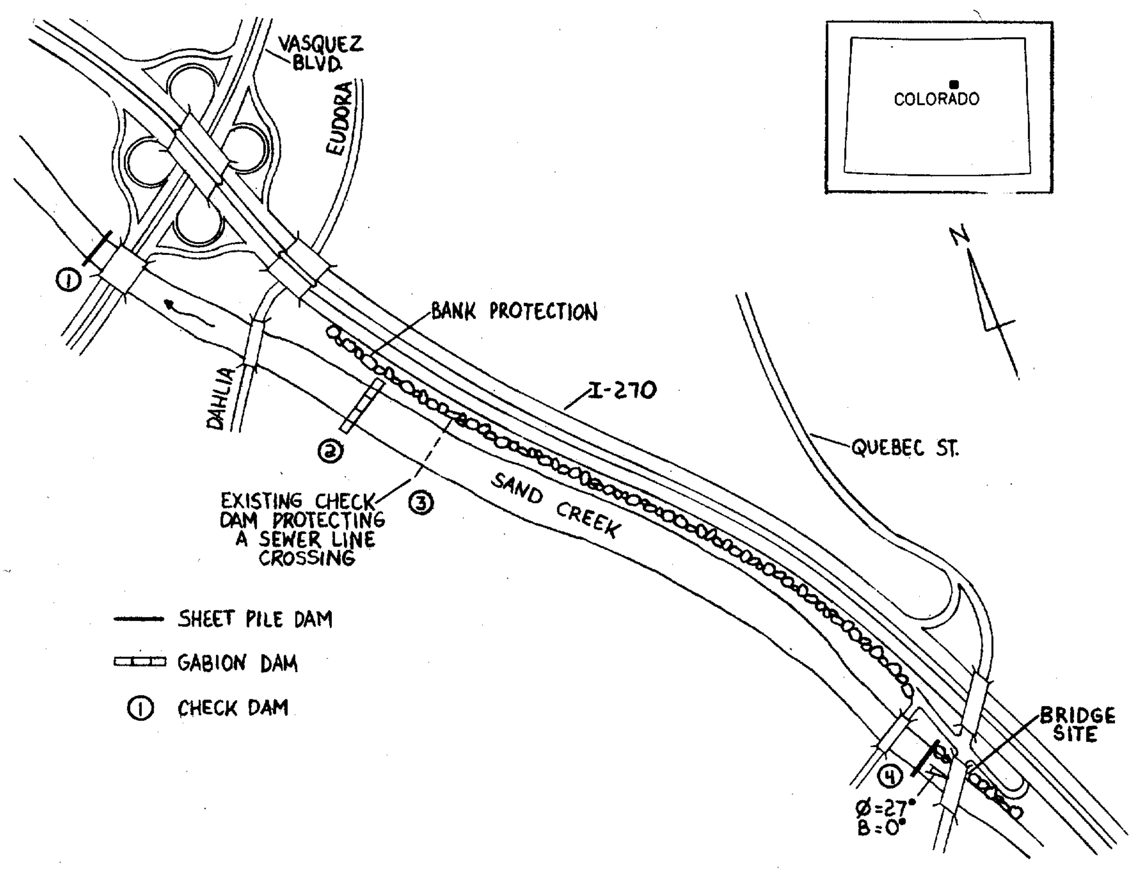


Figure 248. Location of check dams and bank stabilization works on Sand Creek in the vicinity of Quebec Street bridge, Denver, Colo.

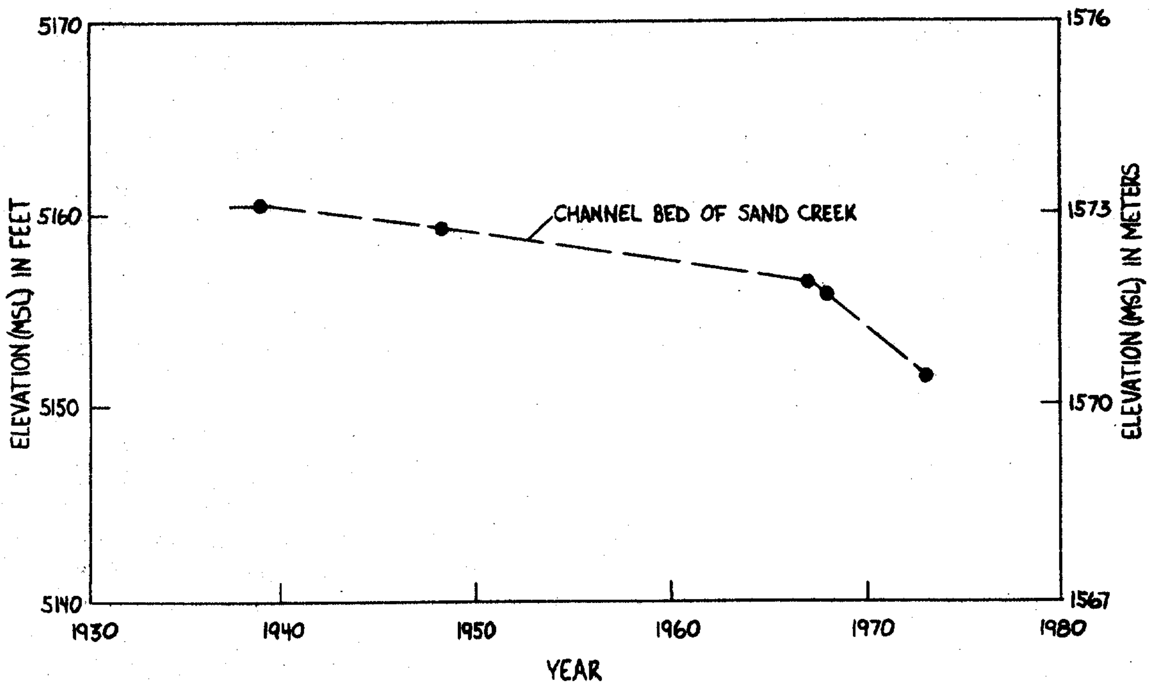


Figure 249. Rate of channel degradation of Sand Creek at Vasquez bridge between 1940 and 1973.

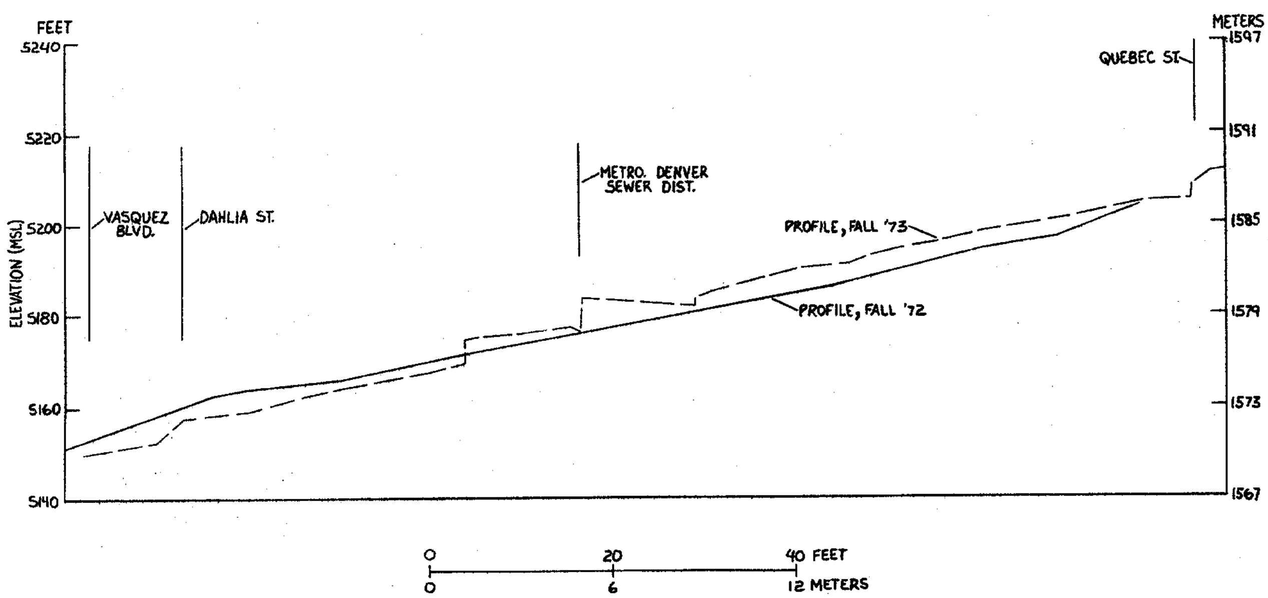


Figure 250. Streambed profile of Sand Creek in vicinity of Quebec Street bridge in 1972 and 1973.

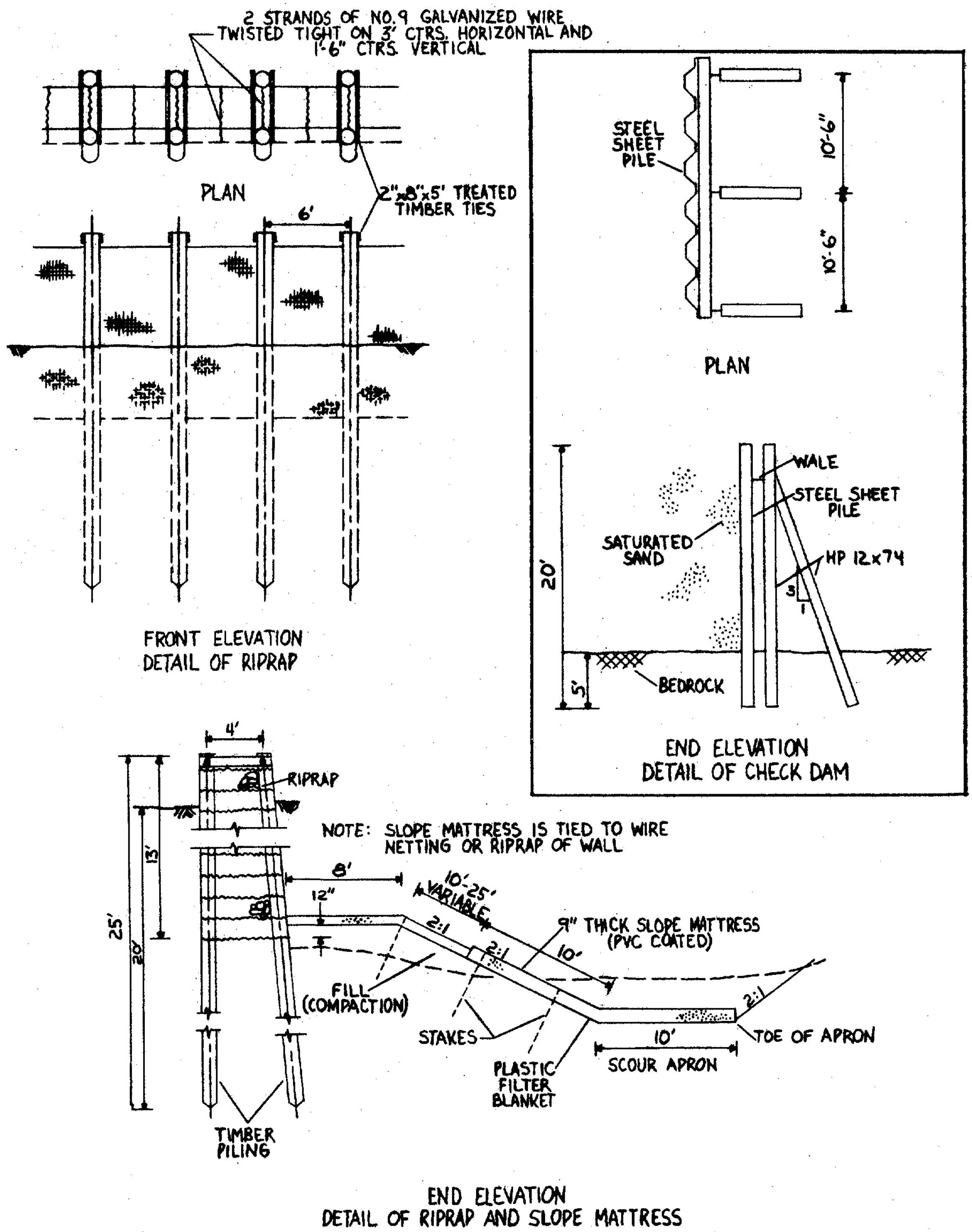


Figure 251. Details of bank protection and (inset) sheet-pile check dam.

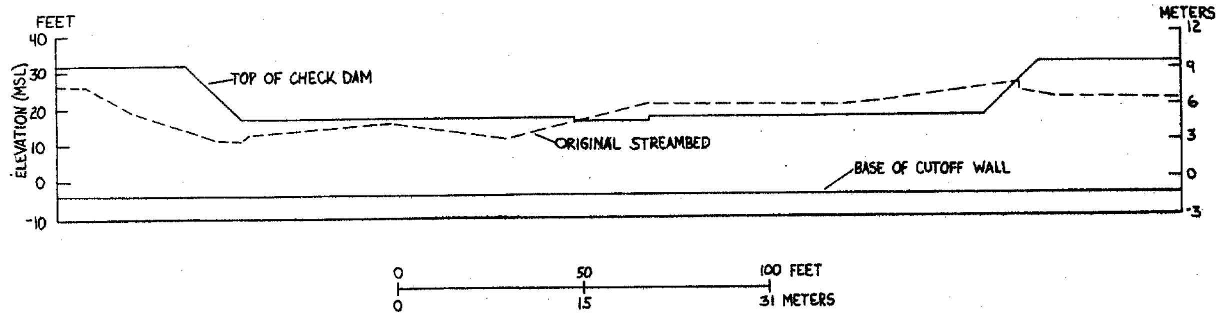
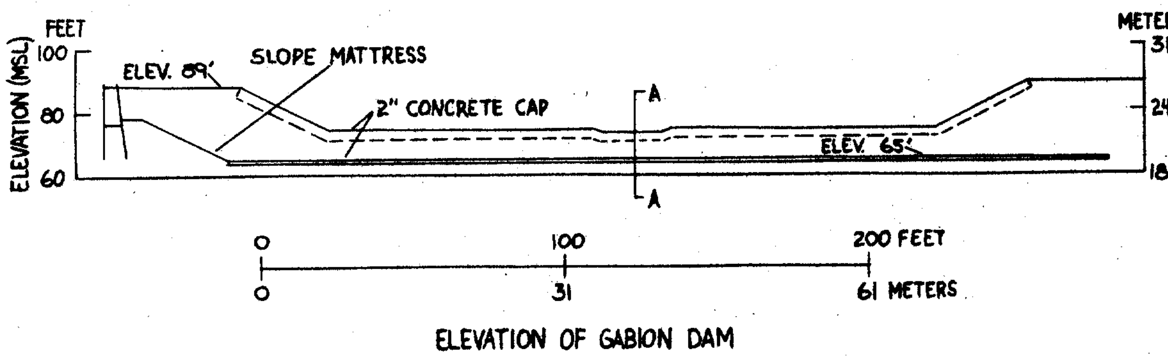


Figure 252. Elevation of check dam at Quebec Street bridge.



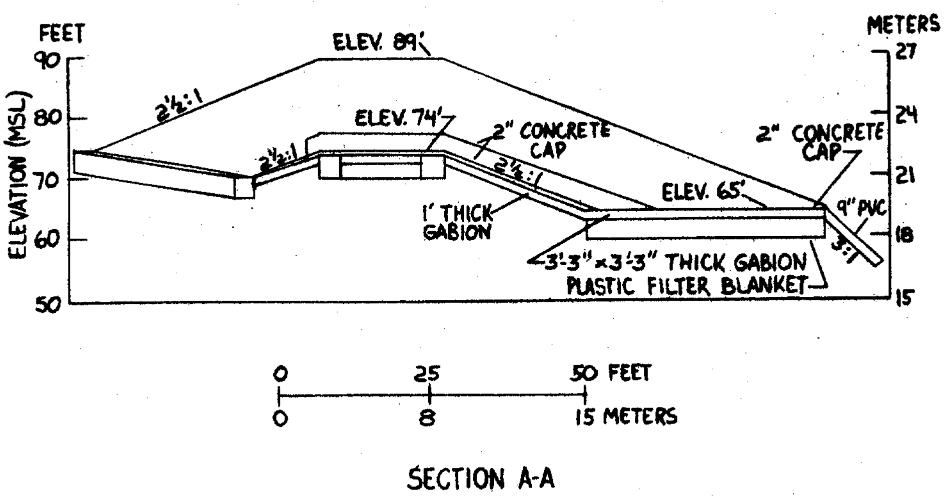


Figure 253. Details of gabion check dam.

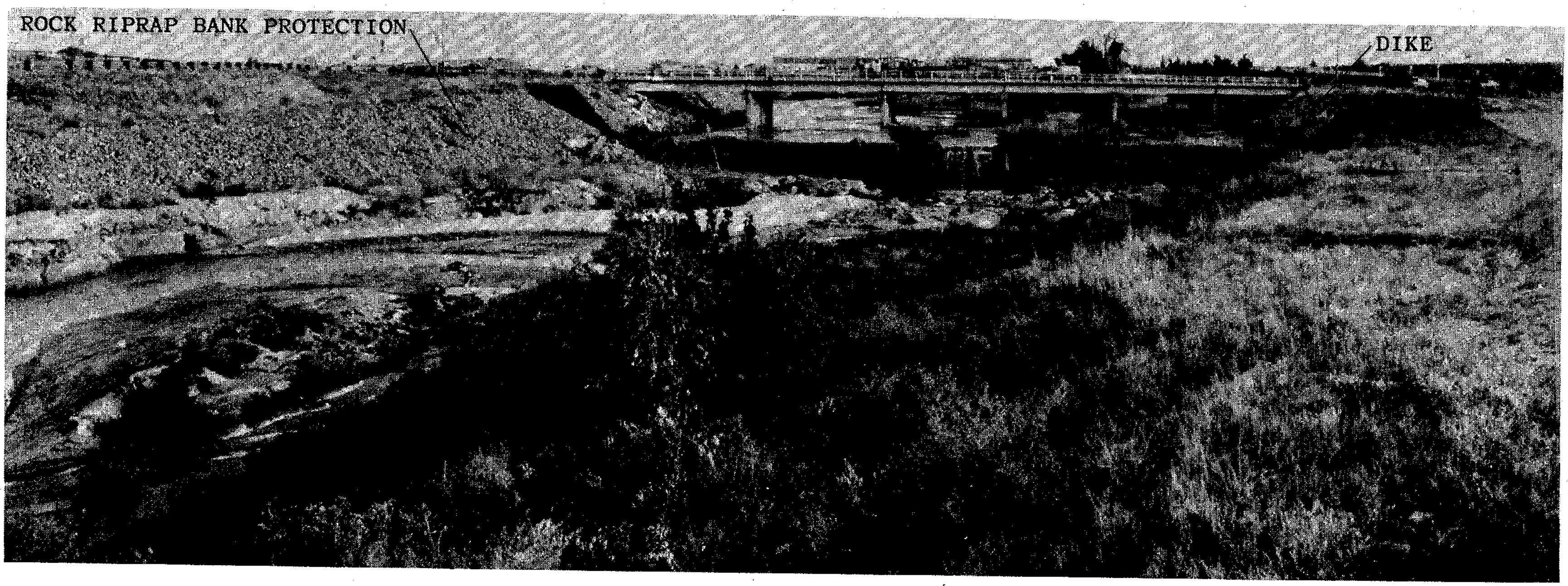


Figure 254. View upstream in 1977 of steel sheet-pile check dam, built in 1974, at Quebec Street bridge.

SITE 88. HENRYS FORK AT SR-88 NEAR REXBURG, IDAHO

Description of site: Lat 43°50', long 111°54, location as shown in fig. 255. Existing bridge is prestressed-concrete girder structure, 323 ft (98 m) in length, spillthrough abutments, three wall type piers with pointed nose and founded on piles driven in alluvial bed material. Bridge skew is 30 degrees, piers are alined with the flow.

Drainage area, 2,920 mi^2 (7,562 km^2); valley slope, 0.0002; channel width, about 250 ft (76 m). Stream is perennial, alluvial, sand bed, in valley of low relief, wide flood plain. Channel is meandering, locally braided, locally anabranched, cut banks rare, silt-sand banks.

Hydraulic problems and countermeasures:

1960 Concrete girder bridge built, 297 ft (91 m) long, supported by six bents of concrete-filled steel pipe-piles driven a minimum of 15 ft (4.7 m) below the alluvial streambed.

1976 Flooding from failure of Teton Dam upstream from the bridge overtopped the bridge deck by a depth of about 5 ft (1.5 m). After the flood crest, debris became caught on the individual piling of bents and built up at the upstream side of the bridge (fig. 256). Attempts to remove the debris were unsuccessful, and channel bed scour reduced the carrying capacity of the piles sufficiently to cause subsidence of bents 3, 4, and 5 and the bridge structure (fig. 257) about five days after the flood peak. The depth of scour is unknown but soundings obtained on June 18 indicate only about 5 ft (1.5 m) of scour.

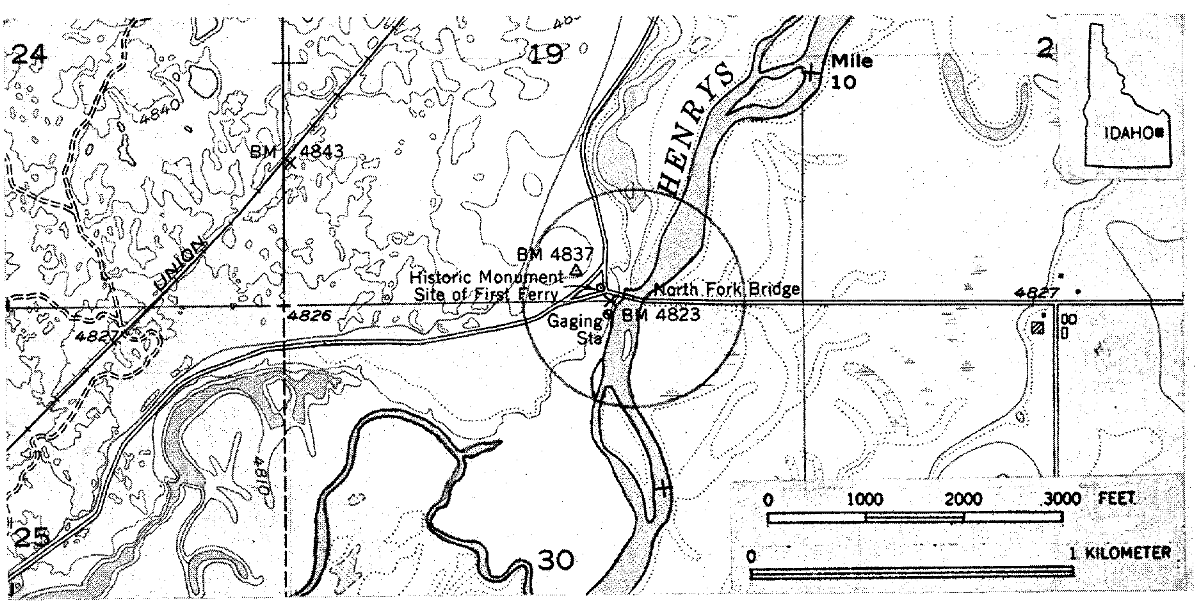


Figure 255. Location of SR-88 crossing (circled) at Henrys Fork. (From U.S. Geol. Survey Menan Buttes, Idaho, 7.5' quadrangle, contour interval 10 ft, 1951.)

Late 1976 — A new precast-concrete bridge, 26 ft (8 m) longer than the older bridge, three piers, was constructed. Channel surveys made for bridge design indicate that the lowest point in the streambed was 3 ft (0.9 m) lower than the channel bed surveyed on June 18, 1976. This indicates that the channel bed is unstable.

Discussion: The older bridge was overtopped by about 5 ft (1.5 m) during the June 1976 flood but was not damaged by lateral forces or local scour during the flood peak. Debris accumulation at the bridge during or after the flood set up severe local scour conditions that undermined the piling sufficiently to cause settlement of the pile bents and bridge. Settlement of the pile bents and the bridge occurred about five days after the flood peak. The pile bents were especially susceptible to lodgment of debris between individual piling, and the debris was difficult to remove.

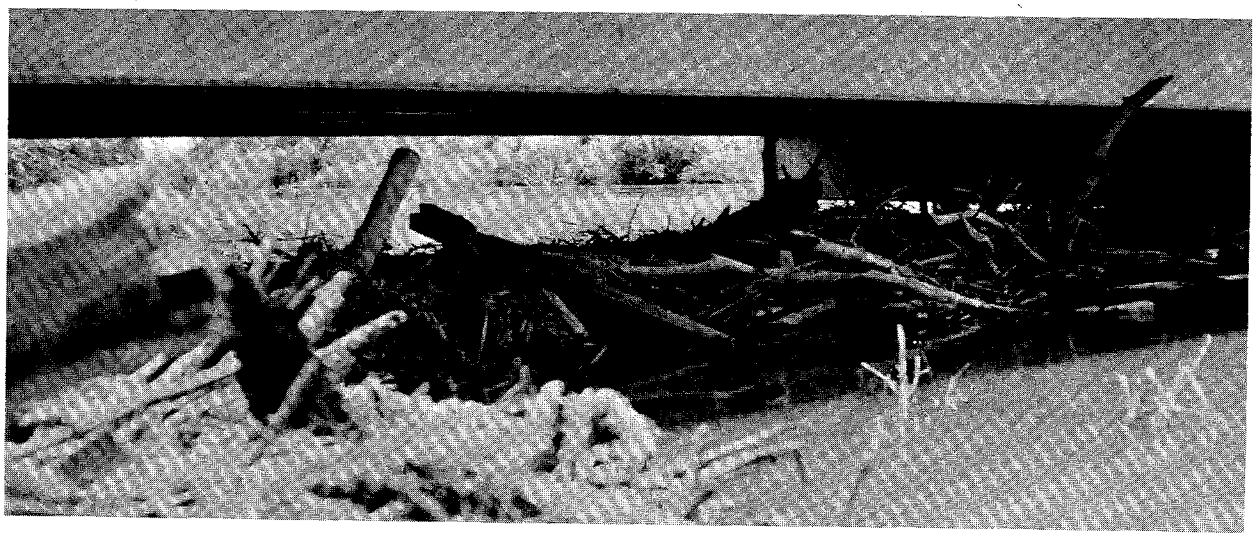


Figure 256. View of debris lodged under upstream side of SR-88 bridge following flood caused by failure of Teton Dam.

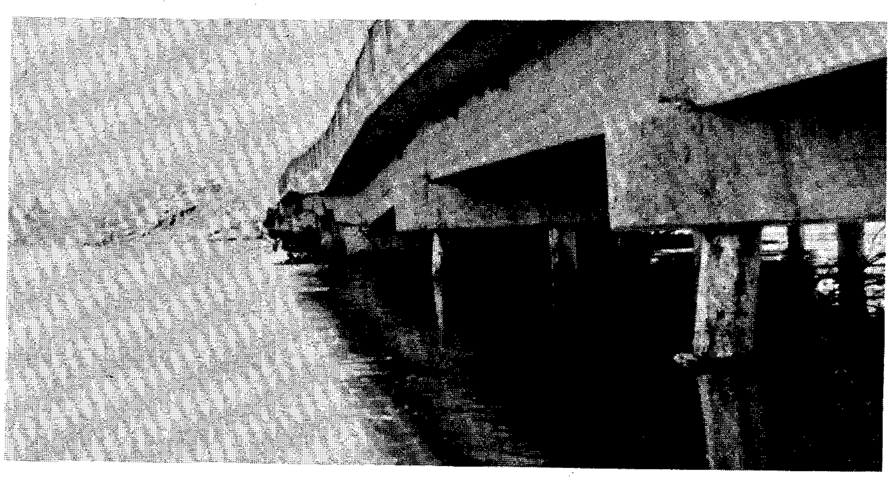


Figure 257. View of upstream side of bridge showing subsidence at bents 3, 4, and 5.

SITE 89. LAPWAI CREEK TRIBUTARY AT CAMAS PRAIRIE RAILROAD NEAR CULDESAC, IDAHO

Description of site: Lat 46°21', long 116°37', location as shown in fig. 258. Timber-trestle railroad bridge, 140 ft (42.7 m) in length, supported by timber-pile bents and founded on spread footings of concrete placed on basaltic bedrock. Skewness of bridge and piers to flow considered insignificant.

Valley slope, 0.45. Stream is ephemeral, semi-alluvial, gravel and boulder bed on basaltic bedrock in valley of moderate relief, no flood plain. Channel is straight, gravel and boulder banks.

Hydraulic problems and countermeasures:

1965 US-95, 180 ft (54.9 m) downstream from the Camas Prairie railroad bridge, was improved and a culvert 5-ft (1.5-m) in diameter placed to convey flows from a tributary to Lapwai Creek. The average slope of the channel between the culvert and railroad bridge is about 0.45 ft/ft. The stream has scoured the channel above the railroad trestle and carried bed material downstream to the highway culvert. Over a period of time, the scouring action exposed the bottom of the concrete footings at the railroad trestle. The highway department periodically removed the deposited bed material from the channel to prevent plugging of the culvert inlet.

1971 To prevent continued maintenance at the highway culvert and scour of the channel at the railroad bridge, a gabion check dam was constructed downstream from the railroad bridge, (fig. 259) and the channel behind the dam filled with rock. The gabion check dam was selected because it was more economical than other types of material used for check dams, and it provided better drainage of water behind the dam and less hydrostatic pressure. The gabion wall could be constructed in wet weather more easily than other types and a convenient source of excellent rock material was available nearby. Finally, other gabion type walls constructed in 1970 near the site were performing satisfactorily.

1977 Inspection of the gabion check dam and adjacent channel indicates the structure is performing satisfactorily. No flow records are available at the site to indicate the severity of flooding since construction of the dam. Some channel degradation between the check dam and railroad trestle has occurred. The base of the concrete railroad trestle footings are exposed but the present streambed level appears stable (fig. 261).

Discussion: Placement of the rock gabion check dam has effectively prevented further channel degradation and undermining of bridge piers upstream from the gabion.

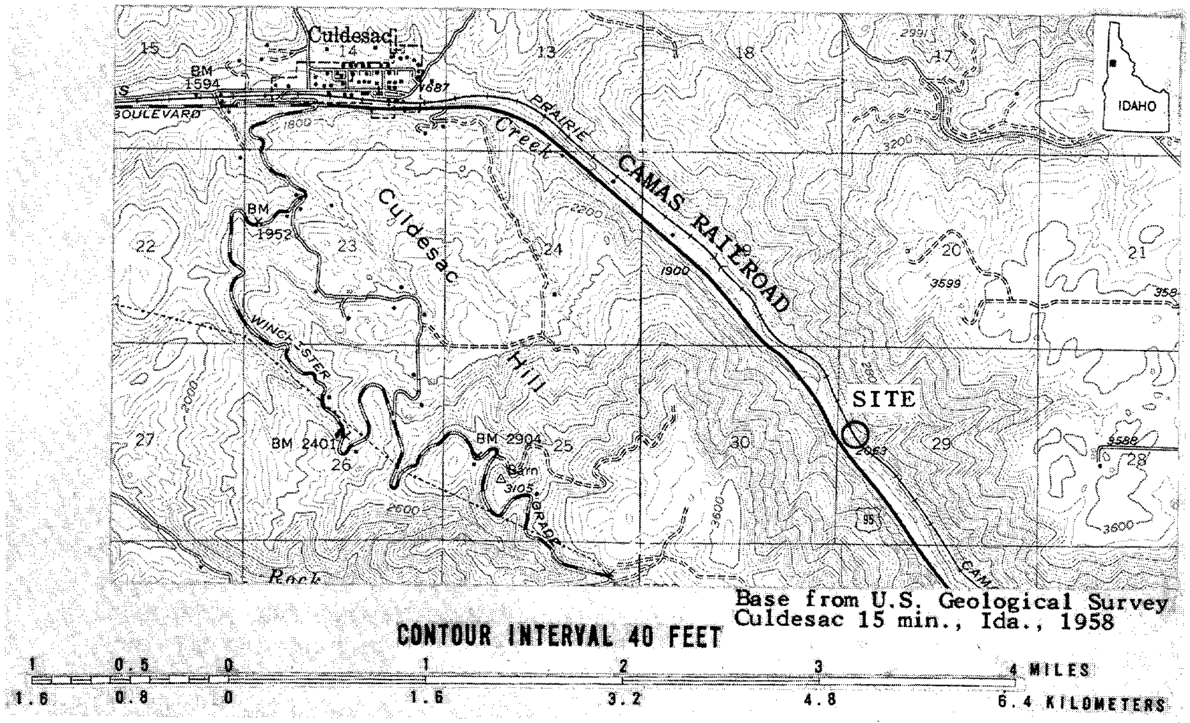


Figure 258. Location of Lapwai Creek tributary at Camas Prairie railroad crossing.

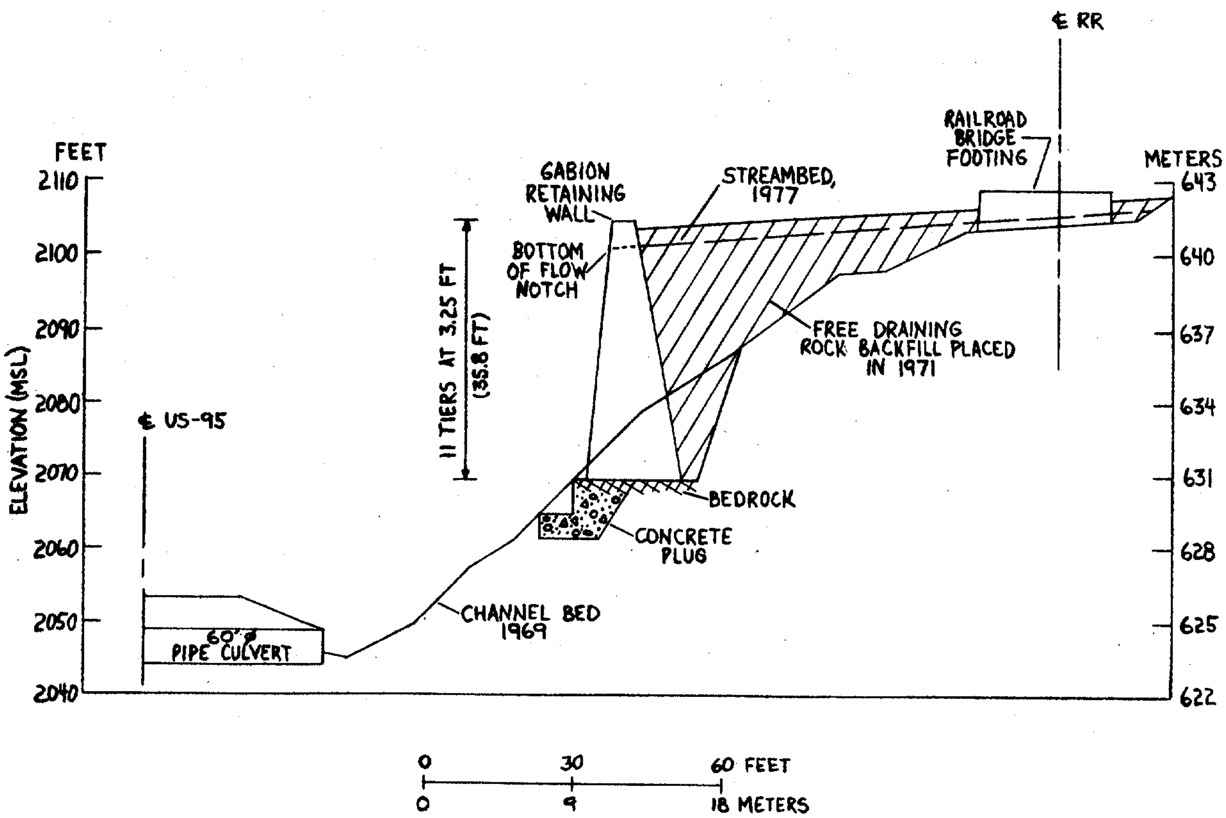


Figure 259. Profile along center of channel between railroad and highway after construction of check dam in 1971.

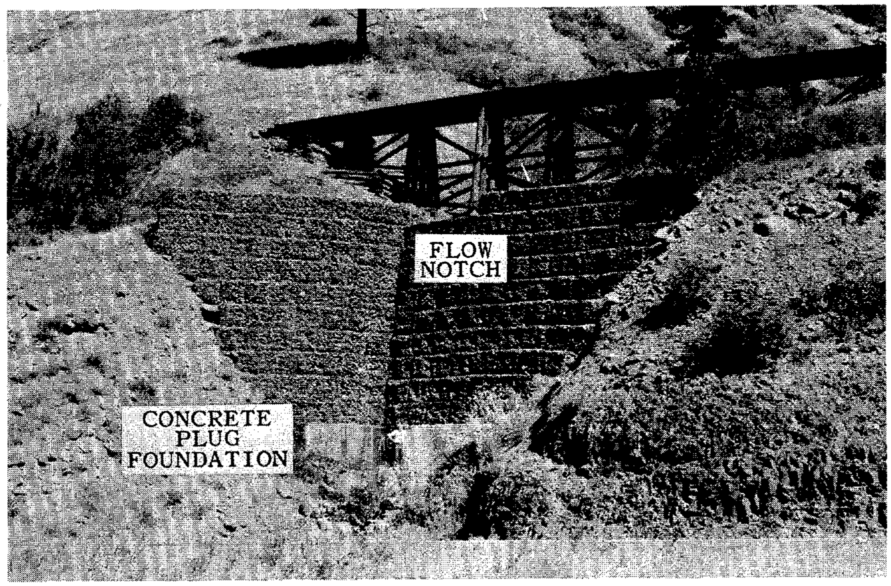


Figure 260. Gabion retaining wall in 1977, 6 years after construction.

Figure 261. Streambed material between gabion retaining wall and railroad trestle in 1977. Note growth of vegetation in streambed.

SITE 90. BOISE RIVER AT FAIRVIEW AVENUE AT BOISE, IDAHO

Description of site: Lat 43°37', long 116°14'. Concrete arch bridge, 382 ft (116 m) long, 5 spans. Piers are founded on spread footings. Bridge and piers skewed 30 degrees. Piers are wall type with pointed nose. Abutments are vertical (full-height) with zero degree angle wingwalls.

Drainage area, 2,760 mi^2 (7,150 km^2); valley slope, 0.0033; channel width, about 300 ft (90 m). Stream is perennial, regulated, gravel bed, in valley of moderate relief, wide flood plain. Channel is sinuous, locally braided, generally anabranched.

Hydraulic problems and countermeasures:

1933 Bridge built with base of pier footings at least 5 feet (1.5 m) below the streambed.

1943 Flood of April 20, 1943, discharge 21,000 ft^3/s (595 m^3/s), eroded streambed and pier footings. A check dam was constructed downstream from the bridge to prevent further general channel scour. The check dam was placed with the base 6 ft (2 m) below the streambed and the footing of pier 1 extended deeper.

1947-48 Gravel removal operations in the channel during the 1940's caused streambed degradation. In 1947-48, the check dam failed, but was soon repaired. The entire channel bed was regraded for a distance of about 600 ft (183 m) in the vicinity of the bridge to prevent abrupt changes in channel slope and subsequent scour.

1967 The check dam held satisfactorily until it was removed in 1976 as part of a project to widen the bridge.

Discussion: A rock riprap check dam effectively prevented channel degradation upstream from the bridge because the base of the wall extended below degraded channel bed levels. Use of rock riprap in the check dam construction contributed to good performance because the rock will tend to settle deeper as the channel degrades. This "self-healing" ability of rock riprap partly alleviates the need to estimate accurately the rate of degradation.

SITE 91. BOISE RIVER AT US-95 NEAR PARMA, IDAHO

Description of site: Lat 43°45', long 116°55'. Precast-concrete bridge, 425 ft (130 m) long, supported on steel-shell pile bents, driven at least 10 ft (3 m) below the channel bed, with concrete webs. Bridge skew is 45 degrees but pile bents are alined with flow. Abutments are spillthrough.

Drainage area, 3,970 mi^2 (10,282 km^2); bankfull discharge, about 10,000 ft^3/s (283 m^3/s); valley slope, 0.0025; channel width, about 200 ft (61 m). Stream is perennial, regulated, alluvial, gravel bed, wide flood plain. Channel is meandering, locally anabranched, tree cover at less than 50 percent of bankline.

Hydraulic problems and countermeasures:

1960-61 Bridge built across Boise River with a 45 degree skew angle. The channel was regraded and widened to improve the waterway. Rock riprap, covered with concrete, was placed on both banks upstream and downstream from the bridge to prevent lateral erosion. The bridge and regraded channel was designed for a flow of 20,600 ft^3/s (583 m^3/s), and a depth of 9.5 ft (2.9 m).

1976 Site inspection indicates the bank protection is failing on the right bank downstream from the bridge. Total failure of the concrete-and-riprap revetment occurred within 40 ft (12 m) of the bridge (fig. 262). Farther downstream, damage is less, probably because flow turbulence induced by the bridge constriction is less.

The maximum flow between 1960 and 1976 occurred on June 19, 1974, (discharge about 8,100 ft^3/s or 229 m^3/s), and was 61 percent less than the design discharge. The left-bank revetment probably was not damaged because the channel alinement near the bridge tends to force the main part of the stream towards the right bank.

Discussion: Factors involved in the failure of the concrete-covered riprap revetment are as follows: (1) The toe and top surface of the slope protection was inadequate to prevent leaching or undermining of supporting material on the embankment. (2) Flow turbulence induced by the bridge constriction, plus concentration of flow near the right bank caused sufficient erosive action to undermine the revetment. When failure occurred, the flat slabs were moved by action of the flowing water away from their original position, leaving the bank exposed.

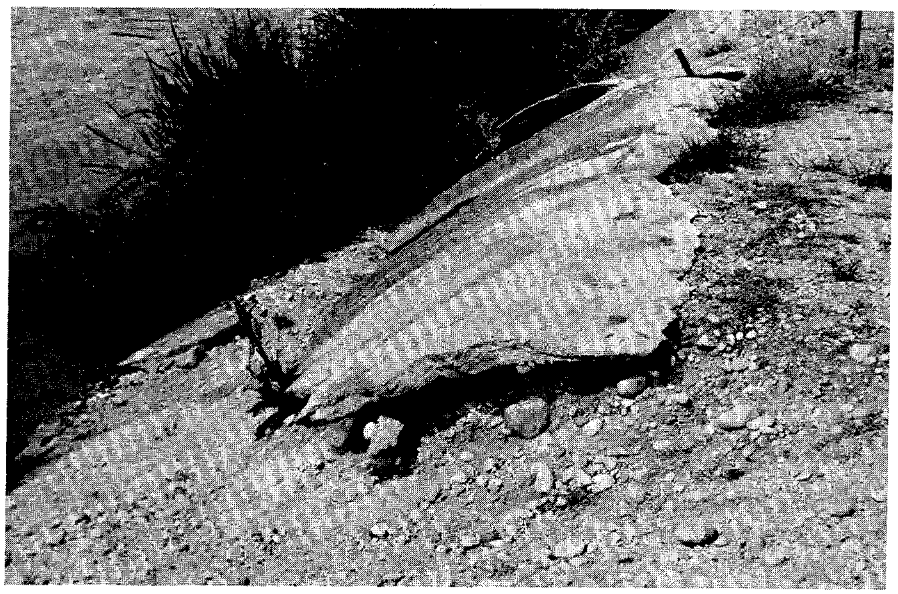


Figure 262. Damaged right-bank revetment, about 40 ft (1 m) downstream from bridge, in 1976.

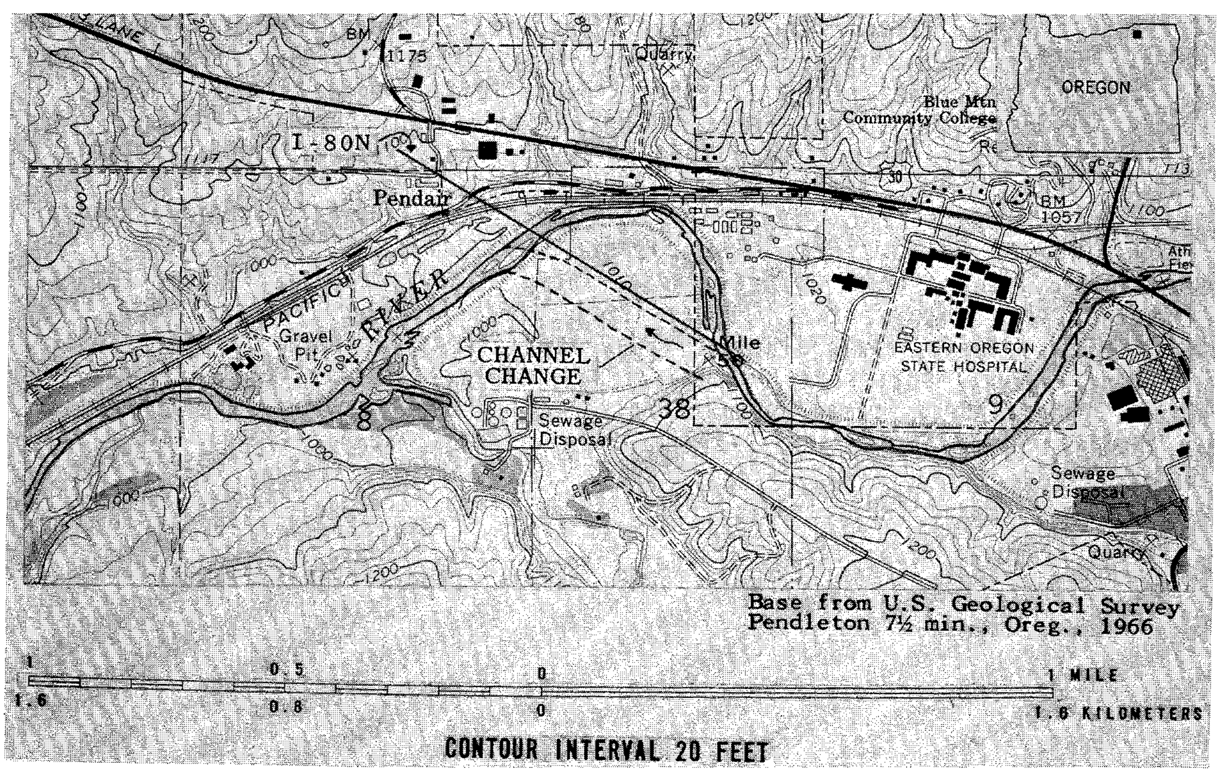


Figure 263. Location of Umatilla River channel relocation at I-80N.

SITE 92. UMATILLA RIVER AT I-80N AT PENDLETON, OREG.

Description of site: Lat 45°40', long 118°50', location as shown in fig. 263. Channel at site has been relocated to avoid two crossings, and there is no bridge.

Drainage area, 637 mi^2 (1,650 km^2); valley slope, 0.0052; channel width, about 200 ft (61 m). Stream is perennial, alluvial, gravel bed, in valley of moderate relief, narrow flood plain. Channel is meandering, locally braided, random width variation.

Hydraulic problems and countermeasures:

1969 As part of the I-80 N construction, the Umatilla River at Pendleton was relocated. The channel change increased the natural streambed slope from about 0.00392 ft/ft to an average of 0.00457 ft/ft. To prevent scour or lateral erosion problems, several measures were taken at the time of construction: (1) Eleven spurs, about 3 ft higher than the streambed but designed to be overtopped during floods, were constructed (fig. 264), (2) a dike faced with rock riprap was constructed along the left channel bank, and (3) the rock riprap revetment at both banks was placed with the toe about 5 ft (1.5 m) below the streambed.

1973 Aerial photographs of the site show that the channel has remained stable since construction and gravel bars are building up in protected parts of the channel between the spurs (fig. 265). Between 1969 and 1976, effectiveness of the spurs and bank protection was tested by several floods. The largest flood occurred in 1975 (R. I. of about 15 yr) and slightly smaller floods occurred in 1970 (R. I. of about 9 yr) and in 1972 (R. I., about 6 yr).

1977 The measures taken to stabilize the relocation channel are considered to be effective, and no maintenance has been required.

Discussion: Spurs oriented in a downstream direction and designed to be overtopped during floods, have effectively stabilized the relocated channel during floods with R. I. up to 15 years. Rock riprap revetment, with the toe placed below the streambed to prevent undermining, has probably prevented lateral erosion problems associated with control of flow alinement by the spurs.

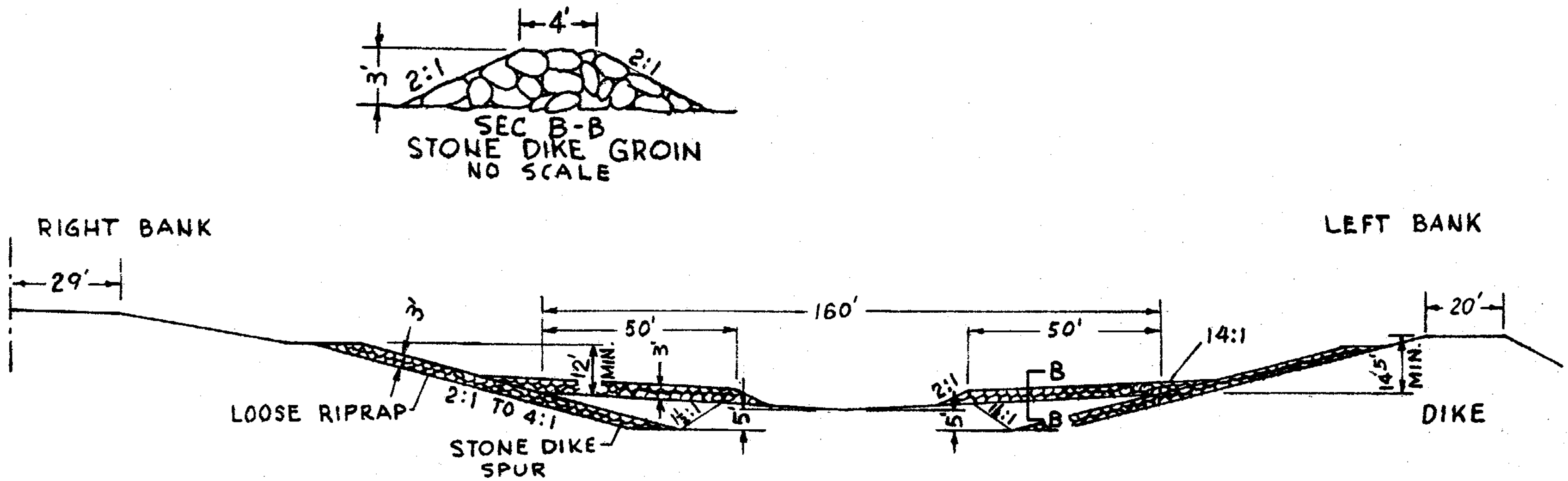


Figure 264. Details of bank protection and spur construction.

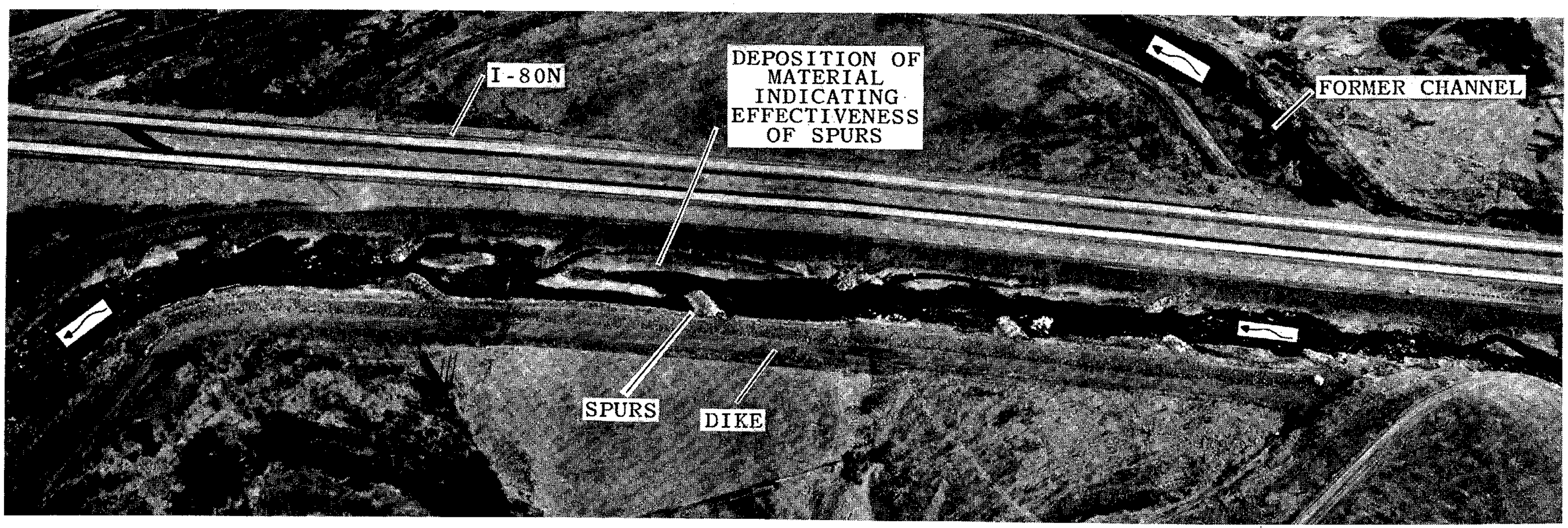


Figure 265. Aerial view of channel change in 1973, four years after construction. (From Oregon Dept. of Highways.)

SITE 95. LAPWAI CREEK AT US-95 NEAR CULDESAC, IDAHO

Description of site: Lat 46°22', long 116°38', location as shown in fig. 266. Concrete bridge 103 ft (31 m) in length, supported on two concrete wall-type piers with rounded nose, and founded on spread footings. Abutments are spillthrough type. Bridge skewed 42 degrees, but piers are alined with flow.

Valley slope, 0.0063. Stream is perennial, semi-alluvial, gravel and cobble bed, in valley of high relief, narrow flood plain. Channel is straight, gravel and cobble banks.

Hydraulic problems and countermeasures:

1965 Flood of January 29 (R. I. about 10 yr) on Lapwai Creek washed out the highway embankment at many locations. Countermeasures consisted of protecting the embankment by use of loose rock riprap and concrete-grouted rock riprap (fig. 267).

1966-77 Two floods, each of about 5-yr R. I., have not damaged the countermeasures (fig. 268). The stream carries large amounts of suspended material and abrasion damage to the upstream nose of the right bank pier is evident (fig. 269).

Discussion: The road embankment and bridge on the relocated channel were damaged by floods of 10-yr R. I. because the bank protection was not sufficient, especially on the outside bank of channel bends. Rock riprap and concrete-grouted rock riprap, placed with the toe several feet below the streambed, provided protection for the embankment and bridge abutment during two floods of about 5-yr R. I.

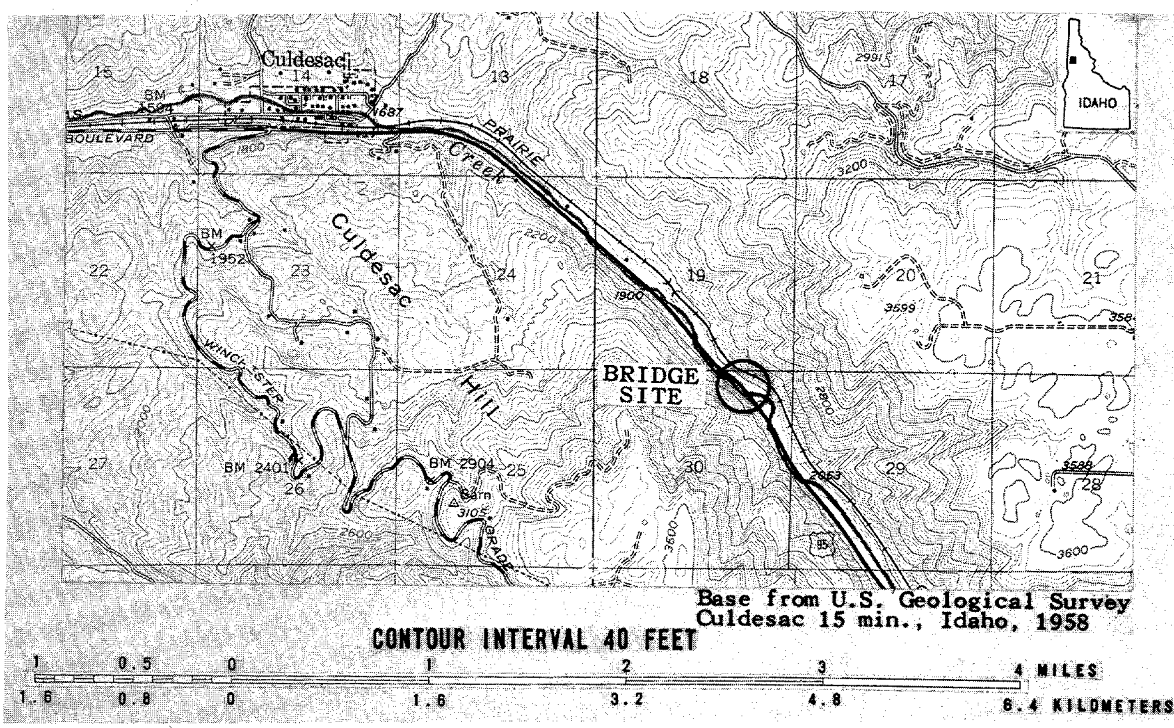


Figure 266. Location of Lapwai Creek at US-95 near Culdesac, Idaho.

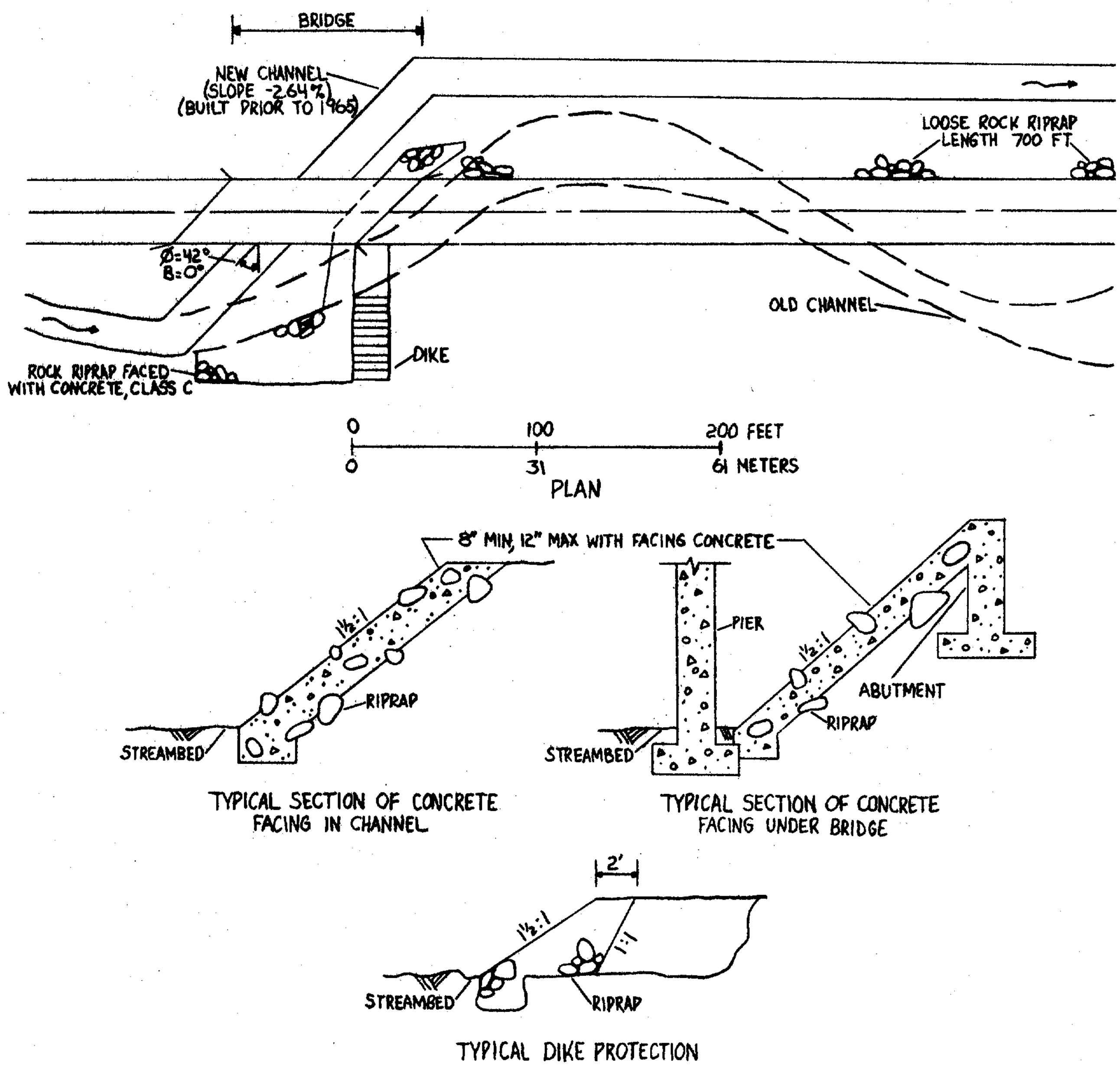


Figure 267. Details of bank protection constructed in 1965 on Lapwai Creek.

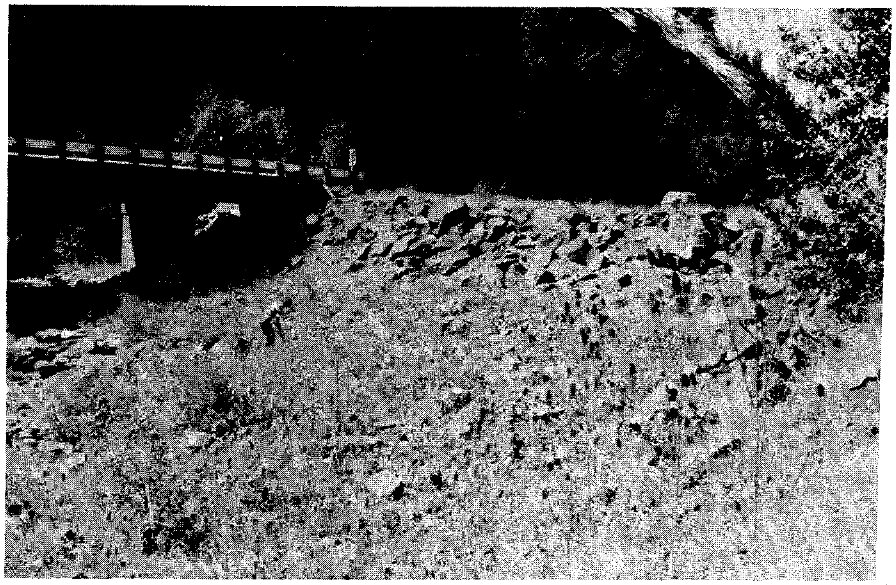


Figure 268. Condition of rock riprap placed in 1965 on right bank upstream from Lapwai Creek bridge, as photographed in 1977.

Figure 269. Abrasion damage on upstream nose of pier at right bank, Lapwai Creek, 1977.

SITE 97. LITTLE SALMON RIVER AT US-95 NEAR NEW MEADOWS, IDAHO

Description of site: Lat 45°58', long 116°18'. Steel girder bridge, 168 ft (51.2 m) in length, single span, with spillthrough abutments supported on steel piles.

Valley slope, 0.004. Stream is perennial, semi-alluvial, gravel-boulder bed, in valley of moderate relief, wide flood plain. Channel is sinuous, gravel banks.

Hydraulic problems and countermeasures:

1923 Bridge built, without countermeasures.

1944 Bridge was damaged by flooding and length was increased from 123 ft (37.5 m) to 138 ft (42 m) in order to increase the flow capacity. Heavy rock riprap was placed around the left-bank abutment.

1955 Ice jam caused water to overtop bridge deck, but bridge was not damaged.

1975 New bridge built to improve traffic capability. Design of new bridge based on previous hydraulic problems at the site as follows: (1) Bridge length was increased from 138 ft (42 m) to 168 ft (51.2 m). (2) To reduce possibility of debris or ice accumulation, bridge was built without piers. (3) Gabion revetment was provided at both abutments and at highway embankment (fig. 270).

1977 No significant flood has occurred since construction of the new bridge and gabions.

Discussion: The roadway embankment and bridge abutments were located at a bend in the stream and subject to lateral erosion. Extensive revetment protection was used to prevent roadway and bridge damage during floods. The bridge crossing has a large angle of skew. The absence of piers reduces the possibility of debris or ice accumulation at the bridge.

Figure 270. View of upstream right bridge abutment and embankment showing gabion revetment. (From Idaho Dept. of Transportation, 1975.)

SITE 99. PINE CREEK AT I-90 AT PINEHURST, IDAHO

Description of site: Lat 47°33', long 116°13'. Prestressed-concrete girder bridge, 105 ft (32.0 m) long, with 3 spans of 35 ft (10.7 m). Bridge is supported by two steel-pile bents and spillthrough abutments. After the flood of January 1977, a new bridge was built with a length of 125 ft (38 m) and only one concrete pier.

Valley slope, 0.006. Stream is perennial, alluvial, gravel bed, in valley of moderate relief, narrow flood plain. Channel is sinuous, locally braided.

Hydraulic problems and countermeasures:

1964 Construction of I-90 in the vicinity of Pinehurst required changes in the alinement of Pine Creek and construction of two access ramp bridges (fig. 271) across the creek. Countermeasures applied at the site at the time of construction included placement of rock riprap along the channel banks, and design of the off-ramp bridge A-B for clearance of about 1 ft (0.3 m) above the historical high water of 1933.

1974 Flood of January, R. I. about 100 yr, caused several hydraulic problems at the site: (1) The abrupt channel alinement change downstream of the main street bridge (fig. 271) washed out the highway embankment. (2) A restricted channel size and a sharp channel bend upstream from the ramp A-B bridge caused overtopping of the right-bank levee and inundation of the right-bank flood plain. (3) An inadequate waterway area under the ramp A-B bridge caused overtopping and subsequent failure of the right-bank riprap revetment near the bridge.

1975 Proposals to repair the damaged road, and prevent inundation of the flood plain required consideration of several factors: (1) Increasing the channel gradient by lowering the streambed to improve flow capacity may cause undesirable scour or deposition problems in the new channel or at the point of tangency between the old and new channels. (2) Prevention of scour in the new steeper channel could be accomplished by use of a concrete-lined channel, but this would produce undesirable water-surface superelevations at the curves and excessive velocities. (3) Additional bank protection and diking was needed to prevent inundation of the frontage road. (4) A new ramp A-B bridge design was needed to increase the clearance above the highwater level and provide a greater waterway capacity. Raising the bridge to provide the additional height was limited by existing freeway and frontage road grades and a desire to keep the ramp gradient to a minimum.

1977 Countermeasures constructed at the site include: (1) Realining the channel (fig. 271) to reduce the number of sharp bends. (2) Using bank revetment throughout the newly alined channel. Material consisted of grouted rock riprap, rock riprap, and a concrete retaining wall. (3) Constructing a new ramp A-B bridge with shallower beams to give greater clearance increasing the bridge length from 105 ft (32 m) to 125 ft (38.1 m), and using one support pier at midspan instead of two intermediate pile bents. The channel bed elevation at the ramp A-B bridge was unchanged. No floods have occurred to test the countermeasures.

Discussion: Channel relocation and elimination of the flood plain as an overflow area, caused lateral erosion of the roadway embankment and overtopping of dikes when the January 1974 flood exceeded the capacity of the new channel. An additional factor causing hydraulic problems at the site was the capacity of the waterway under ramp A-B bridge, which was less than the capacity of the unobstructed channel adjacent to the bridge. Because of highway grade limitations, a special design for the ramp A-B bridge, using shallow stringer depths and fewer piers, was required to provide adequate clearance and channel capacity for the design flood.

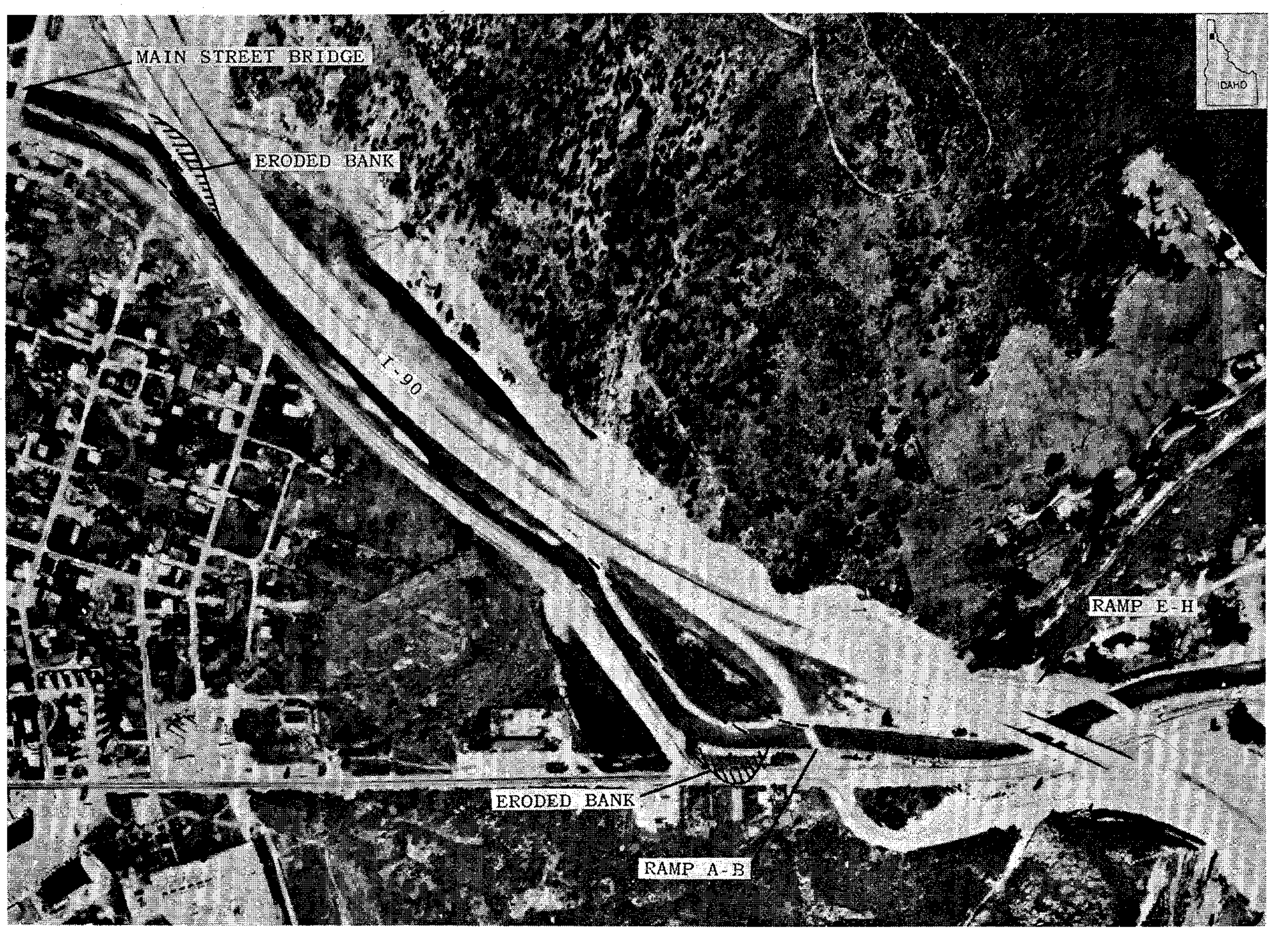


Figure 271. Aerial view of Pine Creek channel in 1971, and proposed relocations. (From Idaho Dept. of Transportation.)

SITE 100. MIDDLE PATUXENT RIVER AT CARROLL MILL ROAD NEAR MAYFIELD, MD.

Description of site: Lat 39°16, long 76°56'. Two adjacent concrete box culverts with spans of 25 ft (8 m) each and a 740-ft (226-m) length of road embankment designed for overflow during severe floods. Culvert abutments are vertical with 45 degree wingwalls to guide flow through the openings. In the future, six additional culverts will be added to increase the flow capacity of the structure.

Drainage area, 21 mi^2 (54 km^2); valley slope, 0.0042; channel width, about 50 ft (15 m). Stream is perennial but flashy, alluvial, in valley of low relief, wide flood plain. Channel is nearly straight.

Hydraulic Problems and countermeasures:

Much of the following is from Johnson (1976):

1972 Flooding during Hurricane Agnes in June 1972 destroyed the original bridge. The old bridge had a span of 34 ft (10 m) and a clear height above the waterway of about 6 ft (2 m) with an estimated hydraulic capacity of 2,000 ft^3/s (57 m^3/s).

1975 Twin-cell concrete box culvert (combined width 50 ft or 15 m) built with maximum flow capacity of 3,050 ft^3/s or 86 m^3/s). Estimated flows for a 100-yr R.I. flood are 7,080 ft^3/s (201 m^3/s). Flows in excess of the culvert capacity pass over the road embankment which was designed as an overflow structure (fig. 272). Countermeasures constructed at the site include: (1) Placement of gabion revetment on the downstream side of embankment to prevent erosion as water drops over the roadway to its natural level on the flood plain (fig. 272). (2) Design of the road profile as a broad-crested weir without a low dip (the level length of roadway is 560 ft or 171 m), to prevent flooding of the road until the full hydraulic capacity of the culvert is reached. In this way, periods of road overflow will have the least effect on traffic because flow depths are minimized. (3) Placement of rock riprap around the culvert abutments and along the channel banks to prevent lateral erosion as flows are constricted through the openings (fig. 273).

1977 Since the project was completed, one flood has occurred which exceeded the capacity of the culvert, and excess flows passed over the road embankment. The road embankment and culvert performed satisfactorily, and with minimum traffic disruption.

Discussion: Use of road embankments as a broad-crested overflow section is most effective if the depth of overflow at any point on the embankment is minimized--this means that the embankment profile should be level. Road embankments used as overflow sections should be protected on the downstream shoulder to prevent erosion. An apron extension of the gabion revetment 6 ft (2 m) in width is used to prevent scour of the flood plain, where flows over the road return to subcritical velocity and depth.

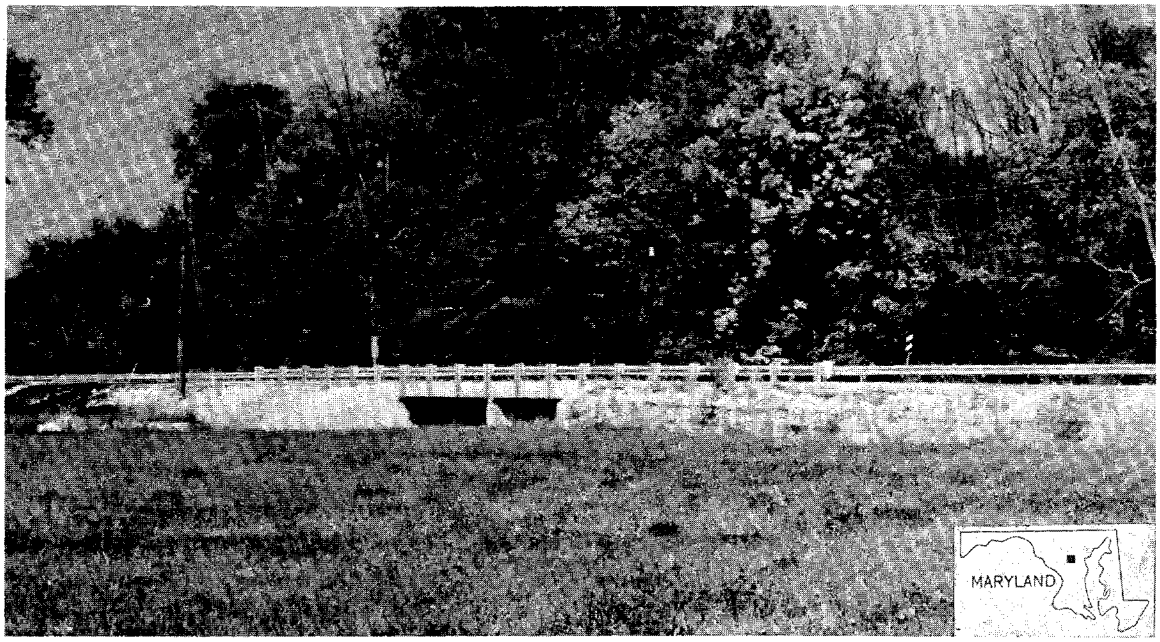


Figure 272. View of downstream side of approach embankment designed as an overflow structure. Note gabions along embankment for protection against scour when flows overtop roadway.

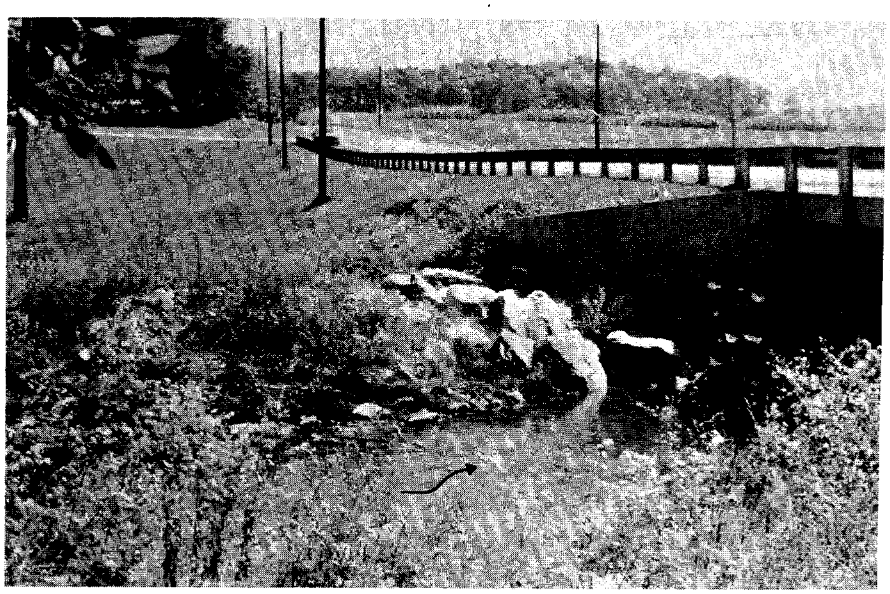


Figure 273. View of rock riprap placed on bank upstream from box culvert entrance to prevent lateral erosion when flows on the left-bank flood plain enter the main channel.

SITE 101. MUSSELSHELL RIVER AT SR-200 AT MOSBY, MONT.

Description of site: Lat. 47°00', long 107°53', location as shown in fig. 274. Prestressed-concrete bridge, length 374 ft (114 m), two spillthrough abutments founded on steel H-pile, three round-nose concrete wall piers with spread footings in hard, dark gray shale.

Drainage area, 7,846 mi^2 (20,306 km^2); valley slope, 0.0021; channel width, about 200 ft (60 m). Stream is perennial, alluvial, sand-silt bed, in valley of moderate relief, wide flood plain. Channel is meandering, wider at bends, point bars, tree cover at less than 50 percent of bankline (fig. 275).

Hydraulic problems and countermeasures:

1929 Steel truss bridge built, 436 ft (133 m) in length, with spillthrough abutments and round nose, wall-type piers.

1944 Maximum flood of record, discharge 18,000 ft^3/s (510 m^3/s, R.I. of 26 yr.

1971 Flood having peak discharge of 11,600 ft^3/s (329 m^3/s created no problems at the bridge.

1972 New bridge built, designed for 50-yr R.I. flood of 22,400 ft^3/s (634 m^3/s). The main channel was relocated through a previous overflow channel, which shortened the bend upstream of the bridge and increased the channel slope. The new channel banks and bridge abutments were lined with rock riprap, and a spur dike was constructed on the right bank upstream side of the new bridge, (fig. 276).

1975-77 Flood on May 7 discharge 11,600 ft^3/s (314 m^3/s), R.I. about 9 yr, caused no damage to the bridge. The effectiveness of the new channel in conveying flood flows efficiently was determined by measurements made on May 7, 1977 (table 11). During the measurement, flows were at bankfull stage (assumed to be elev. 2505 ft). Although some channel scour or lateral erosion normally occurs when channels are realined and straightened, flood stages and velocities were less than anticipated. In addition, the channel bed at the bridge aggradated between 1972 and 1975.

Discussion: Relocation of the channel, protection of the channel banks and bridge abutments with rock riprap, and construction of a spur dike improved the efficiency of the bridge opening as compared with the old bridge. Data obtained during a flood (R.I. about 9 yr) indicated that flow conditions in the old and new channels were within 11 percent of each other. Although some channel scour or lateral erosion usually occurs when channels are shortened, flood stages and velocities for the old and new channels were very similar and problems of scour and lateral erosion did not occur. These data indicate that changes in a channel may be made without causing hydraulic problems if flow conditions are not significantly altered.

Table 11. Measured and estimated values of flow factors at the old and the new bridge

Location	Discharge [1](ft³/s)	Water Surface Elev. (ft)	Area of Channel [2](ft²)	Mean Velocity [3](ft/s)
Old bridge (estimated values)	11,100	2505.5	2700	4.1
New bridge (estimated values)	11,100	2506.0	2410	4.7
New bridge (values measured 5-7-77)	11,100	2505.65	2960	3.8

[1]Conversion factor, ft^3/s to m^3/s, is 0.0283
[2]Conversion factor, ft^2 to m^2, is 0.093
[3]Conversion factor, ft/s to m/s, is 0.305

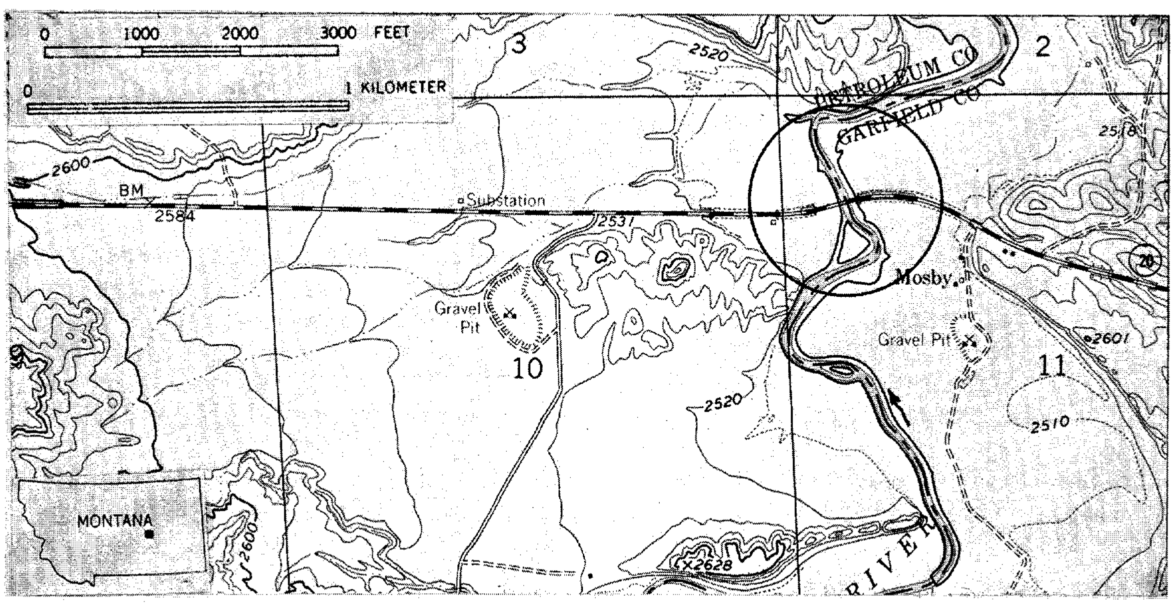


Figure 274. Location of Musselshell River at SR-200 (circled) prior to construction of new bridge. (From U.S. Geological Survey Mosby, Mont., 7.5' map, contour interval 20 feet, 1962.)

Figure 275. Aerial photograph in 1968 of Musselshell River at SR-200, prior to construction of new bridge. (From U.S. Dept of Agriculture.)

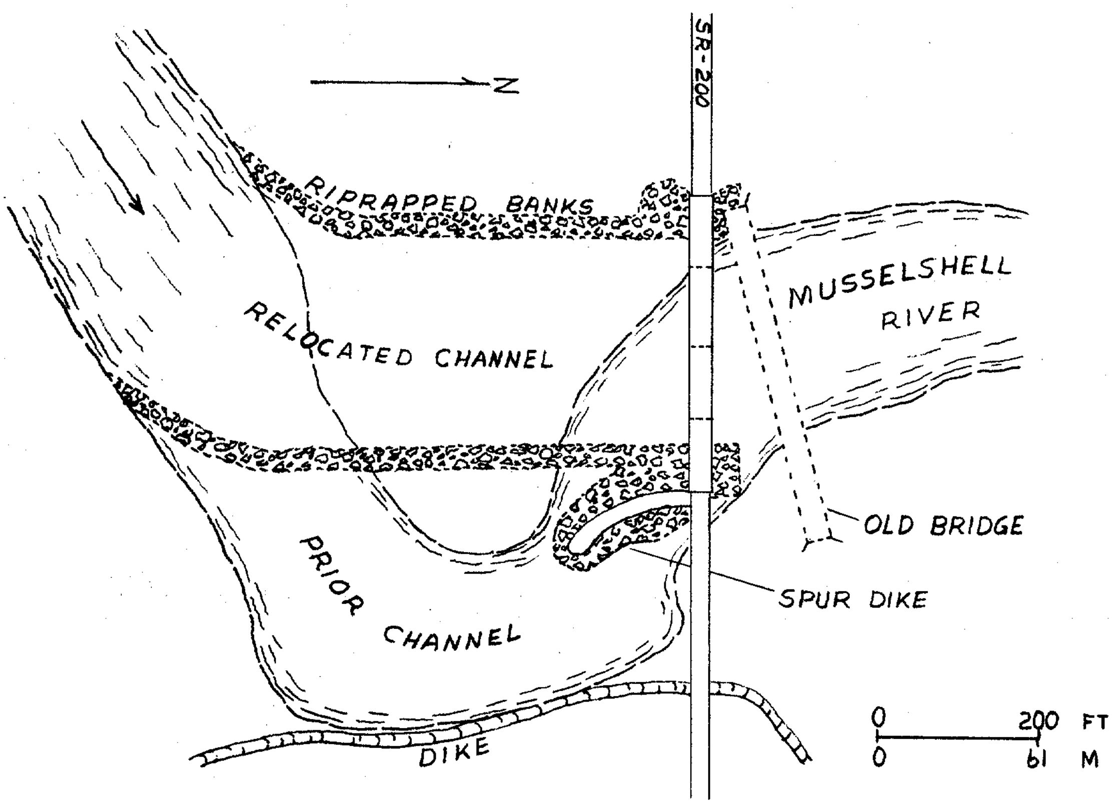


Figure 276. Plan of channel relocation and countermeasures, Musselshell River at SR-200.

SITE 103. RIO GRANDE AT US-380 AT SAN ANTONIO, N. MEX.

Description of site: Lat. 33°55', long. 106°51', location as shown in fig. 277. Timber bridge, 1,335 ft (407 m) in length, 41 timber-pile bents, two steel-and-concrete pile bents at 31-ft (9-m) centers. Abutments are spillthrough protected by wire-fabric retards.

Drainage area, 24,500 mi^2 (63,455 km^2); valley slope, 0.0008; natural channel width, about 1,750 ft (534 m). Stream is perennial, regulated, alluvial, sand bed, in valley of high relief, narrow flood plain. Channel is sinuous, generally braided, wandering thalweg.

Hydraulic problems and countermeasures:

1938 Timber bridge built; each abutment was protected by a wire-fabric retard and a blanket of light rock riprap encased in wire fabric.

1941 Flood, R.I. about 8 yr, caused by rainfall and snowmelt. Unpublished notes from U.S. Geological Survey files describe conditions at the bridge site as follows:

> "About 9 p.m. on May 16, 1941, the lowest pile in one of the bents was swinging loose which showed that all of the materials around it was completely removed by erosion. The discharge at the time of failure was about 24,000 ft^3/s and an estimate made before the failure indicated about 18,000 ft^3/s were flowing in a part of the channel with a width of 75 ft, and the thread of this particular channel was against one of the pile bents of the bridge. Many piles that were 25 feet long were eroded along the edge of the river by lateral erosion. About noon on May 17, two spans of the bridge that had been constructed in 1938 were washed out. Measurements made shortly after the failure showed a depth of water of 45 feet. The scouring of the stream channel must have extended about 40 feet below normal stream bed."

The retard constructed around each abutment was apparently effective because there was no abutment damage. The bridge was repaired late in 1941.

1949 Bridge was replaced and a wire-fabric mattress, held in place with steel railroad rails, was built to prevent bed scour in vicinity of bridge.

1973 Upstream diversions and regulation, in addition to levees built along channel confine the stream to a main channel and overflow channel (fig. 277). The largest flood recorded during the period 1950-64 at the Rio Grande at San Marcial gage (19 miles downstream) was 9,570 ft^3/s (371 m^3/s).

Discussion: Light riprap encased in wire fabric prevented damage to abutment and embankment during several floods between 1938 and 1964. Local scour depths of about 40 feet (12 m) below normal streambed level were observed during a flood with a recurrence interval of about 8 years. A wire fabric mattress held in place with steel piling and placed 80 ft (24 m) downstream from the bridge has apparently prevented excessive bed scour at bridge.

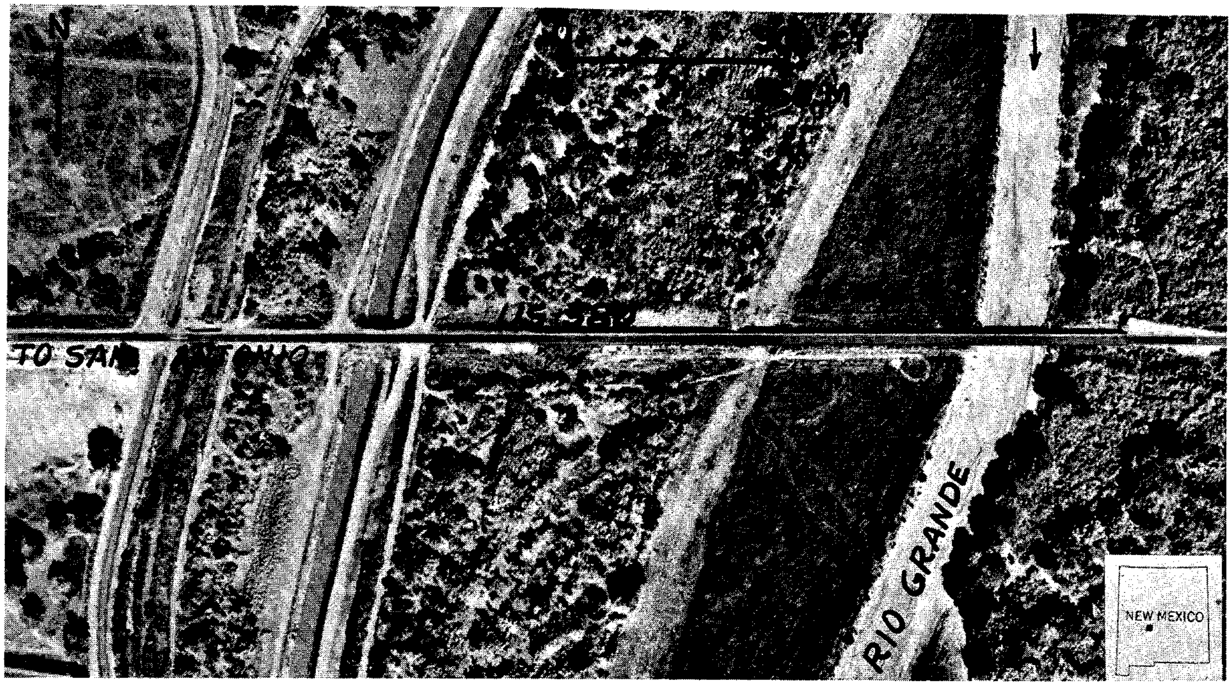


Figure 277. Aerial photograph of US-380 crossing, Rio Grande River, in 1973. (From New Mexico Dept. of Highways.)

SITE 115. WASHITA RIVER AT I-35 NEAR WHITEBEAD, OKLA.

Description of site: Lat 34°47', long 97°18', location as shown in fig. 278. Dual bridges of reinforced concrete and steel beams, 724 ft (221 m) in length; supported by seven paired round-column webbed-concrete piers and spillthrough abutments set on steel piles.

Drainage area, 5,207 mi^2 (13,486 km^2); valley slope, 0.00056; channel width, 150 ft (46 m). Stream is perennial, alluvial, sand bed, in valley of moderate relief, wide flood plain. Channel is meandering, locally braided, random width variation, cut banks general, tree cover at less than 50 percent of bankline.

Hydraulic problems and countermeasures:

(In part from Keeley, 1971)

1969 Dual bridges built. The right bank of the channel showed evidence of lateral erosion, and bank stabilization measures were considered when designing the bridge. Measures taken to prevent damage to the bridge include: (1) Placement of bridge piers on piling driven to hard shale. (2) Clearance of low steel 7 ft (2 m) above high water. (3) Use of webs between columns in the piers to prevent lodging of debris (fig. 279). (4) Construction of spur dikes on both banks and protection with rock riprap (fig. 280). (5) Construction of a 560-ft (171-m) timber-pile retard along the right bank upstream from the bridge to prevent lateral erosion (fig. 281).

Floods in 1969 caused the right bank to erode upstream from the anchor end of the timber-pile retard. This part of the bank was then protected with rock riprap. Drift, lodged on the right bank upstream from the bridge during the flood, caused eddy currents and local scour. The debris was removed during low-flow periods.

1971 Flood caused water to overtop the timber-pile retard by about 2 ft (0.6 m), but no damage to the retard or rock riprap bank were observed.

1976 The presence of dense vegetation indicates the effectiveness of the spur dike in preventing scour at the bridge abutment (fig. 280) and of the timber-pile retard in stabilizing the channel bank.

Discussion: Keeley (1971) reported lateral erosion of the right bank immediately downstream from the bridge after construction of the upstream protective devices in 1968 and 1969. He concluded on the basis of several cases that this condition may be a cause-and-effect relation where reduction of erosive action at one meander loop, through protective measures, will increase the erosion potential at the next meander loop downstream. If this conclusion is correct, bank protection measures both upstream and downstream from the bridge may be needed.

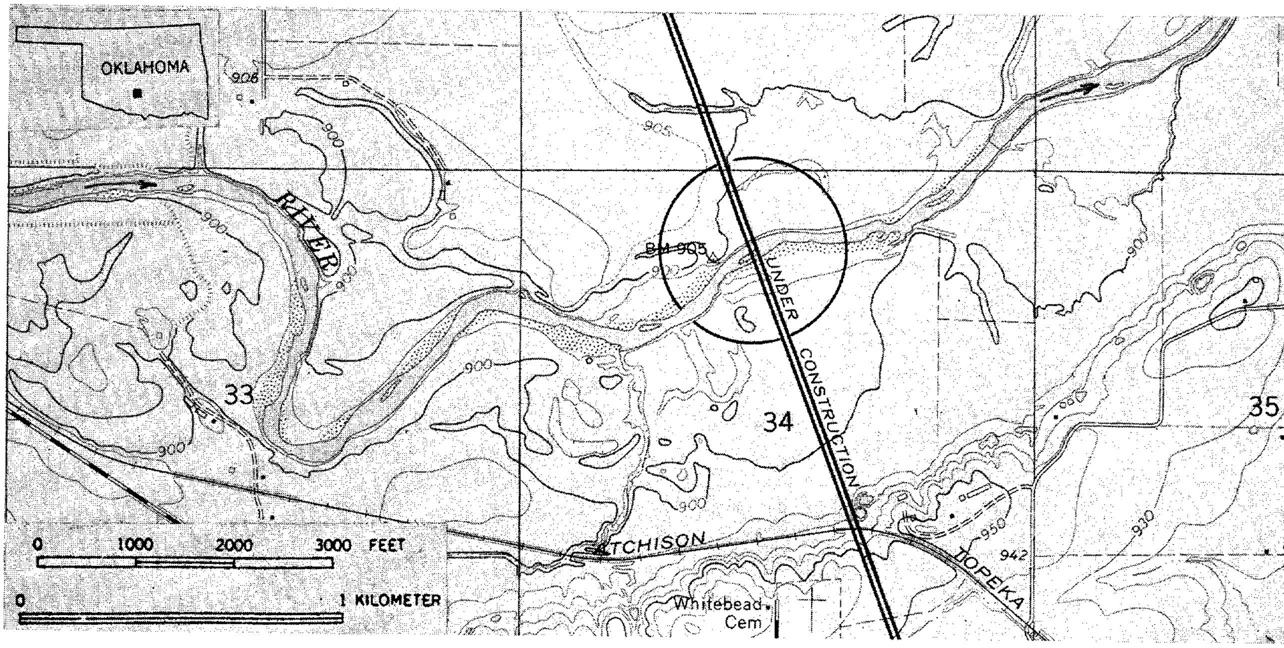


Figure 278. Location of Washita River at I-35 near Whitebead. (From U.S. Geological Survey Paoli, Okla., 7.5' quadrangle, contour interval 10 feet, 1965.)

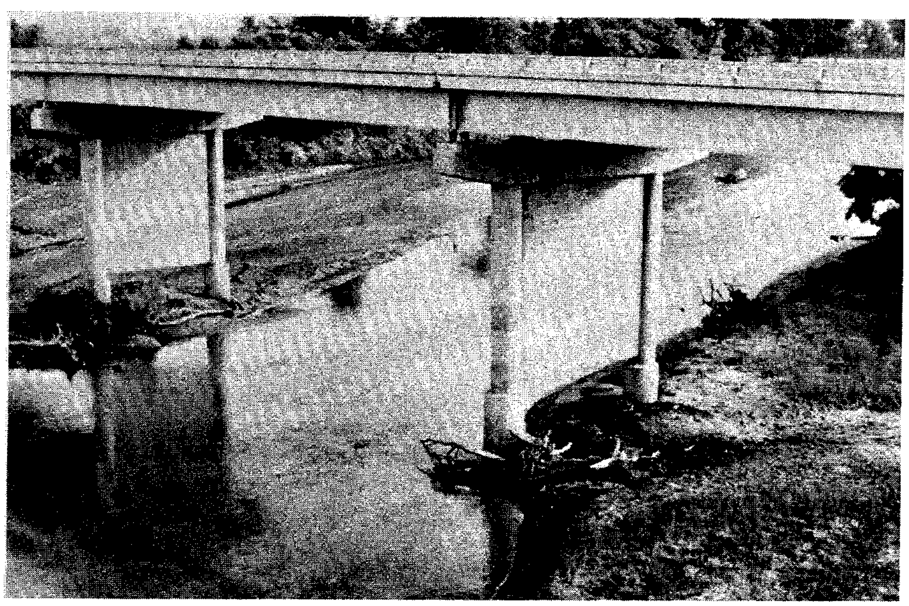


Figure 279. Webbed piers supporting Washita River bridge that was built in 1969, as photographed in 1976.

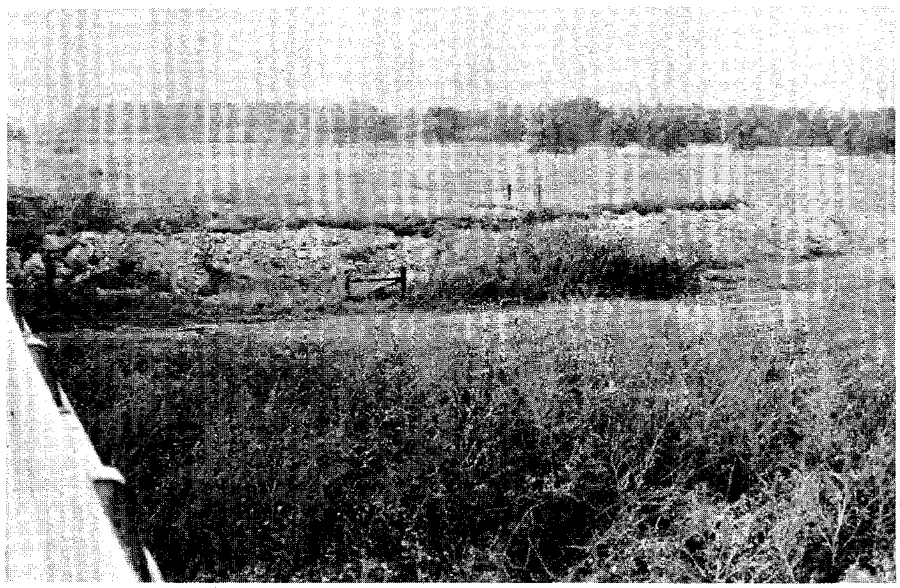


Figure 280. Spur dike on upstream right bank, Washita River, as photographed in 1976.

Figure 281. Timber-pile retard on right bank upstream of Washita River bridge, as photographed in 1976.

SITE 116. WASHITA RIVER AT SR-7 NEAR DAVIS, OKLA.

Description of site: Lat 34°30', long 97°08', location as shown in fig. 282. Bridge is reinforced concrete, 702 ft (214 m) in length, with six 100-ft (30-m) spans and two 50-ft (15-m) spans supported by seven pairs of circular-column webbed piers on spread footings. Abutments are spillthrough type, founded on piles. An existing bridge located 100 ft (30 m) upstream is used for northbound traffic.

Drainage area, 6,570 mi^2 (17,016 km^2); valley slope, 0.0007; channel width, about 350 ft (107 m). Stream is perennial, alluvial, sand bed, in valley of low relief, wide flood plain. Channel is meandering, locally braided, wider at bends, point bars, tree cover at less than 50 percent of bankline.

Figure 282. Aerial photograph of Washita River at SR-7 in 1962. Channel alinement at time of new bridge construction in 1976 is indicated in dashed lines. (From Oklahoma Dept. of Transportation.)

Hydraulic problems and countermeasures:

Pre-1964 Steel truss bridge built, 485 ft (148 m) in length; supported by four circular-column webbed piers and spillthrough abutments.

Pre-1969 Left bank of main channel near the bridge abutment was protected with rock riprap for a distance of about 350 ft (107 m) upstream from the bridge. Old car bodies were dumped along the right bank upstream of the bridge to reduce lateral erosion and movement of the channel toward the approach embankment.

1975-76 New bridge built about 100 ft (30 m) downstream from existing bridge. Countermeasures included increasing bridge length, clearing and excavating the main channel, construction of a spur dike on the right bank, placing rock riprap along the channel banks, and building three spurs (fig. 283) to prevent lateral erosion of the channel upstream from the bridge.

Discussion: A consideration in the use of revetted earth-embankment spurs rather than timber-pile spurs was the reduced possibility of damage by fire and vandalism. The channel modification and bank protection works were intended to maintain the stream alinement relative to the dual bridges, to prevent lateral migration of the meander against the approach embankment, and prevent damage to the right-bank abutment from overbank flow returning to the main channel.

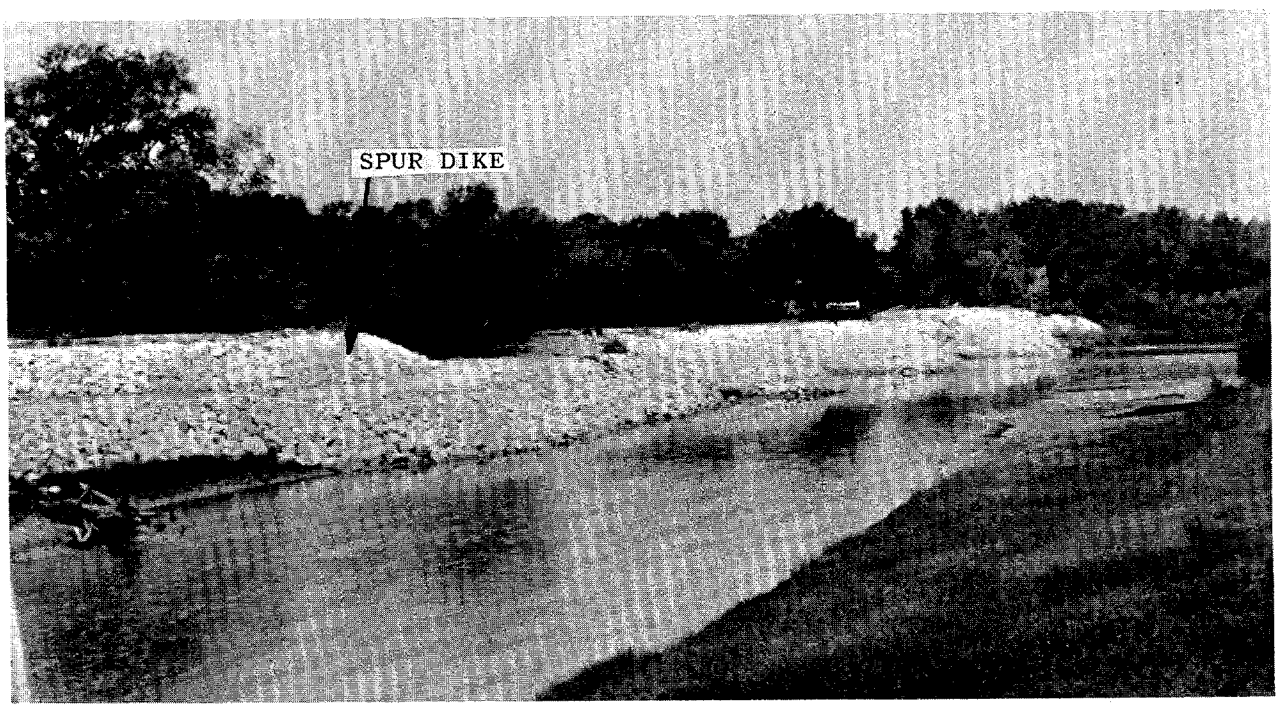


Figure 283. View upstream, in 1976, from old SR-7 bridge toward right bank, showing spur dike, three spurs, and riprapped bank.

SITE 120. WASHITA RIVER AT US-77 NEAR JOLLYVILLE, OKLA.

Description of site: Lat 34°27', long 97°08', location as shown in fig. 284. Bride is reinforced concrete, 890 ft (271 m) in length, supported by concrete piers and spillthrough abutments.

Bankfull discharge, about 25,000 ft^3/s (708 m^3/s; valley slope, less than 0.0006; channel width, about 250 ft (76 m). Stream is perennial, alluvial, sand-silt bed, in valley of moderate relief, wide flood plain. Channel is meandering, locally braided, random width variation, cut banks general (fig. 284) silt-sand banks, tree cover at less than 50 percent of bankline.

Hydraulic problems and countermeasures:

(Much of the following historical data is from Keeley, 1971, p. 244-251).

Pre-1940 Two steel truss bridges, each 360 ft (110 m) in length, were built across the main channel and a nearby overflow channel.

1940-49 Streamflow data indicate that streamflow was higher than normal for a sustained period, causing continuous changes in stream alinement.

1957 Floods during April, May and June caused another period of channel adjustment.

1966 The main channel between the upstream and downstream meander loops was cleared and straightened.

1967 Flood during spring runoff again caused the channel alinement to shift and threatened the embankment of I-35 adjacent to this reach of river. The upstream meander loop was revetted with rock riprap and plans were made to stabilize the downstream loop by use of a large Kellner jack field. During the ten years prior to 1967, the banks migrated about 300 ft (100 m) towards the I-35 alinement.

1968 A Kellner jack field (fig. 285) was constructed and the new US-77 bridge was built about 700 ft (213 m) upstream from the original bridge. Countermeasures included: (1) The new bridge is about 170 ft (52 m) longer than the combined length of the two older bridges. (2) Spur dikes were placed on both banks upstream from the bridge to prevent lateral erosion of the approach embankment as flows on the flood plain return to the main channel. (3) The approach embankments were protected from lateral erosion with rock riprap.

1969 Aerial photograph of channel indicates that the Kellner jacks are beginning to stabilize the channel (fig. 285).

1971 J. W. Keeley (1971) reported that the rock riprap installed in 1967 at the upstream meander loop was performing as intended.

1976 Surveys of the downstream meander loop and Kellner jack field indicates that the jacks have continued to stabilize the right bank and prevent further channel migration. The effectiveness of other countermeasures placed at the bridge site has not been evaluated.

Discussion: Steel jacks (Kellner jetties) were very effective in stabilizing the alinement of the Waschita River between 1968 and 1976. The rock riprap bank revetment was also effective in stabilizing the channel banks, apparently because of measures to prevent undermining.

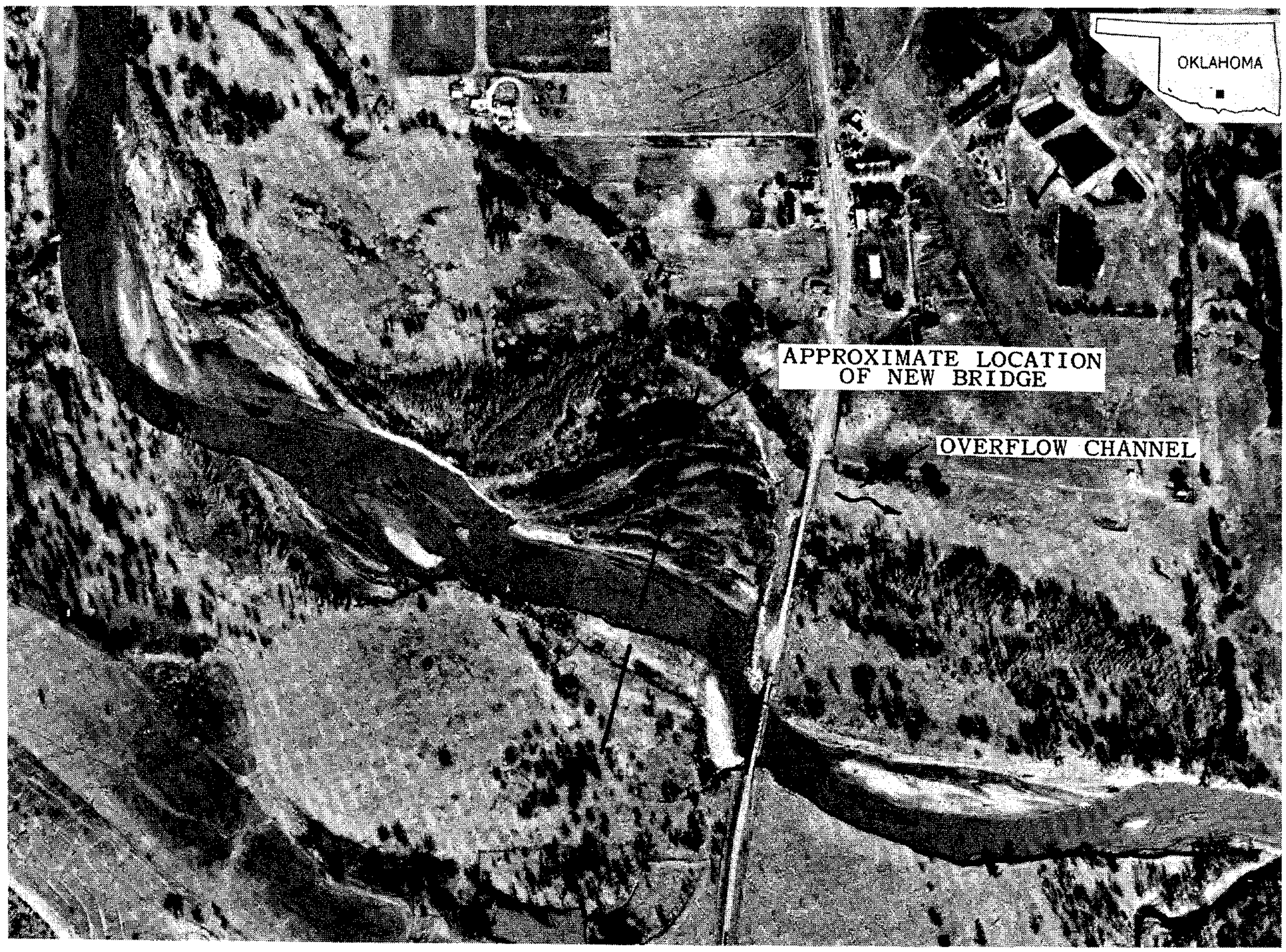


Figure 284. Aerial photograph of US-77 crossing, Washita River, in 1962 before countermeasures were installed. (From Oklahoma Dept. of Transportation.)

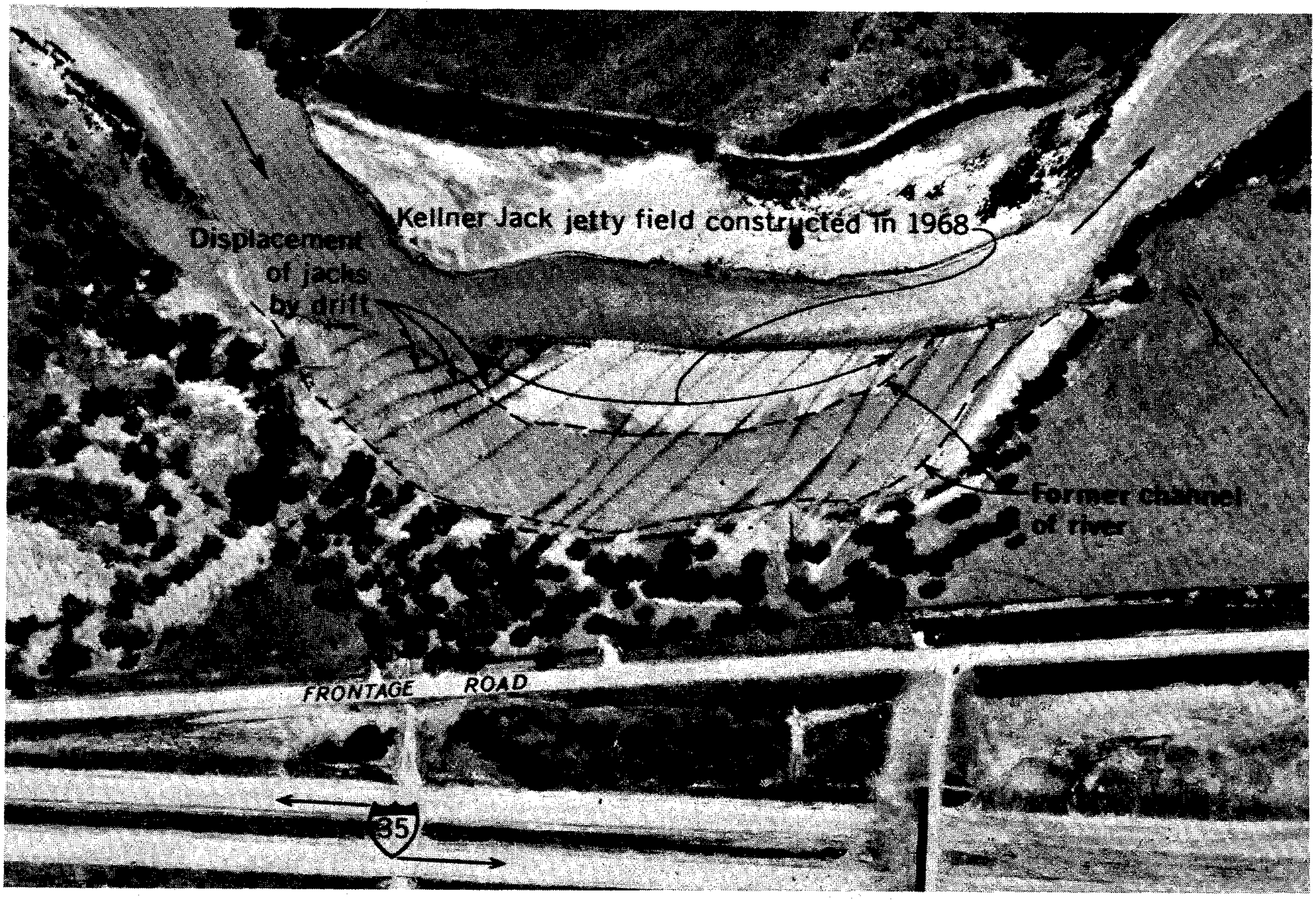


Figure 285. Aerial photograph in 1969 of jack field installed in 1968 on right bank of Washita River about 1,400 ft (427 m) upstream from US-77 bridge. Note displacement of jack lines by drift and deposition of sediment in jack field. (From Oklahoma Dept. of Transportation.)

SITE 122. McKENZIE RIVER AT I-5 NEAR COBURG, OREG.

Description of site: Lat 44°07', long 123°03', location as shown in fig 286. Concrete box girder bridge, 860 ft (262 m) long, with 8 spans of 50 ft (15 m) length and 3 main center spans. Bridge is skewed 18°, but piers are alined with flow. Piers are wall type with rounded nose, shaped similar to a large cylindrical column, and supported by spread footings. Abutments are spillthrough type.

Drainage area, 1,337 mi^2 (516 km^2); valley slope, 0.0013; channel width, about 500 ft (150 m). Stream is perennial, regulated, alluvial, gravel bed, in valley of moderate relief, wide flood plain. Channel is sinuous, locally braided.

Hydraulic problems and countermeasures:

1960 Bridge built, with 13 ft (4 m) clearance above the 1945 highwater level.

1971 Channel surveys indicated as much as 10 ft (3 m) of scour occurred around piers 2 and 3 between 1955 and 1971. Floods in 1964, the rectangular shape of the pier footings (which create localized turbulence of flow) and the relative location of piers in an upstream-downstream alinement for the dual bridges, are considered factors causing the scour observed during the 1964 flood.

1971-72 Protection of the two pier footings to prevent further undermining included placement of grout and protective rock riprap. The grout was placed from a temporary platform built on the pier. Rock riprap was placed at the downstream side of the pier footing (fig. 287) to prevent exposure of the grout material to abrasion damage.

1977 Only minor flooding has occurred between 1972 and 1977 and the countermeasure has not been tested.

Discussion: Rectangular spread footings placed for the dual bridges with a continuous upstream-downstream alinement were subject to undermining and local scour. The footings should have been designed for the posssibility of exposure to flows and lodgment of drift because the streambed apparently scours naturally during flood periods. Grout applied to fill the voids under the footings was protected by rock riprap.

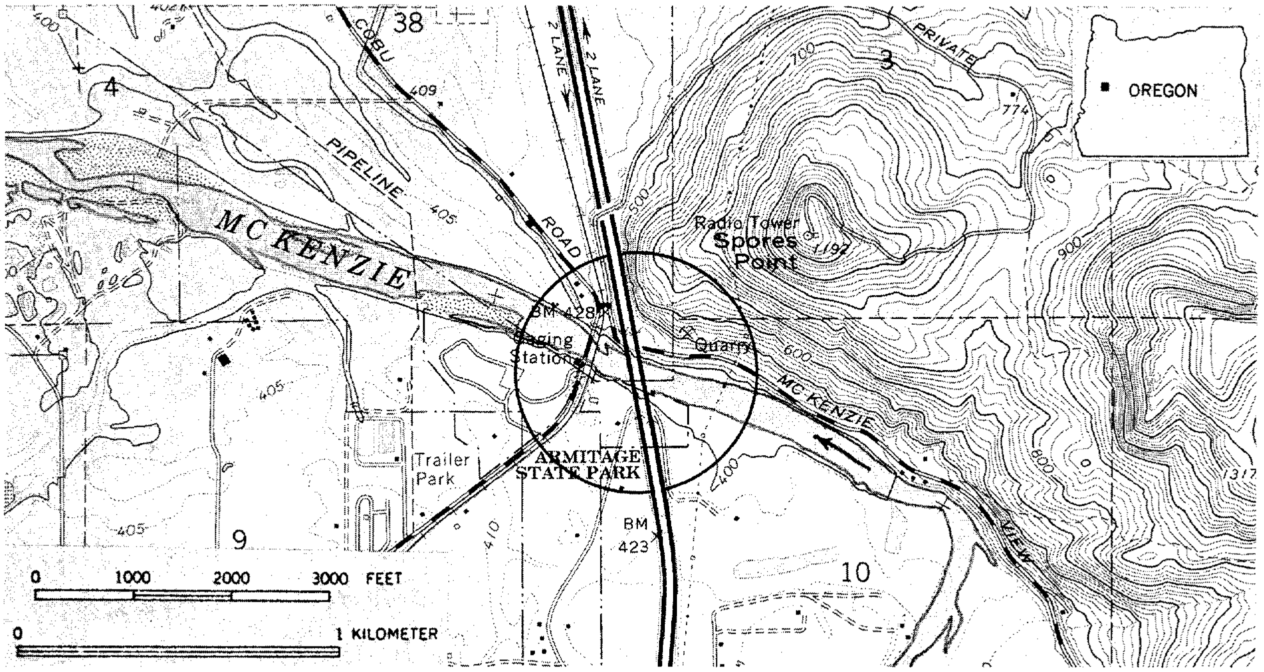


Figure 286. Location of I-5 crossing, McKenzie River. (From U.S. Geological Survey Eugene East, Oreg., 7.5' quadrangle, contour interval 20 feet, 1969, photorevised 1975.)

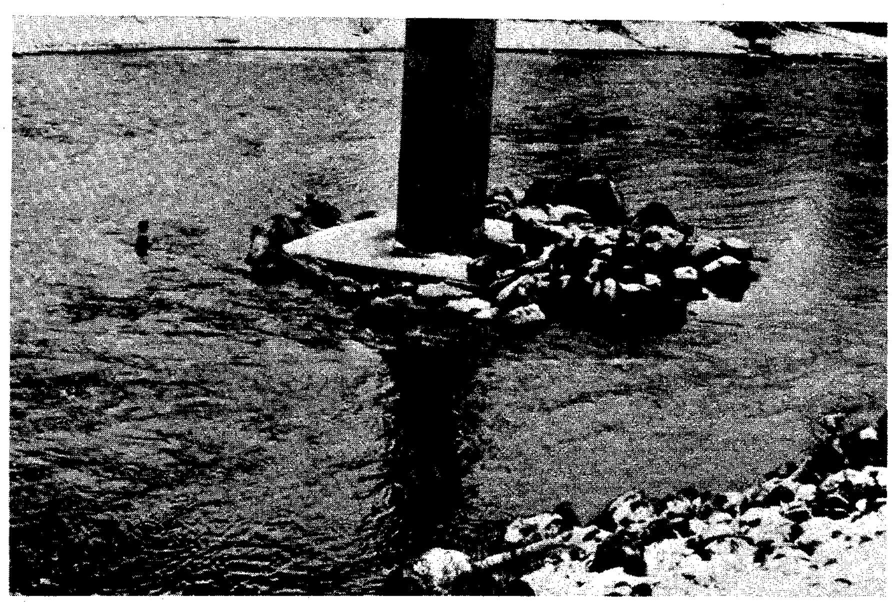


Figure 287. View of rock riprap placed around pier footing after constructing grout curtain, McKenzie River. (From Oregon Dept. of Highways.)

SITE 123. GRANDE RONDE RIVER AT SR-82 AT ISLAND CITY, OREG.

Description of site: Lat 45°20', long 118°02', location as shown in fig. 288. Bridge is reinforced, prestressed concrete, has two 45-ft (14-m) end spans separated by one 120-ft (37-m) main span, total length 210 ft (64 m). Abutments are spillthrough, founded on piles; piers are round nose wall-type located outside the main channel, on piles in gravelly bed material.

Valley slope, 0.029; channel width, about 100 ft (30 m). Stream is perennial, alluvial, gravel bed, on alluvial fan. Channel is sinuous, gravel banks.

Hydraulic problems and countermeasures:

Pre-1965 Bridge built; steel truss bridge with two 100-ft (30-m) spans; spillthough abutments, one center pier and two pile bents at quarter points.

1965 Flood during January, the highest of record (R.I. over 100 yr), created a scour hole between the SR-82 bridge and the Union Pacific Railroad bridge downstream, causing the center pier of the railroad bridge to collapse. Curvature of the channel, channel degradation associated with sand and gravel removal operations upstream and downstream from the bridge site, and constriction of flood flow at the bridge crossings contributed to scour and bridge damage during the flood.

1972 Channel degradation exposed lower parts of the center pier of the SR-82 bridge and threatened to expose the support piling. Attempts had been made earlier to correct the problem but were unsuccessful.

1975 New bridge built. Countermeasures included location of piers outside the main channel (fig. 289); designing for the maximum (January 1965) flood of record; and lining the channel with rock riprap at the bridge for a distance of 45 ft (14 m) upstream and 410 ft (125 m) downstream of the highway centerline.

1977 No large floods have occurred since completion of the construction.

Discussion: Sand and gravel extraction operations in the channel, combined with constriction of flood flows by the bridge piers caused scour of the channel at the bridge site during an extreme flood event. To prevent undermining of the rock riprap channel lining, cutoff walls extending 6 ft (2 m) below the existing streambed were constructed at each end of the revetment.

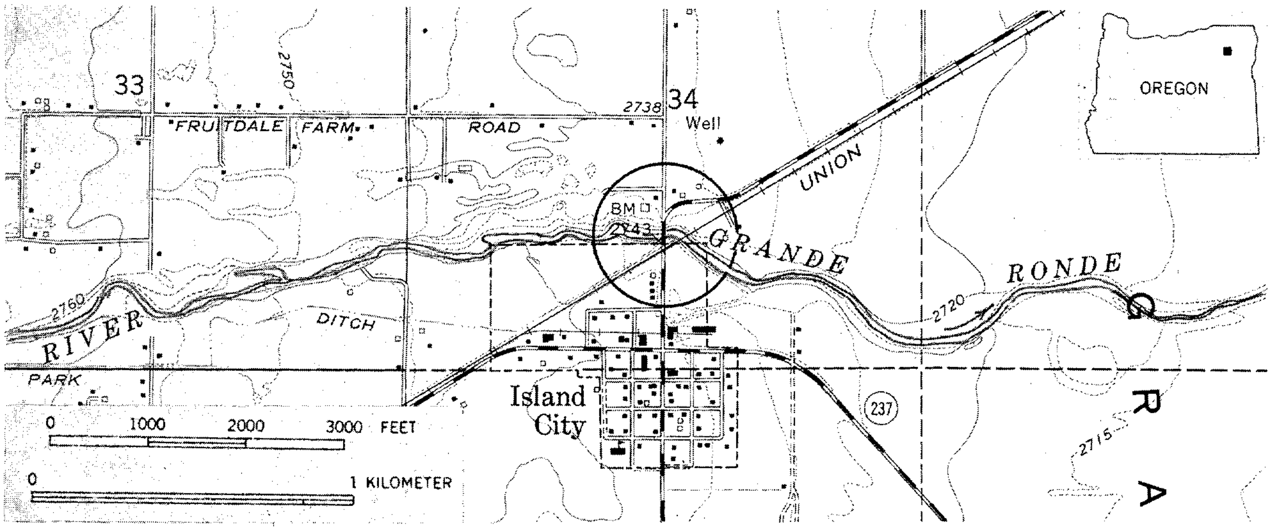


Figure 288. Location of SR-82 crossing, Grande Ronde River. (From U.S. Geological Survey La Grande SE, Oreg., 7.5' quadrangle, contour interval 20 ft, 1963.)

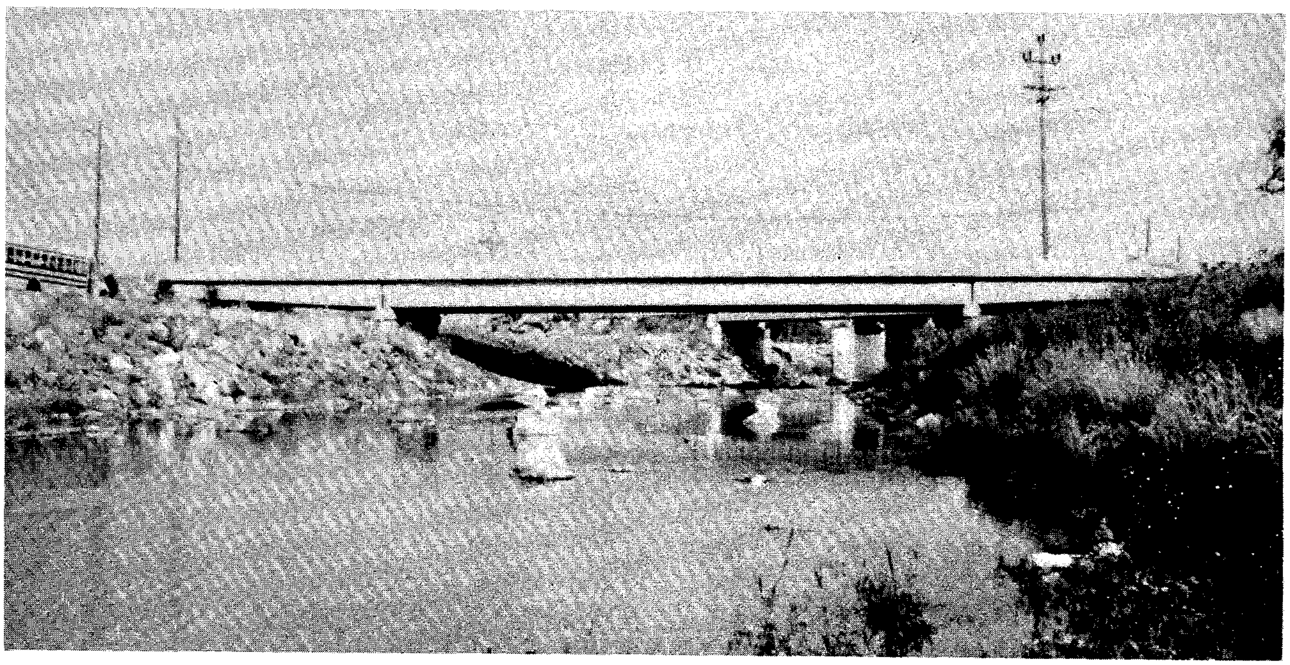


Fig. 289. Downstream view of new SR-82 bridge. Note rock riprap channel lining on left bank and placement of right-bank bridge pier on flood plain away from main channel. (From Oregon Dept. of Highways.)

SITE 124. ROGUE RIVER AT I-5 NEAR ROGUE RIVER, OREG.

Description of site: Lat 42°25', long 123°07', location as shown in fig. 290. Concrete box-girder bridge, length 809 ft (247 m), with 4 main spans and 5 approach spans. Bridge and pier skew is 30 degrees. Piers are rectangular with rounded nose and supported by spread footings placed on compact or cemented gravel. Abutments are spillthrough.

Drainage area, about 2,053 mi^2 (5,317 km^2); valley slope, 0.0013; channel width, about 600 ft (183 m). Stream is perennial, alluvial, gravel bed, in valley of moderate relief, narrow flood plain. Channel is sinuous.

Hydraulic problems and countermeasures

1962 Bridge constructed. Piers placed using spread footings with base about 5 ft (1.5 m) below streambed in cemented gravel, to prevent undermining and to provide adequate bearing capacity.

1964 Flood of Dec. 24, 1964, R.I. about 70 yr, caused substantial changes in channel location and bed elevation.

1968 The river bed had degraded to a depth of 1.5 ft (0.5 m) below the footing base at the southbound bridge pier 3. According to a maintenance department report, maximum erosion occurred 11 ft (3.4 m) downstream from the footing. Repairs to the footing at pier 3 included (1) removal of all loose gravel down to cemented gravels; (2) drilling of holes and placement of 1.75 in (44 mm) diameter reinforcing bars in holes to keep riprap from moving downstream; and (3) placement of large (up to 3.5 yd^3 or 2.7 m^3) rock riprap on the channel bed.

The river bed also degraded, exposing the base of footings at pier 2 on both the north and southbound bridges. The extent of channel bed degradation is described in the work progress report:

> "The northbound bridge footing had eroded 2 ft (0.6 m) below the bottom on 3 sides. Cavitation had started under one corner and extended 3 ft (0.9 m) under the footing base. Numerous channels eroded between two piers and one very large channel about 350 ft (107 m) long extended past both piers, about 6 ft (1.8 m) in depth and 10 ft (3 m) to 25 ft (7.6 m) wide. Approximately 200 ft (61 m) downstream from the southbound bridge the river channel makes an abrupt drop of 6 ft (1.8 m). This situation is moving upstream and will eventually reach the bridge. The construction pads were still in evidence indicating the river bed at the bridge remains about the same."

Repairs to footing 2 on the northbound and southbound bridges included placement of a blanket course of rock riprap averaging 2.5 ft (0.8 m) deep. The riprap was hand placed and keyed together with 1 ft (0.3 m) of concrete. For a large scour hole with maximum depth of 15 ft (4.6 m), riprap was placed to water level. Where eroded channels 3 ft (0.9 m) to 4 ft (1.2 m) deep and about 6 ft (1.8 m) wide occurred, loose gravel was removed and rock riprap 0.75 yd^3 (0.6 m^3) to 1.5 yd^3 (1.1 m^3) in size was placed. Solid blocks of large riprap and concrete were poured to the general channel elevation and aligned slightly toward the mid-channel.The blocks are intended to stabilize the riprap during extreme flood stage and to help deflect the current into midstream and away from the pier footings.

1974 Surveys of the channel bed between 1969 and 1974 indicate that the armoring installed in 1968 has been effective in stabilizing the channel.

1977 Records of flow during the period 1962 and 1976 indicate that much of the channel change at the bridge probably occurred during or after the December 1964 flood (R.I., 70 yr). The channel bed protection and armoring installed in 1968 has been effective during the 1972 flood (R.I., 12 yr) and the 1974 flood (R.I., 10 yr).

<u>Discussion</u>: Armoring the channel bed around the bridge piers with rock riprap has stabilized the channel for the years 1969 to 1977. During this period, maximum floods of 10- and 12-yr R.I. occurred. Gravel bed material overlying cemented gravel was removed and replaced with rock riprap in the 0.75 yd^3 (1.1 m^3) size range to prevent rock movement during future large floods. Steel pins were placed in the cemented gravel channel bed to prevent lateral movement by rock riprap during floods. Channel bed scour, attributed to bridge piers, was observed as much as 120 ft (36 m) downstream from the bridge.

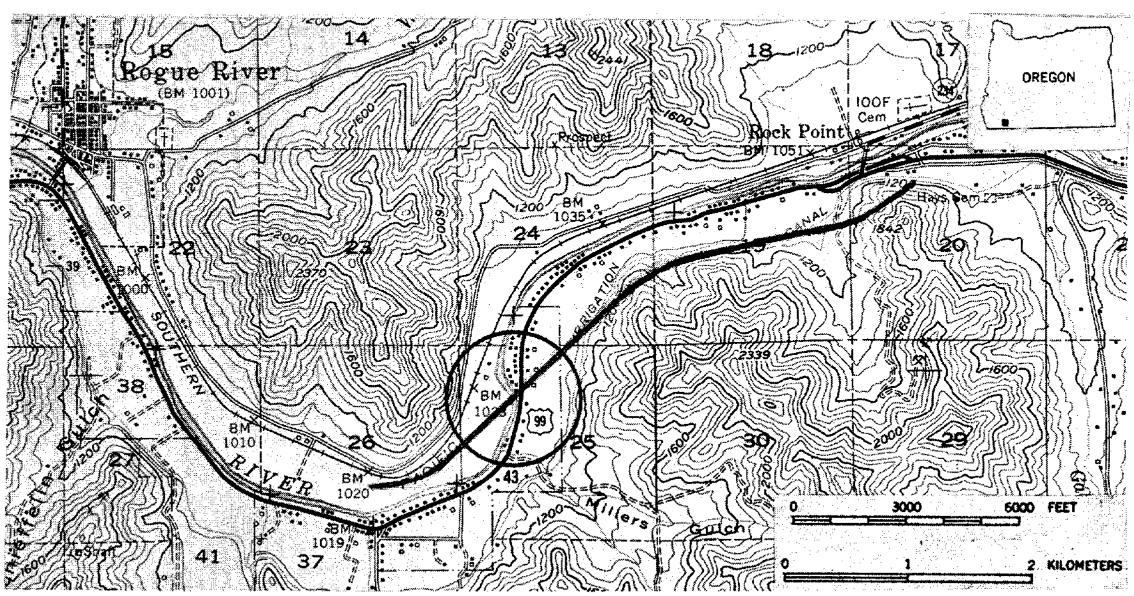


Figure 290. Map showing I-5 crossing of Rogue River (circled). (From U.S. Geological Survey Gold Hill, Oreg., 15' quadrangle, contour interval 100 ft, 1954.)

SITE 125. COW CREEK AT I-5 NEAR AZALEA, OREG.

Description of site: Lat 42°47', long 123°17'. Dual concrete bridges; northbound bridge built in 1948, southbound bridge, in 1974. Bridges are 250 ft (76 m) in length, supported by three concrete columns per bent. Columns are founded on spread footings. Bridge skew is 50 degrees but bents are alined with the flow. Abutments are spillthrough.

Drainage area, about 80 mi^2 (207 km^2); channel width, 100 ft (30 m). Stream is perennial, alluvial, gravel bed, in valley of moderate relief, narrow flood plain. Channel is sinuous, locally braided, wider at bends, point bars, tree cover at 50-90 percent of bankline.

Hydraulic problems and countermeasures:

1948 Construction of the original highway (US-99) required a channel change through a reach of braided channel. The work consisted of straightening and shortening the channel for a distance of about 2,300 ft (701 m) upstream from the bridge site. A dike was built along the right bank to contain flood waters and protect the approach embankment (fig. 291). During the period 1948-56, there appeared to be little change in the Cow Creek channel near the bridge site.

1964 Flooding during the 1964 flood (R.I. about 50 yr) caused a large gravel bar to form along the right bank at the bridge site. Scour near the left bank (fig. 291) partly undermined pier footings on the left bank (south) end of the bridge.

1965 Rock riprap was placed on both banks of the river at the bridge.

1973 Channel changes continued to cause lateral erosion of the banks, particularly downstream from the bridge (fig. 291). These changes are attributed to effects of inflow from Quines Creek and a steep channel gradient downstream from the bridge, in which flows are confined to the main channel causing high flow velocities during floods. A possible cause of the steep channel gradient downstream from the bridge site may be the lowering of the channel bed related to sand and gravel mining on the flood plain near the bridge. Evidence of the former sand and gravel operation, downstream from the bridge site, is visible in photographs. To prevent additional exposure of pier footings, rock riprap was placed in the streambed around selected piers of both bridges. Because of continued local scour, footings for Pier 4 of the northbound bridge were replaced with a foundation supported by steel piles driven alongside the spread footing.

1975 Continued lateral erosion of the right bank downstream from the bridge (fig. 291) required the construction of four rock-riprap spurs (fig. 292).

1977 No floods have occurred to evaluate the effectiveness of the riprap around the bridge piers nor of the four spurs built on the right bank downstream from the bridge.

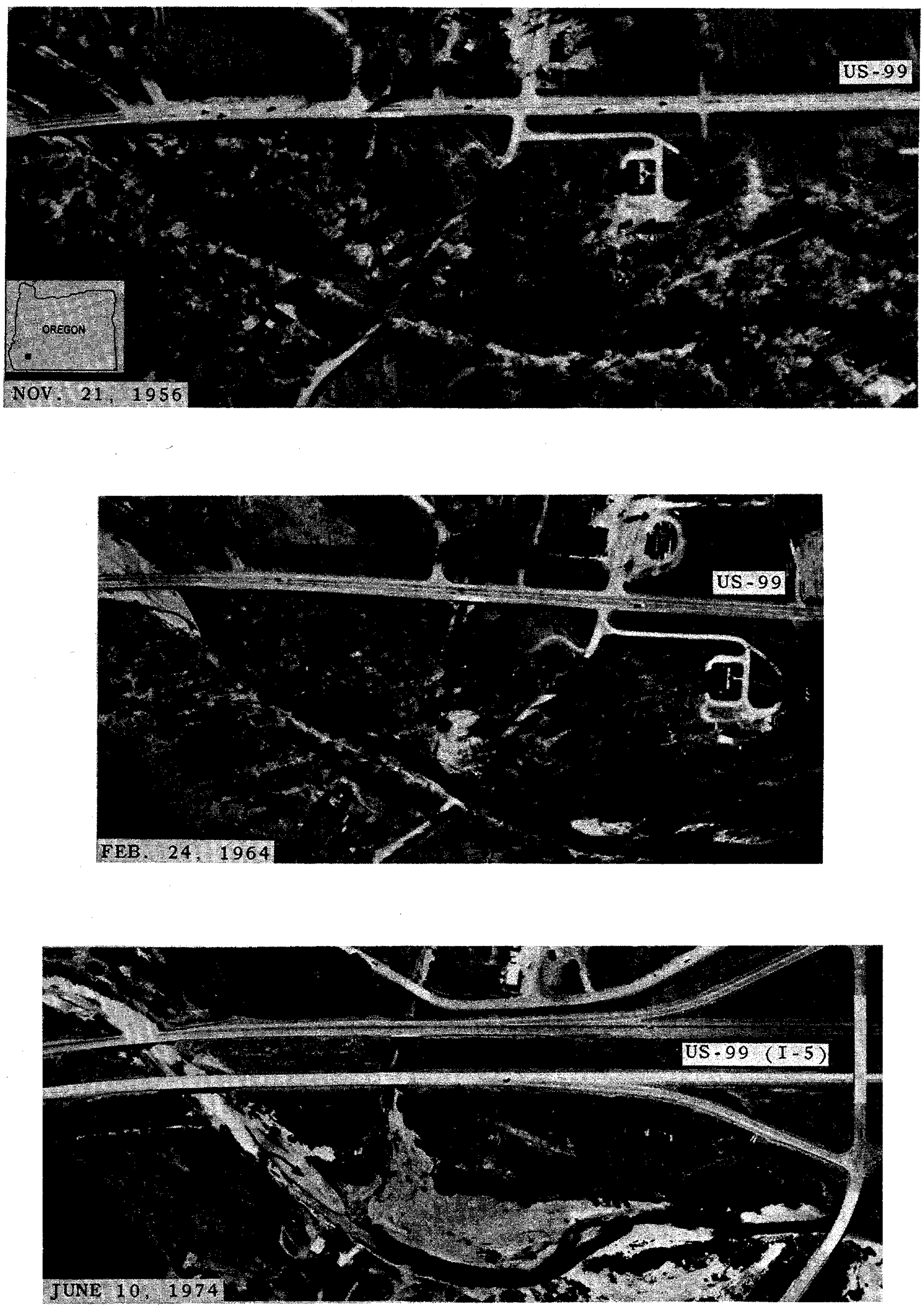


Figure 291. Sequential aerial photographs showing changes in natural channel of Cow Creek, 1956-74. (From Oregon Dept. of Highways.)

Discussion: Channel degradation, local scour, and lateral erosion problems have occurred at this bridge site. These problems are probably associated with realinement and straightening the channel upstream from the bridge; the occurrence of a large flood (R.I. about 50 yr); sand and gravel mining from the channel and flood plain downstream from the bridge site; and depositon of sediment at the mouth of Quines Creek, which is about 1,500 ft (457 m) downstream from the bridge. The effectiveness of countermeasures used at the bridge site, including rock riprap placed on the channel banks to prevent scour at abutments, riprap at piers, and spurs to prevent lateral erosion downstream from the bridge, have not been tested.

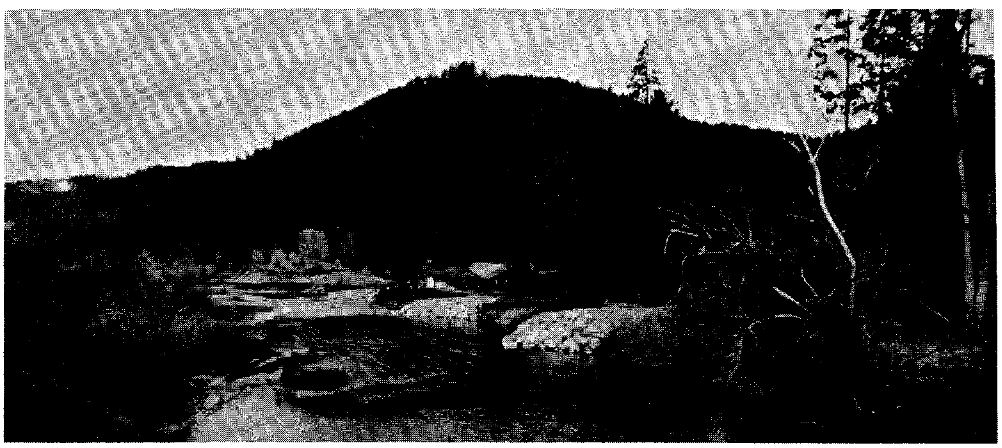


Figure 292. View downstream from bridge, in 1976, of rock spurs built along right bank. (From Oregon Dept. of Highways.)

SITE 126. SOUTH SANTIAM RIVER AT SR-226 NEAR ALBANY, OREG.

Description of site: Lat 44°38', long 122°55', location as shown in fig. 293. Post-tensioned concrete box-girder bridge, 1,080 ft (329 m) in length, three main spans and four end spans, wall-type piers. All piers except one are founded on spread footings. Abutments are spillthrough, supported by steel piling.

Valley slope, 0.0017; channel width, about 500 ft (152 m). Stream is perennial, alluvial, gravel bed, in valley of moderate relief, wide flood plain. Channel is sinuous, generally braided, locally anabranched, cut banks local, tree cover at less than 50 percent of bankline.

Hydraulic problems and countermeasures:

1971 An older bridge, located 0.4 mi (0.6 km) upstream from the proposed new structure (fig. 293) crossed the stream at a point where the main channel alinement was stable. The channel alinement was unstable at the proposed new location. According to a hydraulics study made for the new bridge, floods in December 1964 and January 1965 caused channel changes upstream from the bridge site, and the channel in 1966 was 1000 ft (305 m) wide compared with the old channel width of 500 ft (152 m). During the period 1965-71, the thalweg of the braided channel moved towards the right bank. The study concluded that the main channel would probably form along the right bank and that severe lateral bank erosion could be expected.

1971-73 Sequential aerial photographs of the bridge site record channel changes (fig. 294). During the flood in 1972, the main channel shifted from the center of the proposed new bridge toward the right bank. Subsequent floods moved the main channel further eastward, causing lateral erosion of about 30 ft (9 m) of the right bank. On the basis of observed channel instability, design of the new bridge included several countermeasures: (1) Wall-type bridge piers that nearly approximate the shape of round columns were used to prevent problems of pier skewness and lodging of debris (fig. 295). (2) Pier footings were protected by placement of a rock-riprap scour arrestor beneath the streambed. The size of scour arrestor at a pier depended on the location of the pier relative to the main channel. (3) A revetted dike was constructed at the right bank to prevent the main channel from eroding the bank and eventually exposing the right abutment (fig. 296). The dike and channel banks were protected with rock riprap keyed in place by impacting the rock with a large steel plate to compact the rock. The resulting smooth riprap surface reduces the turbulence of flow and tends to prevent dislodging of the individual rock during high flows. The dike at the left bank, constructed by the Corps of Engineers in 1965 and revetted with rock riprap keyed in place, performed effectively during floods in 1966, 1972, and 1973.

1975 Inspection of the site indicates that the countermeasures constricted in 1973-74 are performing satisfactorily, although no large floods have occurred since bridge construction.

Discussion: The need for countermeasures to protect the new bridge were based on observed channel changes over several years. The original bridge design included plans for a spur dike on the right bank. As additional flow and channel data were collected and the channel moved toward the right bank, there was concern a new channel may form behind (east of) the proposed spur dike, causing eventual abutment failure. The final bridge plans then included a dike and bank revetment protection that extended about 1,200 ft (366 m) upstream from the bridge to high ground.

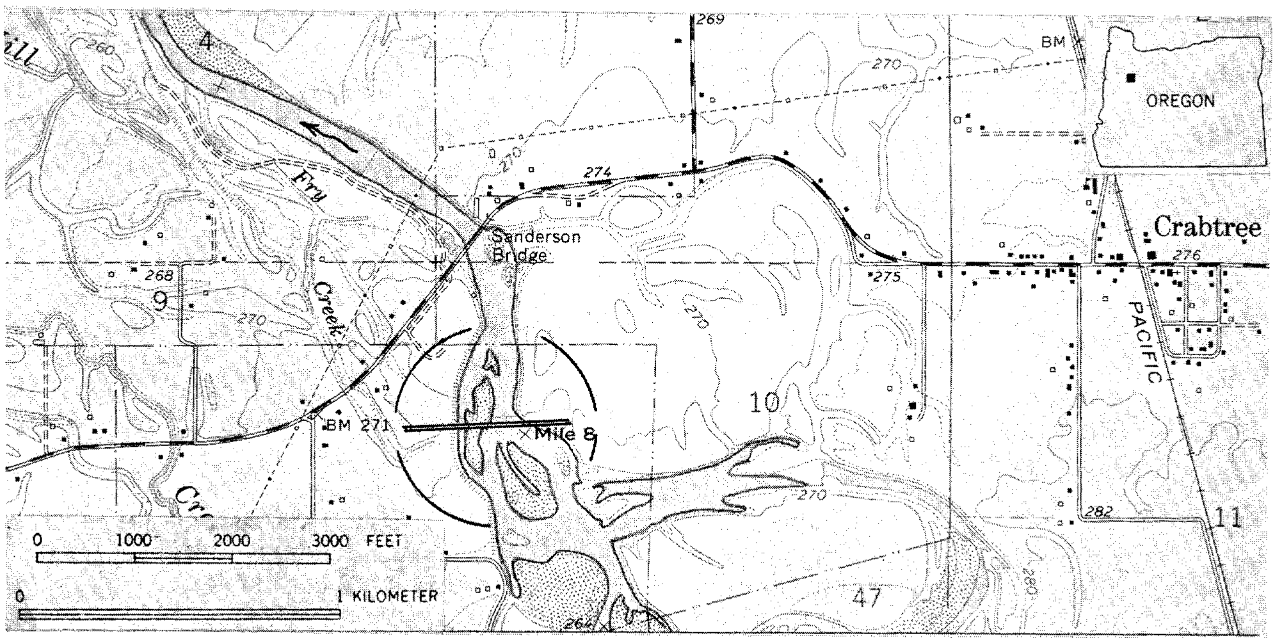


Figure 293. Location of new crossing of SR-226 (circled), South Santiam River. (From U.S. Geological Survey Crabtree, Oreg., 7.5' quadrangle contour interval 10 feet, 1970.)

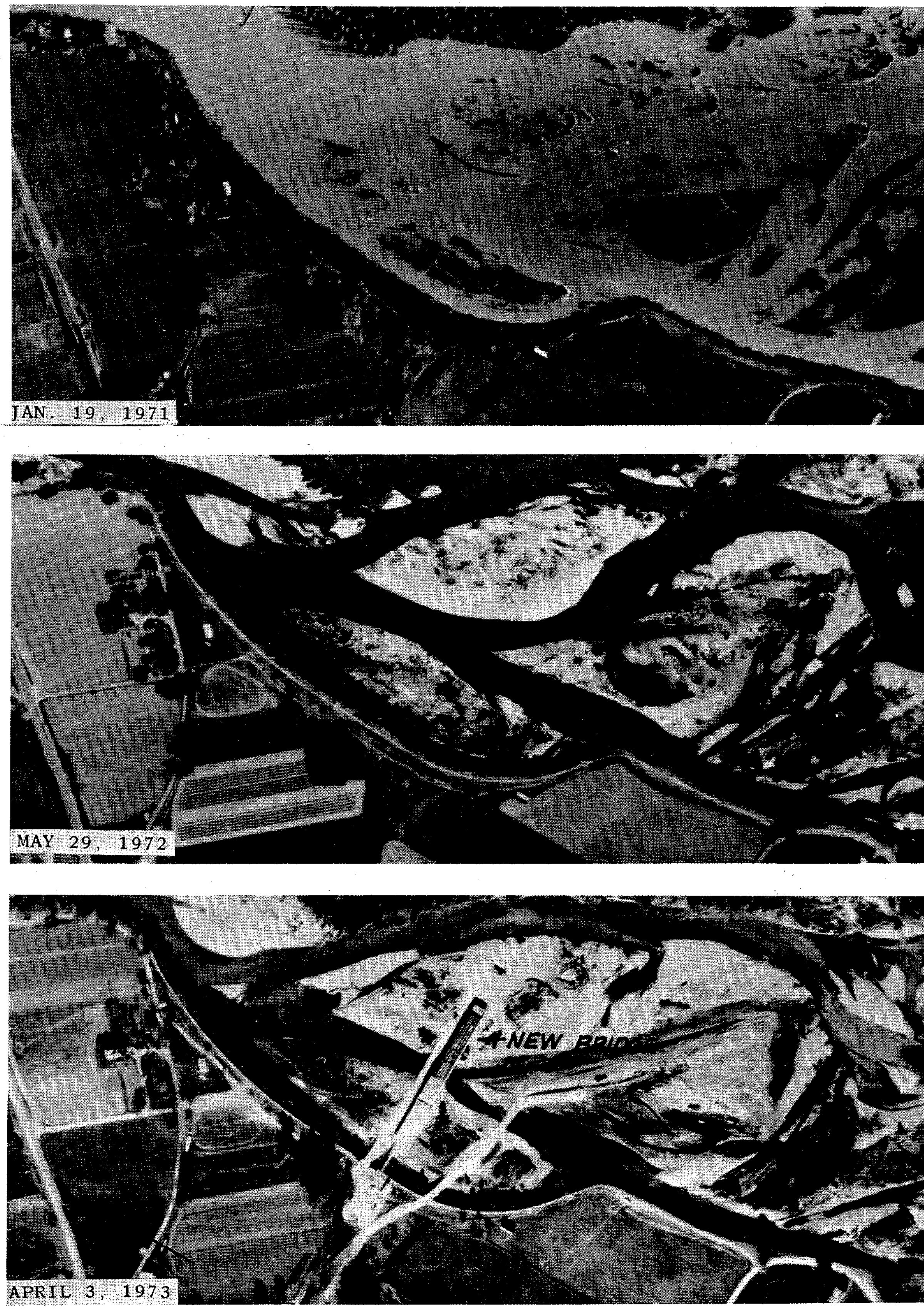


Figure 294. Sequential aerial photographs showing changes in the natural channel of South Santiam River, 1971-73.) (From Oregon Dept. of Highways.)

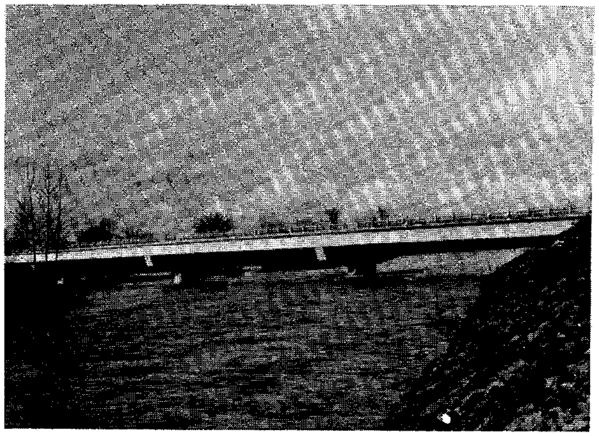


Figure 295. View of upstream side of completed bridge, South Santiam River. Wall-type piers minimize skewness to flow. (From Oregon Dept. of Highways.)

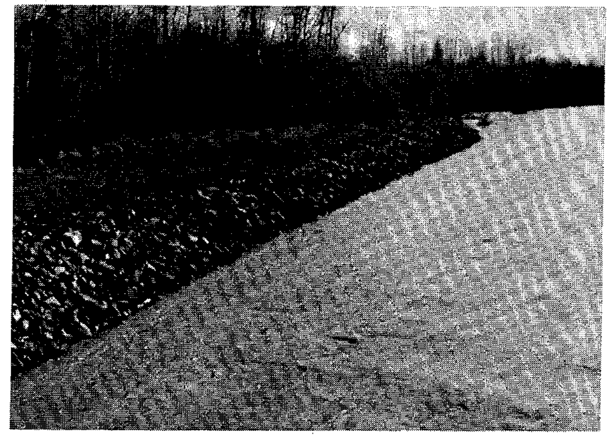


Figure 296. View upstream from South Santiam River bridge toward right bank. Dike is revetted with rock riprap, keyed in place. (From Oregon Dept. of Highways.)

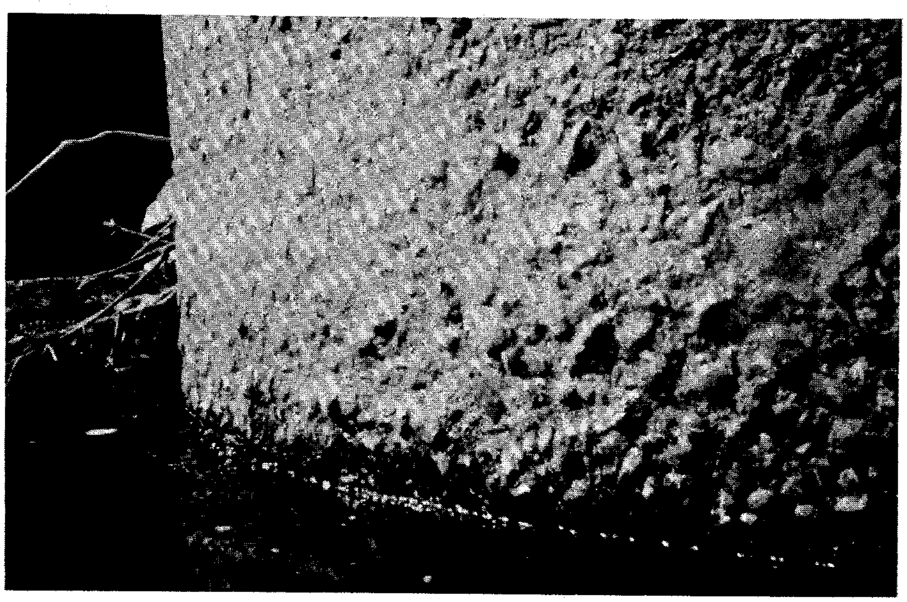


Figure 297. View of downstream face of cylindrical pier showing abrasion damage as photographed about 1971, 13 years after bridge construction. (From Oregon Dept. of Highways.)

SITE 127. SOUTH UMPQUA RIVER AT I-5 NEAR CANYONVILLE, ORE.

Description of site: Lat 43°00', long 123°20'. Steel-girder bridge 588 ft (179 m) long, supported by two cylindrical piers in the main channel and four webbed piers at the approach spans. Bridge is skewed 32 degrees but the center-span piers are alined with the flow. Abutments are spillthrough. Piers are supported by spread footings keyed into solid bedrock.

Valley slope, 0.001. Stream is perennial, alluvial, gravel bed, in valley of high relief, wide flood plain at crossing site. Channel is sinuous in general but nearly straight at crossing.

Hydraulic problems and countermeasures:

1958 Bridge built. The center-span piers are large cylindrical concrete columns, 8 ft (2.4 m) in diameter and supported by spread footings.

(Approx. 1971) Abrasion damage to the downstream face of piers 2 and 3 noted (fig. 297). The damage extends down to the streambed, and some of the reinforcing steel is exposed (fig. 298). The cause of this type of abrasion damage is uncertain, but is apparently due to the wearing effect of suspended bed material associated with flow turbulence during floods at the downstream side of the cylindrical pier. As a countermeasure, large rock riprap was placed around the pier columns.

Discussion: Performance of the rock riprap could not be assessed.

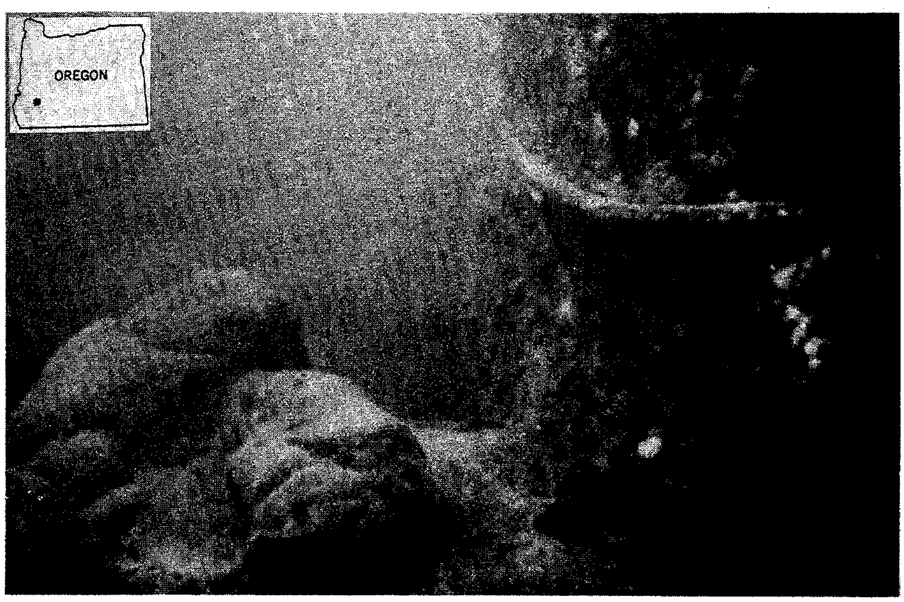


Figure 298. Underwater view of cylindrical pier showing exposed reinforcing steel. Abrasion damage extends to the streambed of cobbles and boulders. (From Oregon Dept. of Highways.)

SITE 130. TRINITY RIVER AT FM-162 NEAR MOSS HILL, TEX.

Description of site: Lat 30°16.5', long 94°48', location as shown in fig. 299. Concrete and steel-girder bridge, about 1,400 ft (427 m) long, supported by wall-type piers placed on piling. Abutments are spillthrough.

Channel width, about 400 ft (122 m). Stream is perennial, alluvial, sand bed. Channel is meandering, wider at bends, point bars, tree cover at greater than 90 percent of bankline.

Hydraulic problems and countermeasures:

1966 Bridge was built at a meander bend of the Trinity River that is undergoing scour and bank erosion. To prevent migration of the meander bend downstream, five timber jetties (spurs) were constructed on the left bank upstream from the bridge (fig. 300). Scour in the deep water channel at the time of construction prompted placement of pier 13 at a lower elevation than originally planned.

1975 Channel surveys by the Texas State Highway Department showed that the right bank had eroded about 80 ft (24 m) as the channel migrated, and about 10 ft (3 m) of scour occurred between 1965 and 1976. As a result, channel bed scour has almost exposed the tops of piles supporting bents 14, 16, 18 and 19. The timber spurs constructed upstream from the bridge prevented further movement of the meander bend at the spurs, but the unprotected channel bank downstream from spur 5 continued to erode. Protection of the channel bank was limited to a distance of about 50 ft (15 m) downstream from spur 5, or about the distance the spur extends into the main channel.

1976 Bank and channel work to protect the bridge proposed by the Corps of Engineers, and scheduled for construction in 1977, included placement of scour arrestors around bents 13, 14 and 15, and revetment at the left bank. The base of the scour arrestor was placed about 1 ft (0.3 m) below the channel bed, and the bank revetment was protected with a toe placed about 3 ft (1 m) below the channel bed. The effectiveness of the new countermeasures has not been determined.

Discussion: Because the bridge crossed the Trinity River at a meander bend, problems of channel stability and scour were anticipated in the bridge design. Specific measures to protect the bridge included: (1) Placement of the main channel pier footings seal level at least 15 ft (5 m) below the streambed, (2) Prevention of channel meander movement by use of five timber spurs placed upstream from the bridge, and (3) In anticipation of future meander bend movement, the seal level for pier 13 was placed about 40 ft (12 m) below the level of the flood plain to prevent scour problems. Protection of the channel bank by the timber spurs was limited to a distance downstream from the lower spur about the distance the spur extends into the main channel. Lateral movement of the meander bend at the bridge was about 70 ft (21 m) during a period of 10 years even though timber spurs were constructed upstream from the bridge to prevent channel movement.

Unanticipated scour about 9 ft (3 m) deep occurred at the point bar on the right side of the channel, exposing the footings of several piers. This occurrence indicates the entire channel geometry is changing and plans to protect the bridge from scour and erosion should consider the entire channel.

Bank and channel bed countermeasures built in 1977 utilized rock-riprap revetment to prevent further channel changes at the bridge rather than flow control structures such as additional spurs. The effectiveness of the new measures has not been determined.

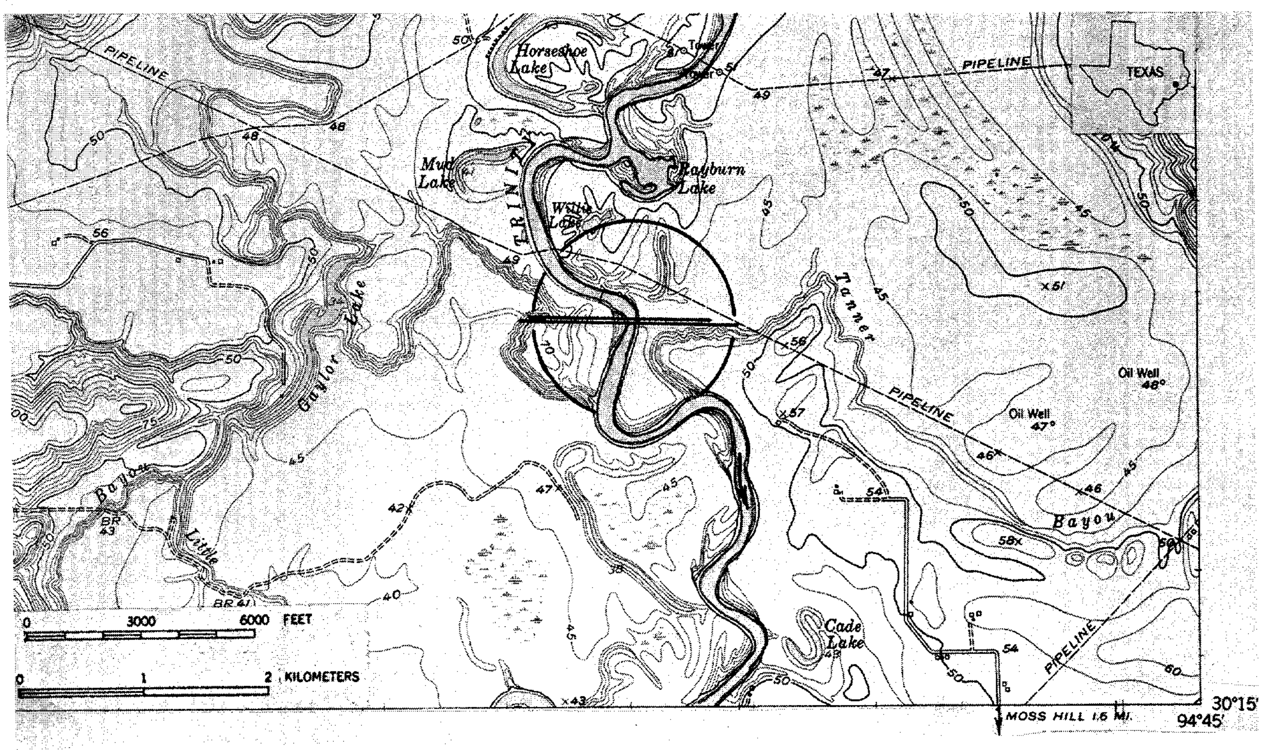


Figure 299. Location of FM-162 crossing (circled) Trinity River. (From U.S. Geological Survey Rayburn, Tex, 15' quadrangle, contour interval 5 ft, 1955.)

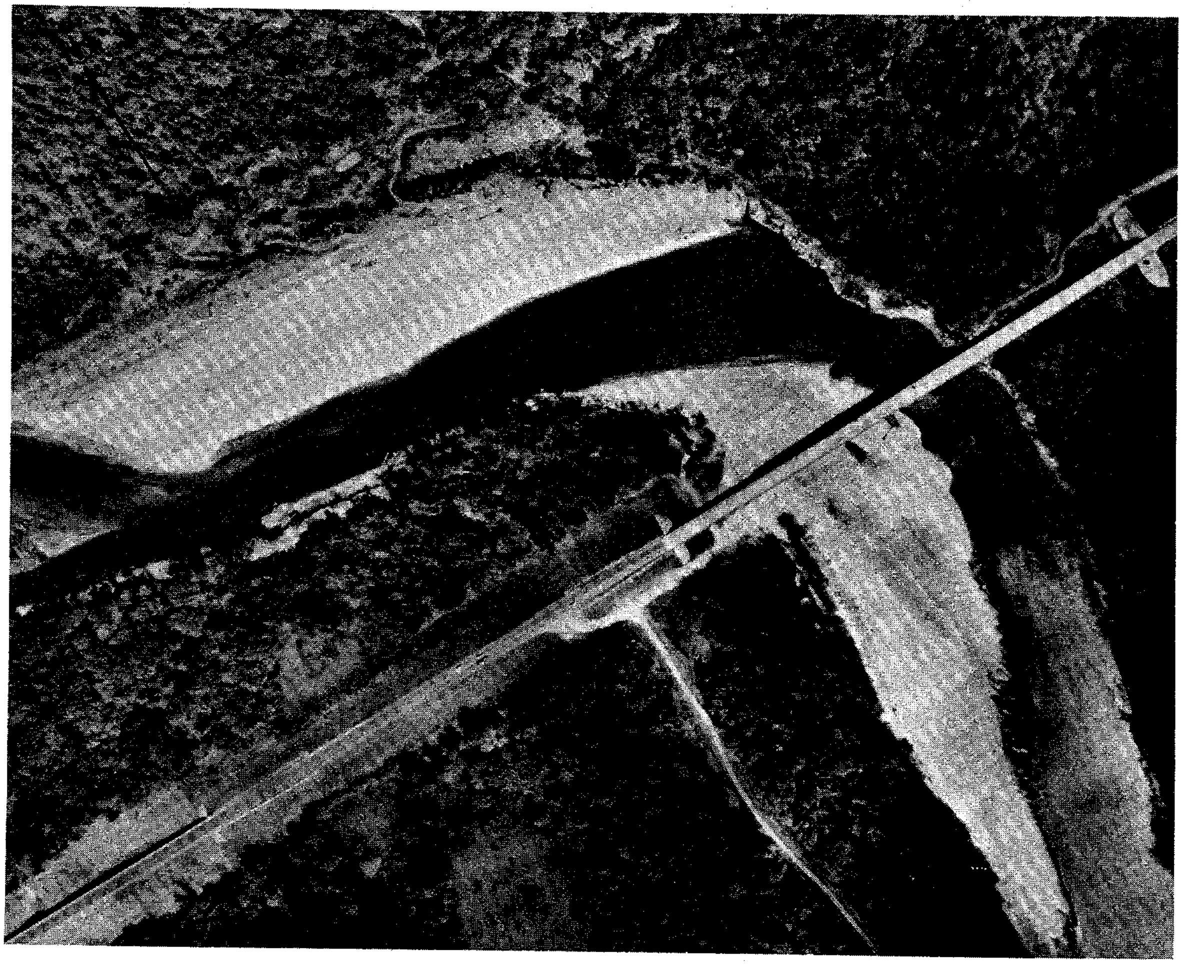


Figure 300. Aerial photograph in 1975 of Trinity River at FM-162 crossing. Note timber spurs on left bank, also bank erosion between spurs and bridge. (From Texas Dept. of Highways.)

SITE 131. BRAZOS RIVER AT FM-1462 NEAR ROSHARON, TEX.

Description of site: Lat 29°21', long 95°35', location as shown in fig. 301. Steel-girder bridge, 720 ft (219 m) in length, with main spans supported by concrete wall-type piers with rounded nose. Bridge is at a meander bend but is approximately normal to flow. Footings supported by piling in a sand bed channel. Abutments are spillthrough, protected with concrete aprons.

Drainage area, 44,340 mi^2 (114,840 km^2), of which 9,240 mi^2 (23,925 km^2) are non-contributing; bankfull discharge, about 80,000 ft^3/s (2,264 m^3/s); valley slope, 0.0002; channel width, 500 ft (152 m). Stream is perennial, regulated, sand bed, in valley of low relief, wide flood plain. Channel is meandering, wider at bends, point bars, probably incised, cut banks general, silt-clay banks.

Hydraulic problems and countermeasures:

Pre-1967 Bridge constructed (fig. 302) at meander bend; the outside (right) bank of the river is unstable and shows evidence of slumping and recession (fig. 303).

1969 Bank erosion was continuing (fig. 304) and concrete rubble was placed along the bank. A sheet-pile revetment was constructed around pier 6, which is nearest to the right bank, and at the right-bank abutment but erosion of the right bank continued.

1977 Plans were made for patented "spur jetties" (fig. 305) to be placed along the right bank upstream from the bridge site to prevent further lateral erosion. This type of spur is patented by K. W. Henson of Hold-That-River Engineering Co., Houston, Texas.

Discussion: Use of steel sheet-piling and concrete rubble as bank and abutment revetment did not prevent continued lateral erosion at a meander bend on the Brazos River. Bank protection is dificult because of the height of the banks (about 30 ft or 9 m) and the tendency of the banks to slump.

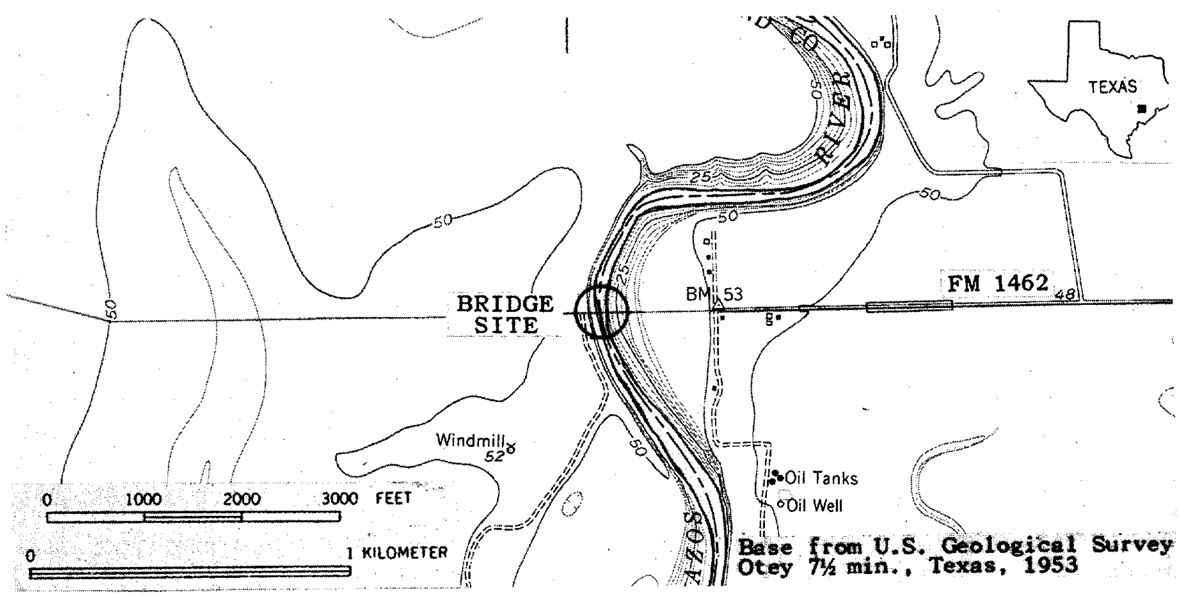


Figure 301. Location of Brazos River at FM-1462 near Rosharon, Texas.

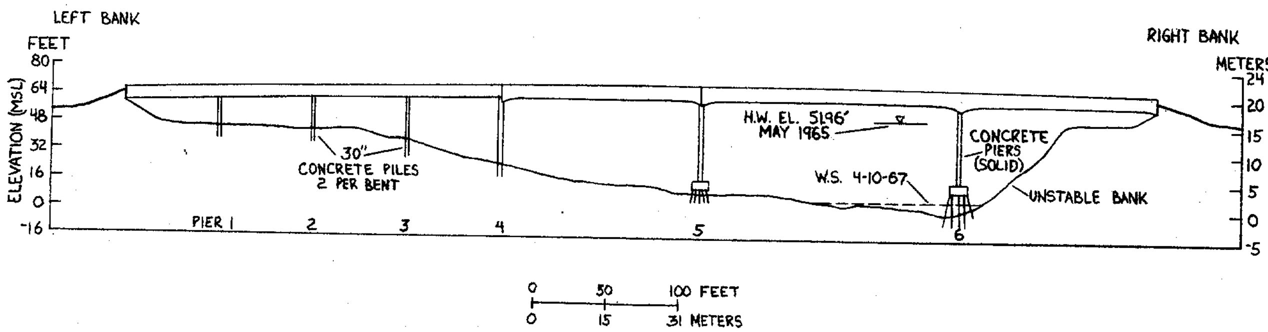


Figure 302. Cross section of Brazos River channel at bridge, in 1967.

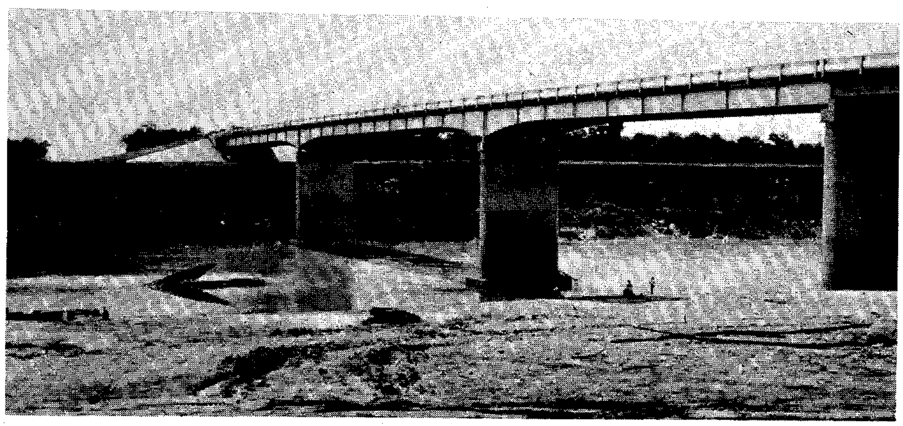


Figure 303. View from left bank showing erosion of right bank at bridge site in 1966. (From Texas Dept. of Highways.)

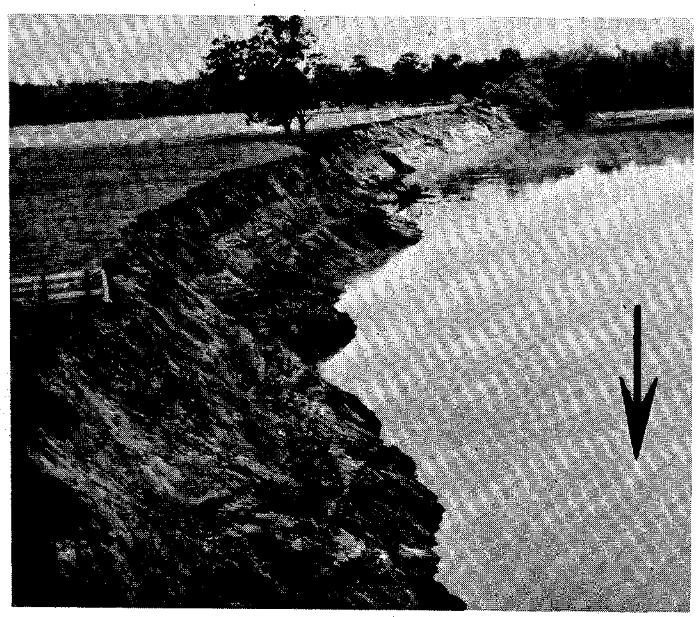


Figure 304. View upstream from bridge showing unstable right bank of Brazos River in 1969. (From Texas Dept. of Highways.)

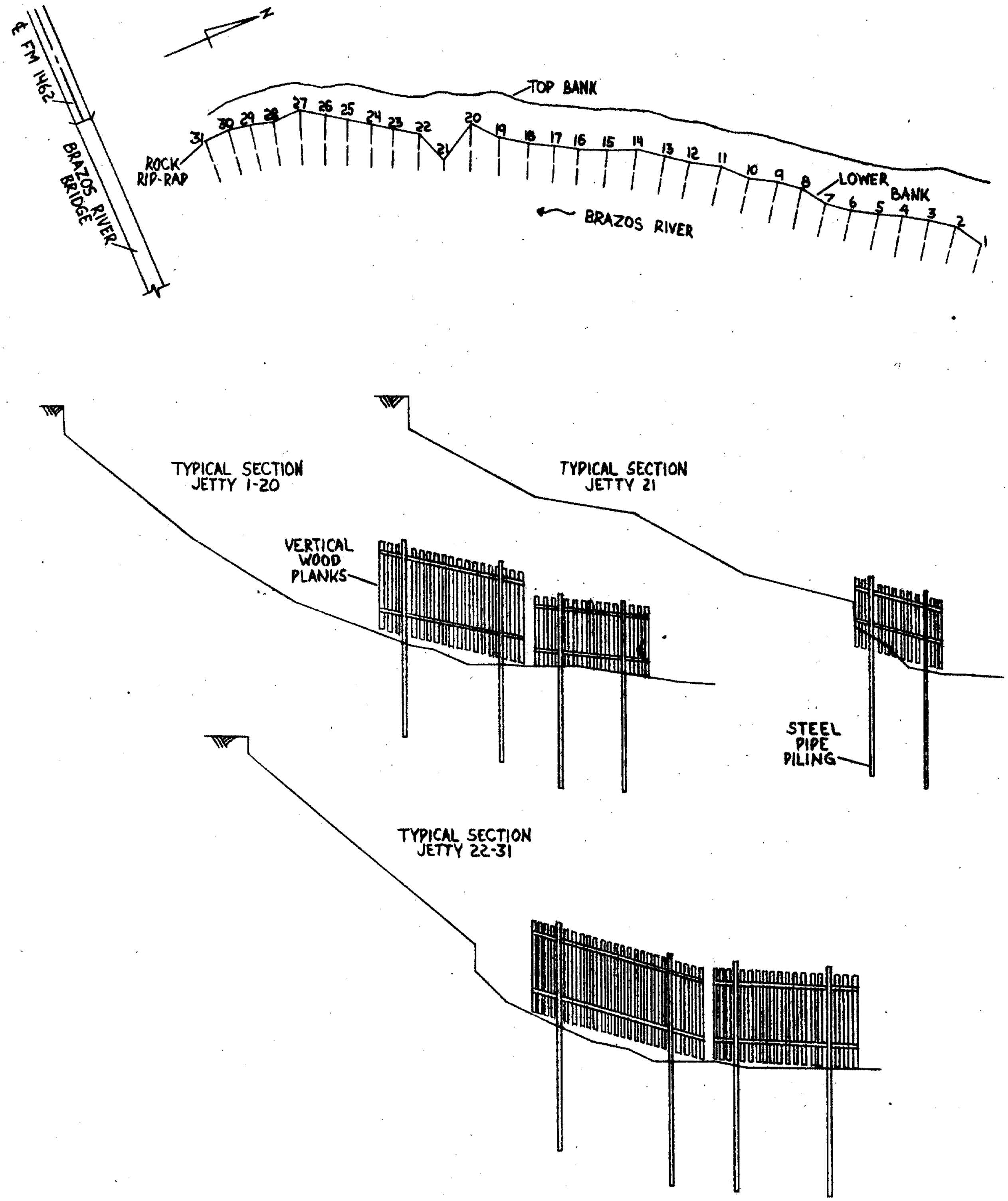


Figure 305. Plan for "spur jetty" installation at FM-1462 crossing, Brazos River.

SITE 132. COLD CREEK AT SR-71 NEAR PONTOTOC, TEX.

Description of site: Lat 30°53', long 98°54'. Concrete bridge, length 158 ft (48 m), spans of equal length supported on steel pile bents. Bridge and piers are alined with flood flow. Abutments are spillthrough, supported by piles.

Drainage area, 17 mi^2 (44 km^2); valley slope, 0.0093; cahnnel width 50 ft (15 m). Stream is ephemeral, alluvial, sand-gravel bed, in valley of moderate relief, narrow flood plain. Channel is straight, locally braided.

Hydraulic problems and countermeasures:

1965 Bridge built, designed for a 25-yr R.I. flood. To permit full penetration of steel piles, pilot holes were drilled. The channel bed was regraded to increase the flow capacity and to confine the flow between abutments. Bridge abutments were protected with concrete slope paving.

1976 Flood of June 4, 1976 exceeded the capacity of the new graded channel and some flow overtopped the bridge or utilized bypass channels. The R.I. of the 1976 flood is over 100 yr. Damage to the bridge included abrasion and impact damage of the upstream piles in a bent (fig. 306) and damage to the guardrail by floating debris. The guardrail design provided minimal resistance to flood flow when the bridge was overtopped. A cross section of the channel surveyed at the downstream side of the bridge in December 1976 indicates a maximum of about 2 ft (0.6 m) of scour has occurred since construction of the bridge.

1977 Damage to the road surfacing was repaired, but no additional countermeasures were installed.

Discussion: The bridge piling were embedded in sandstone that provided resistance to local scour and to the effects of impact or lateral forces associated with submergence of the bridge structure. Several features of the bridge facilitated the passage of flood flows when the bridge was overtopped. The guardrail and support posts provided minimal resistance to flow. The bridge superstructure consists of a flat slab with minimal depth, about 1 ft. (0.3 m) thickness, that reduced the amount of lateral force on the bridge and the potential for lodqment of debris.

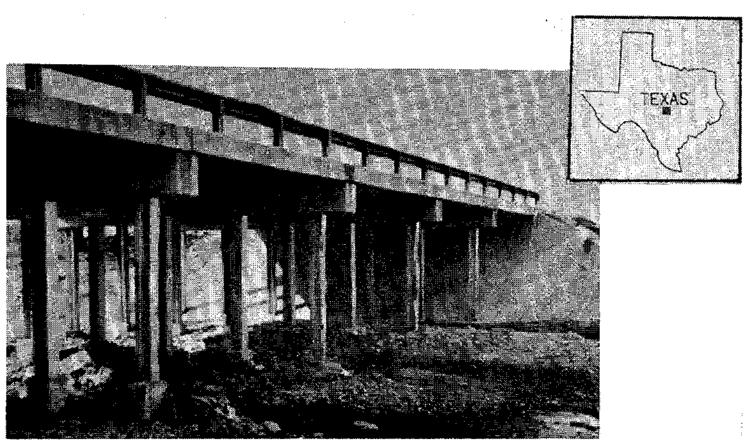


Figure 306. Upstream side of Cold Creek bridge, showing abrasion and impact damage to pile bents.

SITE 133. COLORADO RIVER AT US-90 AT COLUMBUS, TEX.

Description of site: Lat 29°42', long 96°32', location as shown in fig. 306a. Steel truss bridge, length 762 ft (232 m), supported by webbed piers having a pair of round columns, apparently founded on piles. Abutments are spillthrough. The bridge is located on a meander and the bridge and piers are skewed about 20 degrees to flood flow.

Contributing drainage area, 28,190 mi^2 (73,012 km^2); bankfull discharge, about 100,000 ft^3/s (2,830 m^3/s); channel width, about 300 ft (91 m). Stream is perennial but flashy, alluvial, sand bed, in valley of low relief, wide flood plain. Channel is meandering, locally braided, random width variation.

Hydraulic problems and countermeasures:

Pre-1939 Bridge constructed (fig. 307). About 50 ft (15 m) of bank recession occurred on the outside of the channel bend (right bank) between 1939 and 1941.

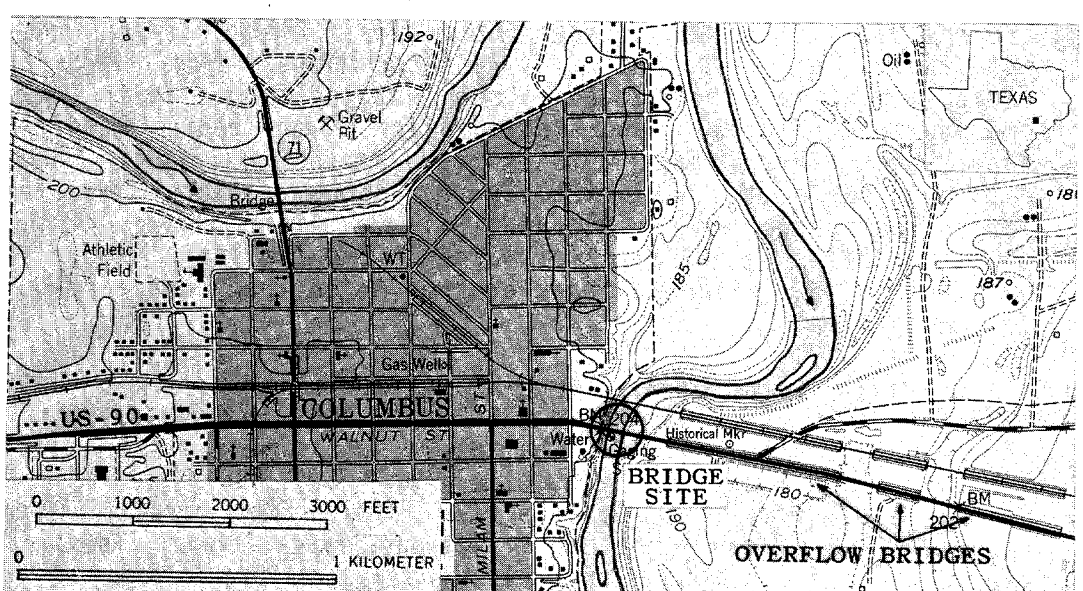
1944 Continuing lateral erosion of the right bank and potential damage to the bridge abutment required construction of bank stabilization measures (fig. 308). According to records of the Texas Department of Highways, "a total of 250 jetty units, J-2 (type A) were placed in 1944. Each unit consisted of three 3 in x 3 in x 1/4 in x 16 ft steel angles and number 6 lacing wire secured by 3/4 in wire cable to railroad ties anchored in the bank. The lacing wire runs through holes in the legs of each unit, apparently for additional resistance to the river current". A plan of the bridge site with the location of the jetty field is shown in fig. 308.

1977 Field inspection of the bridge site showed that the bank is stabilized and the steel jacks are in sound condition. Discharge measurements made from a cableway about 30 ft (9 m) downstream from the bridge indicate notable accumulation of sediment and debris in the former erosion area (fig. 307) and stabilization of the bank. Records of annual peak flows between 1939 and 1976 at the bridge site, based on data obtained at the Geological Survey gaging station, indicate that large floods occurred in 1940 and 1941 with eight subsequent floods of 55,000 ft^3/s (1,558 m^3/s) or larger. The two large floods may have initiated the lateral bank erosion which then continued during following floods.

An analysis of 15 discharge measurements made between 1949 and 1977 for flows of 12,600 ft^3/s (356 m^3/s) to 90,300 ft^3/s (2,557 m^3/s) at the bridge site indicates that pier 6 near the right bank (fig. 307) tends to cause localized high velocities (fig. 309). Maximum point velocities in a vertical up to 9 ft/s (3 m/s) were observed within 100 ft (30 m) of the right bank. The concentration of flow between pier 6 and the right bank probably contributed to the lateral erosion problem.

The effect of the steel jack jetty field installed in 1944 is illustrated by the significant reduction in maximum flow velocities near the right bank (fig. 309). At a discharge of 60,000 ft^3/s (1699 m^3/s) the maximum flow velocity between pier 6 and the right bank was reduced from 8.4 ft/s (2.6 m/s) to 5 ft/s (1.5 m/s). Between 1944 and the mid-70's, deposition of material during floods partially restored the eroded bank, indicating that the steel jack jetty field is effective in stabilizing the bank. The maximum velocity observed at the bridge site was 8.9 ft/s (2.7 m/s), during a discharge of 90,300 ft^3/s (2,557 m^3/s), at a distance of 104 ft (32 m) from the right bank. For a comparable discharge and location in the stream, it is estimated the maximum velocity after construction of the jack jetty field would be about 5 ft/s (1.5 m/s).

Discussion: The bridge was constructed at a meander, and for a period of about 10 years, lateral migration of the meander caused erosion of the channel bank and potential exposure of the right-bank pier and abutment. This case history demonstrates the hydraulic problems that occur when piers constructed near a channel bank cause a concentration of flow between the pier and bank and higher than normal velocities. These higher velocities can cause significant lateral erosion (in this case over 50 ft or 15 m) of the bank over a period of several years. The steel jack jetty field reduced the velocity of flow from 9 ft/s (3 m/s) to 5 ft/s (1.5 m/s) near the streambank during large floods. Following construction of the jack jetty field, the eroded bank was built out about 20 ft (6 m) by the deposition of material. The reduction in flow velocities to about 4 or 5 ft/s (1 to 1.5 m/s) was apparently sufficient to induce deposition of sediment and growth of vegetation.



Base from U.S. Geological Survey Columbus 7½ min., Texas, 1957

Figure 306a. Location of Colorado River at US-90 at Columbus, Texas.

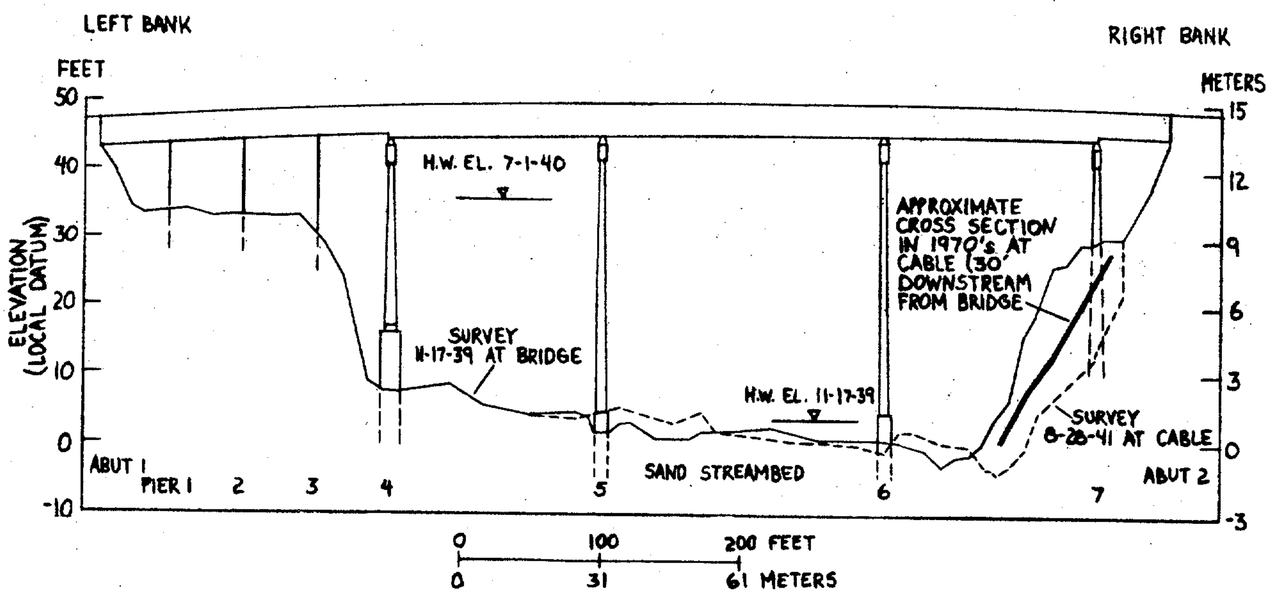


Figure 307. Changes in channel cross section at US-90 bridge between 1939 and the mid-1970's, Colorado River.

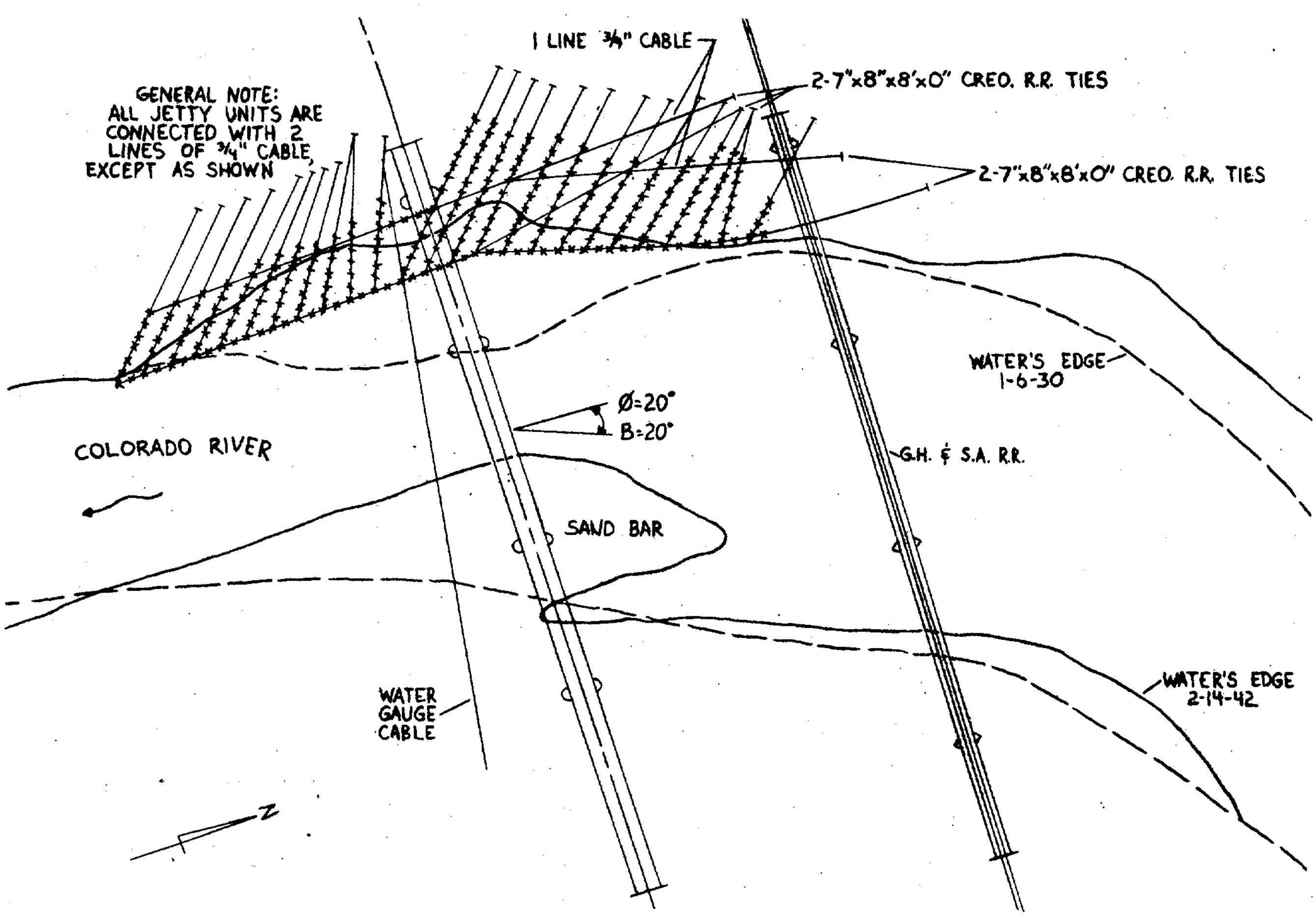
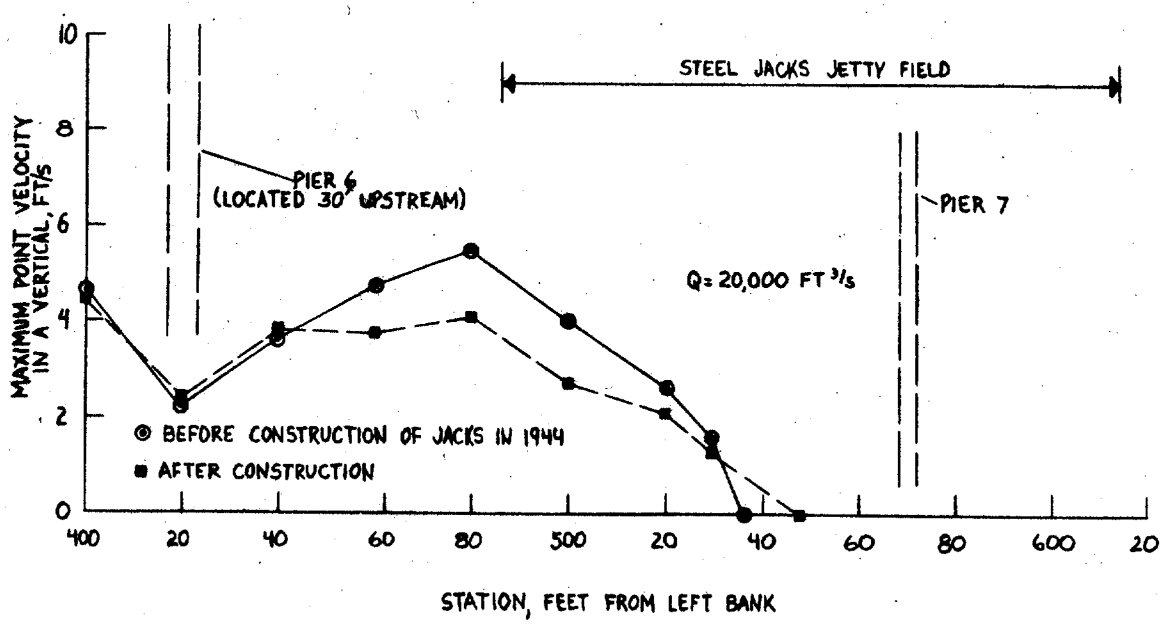


Figure 308. Plan of crossing showing location of steel-jack field, Colorado River.



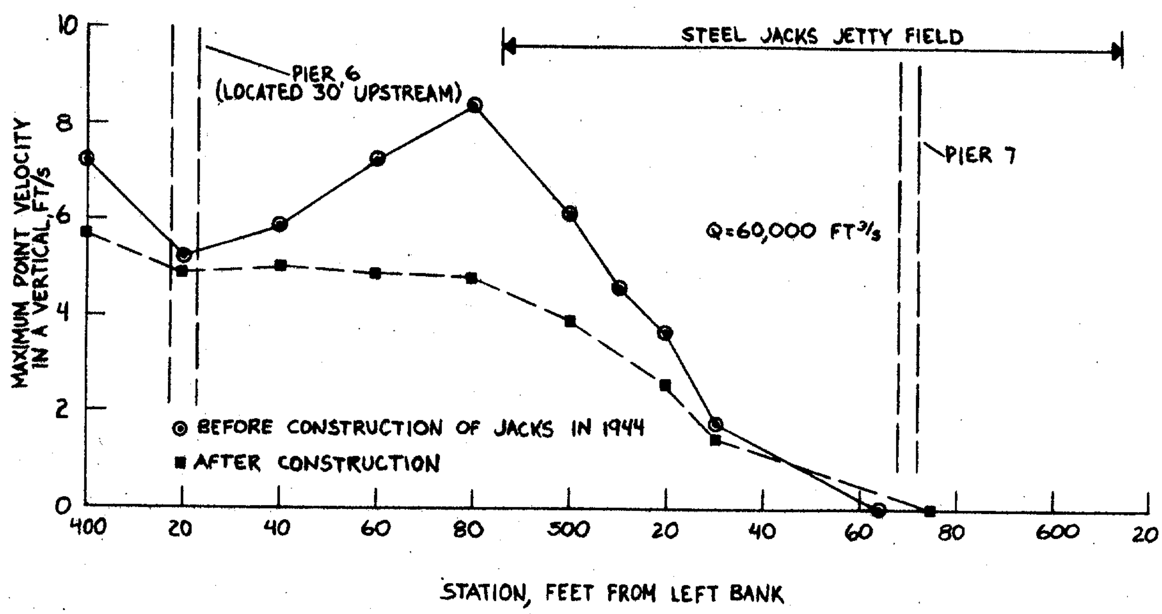


Figure 309. Effect of steel jacks on velocity of flow near streambank, Colorado River at US-90.

SITE 134. GILLS BRANCH AT SR-21 AT BASTROP, TEX.

Description of site: Lat 30°06', long 97°19'. Concrete bridge 80 ft (24 m) long, supported by two piers having dual concrete columns. Columns are placed in shafts drilled about 30 ft (9 m) below the streambed. Abutments are spill-through. Flows are alined with the bridge and piers.

Valley slope, 0.007; channel width, about 50 ft (15 m). Stream is ephemeral, alluvial, sand-gravel bed, in valley of low relief, little or no flood plain. Channel is artificially straightened at bridge.

Hydraulic problems and countermeasures:

1956 Bridge built across a relocated section of the channel (fig. 310). The channel and bridge abutments were lined with concrete (fig. 311) to prevent scour and lateral erosion.

1973-74 Excessive hydrostatic pressure built up during heavy rainfall caused uplift of the concrete floor in the new channel. After the flood, the damaged concrete was broken up and left in place (fig. 312). The broken concrete lining (rubble) increased the bed roughness and reduced flow velocity. Weepholes would probably have prevented the buildup of excessive hydrostatic pressure and uplift of the concrete.

1977 A scour hole developed at the downstream end of the concrete channel (fig. 313). The scour hole serves as an energy dissipator and is at least 4 ft (1 m) deeper than the lined channel invert.

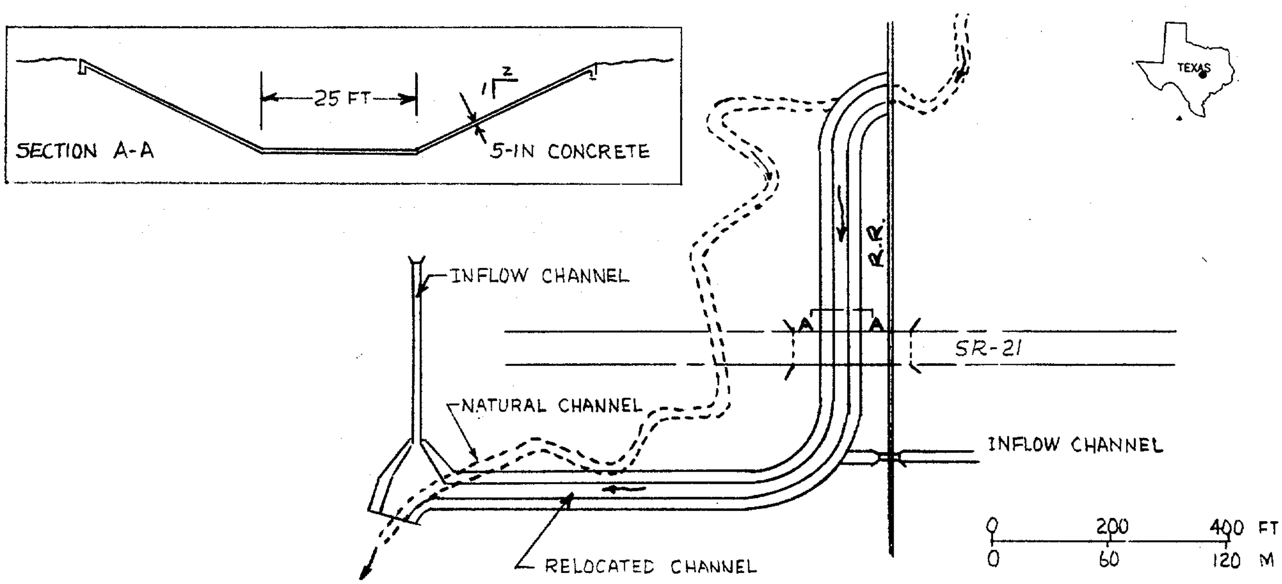


Figure 310. Plan and cross section of relocated channel at SR-21 crossing, Gills Branch.

Discussion: The concrete-lined channel apparently was not designed for the possibility of excessive hydrostatic pressure causing uplift of the lining. The relocated channel, with fewer bends and a shorter length, had a steeper gradient and higher flow velocities than the original channel. A scour hole or plunge pool developed at the point of intersection of the old and new channels and served as an energy dissipator for the increased flow velocities that develop in the new channel. In general, the design of relocated channels should consider the probability of scour and lateral erosion at the exit of a relocated channel. The location, (relative to the bridge), and ultimate size of the scour hole should be evaluated. Channel stabilization measures at the potential scour hole, such as revetment, may be needed.

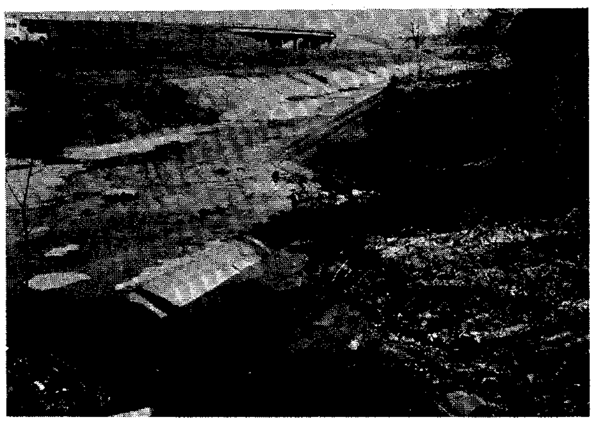


Figure 311. Upstream view of concrete-lined channel, in 1976. Concrete baffles in channel (foreground) are intended to retard flow velocity.

Figure 312. View in 1976 of concrete-lined channel and broken concrete rubble left in place after damage caused by uplift during flood.

Figure 313. View in 1976 of scour hole at downstream end of concrete-lined channel

SITE 138. KLICKITAT RIVER AT SR-142 NEAR KLICKITAT, WASH.

Description of site: Lat 45°50', long 121°05'. Precast, pretensioned-concrete bridge, about 180 ft (55 m) long, two spans supported by one gabion-protected cantiliver-type pier located in mid-channel. Abutments are spillthrough with gabions and wingwalls, protected by dumped rock riprap. The bridge is skewed about 30 degrees to the flow.

Drainage area, 1,017 mi^2 (2,634 km^2); valley slope, 0.0038; channel width about 200 ft (77 m). Stream is perennial, non-alluvial, gravel bed, in valley of high relief, little or no flood plain. Channel is sinuous, meanders deeply incised into volcanic rocks of Columbia Plateau, locally braided.

Hydraulic problems and countermeasures:

1954 Original bridge built, about 140 ft (43 m) in length, prestressed-concrete with two spans of unequal length. One round-nose wall-type pier was located near the channel bank. Abutments were spillthrough with wing walls. Pier protected with rock riprap and bridge was alined with flow. Design flood of 40-yr recurrence interval.

1974 During a flood (discharge about 40,000 ft^3/s or 1,133 m^3/s, R.I. estimated at 170 yr), the right-bank abutment washed out and air was trapped under the bridge between the prestressed-concrete beams, which had a thickness (depth) of about 3 ft (1 m). The bridge was partially submerged, water piled up at the upstream side, and the bridge floated downstream for a distance of 175 ft (53 m). No damage occurred at the adjacent bridge (fig. 314)

1975 New bridge constructed 180 ft (55 m) in length, center pier and abutments protected by gabions and rock riprap (fig. 315).

1977 The new bridge and countermeasures have not been tested by a major flood.

Discussion: Anchor bolts are recommended by Bradley (1973, p. 44) as a precaution against displacement of the bridge superstructure by a combination of buoyancy and dynamic forces. The presence of gabion-protected piers and abutments, and an enlarged waterway opening, may improve the performance of the new bridge in extreme floods. Accumulation of debris, or channel changes upstream from the island (fig. 314), may have caused a change in the distribution of flow between the two bridges during the 1974 flood. Increased flows in the left bank channel may have exceeded the capacity of the bridge causing subsequent failure. Bridges across multiple channels should be designed for the possibility of channel changes and the need to convey a much greater proportion of the total flow than normally expected.

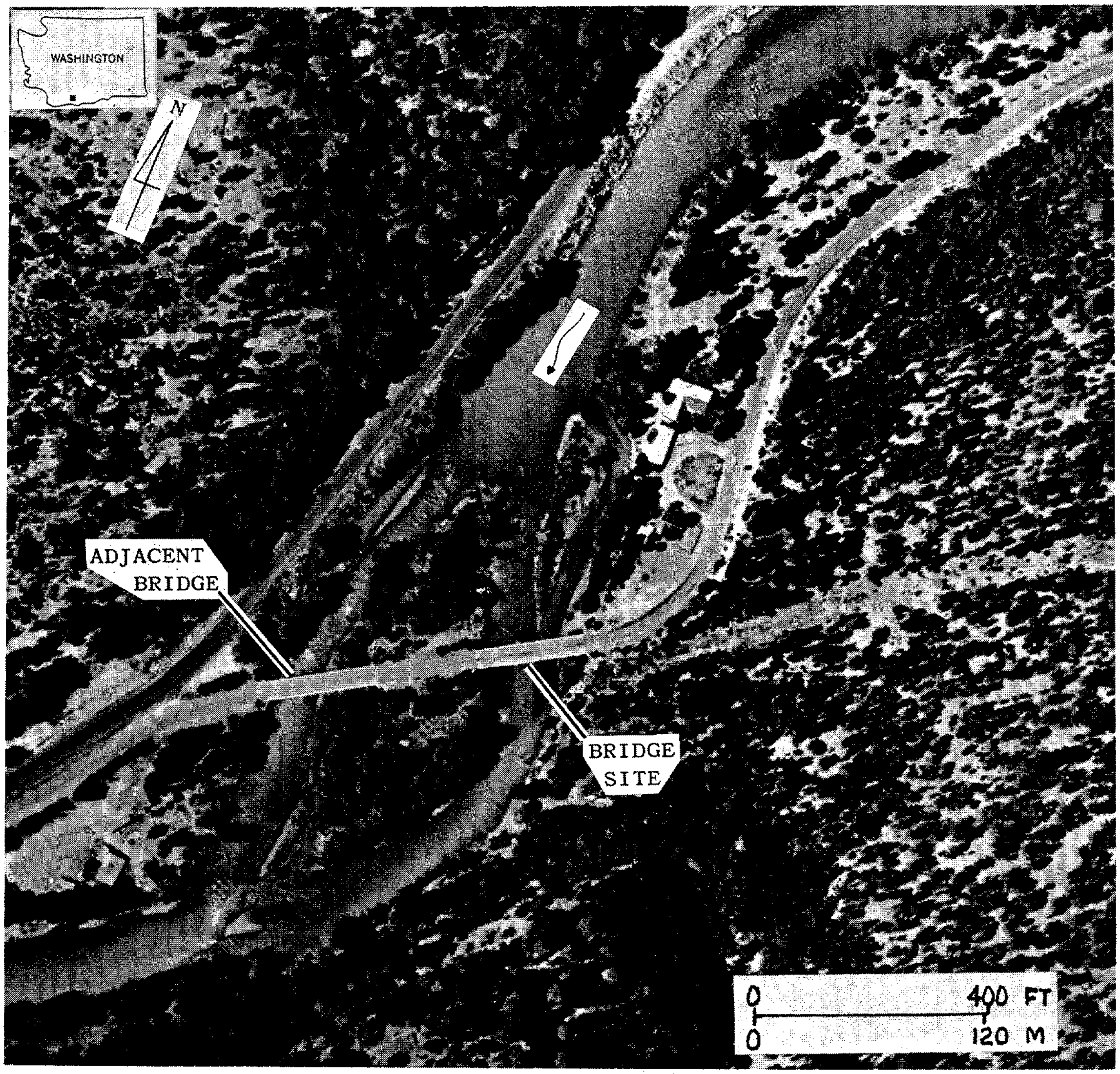


Figure 314. Aerial photograph in 1966 of bridges over Klickitat River at SR-142. (From Washington Dept. of Highways.)

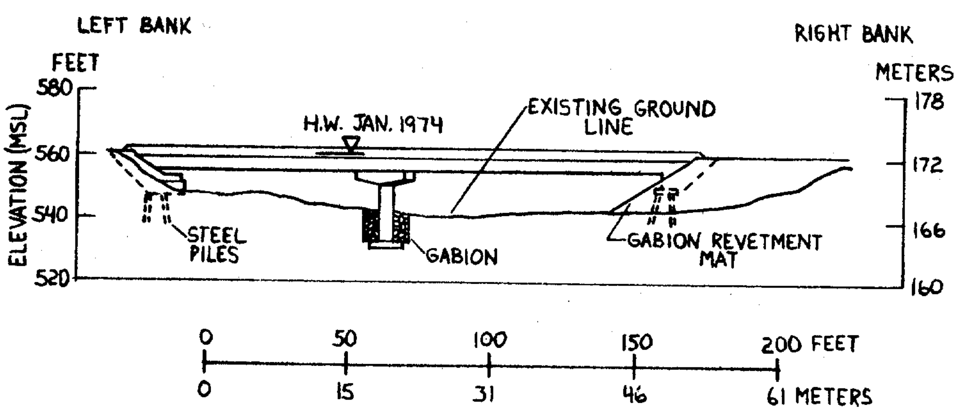


Figure 315. Details of bridge and countermeasures, Klickitat River at SR-142.

SITE 139. COLUMBIA RIVER AT SR-173 AT BREWSTER, WASH.

Description of site: Lat $48^{\circ}05'$, long $19^{\circ}47'$, location as shown in fig. 316. Steel-girder bridge with concrete deck, length 1,235 ft (376 m), seven round-nosed concrete wall-type piers. Piers 3 - 6 were founded on bedrock, pier 2 was founded on spread footing in hardpan, which became undermined. Abutments are spillthrough.

Drainage area, 83,000 mi^2 (214,970 km^2); bankfull discharge, about 58,000 ft^3/s (1,641 m^3/s); valley slope, 0.00045. Stream is perennial, semi-alluvial, little or no flood plain. Flow is regulated by Grand Coulee Dam. Crossing is at a sharp, asymmetrical contraction of the natural channel.

Hydraulic problems and countermeasures:

1927 Timber-deck, truss bridge built.

1948 Flood, R. I. approximately 100 yr, undermined pier 2 causing the pier to settle, elevating the center span, and causing adjacent spans to be laterally twisted. Structural repairs were made and rock riprap dumped around the piers to prevent additional scour (fig. 317).

1949-66 Several major floods occurred; no scour problems were detected.

1967-68 Bridge destroyed by fire. A new concrete bridge was built using old piers 1, 3, 4, 5 and 6 but not 2 and 7.

1977 No further problems have been noted.

Discussion: Rock riprap placed around piers to prevent local scour was effective. Scour of alluvial bed material around piers occurred when velocities were about 10 ft/s (3 m/s), even though the Froude number was only about 0.3.

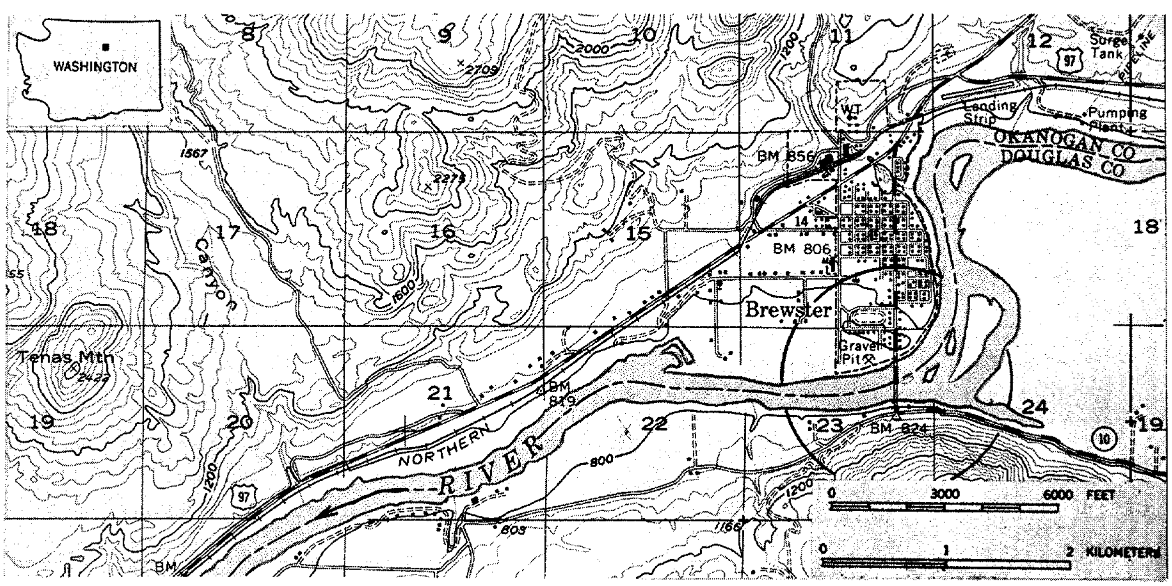


Figure 316. Location of SR-173 crossing (circled), Columbia River. (From U.S. Geological Survey Brewster, Wash., 15' quadrangle, contour interval 80 feet, 1957.)

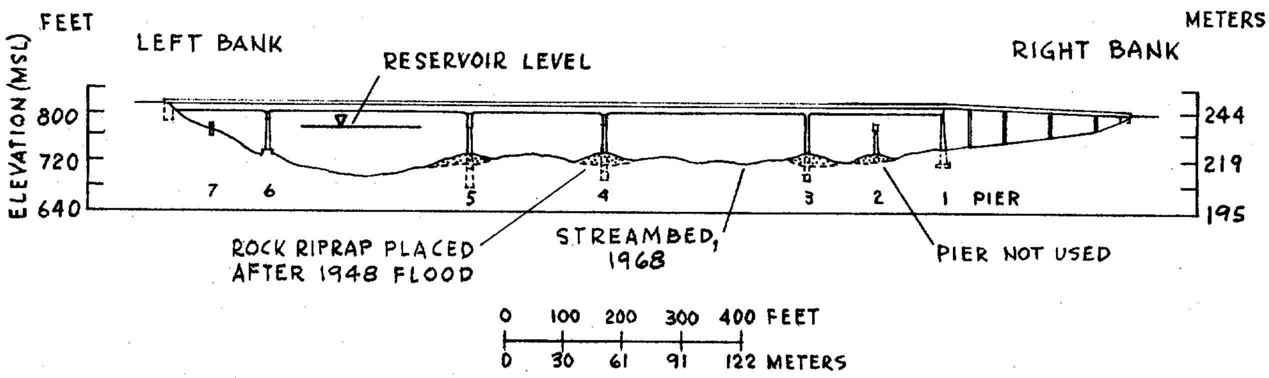


Figure 317. Elevation of SR-173 bridge, built in 1968.

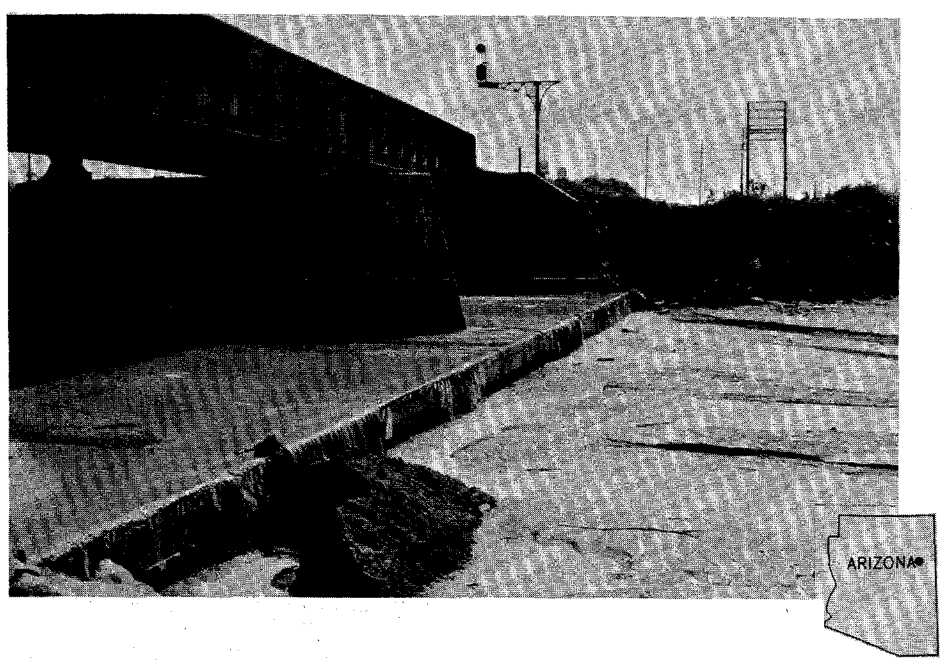


Figure 318. View of concrete cutoff wall built to prevent scour around piers. Note lateral erosion of left bank at the downstream side of abutment.

SITE 141. PUERCO RIVER AT ATCHISON, TOPEKA, AND SANTA FE RAILROAD NEAR CHAMBERS, ARIZ.

Description of site: Lat 35°11', long 109°27'. Steel plate-girder railroad bridge, 172 ft (55 m) in length, supported by one concrete pier 10 ft (3 m) wide. Bridge is normal to flow. Type of pier foundations is unknown. Abutments are vertical.

Drainage area, 2,160 mi^2 (5,600 km^2); bankfull discharge, about 8,000 ft^3/s (226 m^3/s; valley slope, 0.00256; channel width, about 180 ft (55 m). Stream is ephemeral, alluvial, sand bed, in valley of moderate relief. Channel is sinuous with erodible silt-sand banks, and a history of incision by headcutting.

Hydraulic problems and countermeasures:

Date	
Unknown	Pier footings were undermined by scour, and a concrete apron and cutoff wall were built to prevent further scour (fig. 318).
1971	Flood on Sept. 30 (discharge 17,800 ft^3/s or 504 m^3/s) caused no damage to the apron or cutoff wall. Flood flows created an enlarged channel (plunge pool) downstream from the bridge as flows expanded downstream from the bridge constriction. The channel is both wider and deeper downstream from the bridge. Lateral erosion of the left bank (fig.318) is associated with the flow expansion and the bank will probably require protection to prevent future damages.

Discussion: Channel bed scour or degradation caused by the bridge contraction at this site was successfully prevented by use of a concrete apron and cutoff wall, but an enlarged channel has developed downstream. Future floods at this site may cause continued scour and lateral erosion that will require deeper extension of the cutoff wall to prevent undermining, and protection of the channel banks downstream from the bridge to prevent lateral erosion.

SITE 143. ATOSCOSA RIVER AT FM-99 NEAR WHITSETT, TEX.

Description of site: Lat 28°37', long 98°17'. Steel-girder bridge 266 ft (81 m) in length, main spans supported by concrete wall-type piers with rounded nose, and approach spans supported by steel-pile bents. Bridge alinement is normal to flow. All piers are founded on steel piles. Abutments, also founded on steel piles, are spillthrough.

Stream is perennial, alluvial, silt-sand bed, in valley of low relief, narrow flood plain. Channel is straight, incised, about 250 ft (76 m) in width.

Hydraulic problems and countermeasures:

1947 Bridge built with the approach embankment constricting some of the overbank flow. Abutments on both banks were reveted with concrete slope pavement (fig. 319).

1967 Flood from Hurricane Beulah, flows were highly constricted at the bridge site. The concrete slope pavement at both abutments was undermined (fig. 319) and general scour of the channel bed at the bridge exceeded 5 ft (2 m). Repairs following the flood consisted of pumping fill material into the cavities under the concrete slope pavement.

Discussion: The approach embankment on the flood plain caused a significant amount of flow contraction and resultant general scour at the bridge during a large flood. Both toe and end protection are needed to prevent future damage to the concrete slope pavement by scour and undermining.

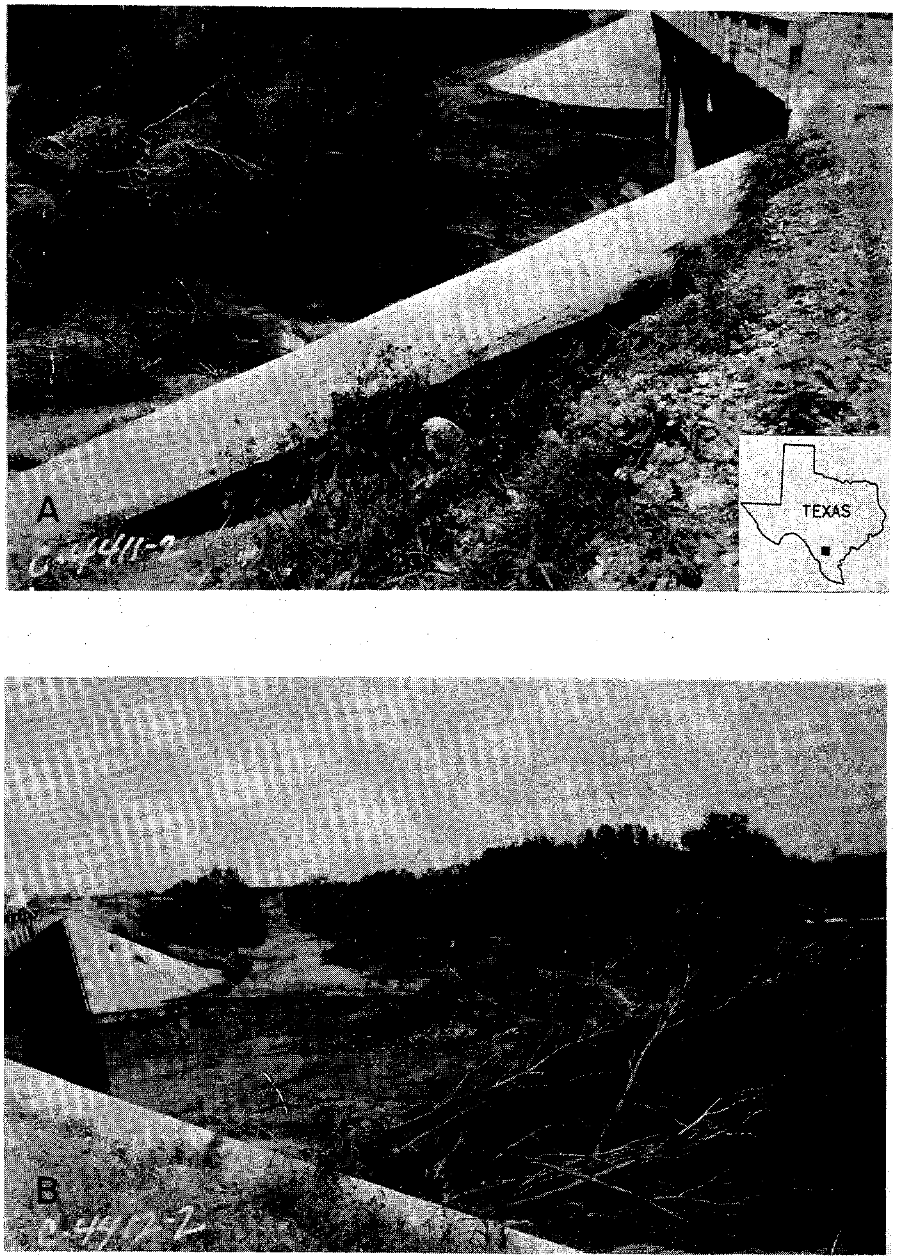


Figure 319. Undermining of concrete slope pavement during flood from Hurricane Beulah, 1967. A, upstream side of bridge; B, downstream side of bridge. (From Texas Dept. of Highways).

SITE 144. TUNIS CREEK AT I-10 NEAR BAKERSFIELD, TEX.

Description of site: Lat 30°53', long 102°22', location as shown in fig. 320. Dual prestressed-concrete bridges, 780 ft (237 m) in length, supported by wall-type piers. Bridge and piers skewed 32 degrees to flow. Abutments are spillthrough.

Valley slope, 0.0026. Stream is ephemeral, alluvial, gravel bed, a wide arid-region drainage channel.

Hydraulic problems and countermeasures:

Pre-1974 Dual bridges built across Tunis Creek, which flows parallel and adjacent to the Interstate (fig. 320) before reaching the bridge site. A spur dike, 150 ft (46 m) in length, was built at the left bank abutment to aline flows with the bridge opening. The spur dike was built as a quarter ellipse and faced with concrete pavement 5 inches (0.13 m) thick (fig. 321).

1974 Flood of September 1974 reached the top of the spur dike and overtopped the frontage road west of the bridge. The dike was reported as functioning properly, but flow velocity increased around the upstream end, undermined the concrete pavement and washed out a hole 6 ft (1.8 m) to 8 ft (2.4 m) deep and 60 ft (18 m) in diameter at the end of the dike (fig. 322). About 50 ft (15 m) of the spur dike washed out. The end of the spur dike was rebuilt using large rock from roadway excavation at the site.

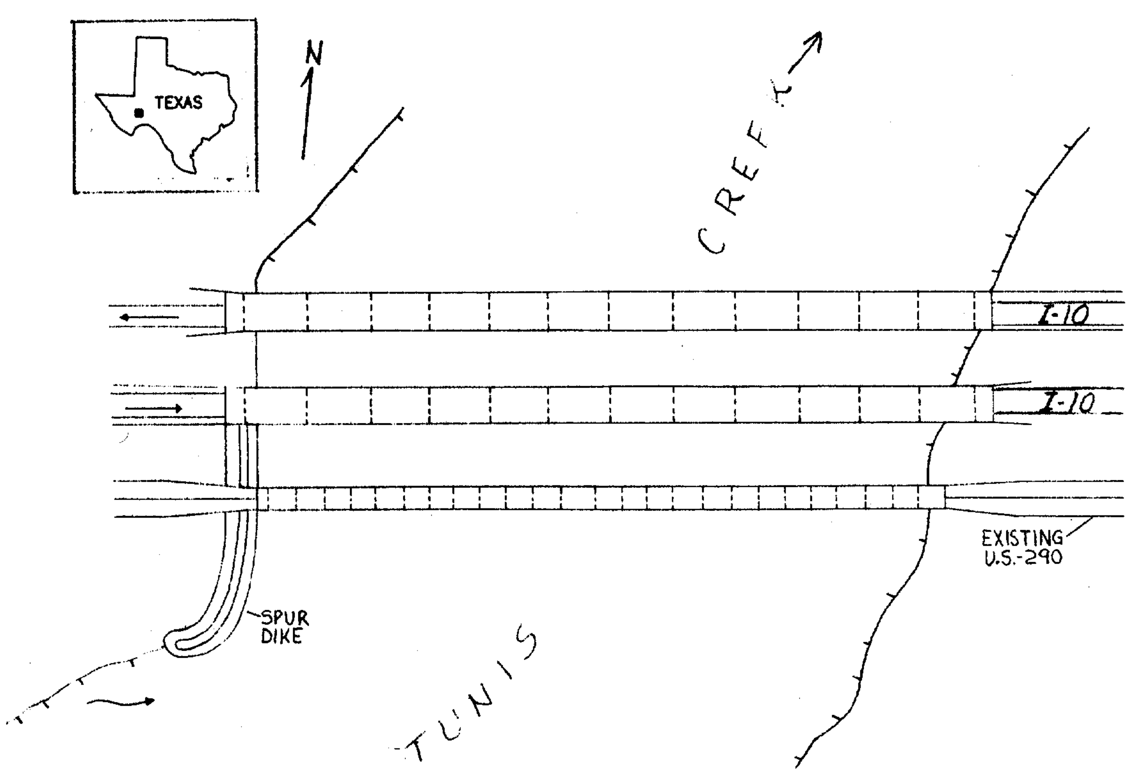


Figure 320. Plan of I-40 crossing at Tunis Creek.

Discussion: The spur dike, built to prevent lateral flow along the approach embankment and to aline flows with the bridge opening, performed as intended during a flood which reached the top of the dike. However, the spur dike was damaged and a scour hole developed at the upstream end as flows passed around the dike and high flow velocities developed. A concrete toe wall placed 3 ft (1 m) below the ground line to prevent undermining of the concrete pavement on the spur dike was unsuccessful during the 1974 flood.

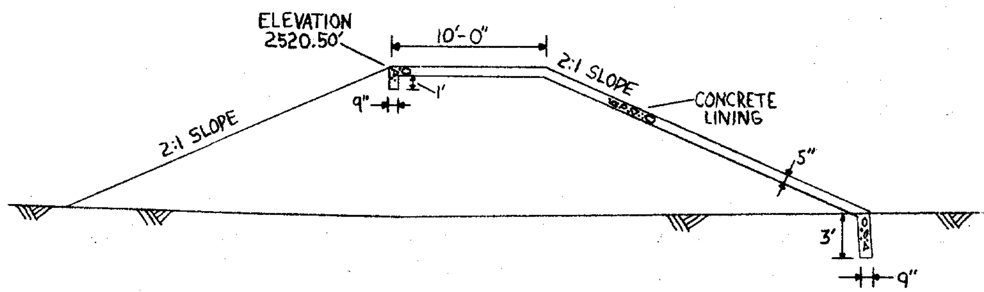


Figure 321. Details of spur dike construction, Tunis Creek.

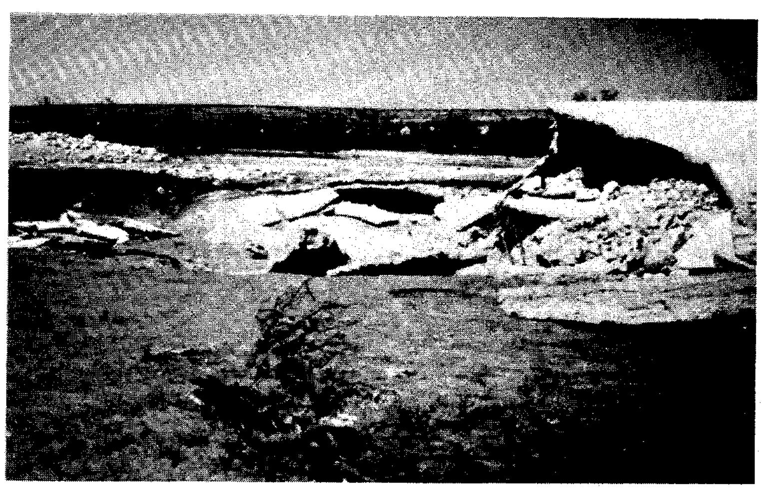


Figure 322. View of damaged upstream end of spur dike after flood of September, 1974. (From Texas Dept of Highways.)

SITE 145. BRAZOS RIVER AT FM-529 NEAR BELLVILLE, TEX.

Description of site: Lat 29°55', long 96°09'. Combination steel-plate girder and prestressed-concrete bridge, 1000 ft (305 m) in length, supported by four concrete column bents and three piers. Bridge and piers are skewed about 6 degrees to flood flow. Piers and abutments are founded on concrete piles. Abutments are spillthrough.

Valley slope, 0.0006; channel width, about 600 ft (91 m). Stream is perennial, alluvial, sand bed, in valley of low relief, wide flood plain. Channel is meandering, wider at bends, point bars, incised, silt-clay banks.

Hydraulic problems and countermeasures:

Pre-1970 Bridge built across Brazos River near meander, with no constriction of the main channel by the approach embankment.

1971 Lateral erosion of the right bank resulted in a loss of 85 (26 m) to 100 ft (30 m) of bank within three years after bridge construction. Control of the bank erosion problem consisted of placing sheet piles near the streambank and pier 2 to act as a bulkhead (fig. 323). The sheet piles were held in place by use of anchors and tie bars. Backfill was placed over the tiebars and anchors, and protected with riprap.

Discussion: A steel sheet-pile bulkhead was built to prevent undermining, by lateral bank erosion, of a pile-supported pier located near the streambank. Performance of this countermeasure was not evaluated.

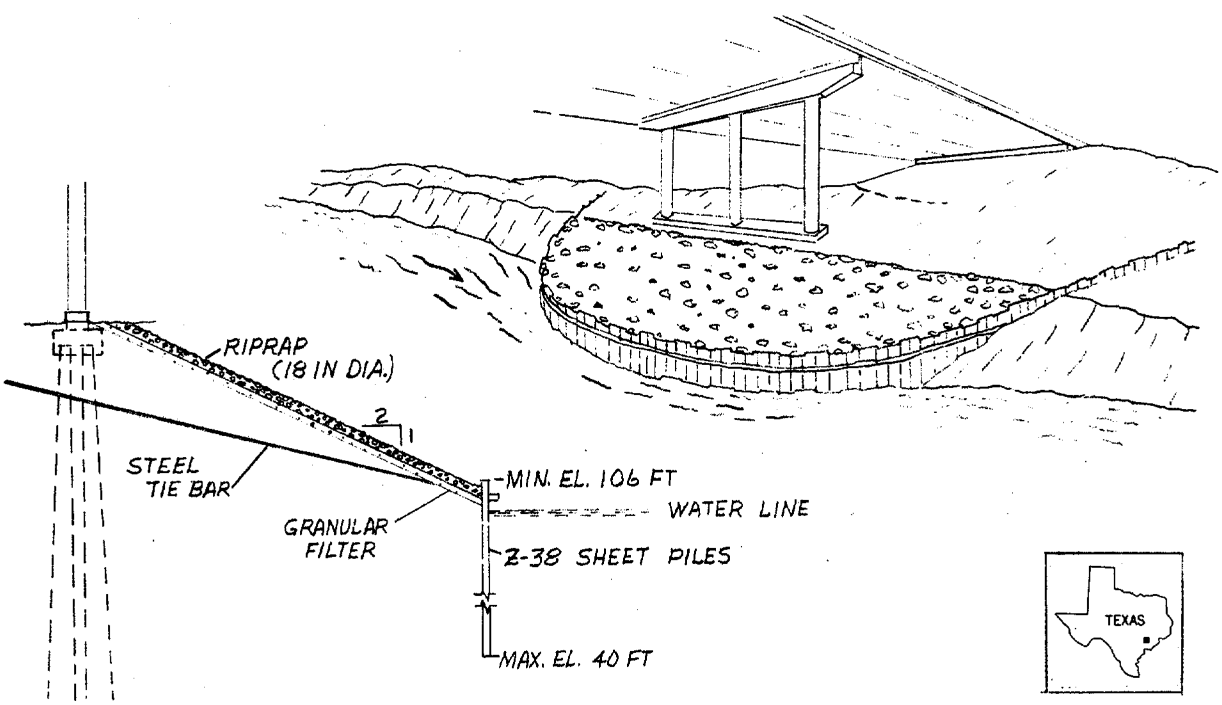


Figure 323. Cross section, left, and perspective sketch, right, of steel sheet-pile bulkhead at FM-529 crossing, Brazos River.

SITE 146. OVERFLOW CHANNEL, NUECES AND FRIO RIVERS AT US-281 NEAR THREE RIVERS, TEX.

Description of site: Lat 28°26', long 98°11', location as shown in fig. 324. Dual concrete-trestle relief bridges, length 334 ft (102 m), 11 spans supported by concrete-pile bents, across overflow channel about 1.2 miles (1.9 km) north of main channel. Flows are normal to the bridge opening. Abutments are spill-through, supported by piles and located near the channel banks.

Valley slope, 0.00058. Nueces River is perennial, alluvial, sand bed, in valley of moderate relief, wide flood plain. Channel is meandering, incised. Overflow channel, crossed by relief bridge, joins Frio and Nueces Rivers (fig. 324).

Hydraulic problems and countermeasures:

Prior to 1967 — Dual bridges (designated as overflow bridges "C") were constructed to pass overflow from the Nueces and Frio River on the left-bank flood plain. The downstream bridge is the older structure, and is of timber-pile and stringer construction.

1967 — Flooding during Hurricane Beulah in September caused a collapse of several spans near the left bank of the timber trestle bridge (fig. 325); flows were highly constricted by the dual overflow bridge openings (fig. 326). The effects of the constriction were more significant at the downstream bridge where, based on flow lines shown on fig. 326, only about 60 percent of the total bridge length was providing an effective opening or waterway for passage of floodflow. The larger amount of flow contraction occurring at the downstream bridge resulted in sufficiently high flow velocities to cause as much as 16 ft (5 m) of general scour below the original streambed at the bridge (fig. 327).

1968 — A new bridge, similar in length to the damaged bridge was built. Concrete piles were used and abutments on both banks were revetted with concrete slope paving (fig. 327).

Discussion: Bridges across the overflow channel caused significant lateral contraction of floodflow. Because the bridge waterway capacity was inadequate, general scour and lateral erosion at the contraction resulted. The contraction of flow at the dual bridges during flooding by Hurricane Beulah was caused by several factors: (1) The dual bridge itself caused an impediment to flow and reduced the hydraulic efficiency of the waterway between abutments. For example, the bridge piling and bents occupy space in the waterway. In addition, the pile bents caused turbulence of flow which affected the flow patterns. The amount of flow turbulence was also affected by the closeness (span length) of adjacent bents (close spacing of bents caused greater amounts of flow turbulence). The approach embankments forced water on the floodplain to move laterally along the embankment and then change directions at the bridge before passing through the opening. Normal flow patterns in the main overflow channel were affected by flows moving laterally along the embankment.

The hydraulic efficiency of the bridge opening would be improved if flow-control structures, such as spur dikes, were installed at the bridge abutments. Spur dikes would, in this case, move the location of maximum flow contraction from the downstream bridge upstream to a point near the upstream bridge. Assuming no blockage by drift, the full hydraulic capacity of both bridge openings could then be utilized.

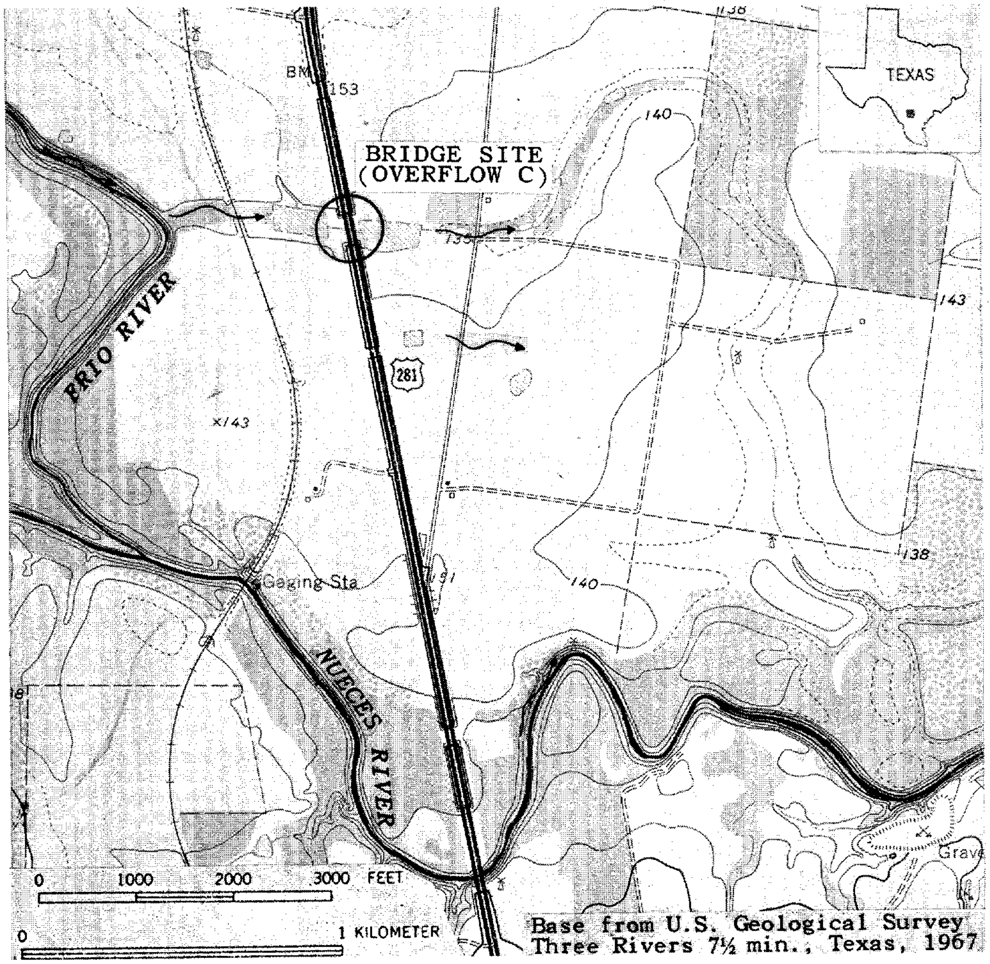


Figure 324. Location of overflow channel at Nueces and Frio Rivers.

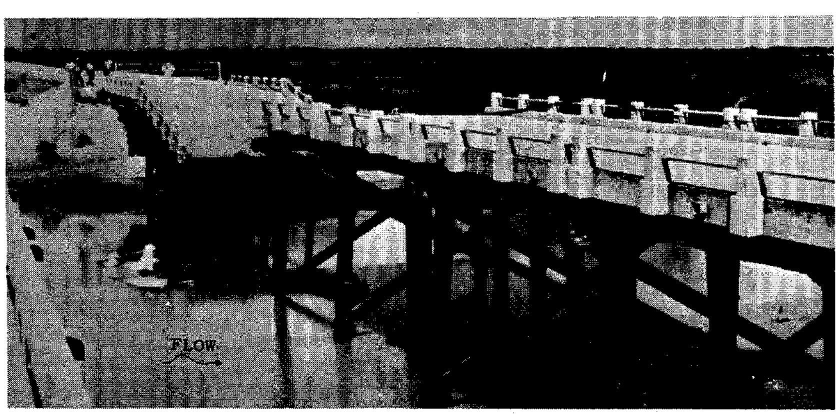


Figure 325. View of damaged bridge, looking north, after flooding by Hurricane Beulah in September 1967. (From Texas Dept of Highways.)

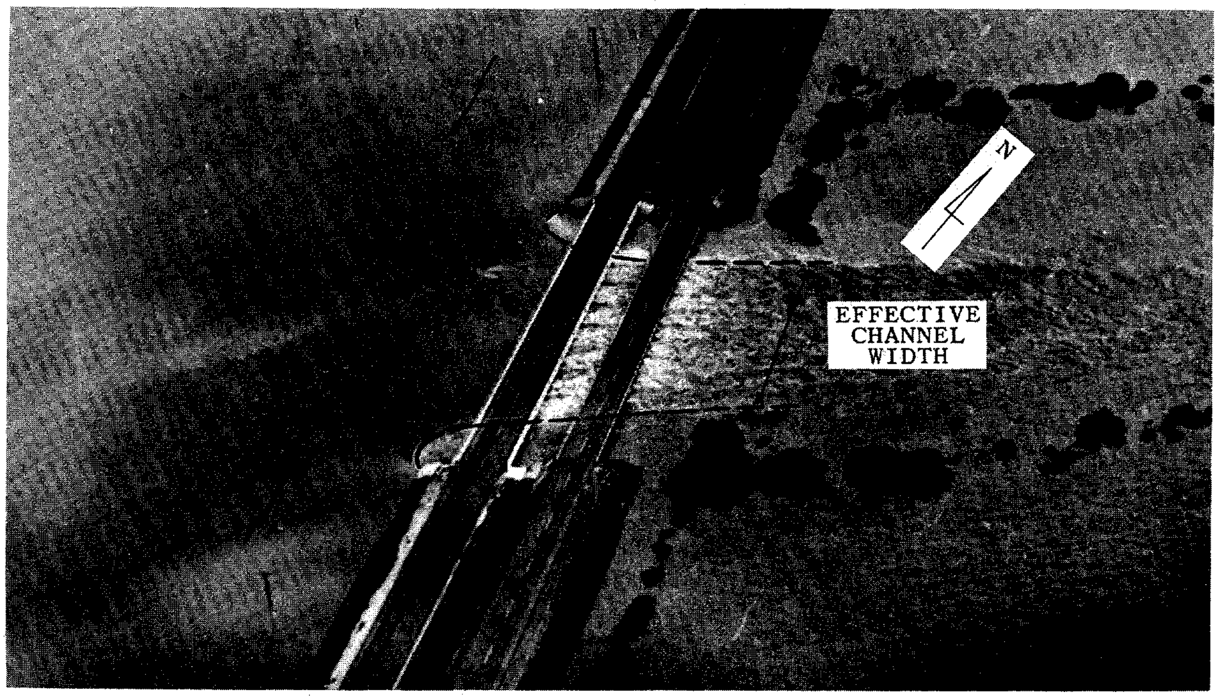


Figure 326. Aerial photograph, on which flow constriction at overflow bridge "C" is indicated, taken before collapse of spans of downstream bridge. (From Texas Dept. of Highways.)

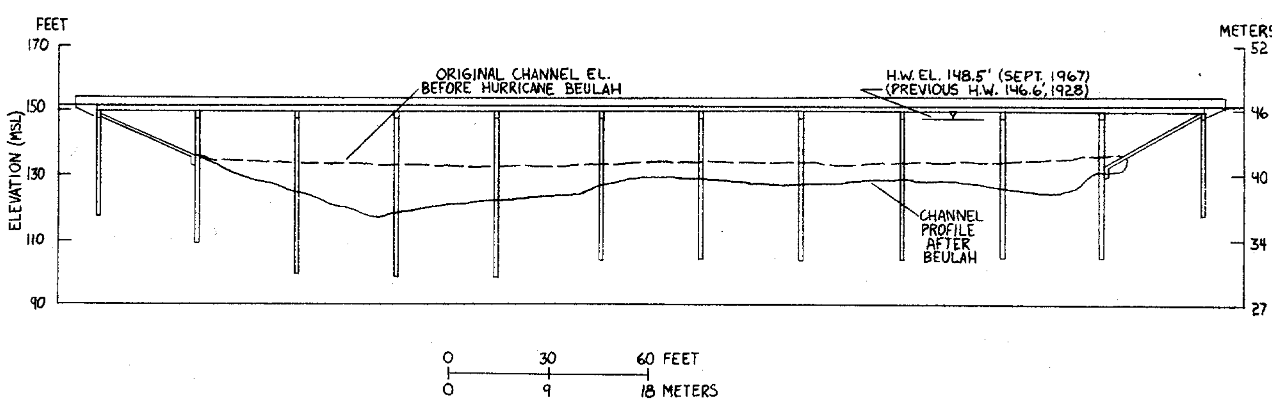


Figure 327. Cross section of overflow channel before and after flooding in September 1967.

SITE 148. MERRILL CREEK AT FM-1550 NEAR LADONIA, TEX.

Description of site: Lat 33°29', long 95°55', location as shown in fig. 328. Reinforced-concrete beam bridge, length 400 ft (122 m), 14 spans supported by 17 steel H-piles her bent. Bridge and piers are skewed 58 degrees to flow. Abutments are spillthrough.

Valley slope, 0.004; channel width, 350 ft (107 m). Stream is perennial but flashy, alluvial, sand-gravel bed, in valley of low relief, narrow flood plain. Channel is artificially straightened, probably incised.

Hydraulic problems and countermeasures:

1920's The North Sulphur River channel (to which Merrill Creek is tributary) was cleared and straightened for a distance of 36 miles (58 km). Following the work, the North Sulphur River channel apparently remained relatively stable until the 1950's. Since then, there has been as much as 5 ft (1.5 m) of channel degradation on the North Sulphur and the channel of Merrill Creek has enlarged over 400 percent at the bridge site, which is 1.7 mi (2.7 km) upstream from the confluence with the North Sulphur (fig. 328). The process of channel degradation has also affected the gradient of North Sulphur tributaries, such as Merrill Creek.

1953 Bridge constructed. The original channel was about 50 ft (15 m) wide with a flat bottom.

1957 Continued channel degradation at the bridge site caused settlement of one pier near the left bank. Bridge modifications included jacking up the pier, replacing timber piles with longer steel piles, extending the bridge with two 25 ft (7.7 m) spans on the left bank and one span on the right bank.

1975 Channel degradation and lateral erosion during high runoff caused the left bank near the bridge to erode (fig. 329) at a rapid rate. Between 1957 and 1975, the lowest part of the channel degraded over 7 ft (2 m), and the channel widened 75 ft (23 m).

About 1976 The bridge was again extended by adding a 25 ft (8 m) span at each end.

Discussion: Channel alinement and clearing work on the North Sulphur River apparently caused changes in Merrill Creek gradient and subsequent channel degradation and lateral erosion. The time interval between the channel change and return to channel stability may require more than 50 years. The amount of channel change and duration of instability depends on flow conditions and geologic features of the basin. In the construction of new bridges across an actively degrading or unstable stream channel, such as Merrill Creek, the possible occurrence of both channel bed lowering and lateral erosion should be considered. Although no countermeasures were applied except for lengthening of the bridge, this case is documented here because it is a typical example of hydraulic problems caused by stream channelization.

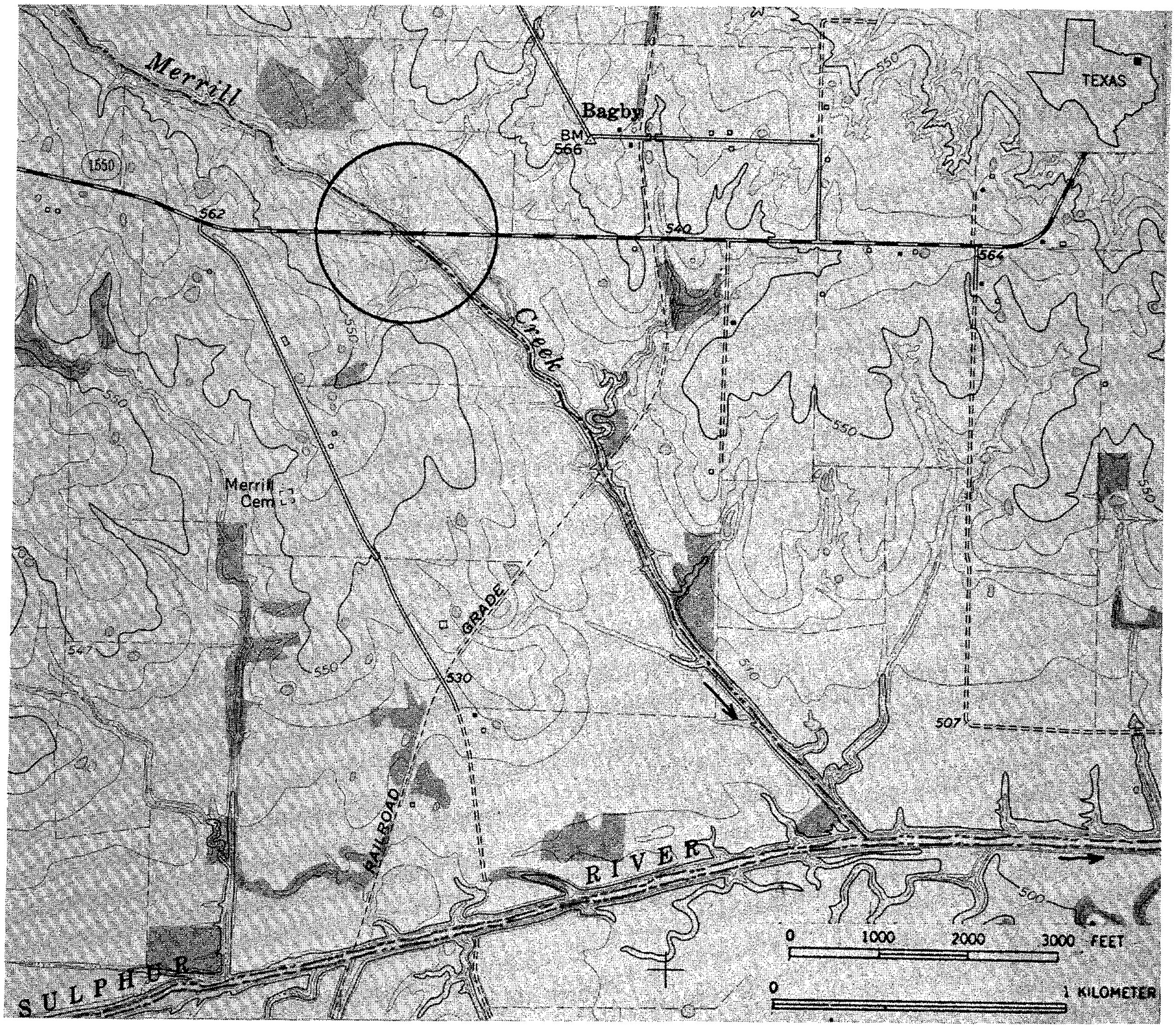


Figure 328. Location of FM-1550 crossing (circled), Merrill Creek, and confluence of Merrill Creek with North Sulphur River. (From U.S. Geological Survey Ladonia, Tex., 7.5' quadrangle, contour interval 10 feet, 1964.)

Figure 329. View in 1975 of eroded left bank upstream from bridge. (From Texas Dept. of Highways.)

CASE HISTORIES
PART II

SITE 150. RED DEER RIVER AT PH-27 AT SUNDRE, ALBERTA, CANADA

Description of site: Lat 114°46', long 51°49', location as shown in fig. 1. Bridge is 431 ft (131 m) in length, four precast, prestressed concrete-girder spans, well-type concrete piers with rounded nose, steel H-pile driven into shale beneath piers. Steel sheet-pile cofferdams at pier footings left in place and filled with concrete.

Drainage area, about 1,300 mi^2 (3,367 km^2); valley slope 0.005; channel width, about 500 ft (152 m). Stream is perennial, alluvial, gravel bed, in valley of moderate relief, narrow flood plain. Channel is straight, generally braided, cut banks local, sand-gravel banks, tree cover at 50-90 percent of bankline (fig. 1).

Countermeasures:

1959 Bridge and guide banks (spur dikes) built. Streamward side of guide banks revetted with concrete pavement, 4 in (10 cm) in thickness, 2:1 slope, riprap launching apron at toe of pavement (figs. 2A and B). Riprap in launching apron consists of rounded glacial boulders, median weight 250 lb (114 kg), placed in two layers, width of apron 34 ft (10 m) along straight side of guide bank and 52 ft (16 m) at curved ends. Height of right guide bank about 3 ft (1 m) above design high water level at bridge. Left guide bank about 6 ft (2 m) lower than right, to prevent formation of pocket of water behind it.

1965 Flood of June 18, high water level 3.2 ft (1 m) below water level at design discharge, maximum point velocities under bridge about 8 ft/sec (2.4 m/sec). Maximum scour depth of about 18 ft (5.5 m) measured at west pier in main channel, attributed in part to steel cofferdam at pier footing, but another factor was drift caught on an old pier 50 ft (15 m) upstream. No damage from the flood occurred at bridge or guide banks.

1977 Guide banks, including concrete pavement, in good condition, vegetation becoming established on launching apron (fig. 3).

Discussion: This case provides a good example of the use of guide banks (spur dikes) to confine and hold in position a wide braided channel. No undue amount of general scour has occurred in the excavated channel (fig. 2A), although the steel cofferdam at the pier footing has induced local scour.

Figure 1. Aerial photograph of Ph-27 crossing, Red Deer River at Sundre, at flood stage on June 19, 1965. Guide banks are indicated by arrows. (From Alberta Highways and Transport).

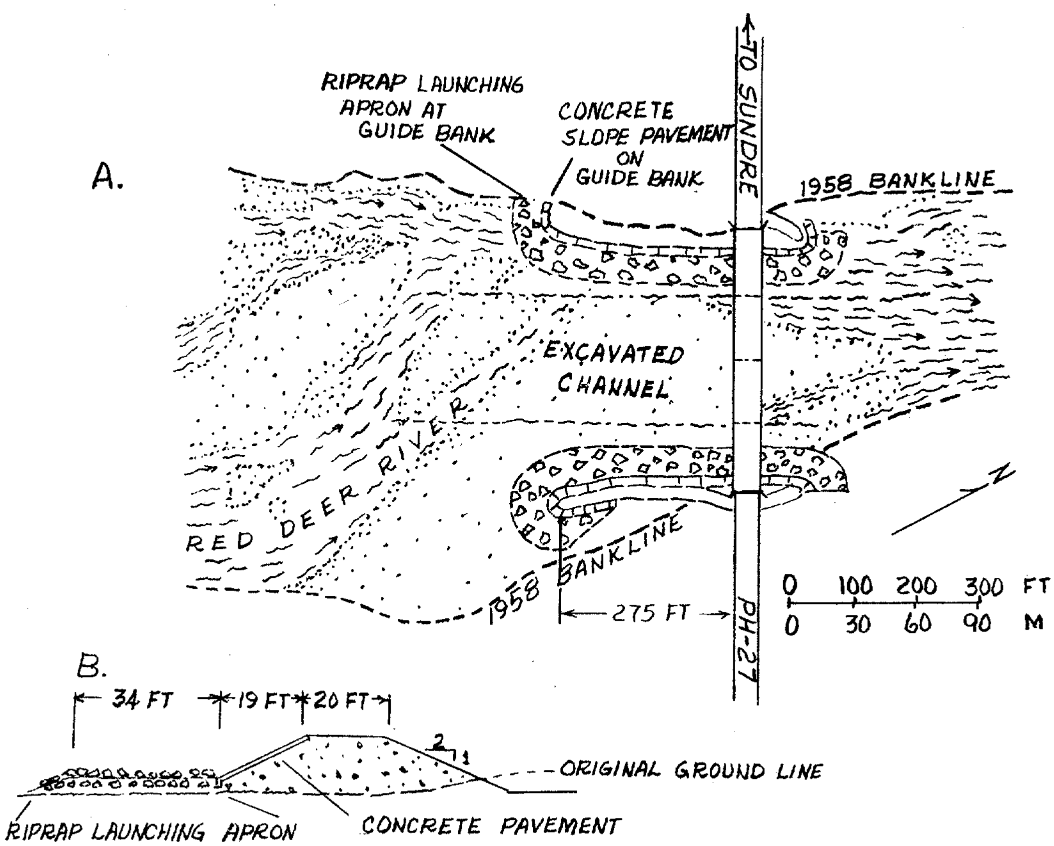


Figure 2. A, Plan sketch of training measures at PH-27 crossing, Red Deer River at Sundre. B, Cross section of guide bank.

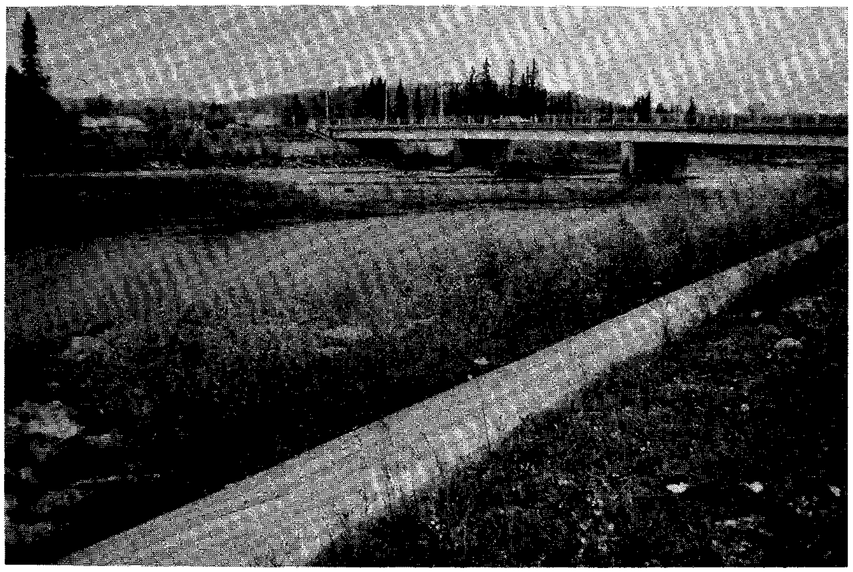


Figure 3. Downstream view of PH-27 bridge from right guide bank, in foreground, on September 29, 1977.

SITE 151. RED DEER RIVER AT SECONDARY ROAD 587 AT GARRINGTON, ALBERTA, CANADA

Description of site: Lat 51°56.5', long 114°29', location as shown in fig. 4. Bridge is 450 ft (137 m) in length, four 112-ft (34-m) precast prestressed concrete girders, concrete wall-type piers, round nose, piers battered and with curved metal shields on upstream side to prevent ice abrasion. Piers on spread footings set in dense gray shale. Left abutment fill slope strongly revetted, straight guide bank (spur dike) at right abutment.

Valley slope, about 0.004; channel width, about 500 ft (152 m). Stream is perennial, alluvial (but shale bedrock is at a depth of about 10 ft or 3 m beneath the streambed), gravel bed, in valley of low relief, narrow flood plain. Channel is sinuous, locally braided, generally anabranched (fig. 4), not incised, cut banks general, sand-gravel banks, tree cover on more than 90 percent of bankline.

Countermeasures:

1962 Bridge and river training measures completed. The dike system with its projecting spurs (fig. 4) was designed and built to confine the width of the stream, to maintain stream alinement, and to prevent the stream from bypassing the bridge. (Note that the river curves and follows the roadway downstream from the bridge). The spurs point upstream, rather than downstream. A "dead water" area is thus formed upstream from the spurs during flood, and revetment is required only at the tip of the spur rather than along the full length of its upstream side (See Neill, 1973, p. 132). The straight guide bank (spur dike) at the right abutment has an upstream length of 365 ft (111 m) and is revetted with concrete pavement, 4 in (10 cm) in thickness, as is the left abutment (fig. 5). A launching apron of heavy riprap, the D_{50} size having a weight of about 250 lb (112 kg), was placed at the toe of all concrete revetment: at the abutment, at the guide bank, and at the tip of the spurs (figs. 6, 7, and 8; see also Neill, 1973, p. 127).

1963-77 Several moderate floods occurred, including the 1970 flood which reached an elevation of 3292 ft (elevation of 50-yr R.I. flood is 3296 ft). Excavations in 1967 showed that the apron at the guide bank had launched to a slope between 2.5:1 and 3:1 but had not reached shale bedrock. It was also concluded that the heavy rock apron should not be placed much higher than required for protection of the bottom of the concrete revetment, as riprap placed above this point may not launch. The river training works have functioned as planned. Some erosion of riprap has occurred at the tip of spur 1 and the right bank of the anabranch upstream from this spur is migrating laterally.

Discussion: This case provides a useful example of effectiveness of river training works on an anabranching and braided stream whose multiple channels occupy much of its flood plain. The anabranching channels tend to migrate laterally, and thus have the potential for bypassing the bridge. Without the training measures, a longer bridge would be required. The upstream-pointing spurs have the advantage of requiring revetment only at the tips, and they serve to divert overbank flow. Under other circumstances, in which the spurs project into a main channel, the advantage of an upstream orientation is not well established.

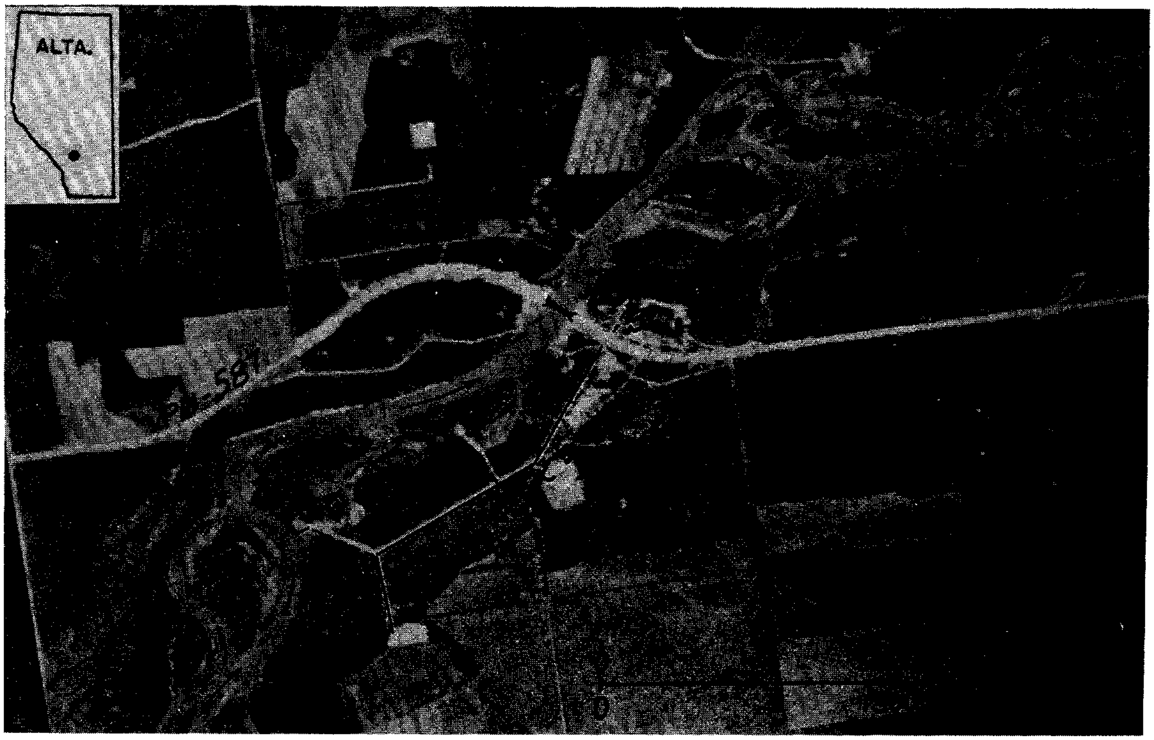


Figure 4. Aerial photograph of Red Deer River at crossing of PH-587, on June 19, 1965. Spurs are indicated by numbers. (From Airphoto Library, Government of Alberta, Lands and Forests).

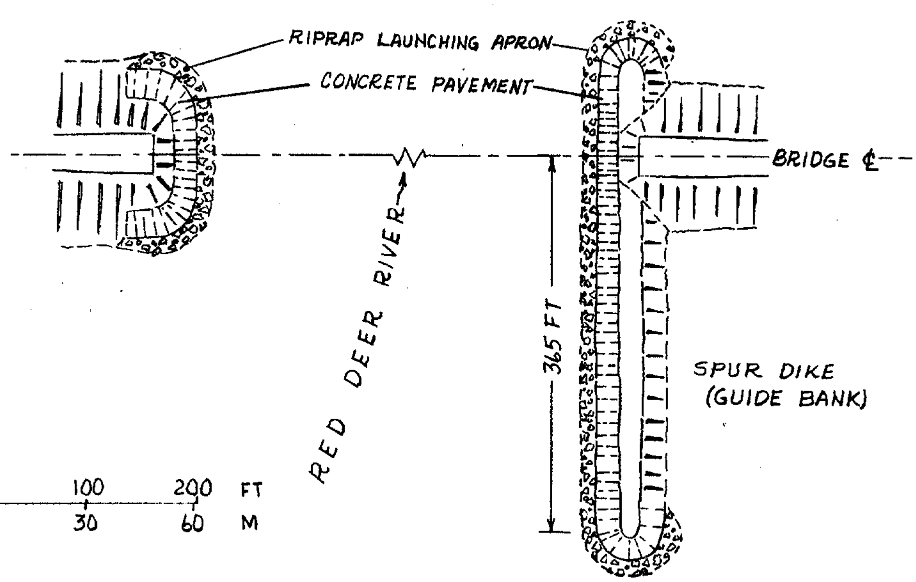


Figure 5. Plan sketch of guide bank (spur dike) and abutment fill-slope protection at PH-587 bridge, Red Deer River.

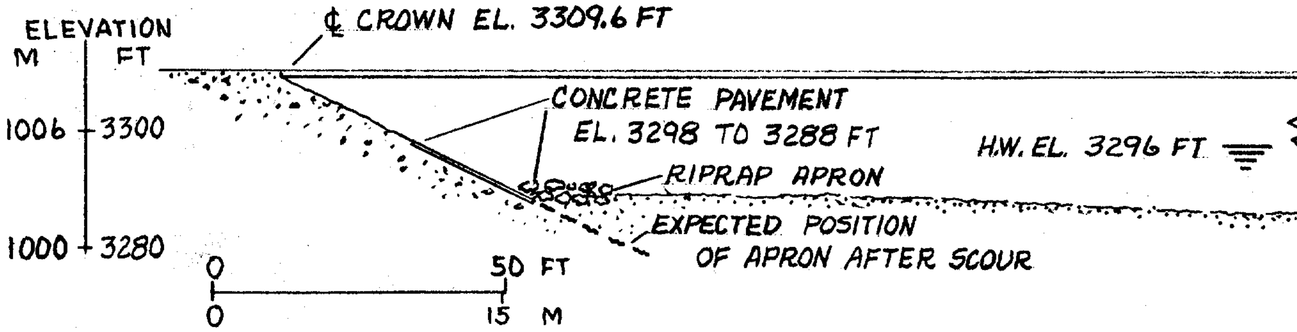


Figure 6. Elevation sketch of left abutment, PH-587 bridge.

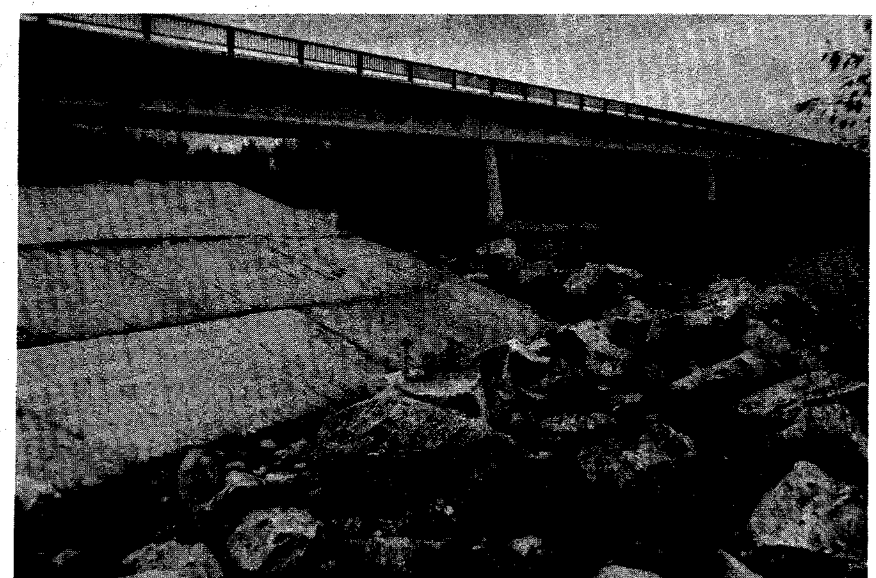
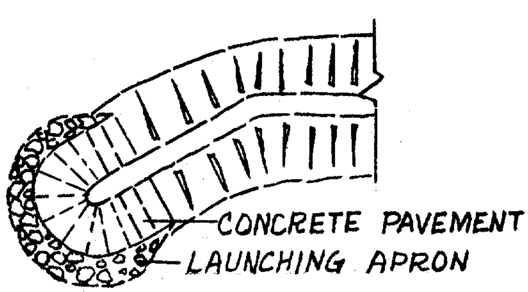


Figure 7. View of riprap launching apron, upstream side of left abutment of PH-587 bridge, on September 29, 1977.



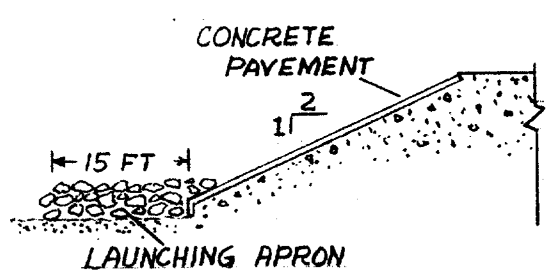


Figure 8. Plan, left, and cross section, right, of revetment at tip of spur No. 1.

SITE 152. OLDMAN RIVER AT PH-2 NEAR FORT MACLEOD, ALBERTA, CANADA

Description of site: Lat 49°43', long 113°27'. Bridge is 440 ft (134 m) in length, four 110-ft (33.5-m) spans, each constructed of precast, pre-stressed-concrete girders; three wall-type concrete piers in channel, spread footings in bedrock (shale), piers battered, rounded nose with rounded metal protectors at upstream ends.

Drainage area, 2,230 mi^2 (5,776 km^2); bankfull discharge, 17,000 ft^3/s (481 m^3/s); valley slope, 0.0017; channel width, about 500 ft (152 m). Stream is perennial, alluvial (but bedrock is at shallow depth, about 11 ft or 3.3 m), gravel bed, in valley of low relief, narrow flood plain. Channel is sinuous, locally braided, locally anabranched, cut banks local, sand-gravel banks, tree cover at 50-90 percent of bankline.

Countermeasures:

1974 Bridge built, with countermeasures that were designed as an integral part of the crossing. For the river-training structures at each abutment (fig. 9) the term "guide bank" is applied and this seems more appropriate than "spur dike", inasmuch as the structures are connected to the stream bank by a berm filled to elevation 3106 ft, or about 1 ft above estimated high water elevation. The streamward side of the guide banks is revetted with concrete slope pavement, 4 in (10 cm) in thickness, 2:1 slope ratio. At the base of the pavement is a riprap launching apron, 12-14 ft (3.5-4 m) wide, consisting of angular limestone and having the following size specifications: 90 percent greater than 150 lb (67 kg), 50 percent greater than 500 lb (225 kg), maximum size 1,500 lb (675 kg). The launching apron consists of 2 layers of stones, without filter blanket.

1975 Extreme flood, peak discharge about 2 times that of the 50-yr R.I. flood. No damage reported at bridge. A small amount of riprap was launched from the apron at the upstream end of the left guide bank. Damage to guide banks was confined to minor undermining at ends of concrete slope pavement.

Discussion: At the crossing site, the channel of Oldman River is braided, anabranching, and relatively wide with respect to the flow. The flood plain has a width of about 3,000 ft and is crossed by abandoned channels (fig. 10). The guide banks, which encroach onto the channel (fig. 9), permitted the design and construction of a shorter bridge than would otherwise have been necessary. The riprap launching apron protects the toe of the concrete slope pavement, which is preferred in Alberta because it is less expensive than riprap and of neater appearance.

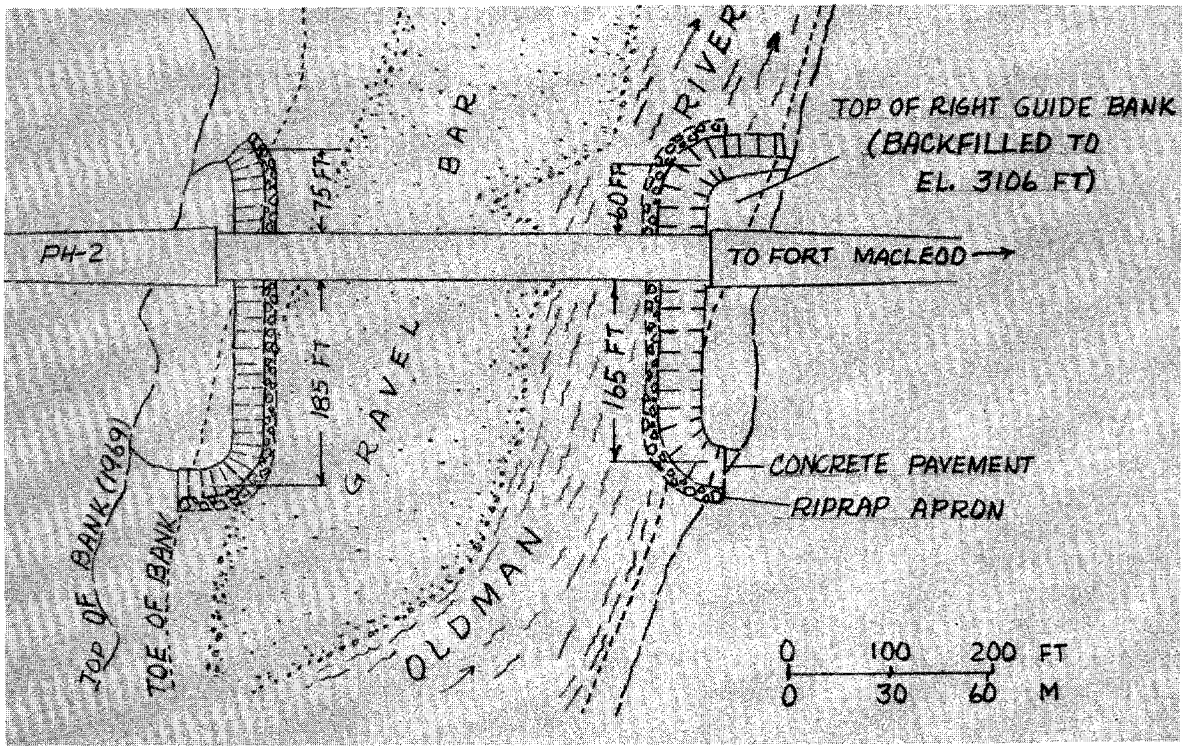


Figure 9. Plan sketch of PH-2 bridge and guide banks, Oldman River near Fort Macleod.

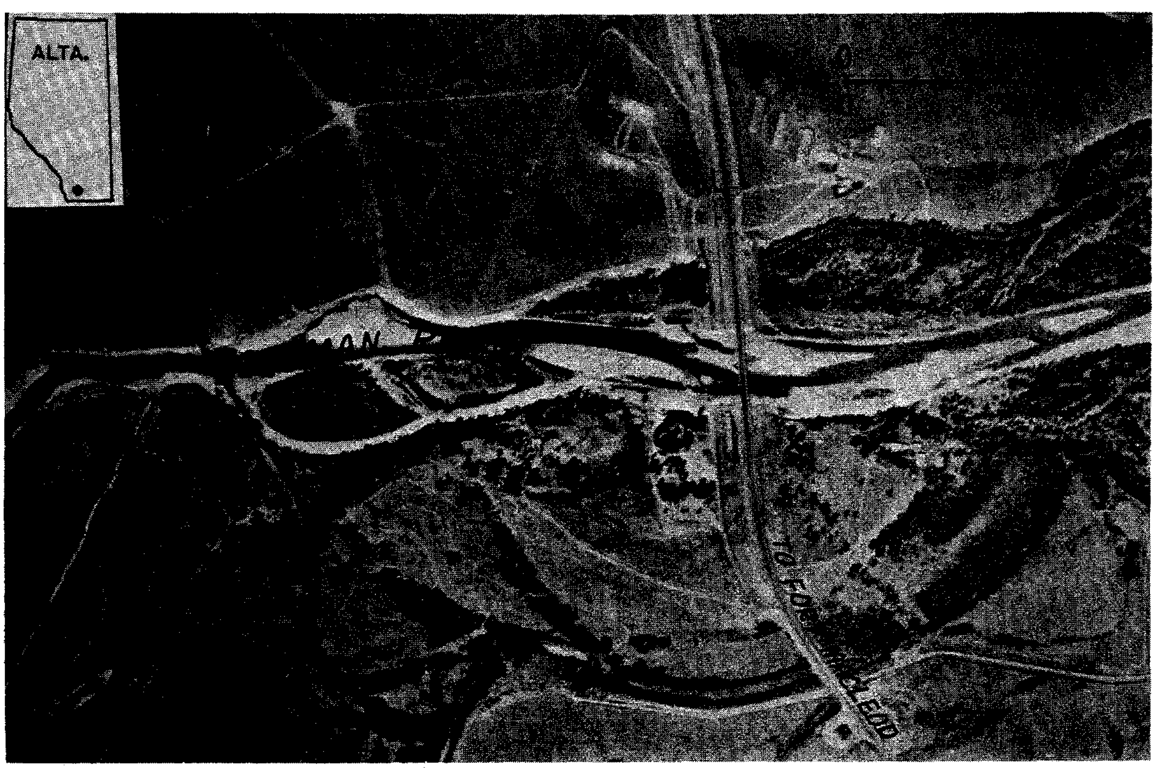


Figure 10. Aerial photograph of PH-2 crossing, Oldman River near Fort Macleod. (Alberta Highways and Transport photograph).

SITE 153. TAZLINA RIVER AT SR-4 NEAR GLENNALLEN, ALASKA

Description of site: Lat 62°04', long 145°26'. Steel truss bridge 400 ft (120 m) in length, one large pointed-nose pier in the right one-third of the channel; pier founded on two concrete-filled sheet-piling caissons 15 ft (4.6 m) in diameter; spillthrough abutments.

Drainage area 2,670 mi^2 (6,910 km^2); bankfull discharge, 56,000 ft^3/s (1,585 m^3/s); valley slope, 0.002 channel; width, 425 ft (127 m). Stream is perennial alluvial, cobble bed, in valley of moderate relief, narrow flood plain. Channel is meandering, locally braided, wider at bends, with point bars, not incised, cut banks local, gravel and cobble banks.

Hydraulic problem and countermeasure: Lateral bank erosion upstream from right abutment, amounting to a recession of about 30 ft (9 m), and erosion of abutment fill, attributed to growth of gravel bar on left side of channel (fig. 11) and consequent shift of thalweg. In 1964, heavy riprap, approximately equivalent to Class II, was placed along the right bank extending for a distance of about 150 ft (45 m) from the abutment. This countermeasure has performed satisfactorily, although a decrease in bed elevation along the right bank amounting to about 5 ft (1.5 m) is attributed to turbulence caused by the riprap. Although the pier and supporting caissons were probably not designed specifically for the purpose of deflecting debris, Norman (1975), p. 89 has noted that, at high stages, a bow or nose wave is formed at the pier that effectively deflects debris (fig. 12). During a flood in September 1971, R.I. about 6 yr, Norman measured 0.9 ft (0.3 m) of general scour at the upstream side of the bridge and 5.5 ft (1.7 m) of local scour at the pier.

Discussion: Comparison of sequential aerial photographs made during the period 1949-1972 indicates that the gravel bar that shifted the thalweg toward the right bank was probably not related to presence of the bridge, but to natural channel processes upstream. In general, bar growth at a particular place in a braided stream is unpredictable.

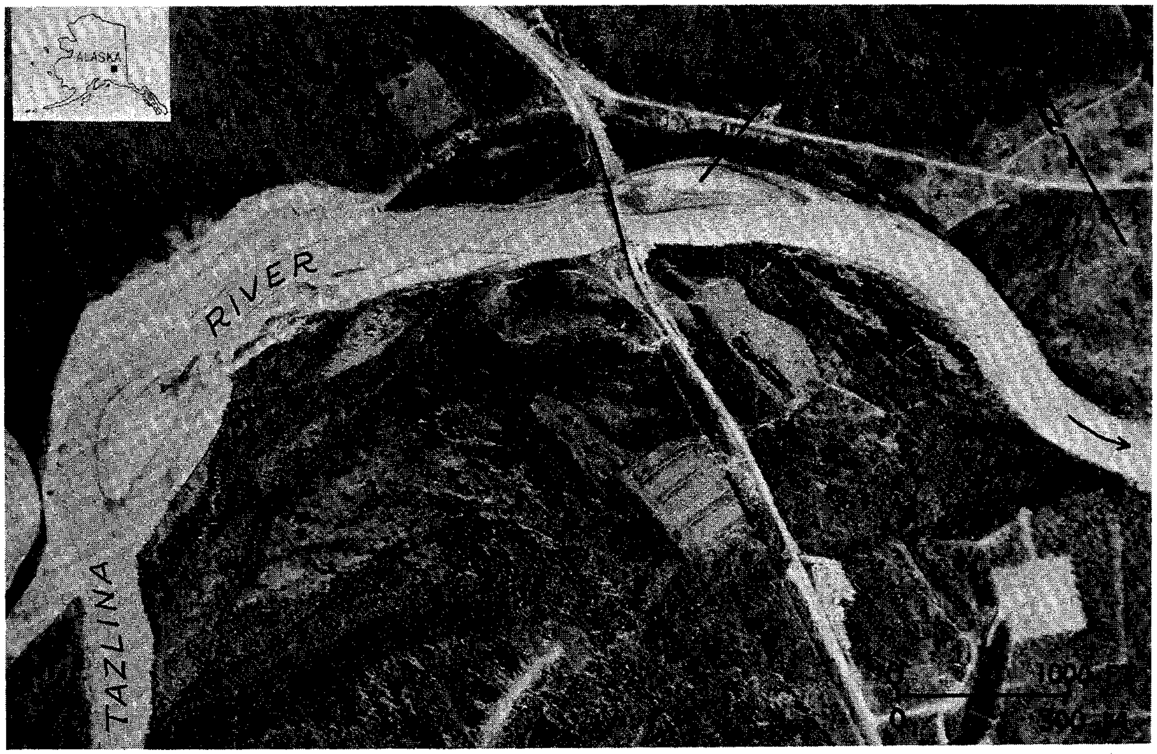


Figure 11. Aerial photograph showing Tazlina River at SR-4 crossing, on August 14, 1969. (From Alyeska Pipeline Service Co.)

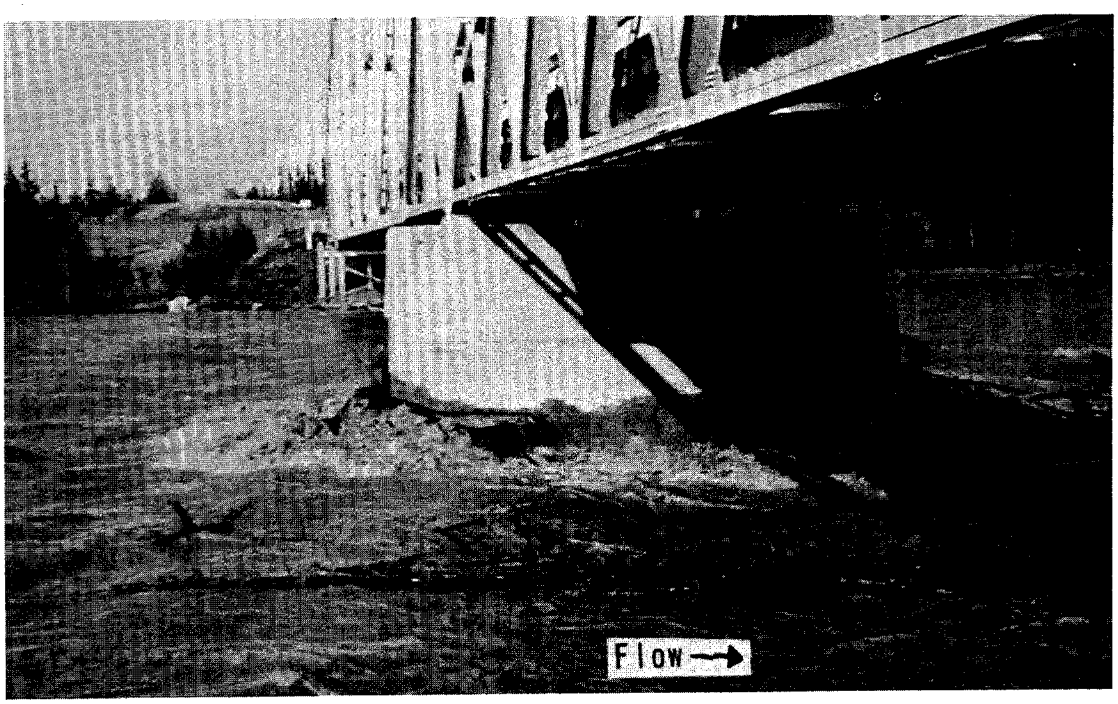


Figure 12. Surface effects of turbulence at pier during high flow on September 4, 1971. (From Norman, 1974, p. 92).

SITE 154. LOWE RIVER AT SR-4 NEAR VALDEZ, ALASKA

Description of site: Lat 61°06', long 145°51', location as shown in fig. 13. Bridge, built in 1951, is 275 ft (82.5 m) in length, three concrete webbed piers in channel on steel pile foundations, vertical concrete abutments protected at toe with heavy rock riprap. Right approach embankment, protected with heavy riprap, constricts braided channel from a natural width of about 850 ft (255 m) to width of bridge waterway, about 275 ft (82.5 m) (fig. 13).

Bankfull discharge, about 12,000 ft^3/s (345 m^3/s; channel width, about 850 ft (255 m). Stream is perennial, semi-alluvial, cobble-boulder bed, in valley of high relief, little or no flood plain. Channel is straight, generally braided, cobble-boulder banks where alluvial.

Hydraulic problem and countermeasure: In view of the substantial constriction of the channel at the bridge, the depth of scour is apparently relatively small. Cross sections plotted from stream discharge measurements at the downstream side of the bridge (fig. 14) indicate that, at near bankfull stage (on August 29, 1974), the minimum elevation of the streambed is at most about 6 ft (2 m) below its elevation at low flow. Foundation pile at the piers have been exposed, however, probably by a combination of local and general scour; but the situation is not regarded as sufficiently hazardous for any countermeasures to be applied. An additional problem has been erosion of the riprapped roadway embankments, and additional large riprap, approximately equivalent Class II, has been placed there from time to time.

Discussion: This case demonstrates that substantial constriction of a wide braided stream transporting coarse bedload (gravel, cobbles, boulders) can be achieved by the use of a riprapped approach embankment, without undue hazard to the bridge. The right approach embankment would be subject to strong lateral erosion during a severe flood, but it could be protected by an upstream spur. A spur dike at the right abutment would seem desirable, but no severe scour has been measured at the right end of the bridge. At this crossing site, however, an alluvial fan from a tributary (fig. 13) tends to hold the main flow near the left bank and forms, in a sense, a natural constriction. It is noteworthy that the local scour measured on August 29, 1974 was not centered around the piers but rather tended to occur between piers.

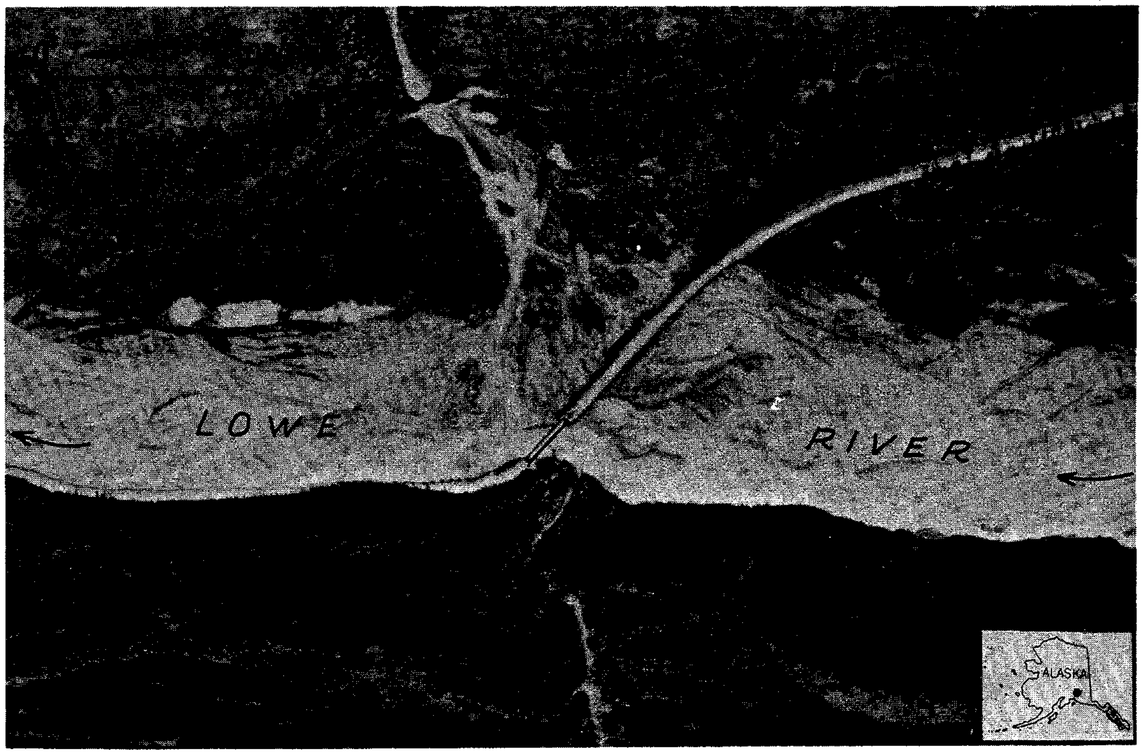


Figure 13. Aerial photograph of Lowe River at SR-4 crossing near Valdez, Alaska. (From Alyeska Pipeline Service Co.)

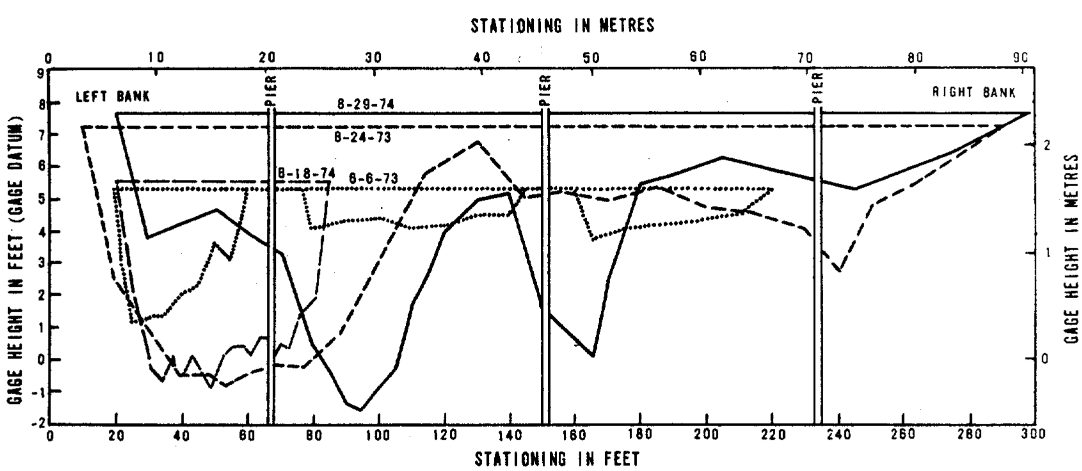


Figure 14. Cross sections of Lowe River near Valdez. (From Childers, 1974).

SITE 155. MOOSE CREEK AT SR-1 NEAR PALMER, ALASKA

Description of site. Lat 61°41', long 149°03', location as shown in fig. 15. Steel girder bridge 180 ft (54.9 m) in length; two concrete piers in channel, 1.75 ft (0.53 m) in width, pointed nose; spillthrough abutments, protected by dumped rock riprap on fill slopes. Piers alined with low flow but skewed up to 40° to flood flow.

Drainage area, about 60 mi^2 (155 km^2); valley slope, 0.012. Stream is perennial, alluvial, gravel bed, in valley of moderate relief, narrow flood plain. Channel is sinuous, locally braided, locally anabranched.

Hydraulic problem and countermeasure. During a flood of August 10, 1972, having a peak discharge of about 18,000 ft^3/sec (510 m^3/s), the right bank approach roadway embankment was breached (figs. 16 and 17). On the previous day, during the rising stage of the flood, the waterway opening beneath both end spans of the bridge was closed by debris, and most of the flow was passing through the center of the opening, where velocities exceeding 25 ft/s (7.6 m/s) were estimated by timing floating trees (Norman, 1975, p. 157). Scour depths extending to the bottom of the pier footings were observed when the discharge was about 5,000 ft^3/s (140 m^3/s).

Discussion. Although breaching of the approach embankment was not intended as a countermeasure, it may have prevented failure of the piers by scour, in view of the very high water velocity and accumulation of debris. Vertical clearance of the bridge was inadequate to prevent obstruction of the opening by accumulation of debris.

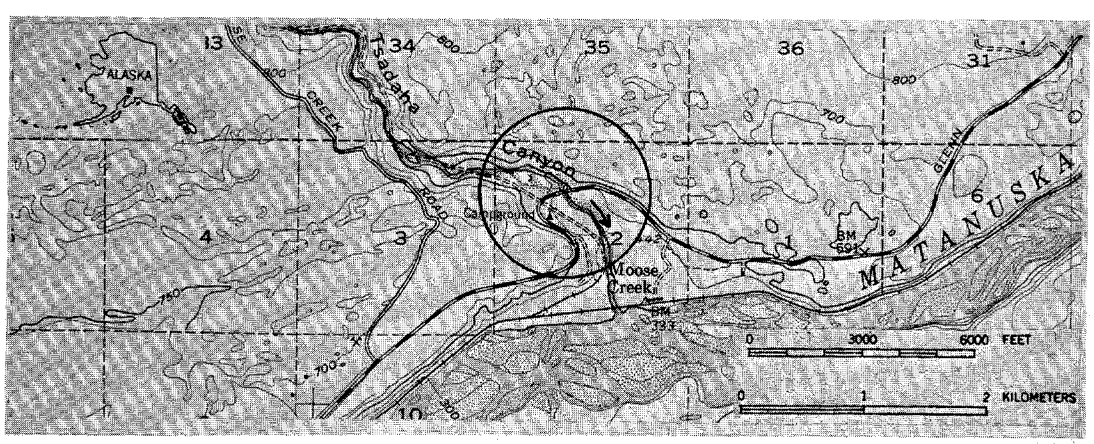


Figure 15. Map showing Moose Creek at SR-1 crossing (circled). (From U.S. Geol. Survey Anchorage (C-6), Alaska, 15' quadrangle, contour interval 50 feet, 1951).

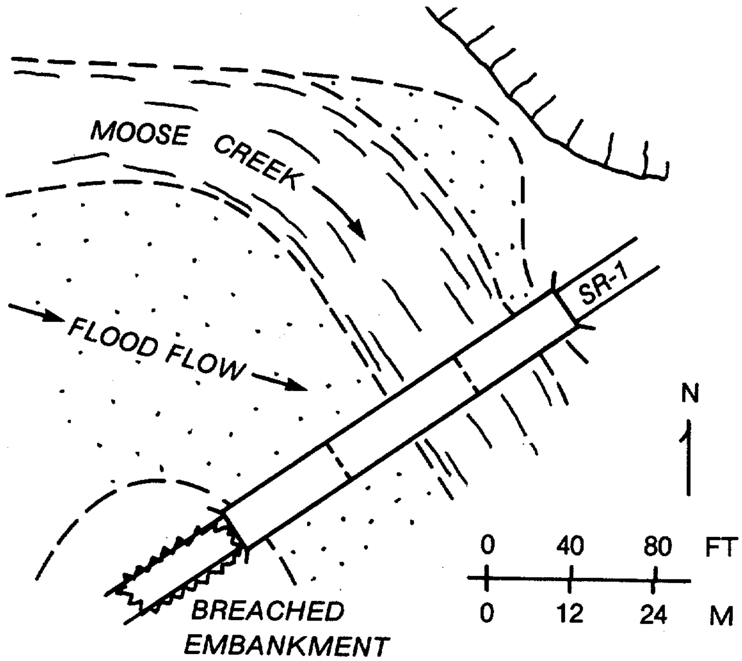


Figure 16. Plan sketch of SR-1 crossing (After Norman, 1975).

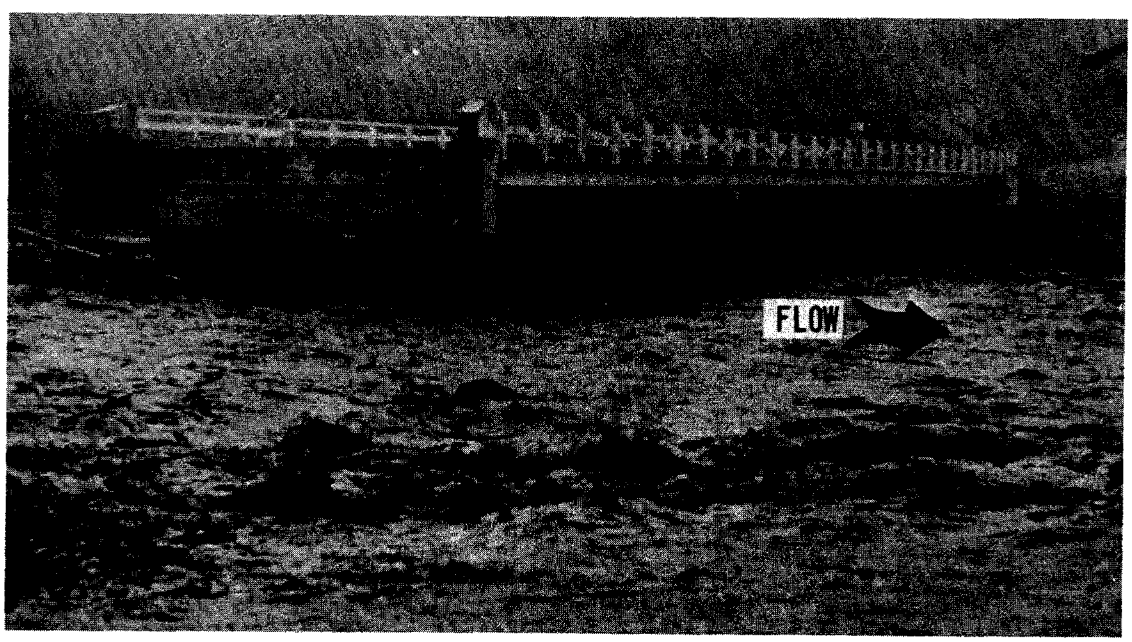


Figure 17. View of SR-1 bridge at Moose Creek after failure of right approach embankment. (From Norman, 1975.)

SITE 156. SNOW RIVER AT SR-9 NEAR SEWARD, ALASKA

Description of site: Lat 60°20', long 149°21', location as shown in fig. 18. Steel girder bridge, 648 ft (198 m) in length, seven spans, six round-nosed cantilever-type piers in channel, spillthrough abutments protected by dumped rock riprap.

Drainage area, 150 mi^2 (388 km^2), much of which is covered by glaciers. Stream is perennial, subject to floods from breakout of glacier-damned lakes. Wide braided channel occupies most of floor of glacial trough, which is about 1 mi (1.6 km) wide, bordered by bedrock slopes and underlain by at least 100 ft (30 m) of silt, sand, and gravel. Bed material of individual channels ranges from sand to gravel, average median diameter of representative samples is about 3 mm.

Hydraulic problem and countermeasure:

1964 Existing bridges and roadway badly damaged by earthquake, but had already been scheduled for replacement.

1966 Two bridges built to replace previous three, right braid blocked by approach embankment and most of drainage diverted to bridge near center of channel (figs. 19 A and B). New embankment designed for unstable foundation of saturated silt and fine sand. Bridge design included two revetted earth spur dikes, of elliptical shape, each 300 ft (91 m) in length upstream from the bridge and about 75 ft (22 m) in length downstream (fig. 20).

1966-70 Streambed along right spur dike scoured to depth of 8-9 ft (2.4-2.7 m) below channel bed existing at time of bridge construction, reaching top of footing at pier adjacent to right bank. Scour is attributed to flow from right braid, which had been blocked by new embankment (Norman, 1975, p. 128). General scour amounting to about 2 ft (0.6 m) at the bridge opening was measured for the period 1966 to September 1970, but no additional scour was measured during the flood of September 1970.

Discussion: Confinement of a wide braided channel has been successfully accomplished by use of spur dikes and revetted roadway embankments. Shortly after construction, the crossing was tested by a major flood, of undetermined recurrence interval but having a discharge of about 55,000 ft^3/s (1,558 m^3/s). General scour in the bridge waterway has been moderate and is evidently not progressing further. Local scour along the right spur dike, although significant, is not regarded as a hydraulic problem.

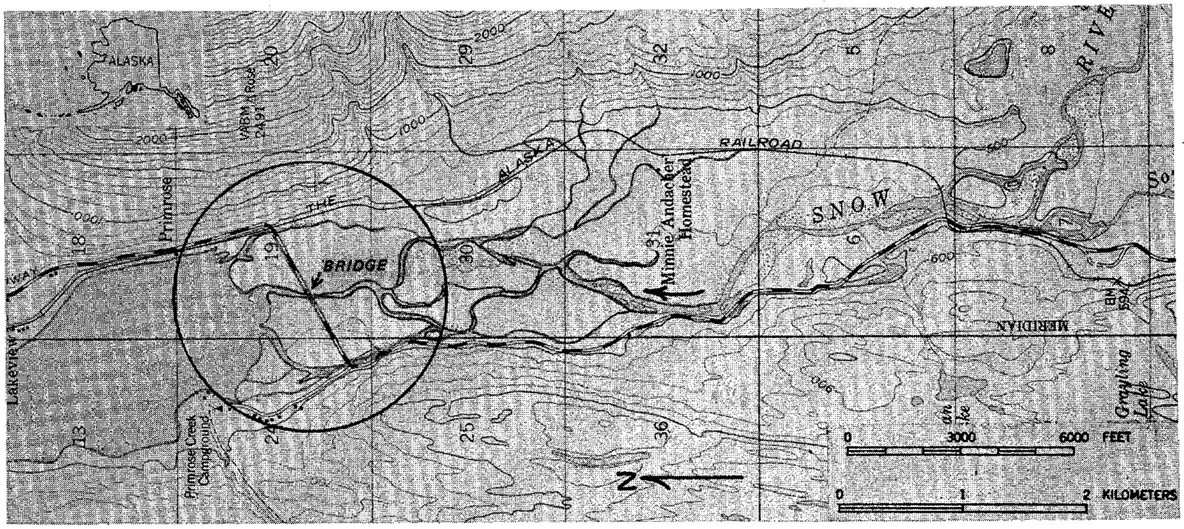


Figure 18. Map showing Snow River at SR-9 crossing (circled). (Base from U.S. Geol. Survey Seward (B-7), Alaska, quadrangle, contour interval 100 feet, 1951.)

Figure 19. (on p. 296)

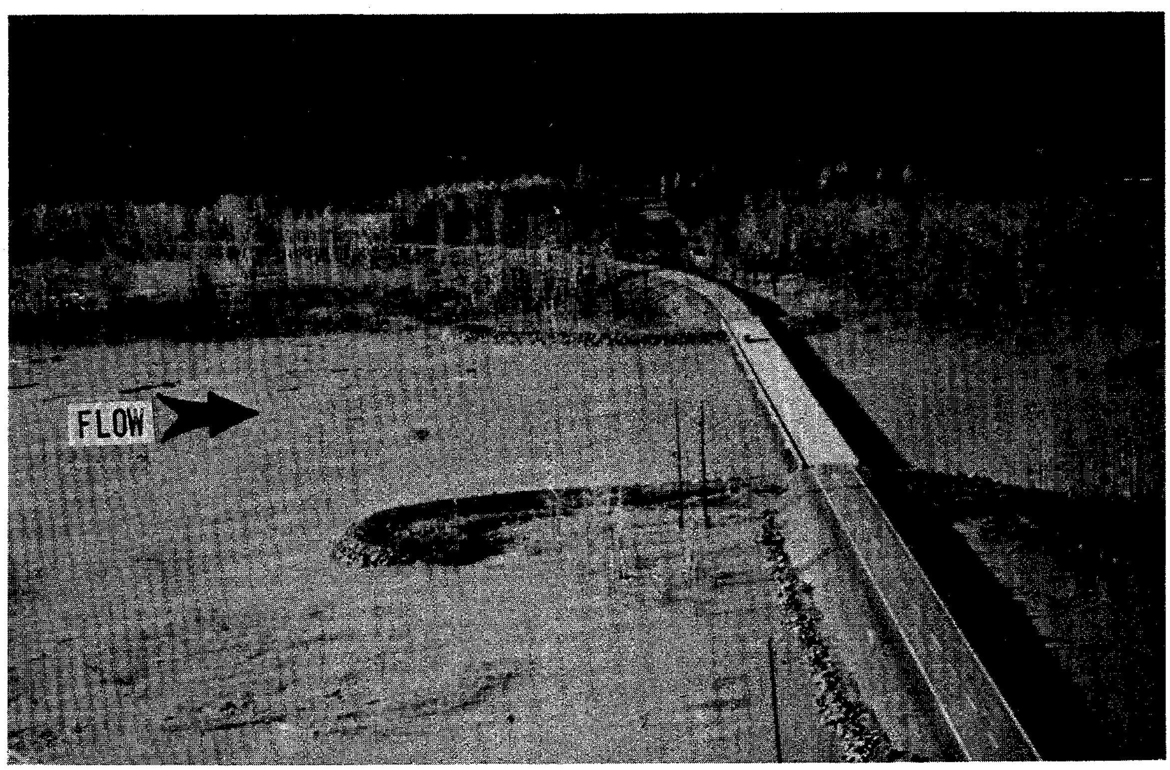


Figure 20. Oblique aerial photograph of SR-9 bridge, on September 22, 1970. (From Norman, 1975, p. 127.)

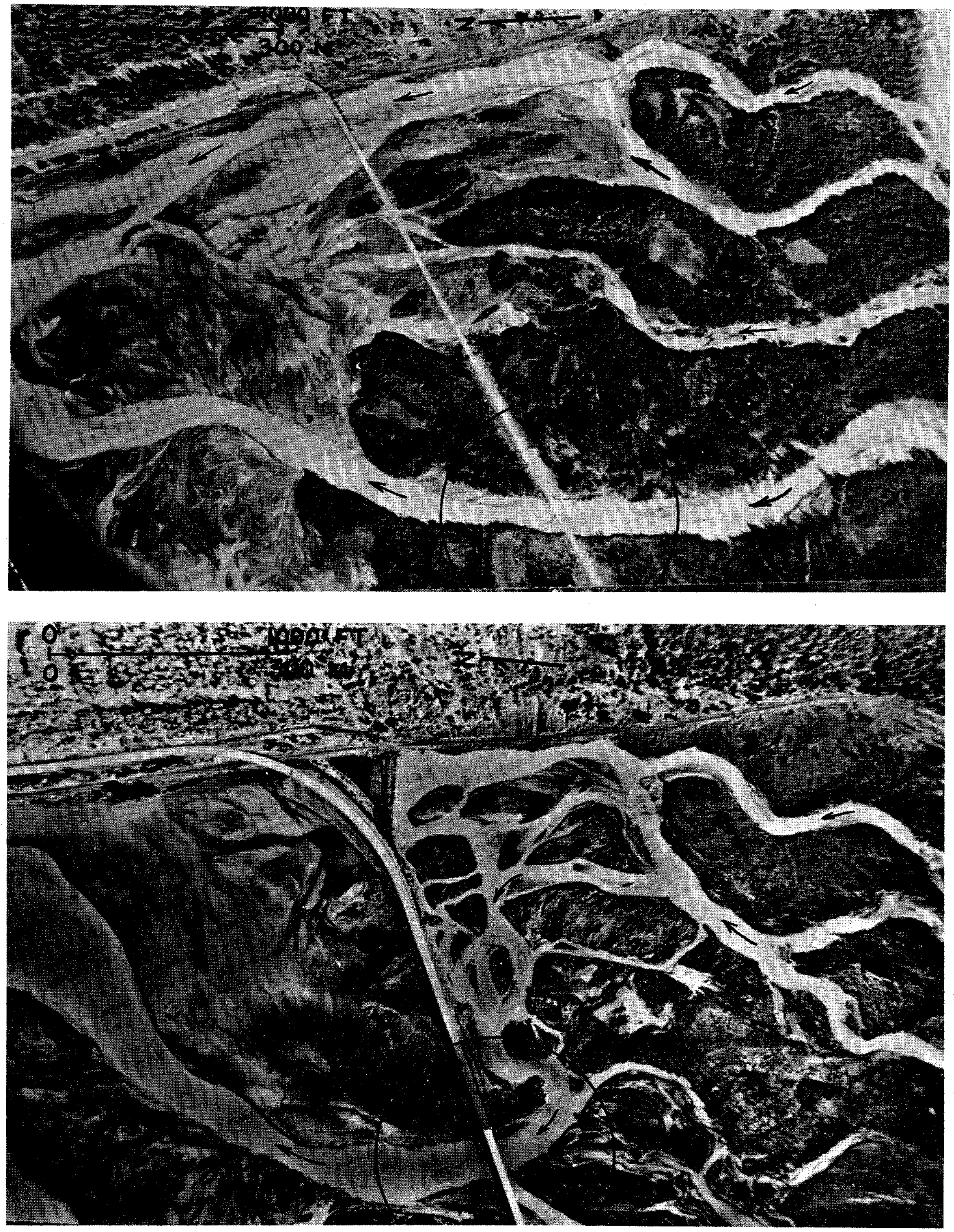


Figure 19. Top, aerial photograph of SR-9 crossing prior to construction of new bridge. Bottom, aerial photograph of SR-9 crossing on September 23, 1975, after construction of new bridge and embankment. (From Alaska Dept. of Highways.)

SITE 157. NORTH FORK CHENA RIVER AT BRIDGE 236, CHENA HOT SPRINGS ROAD NEAR CHENA HOT SPRINGS, ALASKA

Description of site: Lat 64°59', long 146°14', location as shown in fig. 21. Bridge, removed in 1975, was 120 ft (36 m) in length, two steel girder spans, steel H-pile bents encased in concrete to estimated depth of scour, spillthrough abutments. Pile bents were protected from ice damage by steel angle-iron nose.

Drainage area, about 300 mi^2 (800 km^2); valley slope, 0.0038; channel width, about 110 ft (33 m). Stream is perennial, alluvial, gravel bed, in valley of moderate relief, narrow flood plain. Channel is meandering, generally anabranched, wider at bends, point bars, cut banks local, tree cover at more than 90 percent of bankline.

Hydraulic problem and countermeasures:

1962 Bridge built. Design was for flood of 50 yr R.I., but along this reach of the Chena River, ice jams may raise the elevation of the annual flood above that of the 50-yr R.I. flood. The channel accommodates an estimated discharge of about 3,000 ft^3/s (85 m^3/s), and the discharge during annual ice breakup is estimated to be in this range.

1967 Major flood, of undetermined recurrence interval. Both approaches to bridge 236 were breached and the river broke through its right bank at a point about 500 feet upstream from the bridge (fig. 22). The overflow concentrated along the north side of the road where it destroyed nearly a mile of embankment as it excavated a new channel. Additional roadway was lost at a later time during the same flood when the meander upstream from bridge 236 was cut off by the high flow (figs. 21 and 22). Both of the new channels are indicated by lines in fig. 21. Maintenance crews from the Alaska Department of Highways made temporary repairs on the roadway and the old river channel and re-established the flow beneath the bridge.

1968-74 Despite flow-control dikes intended to maintain the channel alinement through bridge 236, the river continued to bypass the bridge during high flows associated with spring ice breakup and ice jams. The decision was made to plug the opening in the embankment, remove the bridge, and use the bridge spans to lengthen existing bridges. In addition, riprap bank protection was provided at existing bridges, and riprap dikes were built to protect the roadway embankment from flood flow in the overflow channels. Between bridges 236 and 235, the river was diverted into a riprapped channel along the north side of the roadway (fig. 21).

1977 River channel appears to have stabilized in alinement along north side of the roadway.

Discussion: Bridge 236 was on a reach of the North Fork Chena River where the trend of the roadway coincided with the slope direction of the flood plain. The river tends to spill overbank readily during spring floods, and to follow bypass channels (anabranches). The river had a natural tendency to bypass the bridge during the period of high water. Under these circumstances, three alternatives were considered: (1) to relocate the roadway, (2) to remove bridge and riprap the roadway embankment, and (3) to increase the size of the bridge and to build adequate flow-control dikes to maintain channel alinement through it. The first alternative was chosen as the most favorable.

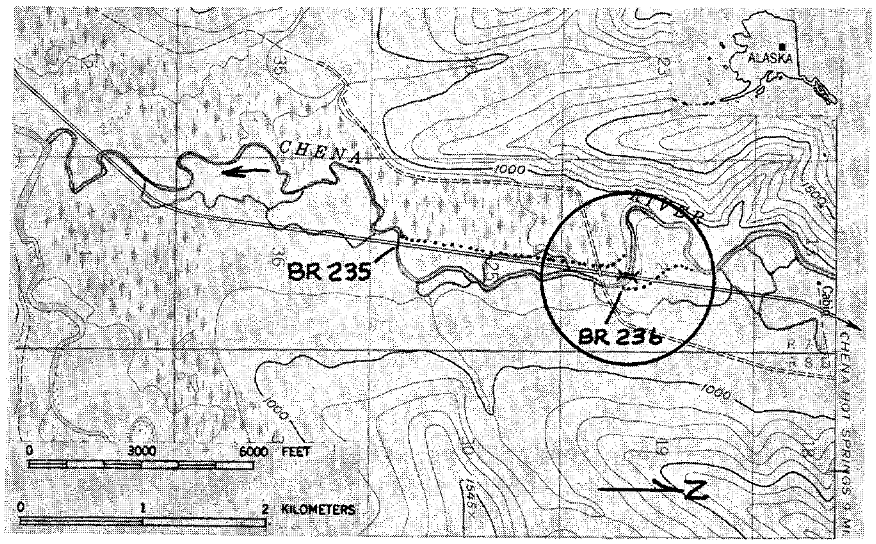


Figure 21. Map showing North Fork Chena River and Chena Hot Springs Road in vicinity of bridge 236 (circled). (Base from U.S. Geol. Survey Big Delta, Alaska, 15' quadrangle, contour interval 100 feet, 1958.)

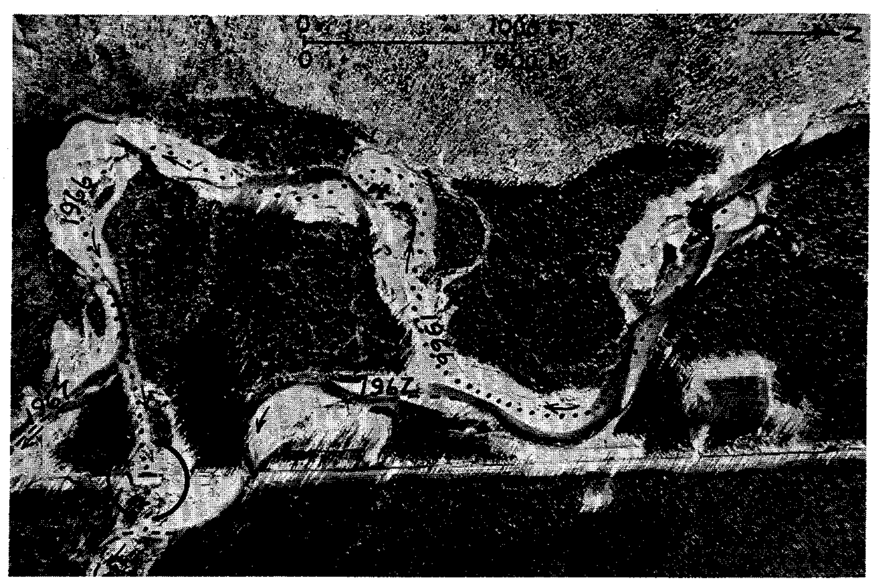


Figure 22. Aerial photograph taken August 29, 1967, showing bridge (circled) and meander cutoff that formed during 1967 flood. (From Alaska Dept. of Highways.)

SITE 158. KNIK RIVER AT SR-1 NEAR PALMER, ALASKA

Description of site: Lat 61°30', long 149°02' location as shown in fig. 23. The old bridge is 2,000 ft (610 m) in length, including a 500-ft (152-m) approach on timber pilings that extends from the right bank to the principal structure, which is a steel truss type supported by six concrete piers. The new bridge, built in 1974, has a total length of 505 ft (154 m), two concrete piers in the channel, spillthrough abutments protected with dumped rock riprap, and two spur dikes included as part of the bridge construction.

Drinage area, 1,200 mi^2 (3,100 km^2), over half of which is mantled with glacial ice; channel width narrows from about 3 mi (4.83 km) at the terminus to Knik Glacier to less than 0.5 mi (0.8 km) at the bridge; valley slope, 0.0025. Stream is perennial, average discharge about 7,000 ft^3/s (198 m^3/s), but peak discharges reaching 359,000 ft^3/s (10,000 m^3/s) have resulted from the breakout of a glacier-dammed lake, Lake George; semi-alluvial, gravel bed, in valley of high relief, little or no flood plain. Channel is straight, generally braided.

Hydraulic problem and countermeasures: No hydraulic problems beyond those requiring routine maintenance are reported for the old bridge, which withstood high flows annually, from breakout, for many years. Norman (1975) p. 53-61) reports no significant general scour and only 3.5 ft (1.1 m) of maximum local depth of scour at a pier where the approach velocity was about 12 ft/s (3.6 m/s). Use of countermeasures incorporated into the design of the new bridge has permitted drastic shortening, relative to the old bridge, from 2,000 ft (600 m) to 500 ft (150 m). These countermeasures include two elliptical, revetted earth-embankment spur dikes, the right having a length of 225 ft (70 m) and the left, 310 ft (95 m) (fig. 24). At the right approach roadway to the new bridge, an overspill section has been provided, the lowest part of which is about 20 ft (6 m) below bridge deck elevation. No breakouts of Lake George have occurred since completion of the new bridge, and thus the countermeasures have not yet been tested by major flood.

Discussion: If the countermeasures prove successful in flood, which seems likely in view of previous hydraulic experience at the crossing, this case will provide an outstanding example of the use of countermeasures to permit design of a shorter bridge. Other examples of successful confinement of a braided stream are provided by Sites 54, 150, 154, 156, 258.

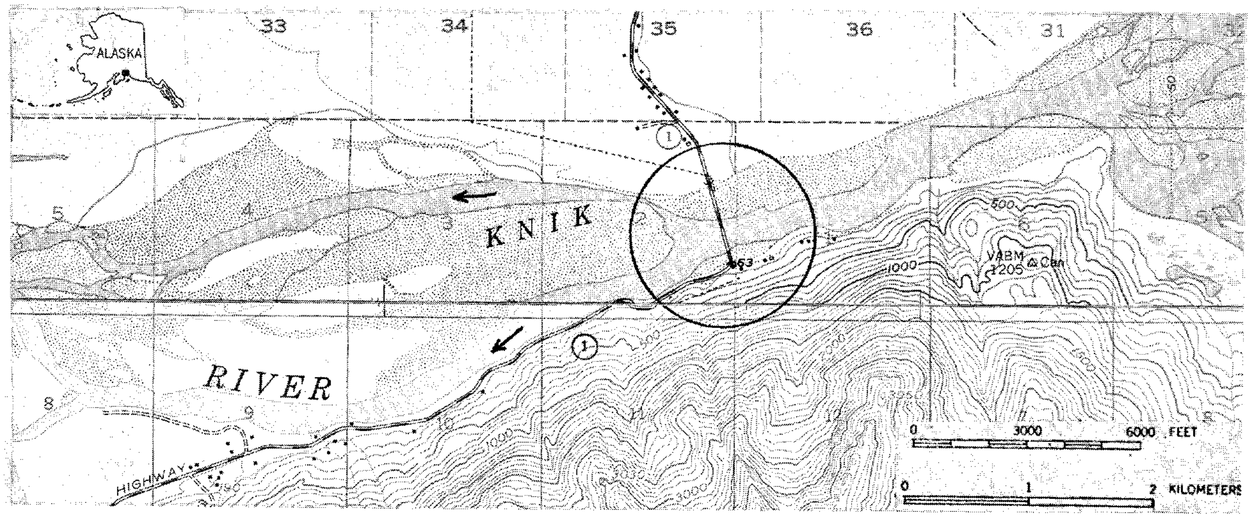


Figure 23. Map showing old bridge on Knik River at SR-1 crossing (circled) near Palmer, Alaska. (Base from U.S. Geol. Survey Anchorage (b-6 and C-6), Alaska, 15' quadrangles, contour interval 100 feet, 1951.)

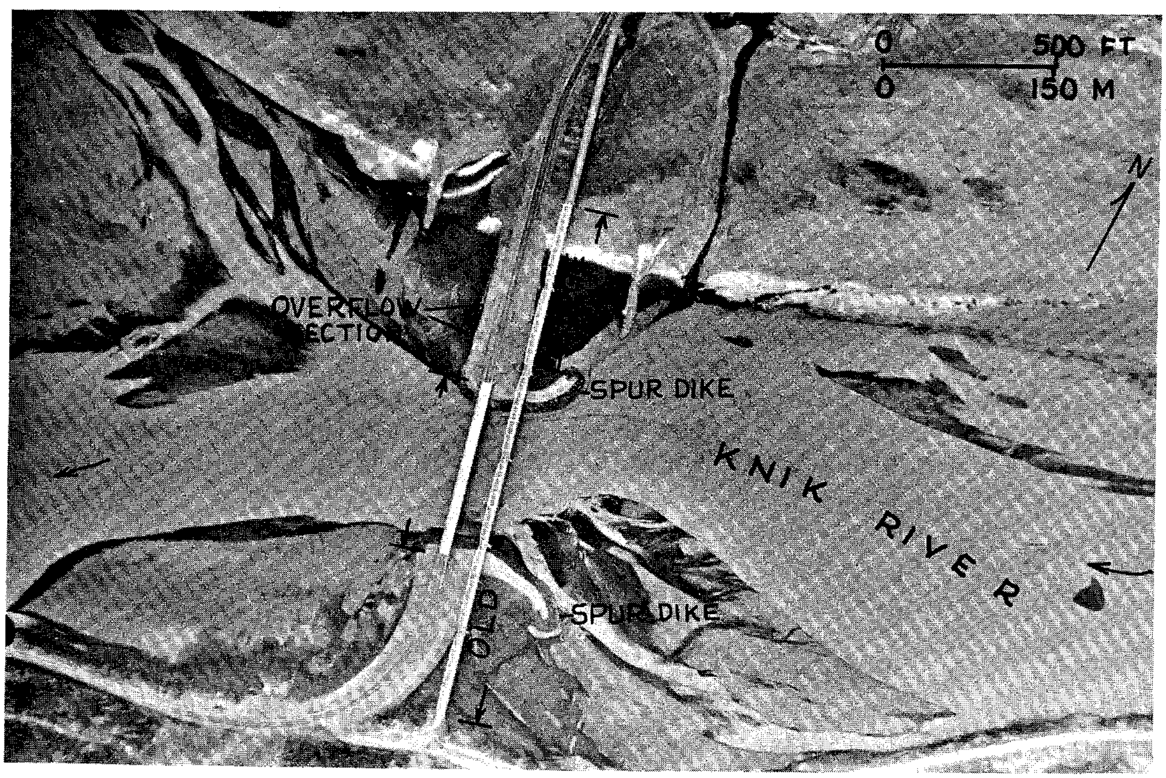


Figure 24. Aerial photograph on May 25, 1976 showing new and old bridges at SR-1 crossing, Knik River. (From Alaska Dept. of Highways.)

SITE 159. BOEUF RIVER AT US-82 NEAR LAKE VILLAGE, ARK.

Description of site: Lat 33°19', long 91°22', location as shown in fig. 25. Bridge, built in 1944, is 150 ft (45 m) in length, three steel I-beam spans, two pile bents in channel, each consisting of six 16-in (41-cm) octagonal precast concrete piles, spillthrough abutments protected by dumped rock riprap.

Drainage area, 355 mi^2 (137 km^2); valley slope, 0.0004; channel width, 150 ft. Stream is perennial, alluvial, in valley of low relief, wide flood plain. Channel is sinuous, artificially straightened, cut banks local, tree cover on 50-90 percent of bankline.

Hydraulic problem and countermeasure: At some time after bridge construction, a steel sheet-pile check dam was built just downstream from the bridge to retard degradation of the channelized stream. Soon afterwards, erosion of channel bed and banks began downstream from the check dam. A large pool has formed, the banks of which are slumping and receding toward the bridge (figs. 25 and 26). Rock riprap (approximately class I) placed on channel banks at the pool has not controlled the problem. The riprap is on a steep slope, the size seems too small, and no toe protection is apparent.

Discussion: Although the check dam, together with riprap beneath the bridge, has protected the bridge substructure from channel degradation, it has caused a bank recession problem downstream that may progress to the abutments.

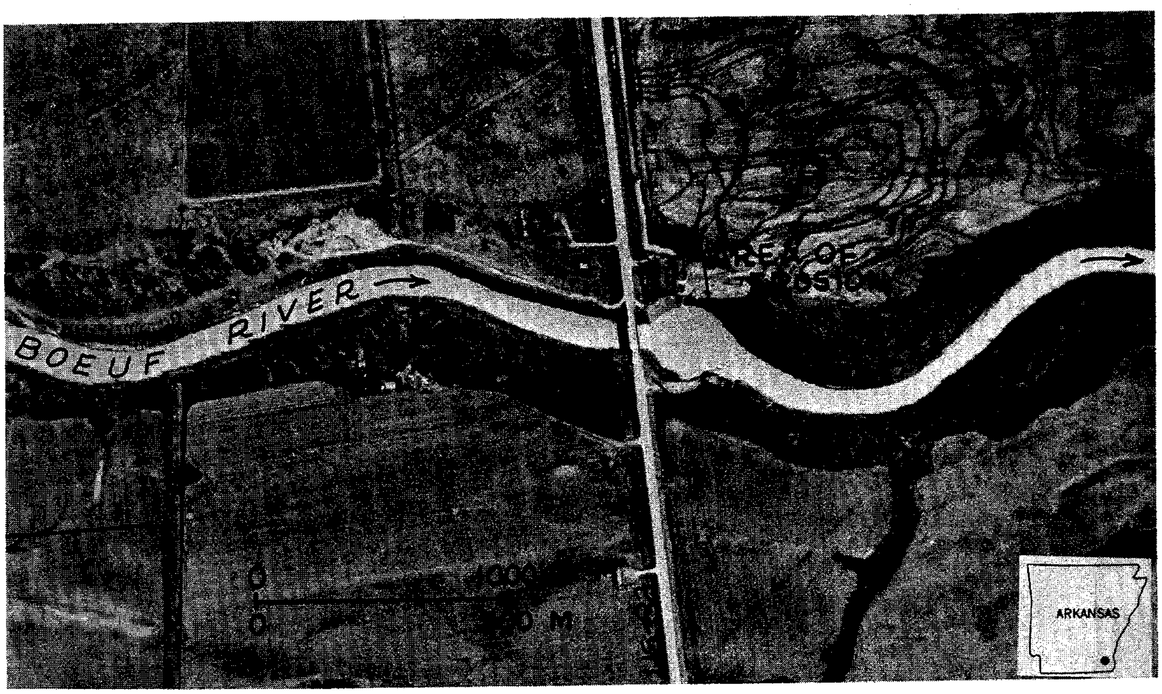


Figure 25. Aerial photograph of Boeuf River at US-82 crossing, on November 27, 1974. (From Arkansas State Highway Department.)

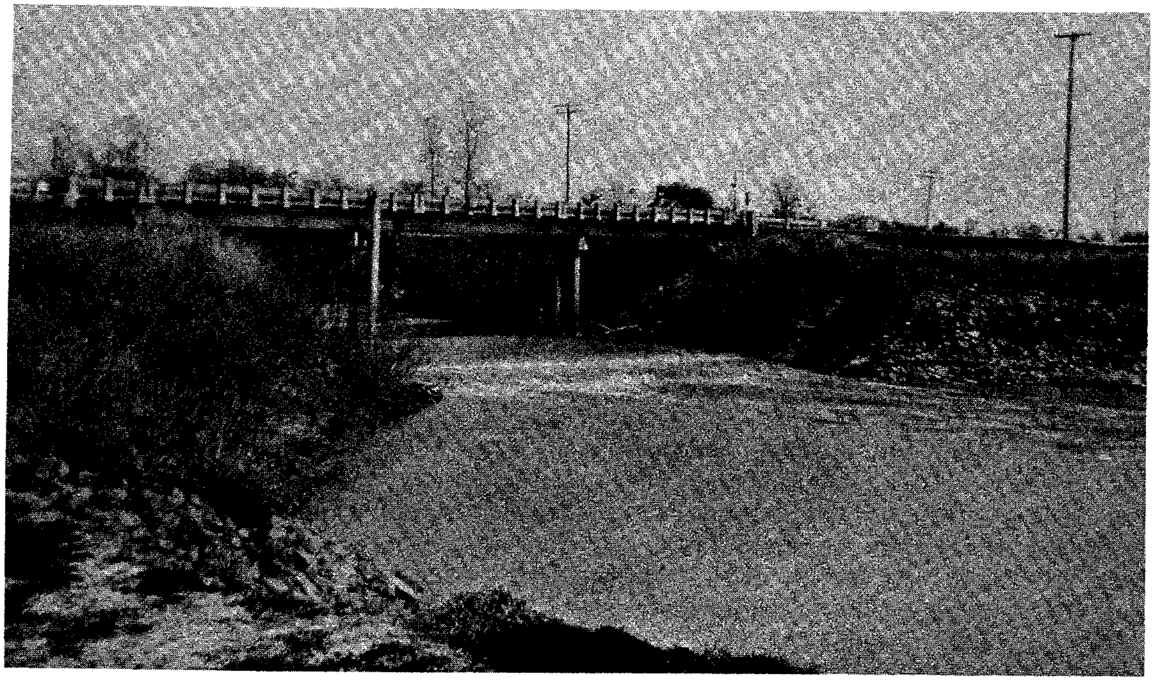


Figure 26. Upstream view of overfall at check dam and riprap on eroded banks below dam, on March 18, 1976.

SITE 160. BURNT CANE LAKE AT SR-38 NEAR WIDENER, ARK.

Description of site; Lat 35°01', long 90°38'. Bridge is 532 ft (160 m) in length, precast concrete pile bents, spillthrough abutments.

Burnt Cane Lake occupies an old meander scar of the Mississippi River, through which flood flow from the St. Francis River has been diverted by means of an artificial channel.

Hydraulic problem and countermeasure: After building of the bridge in the 1960's, upstream diversion resulted in increased magnitude and duration of flow and the channel degraded. Two pile bents were exposed to hazardous depth, and these were "jacked down" 8-10 ft (2.5-3 m).

Discussion: This is the only site at which degradation is attributed to diversion of flow into a channel, although problems at bridges crossing flood diversion channels on the Mississippi alluvial plain were reported also from Missouri.

SITE 161. BOEUF RIVER AT SR-144 NEAR McMILLAN CORNER, ARK.

Description of site: Lat 35°23', long 91°21'. Bridge is 165 ft (50 m) in length, two pile bents in channel, each consisting of six 16-in (40 cm) octagonal posts; spillthrough abutments.

Hydraulic problem and countermeasure: See fig. 27 and description of Site 159, an almost identical case about 5 mi (8 km) downstream.

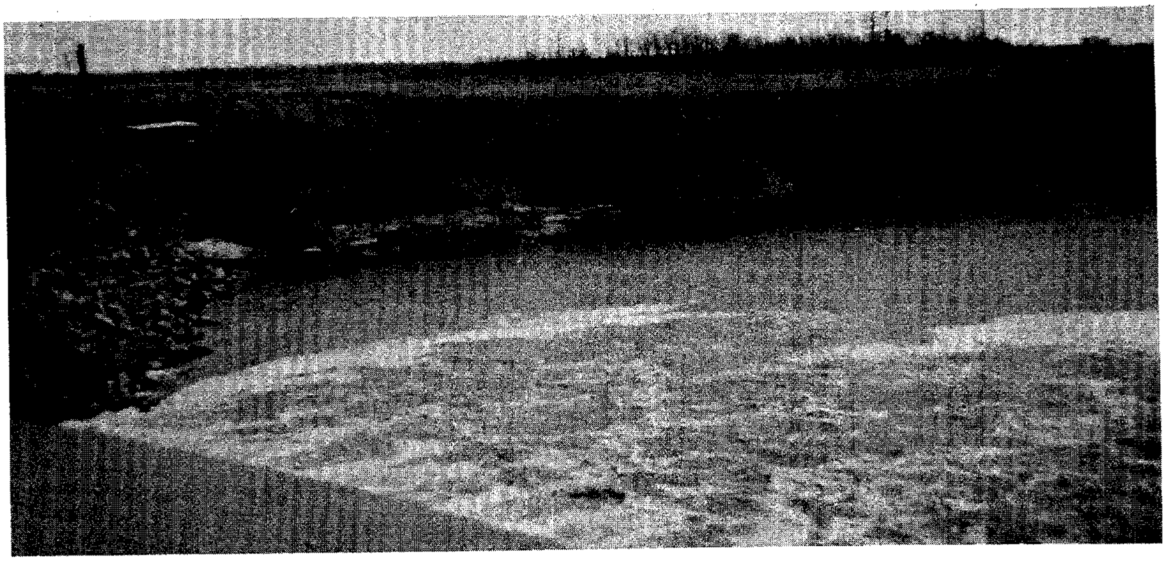


Figure 27. Downstream view of Boeuf River at SR-144, showing overfall at check dam, eroded bank, and remnants of riprap.

Figure 28. Aerial photograph of Cossatot River at SR-24 crossing, on February 18, 1976. (From Arkansas Dept. of Highways.)

SITE 162. COSSATOT RIVER AT SR-24 NEAR LOCKESBURG, ARK.

Description of site: Lat 33°58', long 94°13', location as shown in fig. 28. Bridge is 377 ft (115 m) in length, three concrete piers in channel and four concrete pile bents on flood plain. Crossing is eccentric, located near west edge of flood plain.

Valley slope, 0.0009; channel width, about 135 ft (41 m). Stream is perennial, alluvial, in valley of low relief, wide flood plain. Channel is sinuous, generally anabranched (fig. 28).

Hydraulic problem and countermeasure:

1962 Bridge built, without countermeasures.

1968 Major flood, R.I. estimated at about 50 yr. Owing to constriction of overbank flow by approach embankments, scour occurred at pile bents on flood plain (fig. 28) and at fill slopes of both abutments. Boulders were placed at pile bents and rock riprap on abutment slopes.

1969-76 No major floods, but boulders are judged adequate to protect pile bents.

Discussion: Scour at pile bents is attributed, at least in part, to eccentricity of crossing. Crossing appears to be at bend but, because of the anabranched nature of the channel upstream from the bridge, scour was no more severe at one abutment than the other. Boulders placed at pile bents have increased roughness coefficient from about 0.038 to about 0.12, and thus reduced conveyance. Scour may occur elsewhere in the bridge waterway during a major flood.

SITE 163. COSSATOT RIVER AT US-70 NEAR DE QUEEN, ARK.

Description of site: Lat 34°03', long 94°13', location as shown in fig. 29. Bridge over main channel is 421 ft (128 m) in length, steel I-beam spans, concrete wall-type piers in channel, spillthrough abutments. Crossing is eccentric and is located at a tributary entrance. Relief bridge at which scour occurred (fig. 29) is about 0.6 mi (1 km) east of the main-channel bridge; it is 103 ft (31 m) in length, two concrete pile bents in waterway, spillthrough abutments.

Drainage area, 361 mi^2 (935 km^2); bankfull discharge, about 12,500 ft^3/s (354 m^3/s); channel width, 330 ft (100 m); valley slope, 0.0011. Stream is perennial, alluvial, gravel bed, in valley of low relief, wide flood plain. Channel is sinuous, locally anabranched, random width variation, cut banks rare, silt-sand banks, tree cover on 50-90 percent of bankline. Flood plain has been partially deforested, such that border between cleared and forested areas is just to left of relief bridge.

1937 Bridges built. Vegetated islands, related to anabranching habit of river and to entrance of tributary, were in channel upstream from main-channel bridge, and extended under bridge.

1968 Flood, peak discharge 1.4 times the flood of 50-yr R.I. At main-channel bridge, island caused concentration of flow in deep channel near the right bank, and scour in channel undercut toe of bank, which slumped over an area extending several hundred feet along the channel and several tens of feet from the channel.

At the relief bridge, most flow came from cleared area on flood plain at right of bridge opening. Riprap on right abutment slope was displaced, and fill was eroded, exposing foundation piles. An elongated depression was scoured under the bridge (figs 29 and 30). As a countermeasure, dumped rock riprap, approximately equivalent to Class II riprap, was placed at abutment slope, replacing hand-placed riprap smaller than Class I, which failed to protect the slope.

Discussion: No immediate hydraulic problem occurred at the main-channel bridge, but the large-scale bank slumping is potentially serious. The case demonstrates a hazard of bridge location on streams whose flow is divided by islands. The relief bridge provides an example of the effects of partial clearing of a flood plain on the distribution of flow at a relief bridge. During the 1968 flood, the relief bridge received little flow from the wooded area left (east) of the bridge but a severe concentration of flow near the right upstream abutment from the cleared area to the right (west) of the bridge. This inference on flow distribution was made after the flood by U.S. Geological Survey personnel, from observations of drift caught in fences.

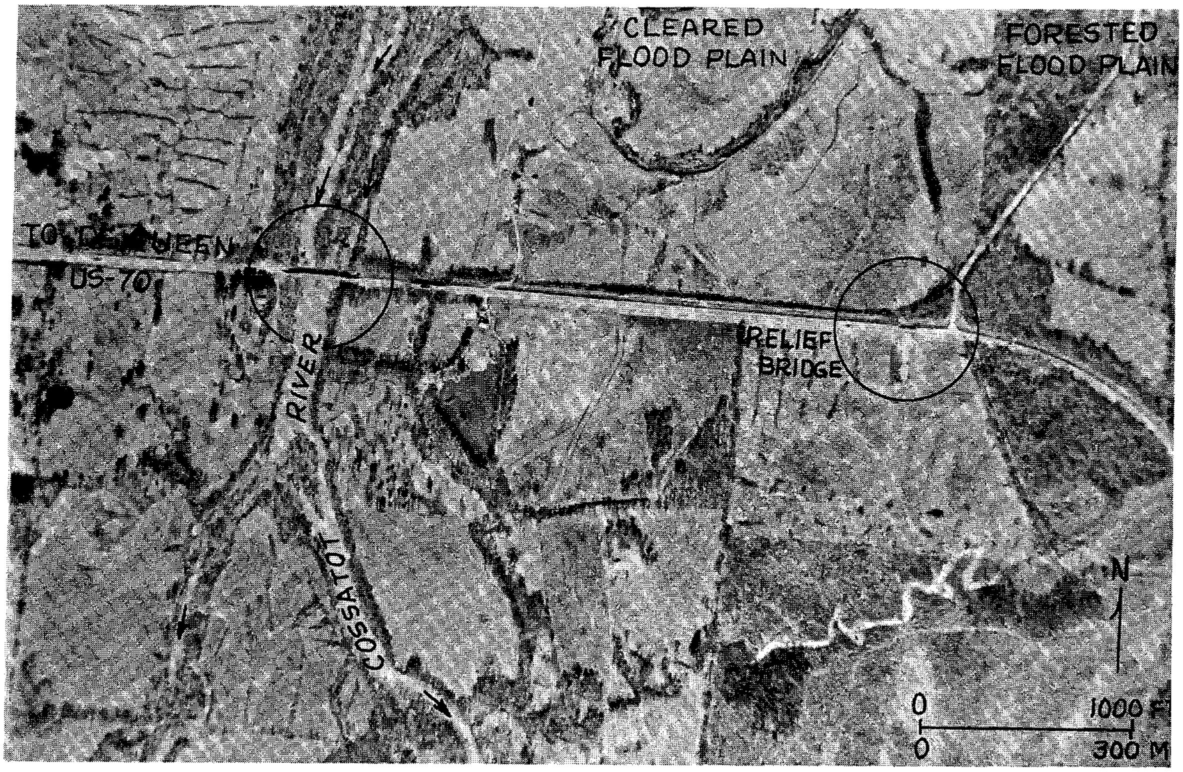


Figure 29. Aerial photograph of Cossatot River and flood plain at US-70 crossing, on February 18, 1976. (From Arkansas Dept. of Highways.)

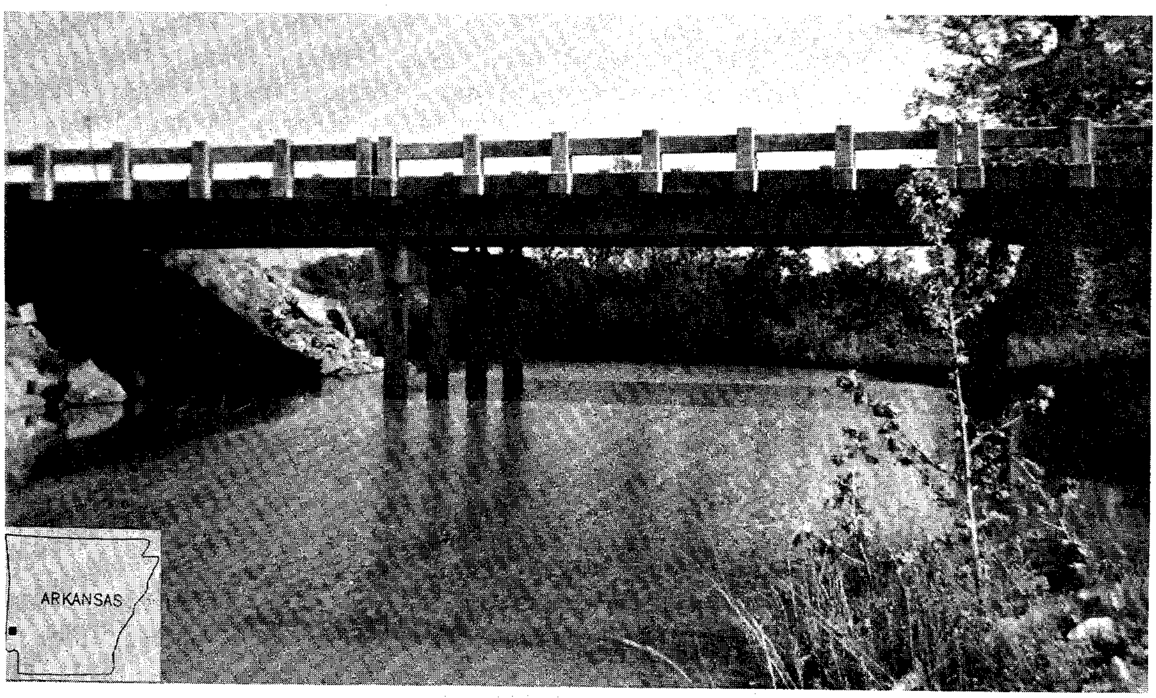


Figure 30. Upstream view of scour channel at US-70 relief bridge on April 27, 1976 and, at left, riprap placed at abutment after erosion of abutment fill.

SITE 164. FORT GEORGE RIVER AT SR-105 NEAR MAYPORT, FLA.

Description of site: Lat 30°25', long 81°25', location as shown in fig 31. Bridge (figs. 32 and 33) is 1,240 ft (378 m) in length, 31 spans of 40-ft (12-m) length, bents of 20-in (51-cm) diameter precast-concrete pile; bents are perpendicular to bridge center line but skewed about 30° to flow.

Crossing is at estuary of Fort George River, subject to tidal currents. Bottom material consists of shell fragments and sand, having a D_{50} of about 0.3 mm, D_{16} of 2.5 mm, and D_{84} of 0.15 mm.

Hydraulic problem and countermeasures:

1950 Bridge built by Fernandina Port Authority, without countermeasures.

1969 General scour, possibly accompanied by local scour, was detected. at the west end of the bridge, affecting bents 4 through 10 (fig. 33). Some subsidence of bents occurred, particularly at bent 6. The scour is attributed to a shift of the thalweg toward the west side of the estuary. To prevent further subsidence, "crutch" pile bents, consisting of two 20-in (51-cm) square concrete pile, were added at each end of bents 4 through 10, and the pile were driven to bedrock, which lies at an elevation of about 47 ft (fig. 33).

1971 Site was studied by Hopkins (1971), who remeasured scour depth (fig. 33). From cross sections measured 100 ft (30 m) upstream and downstream from the bridge, he determined that the normal channel depth was about 15 ft (5 m); and that from each of these cross sections, the channel bed sloped toward the bridge, reaching a depth of about 37 ft (11 m). He did not discern cone-shaped scour holes (of the sort usually associated with local scour) at the bents, but found instead that the entire area of scour was at a fairly uniform depth of 37 ft (11 m). The flatness of the scoured area was attributed by Hopkins (1971, p. 44) to "turbulence interacting between pile groups". The effect of bent skewness was to constrict the flow.

1975 "Deflector plates" were installed at bents 4, 5, and 6 for the purpose of controlling scour and inducing sediment deposition at the pile bents. In theory, a plate controls scour by breaking up the diving currents at the upstream nose of a pile (or pier). A plate consists of a circular sheet of aluminum, 3/16 in (4.8 mm) in thickness and 8 ft (2.4 m) in diameter, with a square opening in the center that is designed to be fitted around a pile. (Some plates have two openings, and are intended to fit around two piles). The plate is split in half and reassembled around the pile by means of aluminum bars bolted to the lower side of the plate (fig. 33, inset). The plates were intended to be installed level and about 5 ft (1.5 m) above the channel bottom, by divers. The main expense of the countermeasure was the cost of installation. There were some difficulties with the fit of the center openings (collars) in the initial installation, due either to encrustations of marine organisms on the piles, or to non-uniform pile size. Also, three plates were torn from piles by tidal currents and lost during installation. Because of the difficulties of working in strong tidal currents, the plates were not installed at a uniform height above the bottom, but at heights ranging from about 1.5 ft (0.5 m) to 10 ft (3 m).

1977 According to an underwater bridge inspection made in May, recent deposition of sand has occurred beneath the plates, but no measurement of the total thickness of deposition since installation was made. Some corrosion of the aluminum fittings was noted, but it would still be possible to remove the plates and re-install them at a different height.

Discussion: Although the amount of deposition induced by the plates is uncertain, no additional scour has been reported. Evaluation of the performance of the plates in preventing scour is uncertain, as scour may have reached an equilibrium depth at the time of installation of the plates. It was originally planned to raise the plates as deposition occurred, but this has so far not been done. For any future installations, C. M. Hopkins (personal communication, 1976) would recommend that the plates be placed at, or just above, bottom elevation.

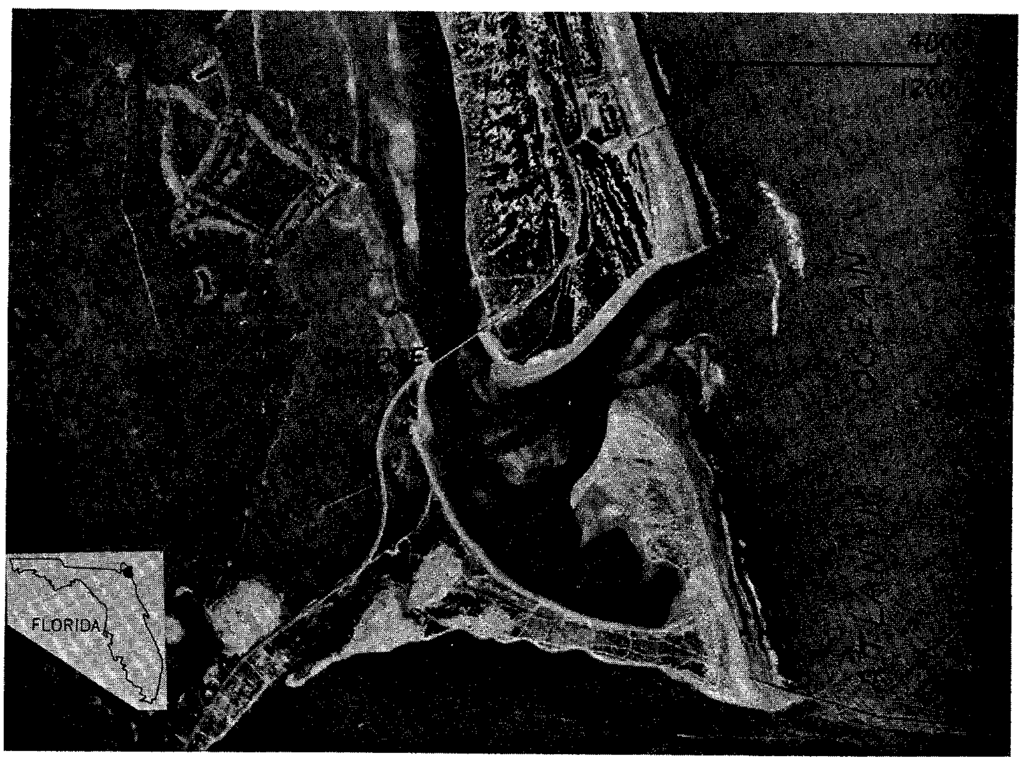


Figure 31. Aerial photograph showing general location of SR-105 crossing, Fort George River, November 8, 1969. (From Florida Dept. of Transportation.)

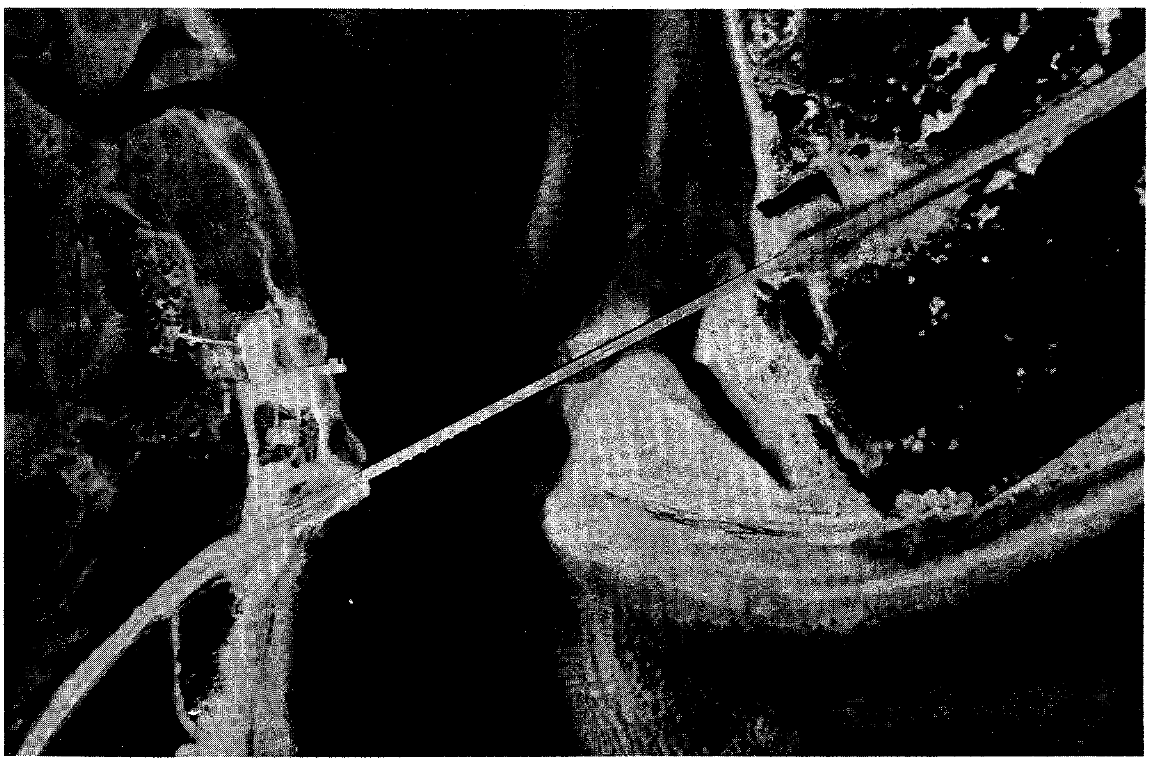


Figure 32. Aerial photograph showing SR-105 crossing, Fort George River, on May 22, 1972. (From Florida Dept. of Transportation.)

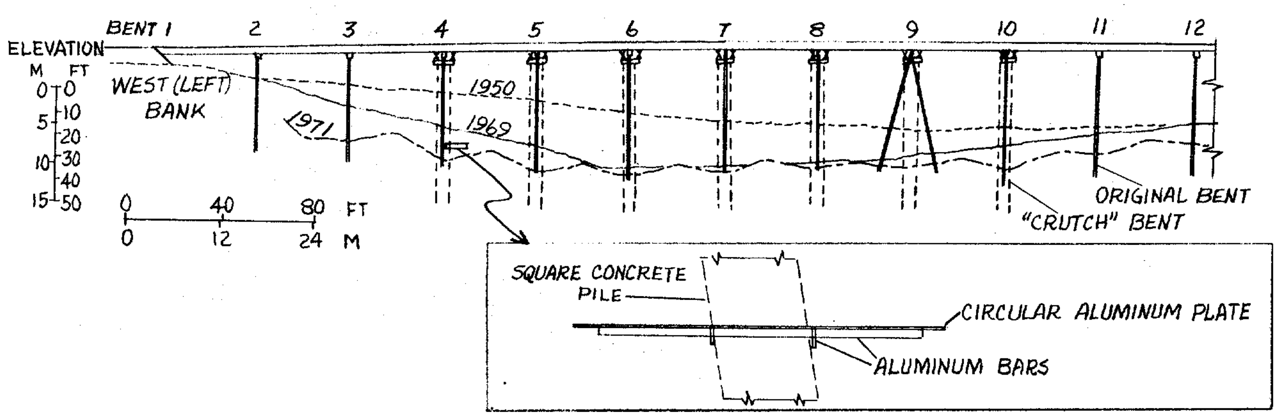


Figure 33. Cross section showing depth of scour at SR-105 crossing. Inset, circular aluminum deflector plate.

SITE 165. HOLLYWOOD CREEK AT SR-26 AT HOLLYWOOD, ARK.

Description of site: Lat $34^{o}06'$, long $93^{o}15'$, location as shown in fig. 34. Bridge is 104 ft (31 m) in length, three open piers with square concrete columns in channel, spillthrough abutments.

Drainage area, about 25 mi^2 (65 km^2); valley slope 0.0024; channel width, 40 ft (12 m). Stream is perennial, alluvial, gravel bed, in valley of low relief, wide flood plain. Channel is sinuous, not incised, sand-gravel banks, tree cover on more than 90 percent of bankline.

Hydraulic problem and countermeasure:

1954 Bridge built, no riprap placed at abutments.

1960 Major flood, of undetermined recurrence interval. Lateral erosion occurred at the downstream left abutment fill slope and embankment fill. Fill was replaced and abutments paved with concrete.

1961-76 Recurrent erosion during small floods at toe of concrete slope paving, left abutment. Pavement and fill eroded to height of about 3 ft (1 m) above streambed (fig. 35).

Discussion: The reason for erosion at the downstream side of the left abutment is not apparent, but a possible cause is turbulence induced by the square pier columns at the toe of the abutment fill-slope. Failure of the concrete slope pavement is attributed to inadequate keying-in or protection at the toe.

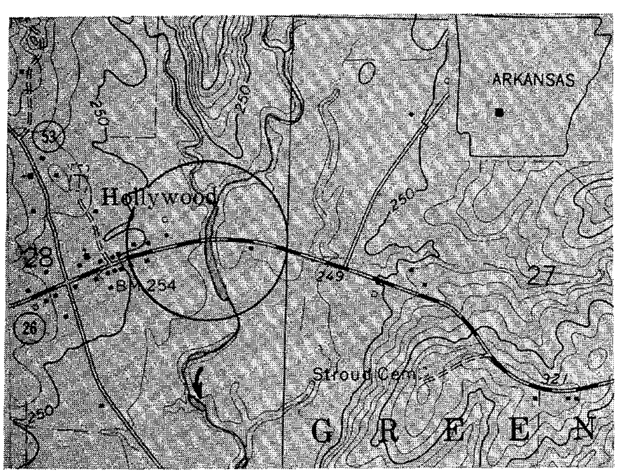


Figure 34. Map showing Hollywood Creek at SR-26 crossing (circled). (Base from U.S. Geol. Survey Hollywood, Ark., 7.5' quadrangle, contour interval 10 feet, scale 1:24,000, 1970.)

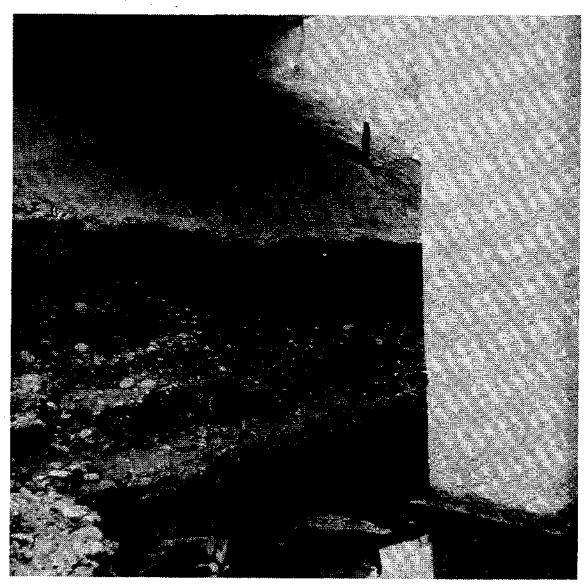


Figure 35. View in 1976 of erosion of concrete slope pavement at left abutment, Hollywood Crk, 1976.

SITE 166. LITTLE DECEIPER CREEK AT SR-8 NEAR ARKADELPHIA, ARK.

Description of site: Lat 34°07', long 93°07'. Bridge is 56 ft (17 m) in length, one concrete pier in channel, spillthrough abutments.

Drainage area, 2 mi^2 (5 km^2); valley slope, 0.0054; channel width, 25 ft (8 m). Stream is perennial, alluvial, in valley of moderate relief, wide flood plain. Channel is straight, probably incised.

Hydraulic problem and countermeasure:

1955 Bridge built. Dumped rock riprap of small size (less than 1 ft or 0.3 m in diameter) placed on abutment slopes.

1960 During flood of undetermined frequency, most of riprap was eroded from abutment slopes. Riprap was replaced and covered with concrete pavement, about 3 in (7.5 cm) thick.

1961-76 Concrete pavement and underlying fill on left abutment slope was undermined by scour, causing collapse of the concrete slope pavement (fig. 36).

Discussion: Failure of riprap is attributed to small size, and failure of concrete slope pavement is attributed to inadequate protection from scour at toe.

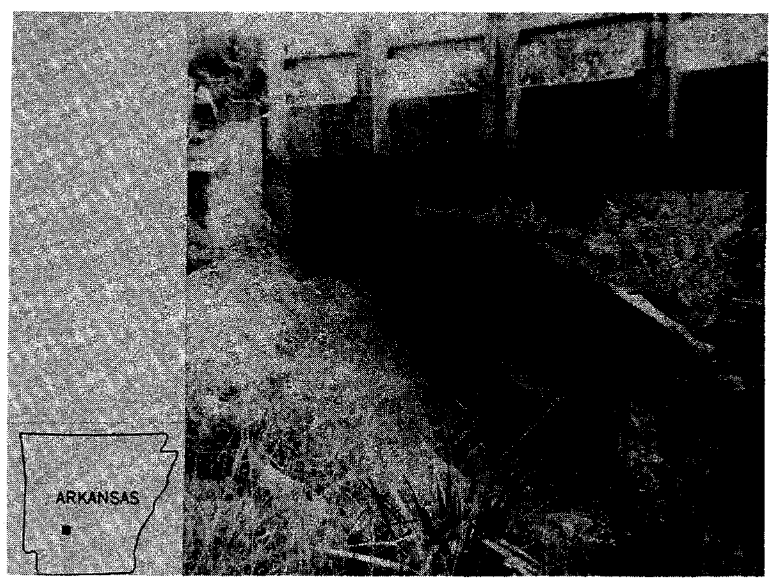


Figure 36. View of failed concrete pavement at left abutment of SR-9 bridge, April 28, 1976.

SITE 167. LITTLE SPADRA CREEK (DARDANELLE RESERVOIR) AT SR-194 NEAR CLARKSVILLE, ARK.

Description of site: Lat 35°26', long 93°30', location as shown in fig. 37. Bridge is 175 ft (53 m) in length, concrete wall-type piers in channel, spillthrough abutments. Bridge and approach embankments were constructed to accommodate flooding of creek (fig. 37) by Dardanelle Reservoir, which is on the Arkansas River.

Hydraulic problem and countermeasure: When reservoir was filled, approach embankment and abutment slopes became saturated and failed by slumping (figs. 38A and 38B). In addition, some wave erosion of embankment and abutment fill slope probably occurred. As a countermeasure, the slope angle of abutment and embankment was flattened and dumped rock riprap, approximately Class II, was placed at the slope toe. No further damage has been reported, although the slump shown in fig. 38A looks potentially active.

Discussion: The approach embankment is about 25 ft (8 m) above normal pool elevation of the reservoir, and the case provides an example of the hazard of constructing high earth fill approach embankments that are subject to saturation and wave action when partially inundated by reservoirs. Riprap cannot be expected to provide protection against internal slope failure.

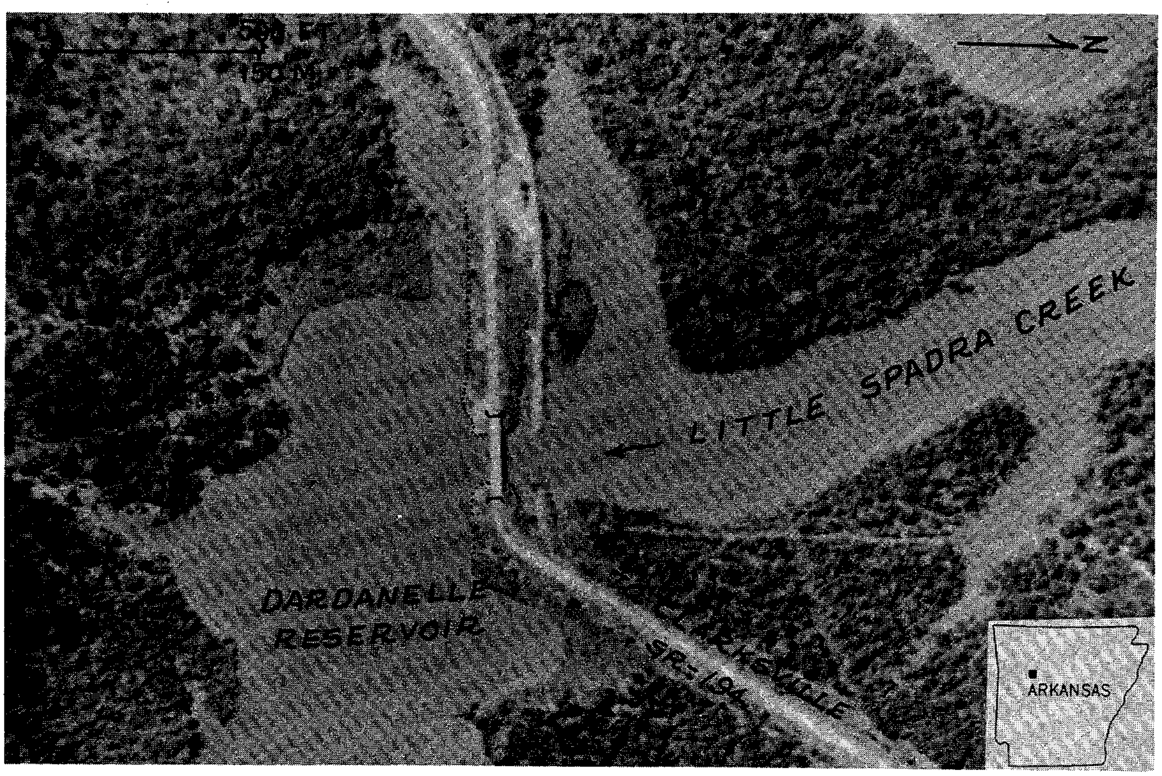


Figure 37. Aerial photograph of Little Spadra Creek at SR-194 crossing, on September 13, 1971. (From Arkansas Dept. of Highways.)

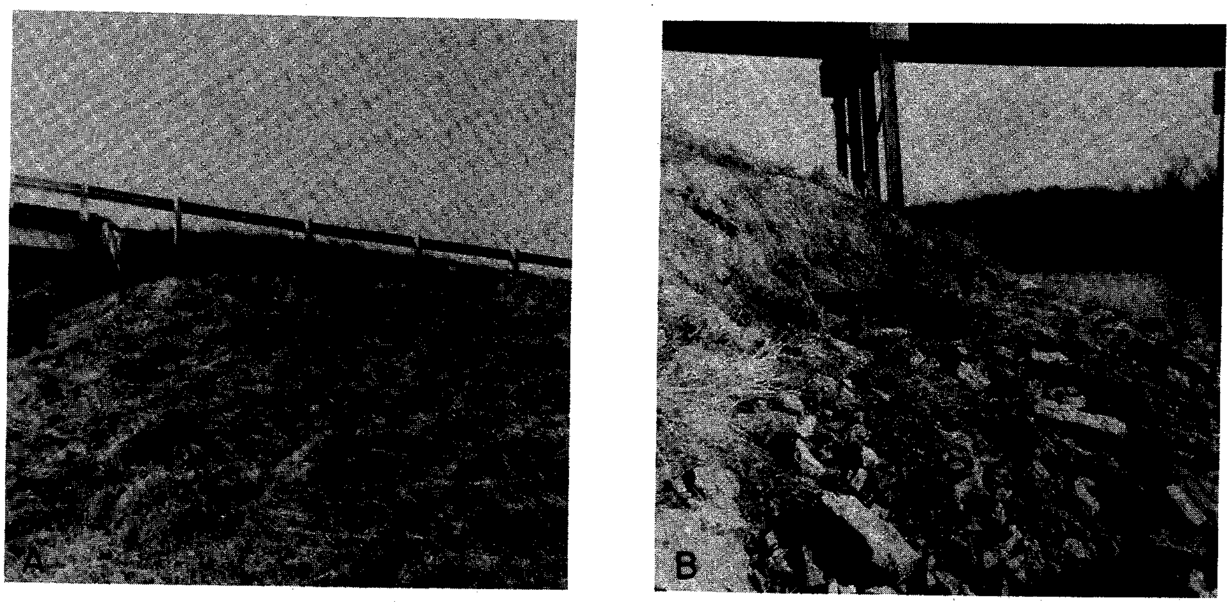


Figure 38. A, view of earth slump on upstream side of fill slope, right abutment of SR-194 bridge, April, 1976. B, view of riprap at toe of eroded fill slope, downstream side of right abutment, April, 1976.

SITE 168. PFEIFFER CREEK AT SR-25 NEAR BATESVILLE, ARK.

Description of site: Lat 35°47', long 91°37', location as shown in fig. 39. Bridge is 175 ft (53 m) in length, a concrete-slab bridge which has been raised by the addition of concrete columns and a concrete cap to each of 13 piers (fig. 40A). Length of spans, 19 ft (5.8 m); spillthrough abutments.

Drainage area, 10.6 mi^2 (27 km^2); valley slope, 0.0058; channel width, 60 ft (18 m). Stream is perennial, alluvial, gravel bed, in valley of moderate relief, wide flood plain. Channel is sinuous, wider at bends with wandering thalweg, cut banks general. Clearing of forests in the drainage basin has contributed an increased supply of floating debris, and mining of gravel in the vicinity of the bridge has contributed to channel degradation.

Hydraulic problem: During a 1975 flood of undetermined frequency, drift accumulated at the piers near the right bank and scour occurred at piers on the opposite side of the channel. The spread footing of the second pier from the left bank was undermined and the pier subsided (fig. 40B).

Discussion: Although the bridge is much longer than the channel width, accumulation of drift at one end of the bridge concentrated the flow sufficiently for scour to occur at piers near the opposite end. Drift accumulation is attributed to short bridge spans, originally designed for a low-water crossing.

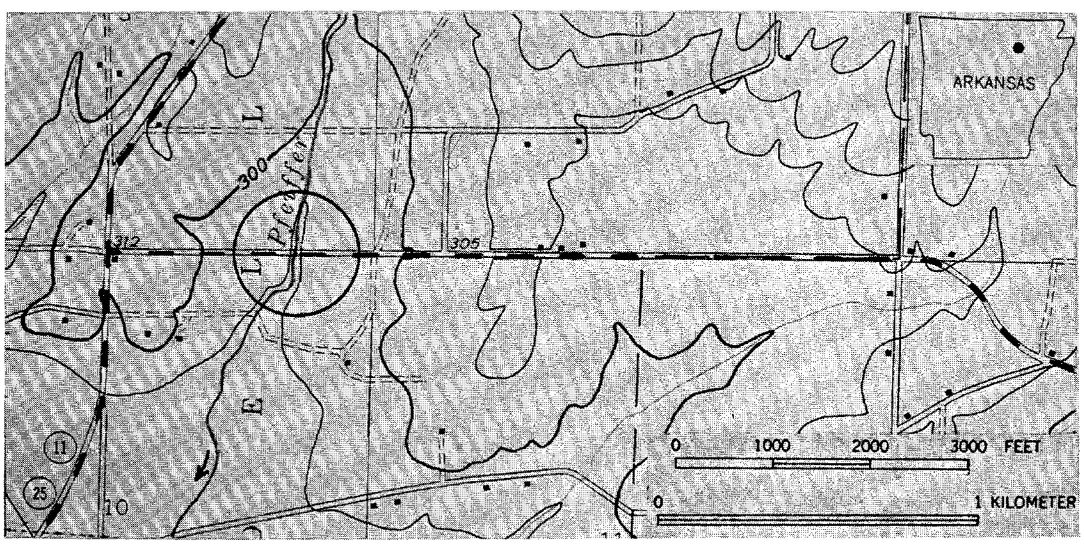


Figure 39. Map showing Pfeiffer Creek at SR-25 crossing (circled). Base from U.S. Geol. Survey Sulphur Rock, Ark., 7.5' quadrangle, contour interval 20 feet, 1943.)

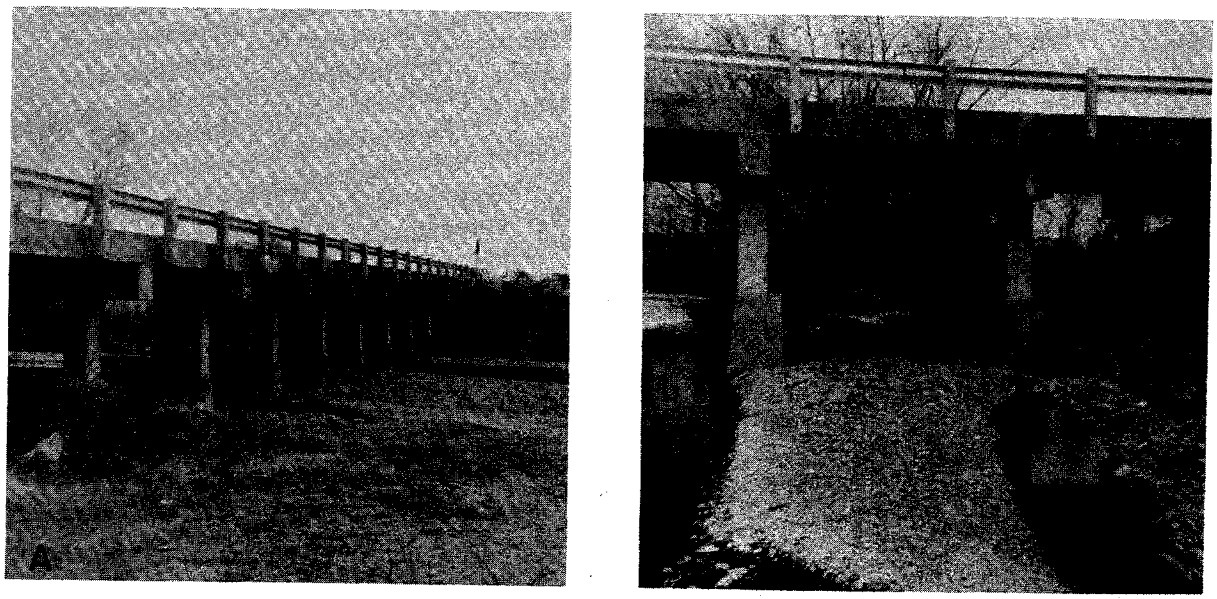


Figure 40. A, view of downstream side of SR-25 bridge, looking toward left abutment. B, upstream view of subsided pier, at right of center, March 24, 1976.

SITE 169. RED RIVER AT I-30 AT FULTON, ARK.

Description of site: Lat 33°36', long 93°49', location as shown in fig. 41. Interstate 30 bridge is 1,294 ft (394 m) in length, eight concrete piers in channel, pier footings set a minimum of 3 ft (1 m) into very hard clay at a minimum depth of about 25 ft (7.6 m) below channel bed; spillthrough abutments. Also at this site is the US-67 bridge, built in 1928, a concrete-arch structure having a length of 1,907 ft (581 m), 5 concrete piers in channel, 19 pile bents outside the channel, and vertical concrete abutments set back from the channel. Upstream from the US-67 bridge is the Missouri Pacific Railroad bridge. The left abutment fill-slope of the railroad bridge is paved with concrete, and upstream from the railroad bridge, at the outside of the sharp bend, is a large timber-pile retard that has been effective in controlling erosion at the bend.

Drainage area, 46,500 mi^2 (120,500 km^2); bankfull discharge, 207,000 ft^3/s (5,862 m^3/s); valley slope, 0.00012; channel width, about 900 ft (274 m). Stream is perennial, alluvial, sand bed, in valley of low relief. Channel is meandering, locally braided, wider at bends, point bars, cut banks general, silt-sand banks, tree cover at less than 50 percent of bankline. The Red River is characterized by a rapid rate of lateral migration, as indicated by the 1975 photorevision of the 1951 Fulton Quadrangle map (fig. 41). Owing to reservoirs on the Red River, the sediment load is stored, flood peaks are decreased, and within-channel flows near average discharge are sustained. The 50-yr R.I. flood is confined to the channel.

Hydraulic problem and countermeasure:

1967 I-30 bridge built.

1968-76 Continued migration of left bank, at outside of bend, encroached on left abutment. Timber-pile retards, consisting of a double row of 3-pile clusters, connected by timber stringers, were installed along the left bank upstream from the I-30 bridge (fig. 42A), at the abutment of the US-67 bridge, and upstream from the US-67 bridge (fig. 42B). These retards, in conjunction with riprap at the bridge abutments, have been effective in controlling lateral migration of the river.

1976 As an additional countermeasure, dumped rock riprap is being placed on the left bank upstream and downstream from the I-30 bridge.

Discussion: Lateral erosion is a problem at many crossings of the Red River. At this site, lateral erosion has been controlled partly by revetment and partly by timber-pile retards, such that it is difficult to assess their relative effectiveness. However, the timber-pile retards have evidently played the major role.

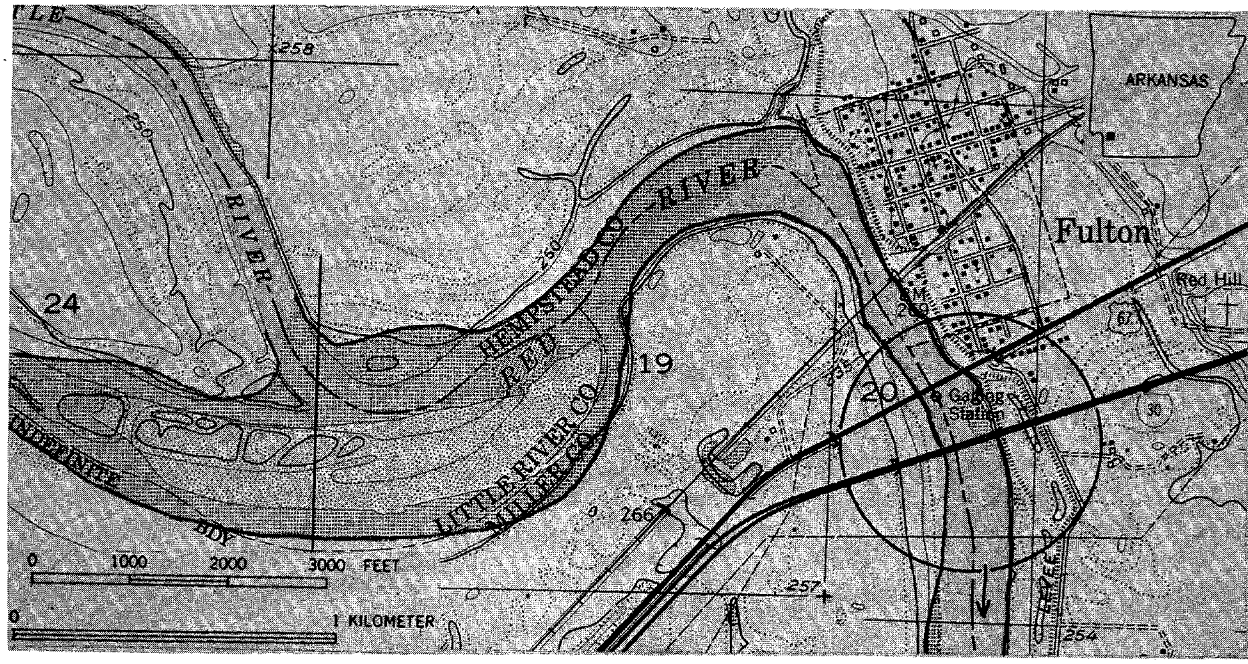


Figure 41. Map showing Red River at I-30 crossing at Fulton, Ark. Shading indicates areas of channel change during the period 1951-1975. (Base from U.S. Geol. Survey Fulton, Ark., 7.5' quadrangle, contour interval 5 feet, 1951, photorevised in 1975.)

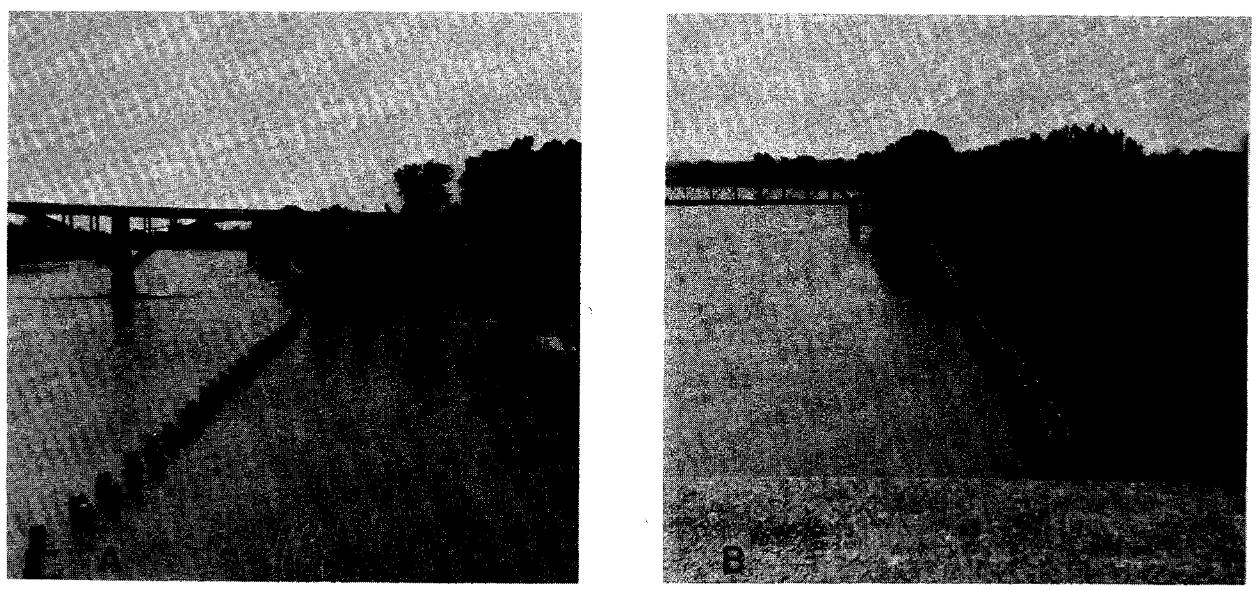


Figure 42. A, view upstream in 1976 from I-30 bridge, showing timber-pile retard at left bank, US-67 bridge in background. B, view upstream from US-67 bridge, showing timber-pile retard at left bank; Missouri Pacific Railroad bridge in background.

SITE 170. RED RIVER AT SR-41 NEAR FOREMAN, ARK.

Description of site: Lat 33°34', long 94°25', location as shown in fig. 43. Bridge is 1,262 ft (385 m) in length, five 2-column concrete piers originally in channel, 11 pile bents originally outside of channel; spillthrough abutments paved with concrete. Each pile bent consists of two cylindrical concrete piles, 2 ft (0.6 m) in diameter, cast in holes drilled to a depth of about 30 ft (9 m) below streambed elevation. Crossing is at bend.

Drainage area, 42,000 mi^2 (108,780 km^2); bankfull discharge, 130,000 ft^3/s (3682 m^3/s); valley slope, 0.0002; channel width, 980 ft (299 m). Stream is perennial, alluvial, sand bed, in valley of low relief, wide flood plain. Channel is meandering, locally braided, wider at bends, cut banks general, silt-sand banks. The Red River in this vicinity has a history of lateral migration, as indicated by the position of the channel in 1970 as compared with the position of a past centerline (heavy dashed line, fig. 43).

Hydraulic problem and countermeasure:

1962 Bridge built, concrete slope pavement installed on abutment slopes.

1962-74 Left bank migrated about 50 ft (15 m) laterally, such that three pile bents originally on flood plain are now in the channel. The main spans are near the right bank and mostly landward of the low-water channel. Three pile bents have been exposed to a depth of about 10-15 ft (3-5 m) below former ground level (fig. 44). According to the Tulsa District of the U.S. Army Corps of Engineers, the left abutment is in danger of being outflanked during a major flood.

1976 A bank protection project was planned by the Corps of Engineers, but bids were excessive and the plan is being revised. The counter-measure proposed was a field of steel jacks along the left bank upstream from the bridge, and a revetment of dumped rock riprap at the left abutment.

Discussion: This case demonstrates that lateral bank erosion may occur on the inside of a large bend. Flow is being deflected toward the eroding bank by a secondary bend upstream from the bridge (fig. 45). Secondary bends commonly develop at large bends of meandering rivers.

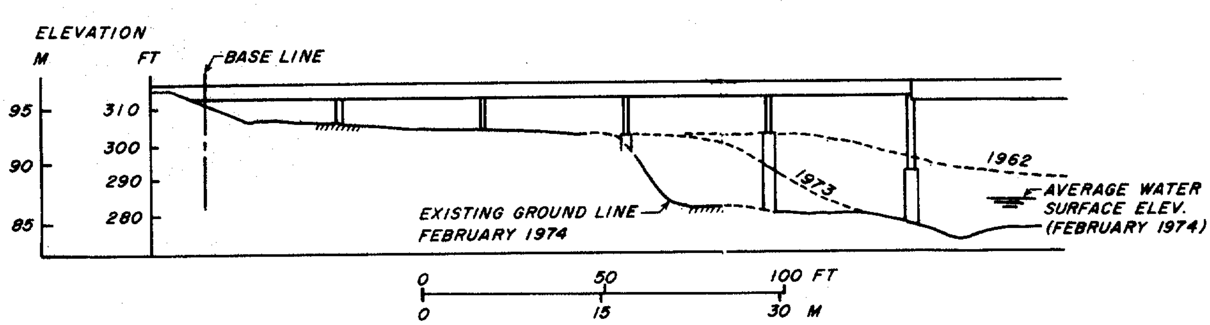


Figure 44. Cross section showing lateral erosion of left bank at SR-41 bridge. (From a survey made in 1974 by Arkansas Dept. of Highways.)

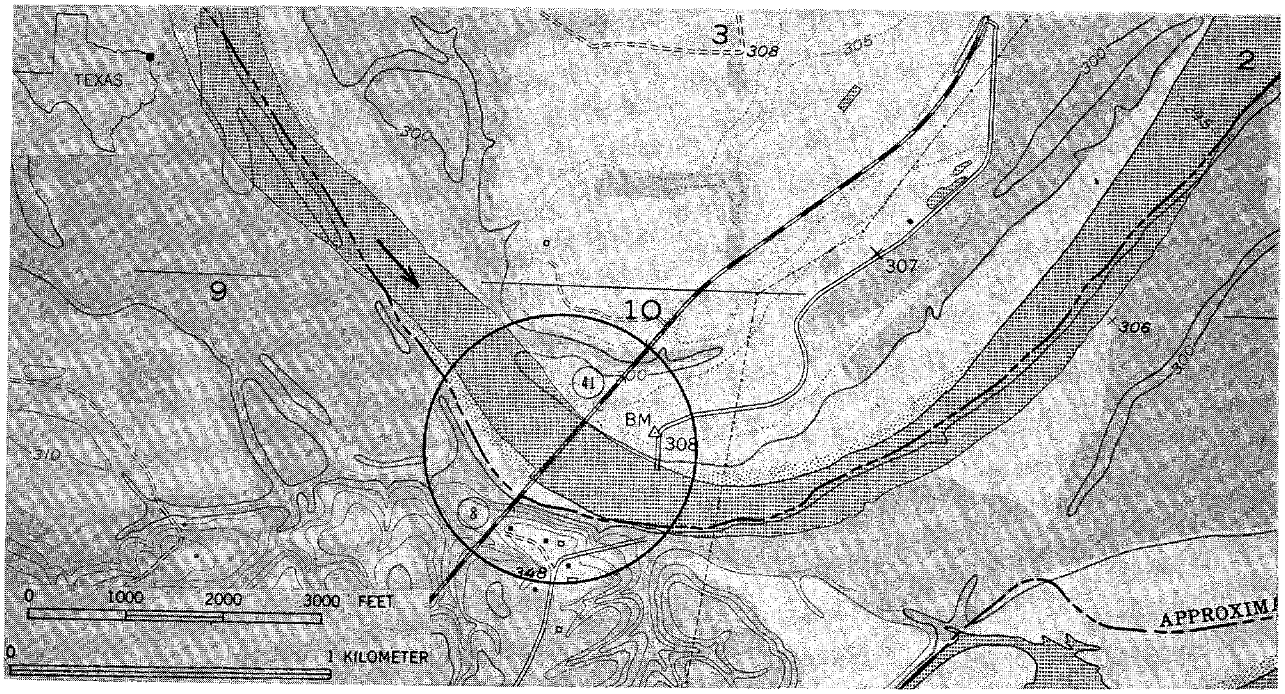


Figure 43. Map showing Red River at SR-41 crossing near Foreman, Ark. (Base from U.S. Geol. Survey Daniels Chapel, Tex.-Ark. 7.5' quadrangle, contour interval 10 feet, 1951 photorevised 1970.)

Figure 44. (on p. 318)

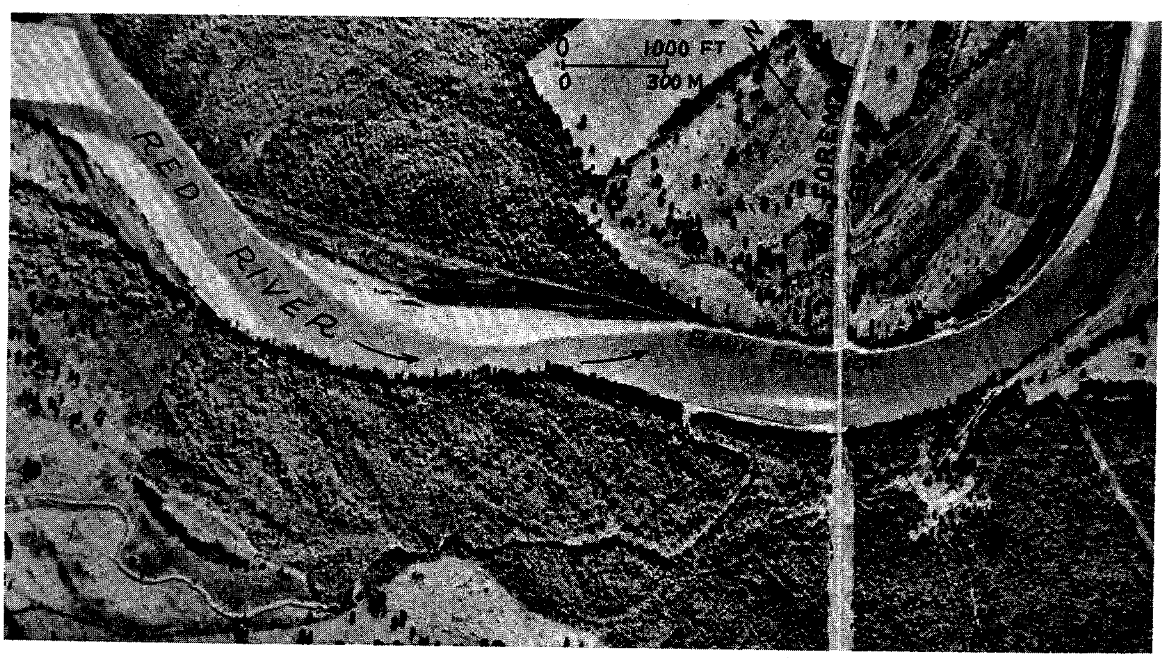


Figure 45. Aerial photograph of Red River at SR-41 crossing on October 18, 1974. (From Arkansas Dept. of Highways.)

SITE 171. RED RIVER AT US-82 AT GARLAND, ARK.

Description of site: Lat 33°21', long 93°42', location as shown in fig. 46. Bridge is 1,849 ft (564 m) in length, three steel-truss main spans, four webbed concrete piers in channel, vertical concrete abutments set back from channel. Crossing is at slight bend in channel.

Drainage area, 46,700 mi^2 (121,000 km^2); bankfull discharge, 185,000 ft^3/s (5,235 m^3/s); valley slope, 0.00022; channel width, about 1,000 ft (300 m). Stream is perennial, alluvial, sand bed, in valley of low relief, wide flood plain. Channel is meandering, locally braided, wider at bends, point bars, cut banks general, silt banks, tree cover at 50-90 percent of bankline. Owing to reservoirs on the Red River, the sediment load is stored, flood peaks are decreased, and within-channel flows near average discharge are sustained. Since regulation, the point bars have become vegetated, thus changing the aspect of the channel.

1931 Bridge built.

1931-76 The left bank at the crossing site receded about 210 ft (64 m) because of its location at the outside of a small bend. Timber-pile retards were installed upstream from the bridge, at the St. Louis Southwestern Railroad bridge (figs. 47 and 49). At the US-82 bridge, the left streambank (fig. 48) has been revetted with dumped rock riprap, some of which has been removed by erosion.

Discussion: The timber-pile retards at the railroad bridge have controlled, but not prevented, lateral migration of the river. Downstream from the railroad bridge, the retards are operating in conjunction with riprap bank revetment. Although the relative effectiveness of the two countermeasures cannot be assessed, the retards are located where the most severe bank erosion would be expected.

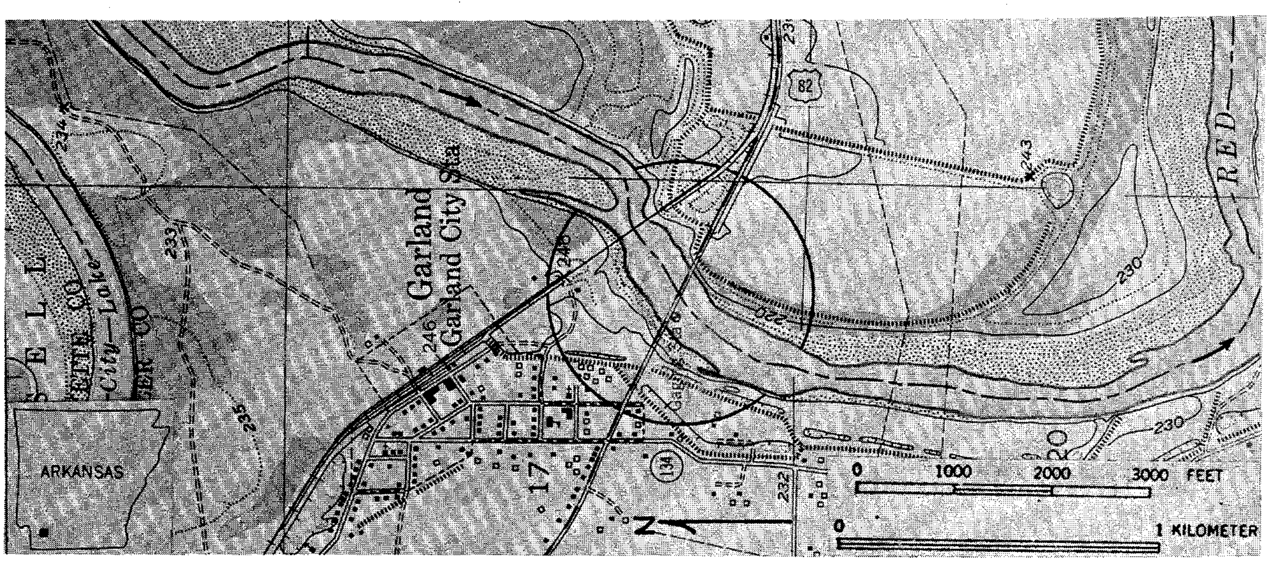


Figure 46. Map showing Red River at crossing of US-82 and the St. Louis southwestern railroad bridge. (Base from U.S. Geol. Survey Garland, Ark., 7.5' quadrangle, contour interval 10 feet, 1952.)

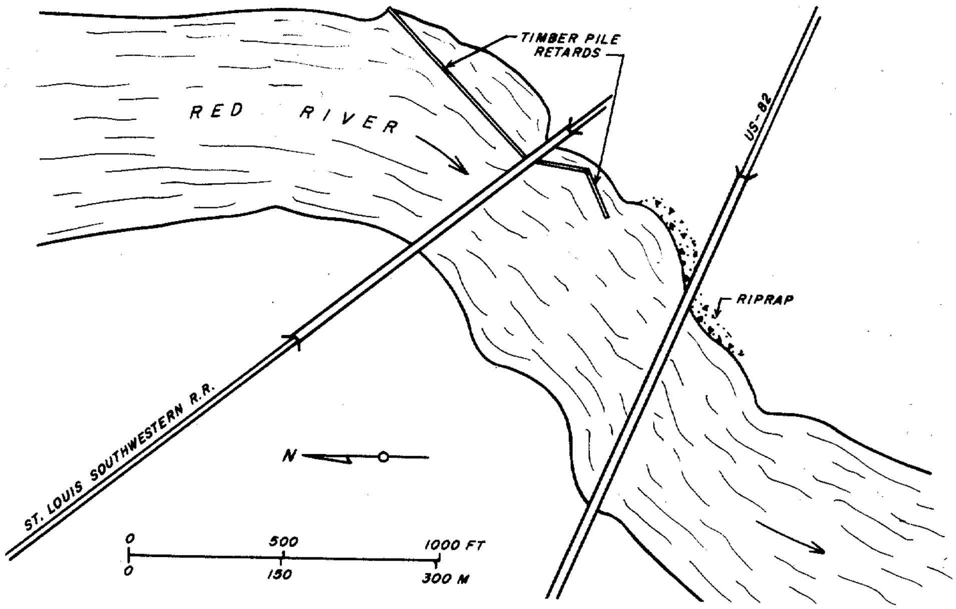


Figure 47. Plan sketch showing countermeasures at US-82 crossing. (Traced from aerial photograph taken in in 1974.)

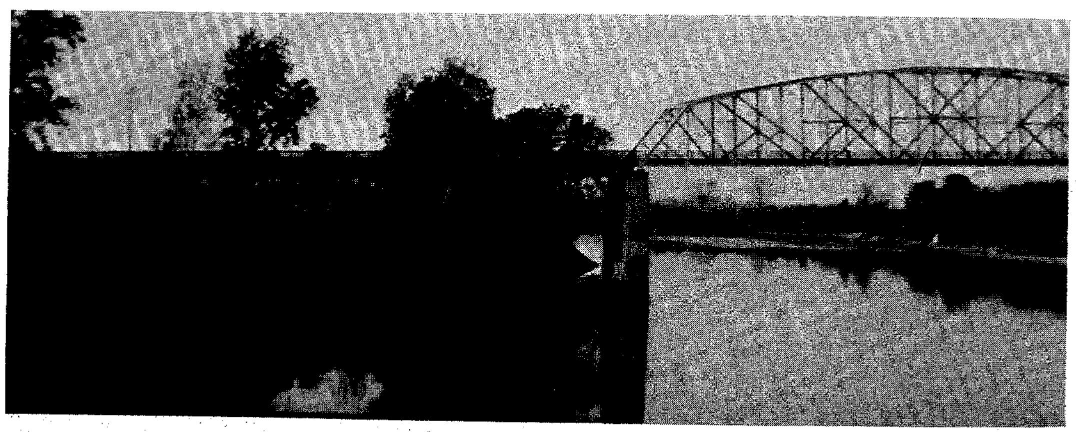


Figure 48. Downstream view of pier at left bank, US-82 bridge, in April, 1976. Pier was 210 ft from streambank at time of bridge construction.

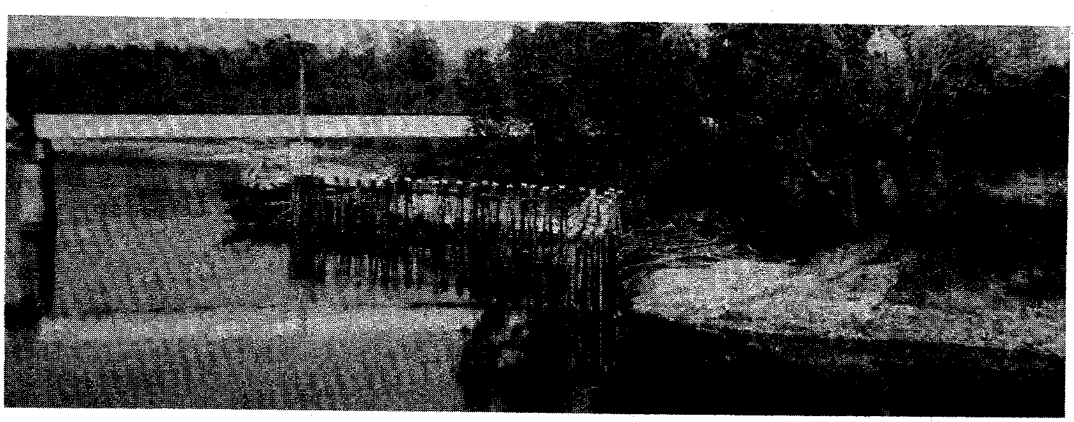


Figure 49. View of timber-pile retard at left bank upstream from US-82 bridge, April, 1976.

SITE 172. SMACKOVER CREEK AT SR-57 RELIEF BRIDGE NEAR STEPHENS, ARK.

Description of site: Lat 33°24', long 93°04', location as shown in fig. 50. Relief bridge is 90 ft (27 m) in length, concrete-slab spans, four pile bents in channel, octagonal concrete pile. Crossing is at a small anabranch (overflow channel) of Smackover Creek.

Drainage area, 6,960 mi^2 (18,000 km^2); valley slope, 0.00095.

Hydraulic problem and countermeasure:

1954 Bridge built, no revetment at abutments.

1958 During spring flood of undetermined frequency, but estimated to be well in excess of design flood, abutment fill-slopes were removed by general scour and channel was widened and deepened at bridge (fig. 51A). The abutment fill-slopes were replaced and concrete slope pavement was installed on the slopes.

1959-76 Recurrence of general scour. Upstream end of concrete pavement on right abutment was undermined, and fill slope was eroded (fig. 51B).

Discussion: Although no details were obtained as to the construction of the concrete pavement, field inspection indicates that the pavement was inadequately protected at the toe and that it was not carried far enough around the abutment fill-slope.

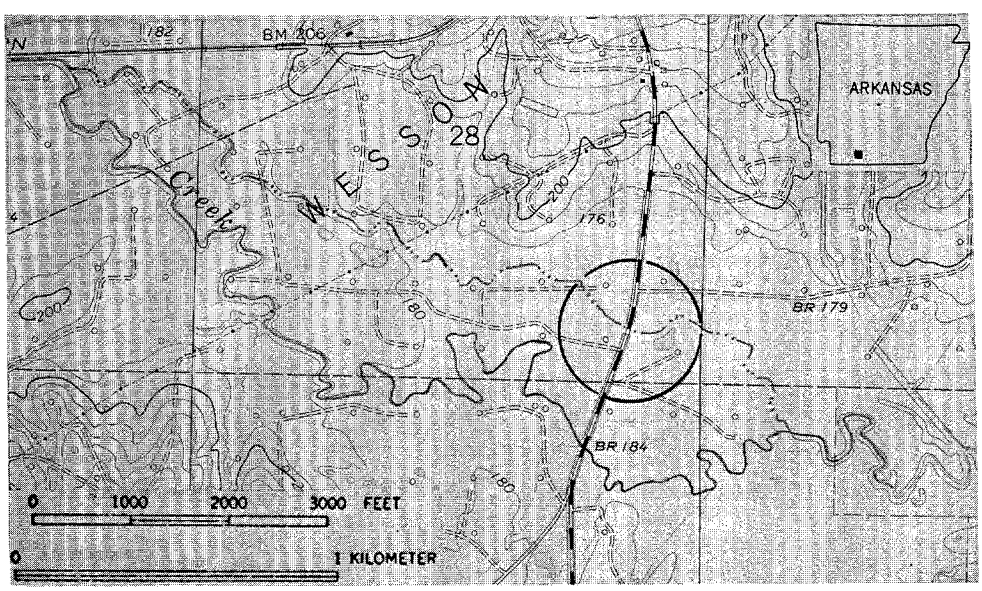


Figure 50. Map showing Smackover Creek at SR-57 relief bridge (circled). (Base from U.S. Geol. Survey Stephens, Ark., 7.5' quadrangle, contour interval 10 feet, 1969.)

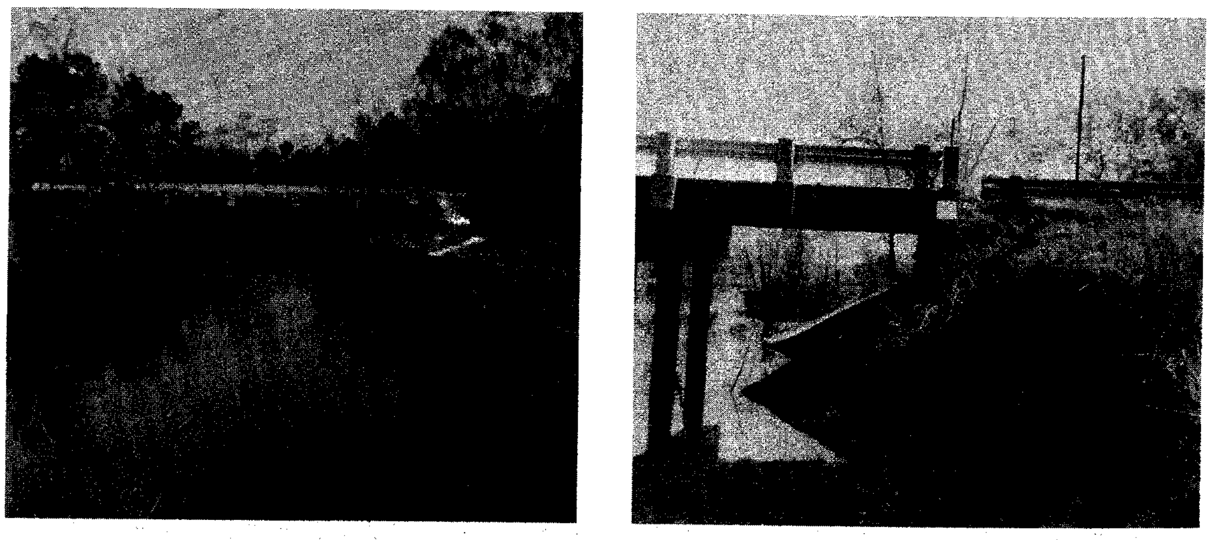


Figure 51. A, upstream view of relief bridge and channel scoured by flood flow. B, erosion of fill and concrete slope pavement at right abutment. Photographs taken in 1976.

SITE 173. SULPHUR RIVER AT US-71 NEAR FT. LYNN, ARK.

Description of site: Lat 33°09', long 93°54', location as shown in fig. 52. Bridge is 463 ft (141 m) in length, two steel-truss spans, one concrete webbed pier with spread footing in channel; spillthrough abutments, with concrete wingwalls, on 60-ft (18-m) precast-concrete pile foundations. Crossing is at inflection point between two bends in channel (fig. 52).

Bankfull discharge, 25,000 ft^3/s (708 m^3/s); channel width, 150 ft (46 m); valley slope, less than 0.0002. Stream is perennial, alluvial, silt-clay bed, in valley of moderate relief, wide flood plain. Channel is meandering, equi-width in its natural state, not incised, cut banks local where forested but general where banks are cleared (fig. 53). Logs of preconstruction test holes at site showed surficial stratum of clay overlying sand-silt, and surface fissures (fractures) on banks indicated tendency of banks to slump (fig. 54). Owing to upstream reservoirs, sediment load has been stored, flood peaks have been decreased, and flows near average have been sustained for a long period.

Hydraulic problem and countermeasures:

1931 Bridge built, no countermeasures for bank erosion.

1953 Regulation by upstream reservoir (Lake Texarkana) began.

1958 Major floods, accompanied by large flow releases from reservoir. Lateral erosion at right abutment and approach embankment noted. In April, right embankment adjacent to abutment washed out. Thirty-three freight-car loads of rock riprap were dumped at embankment and abutment, but the riprap was displaced by erosion.

1958-73 Continued erosion at abutment, timber-pile bulkheads installed (figs. 55A and 55B).

1975 Plans to install concrete erosion-control mat abandoned, because of recurrent slumping of embankment and continued stream erosion. The timber-pile bulkhead was holding temporarily, and plans for lengthening bridge were under consideration.

Discussion: The channel degradation and lateral erosion is attributed to sustained releases of water from reservoir, depletion of sediment load by reservoir, and to lack of vegetal cover on banks. Failure of riprap is attributed to slumping of unstable streambank and fill material. The erosion occurred at the right abutment because of its location at the outside of a bend, but bend was evidently more stable before flow regulation. Surficial fractures (fissures) in soil along streambanks, observed at time of bridge construction, indicated susceptibility of banks to slumping. Banks susceptible to slumping are evidently difficult to stabilize with riprap, particularly if they are high.

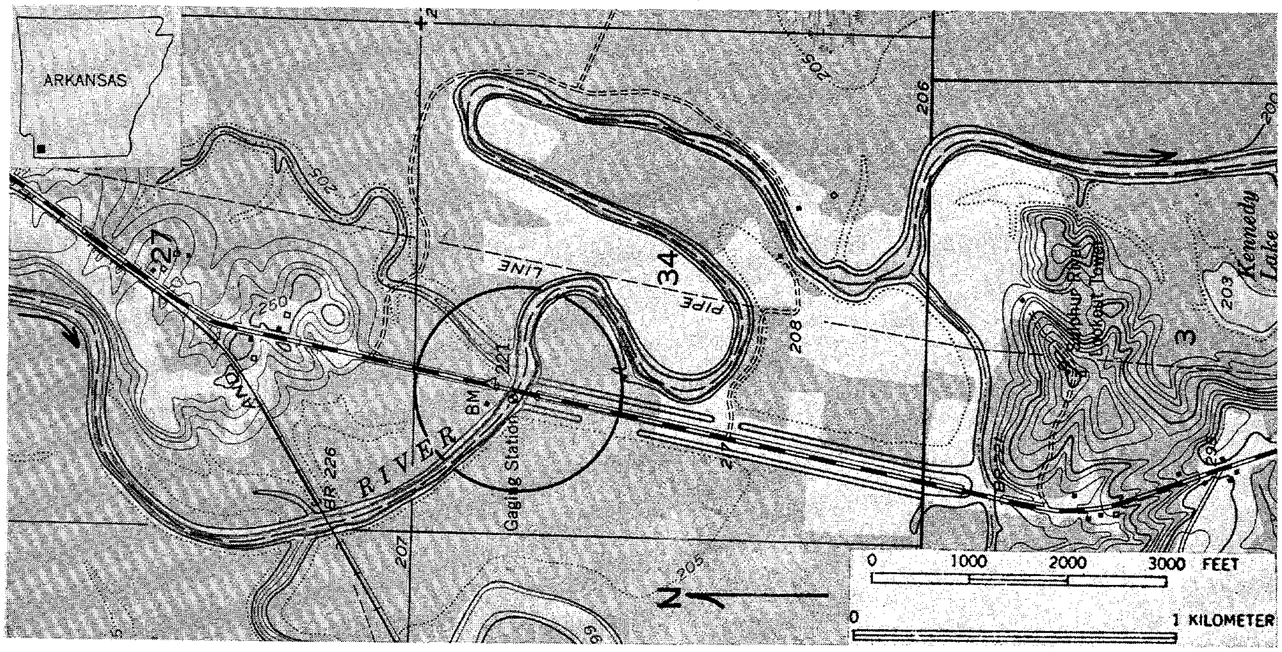


Figure 52. Map showing Sulphur River at US-71 crossing (circled). (Base from U.S. Geol. Survey Doddridge NW, Ark., 7.5' quadrangle, contour interval 20 feet, 1952.)

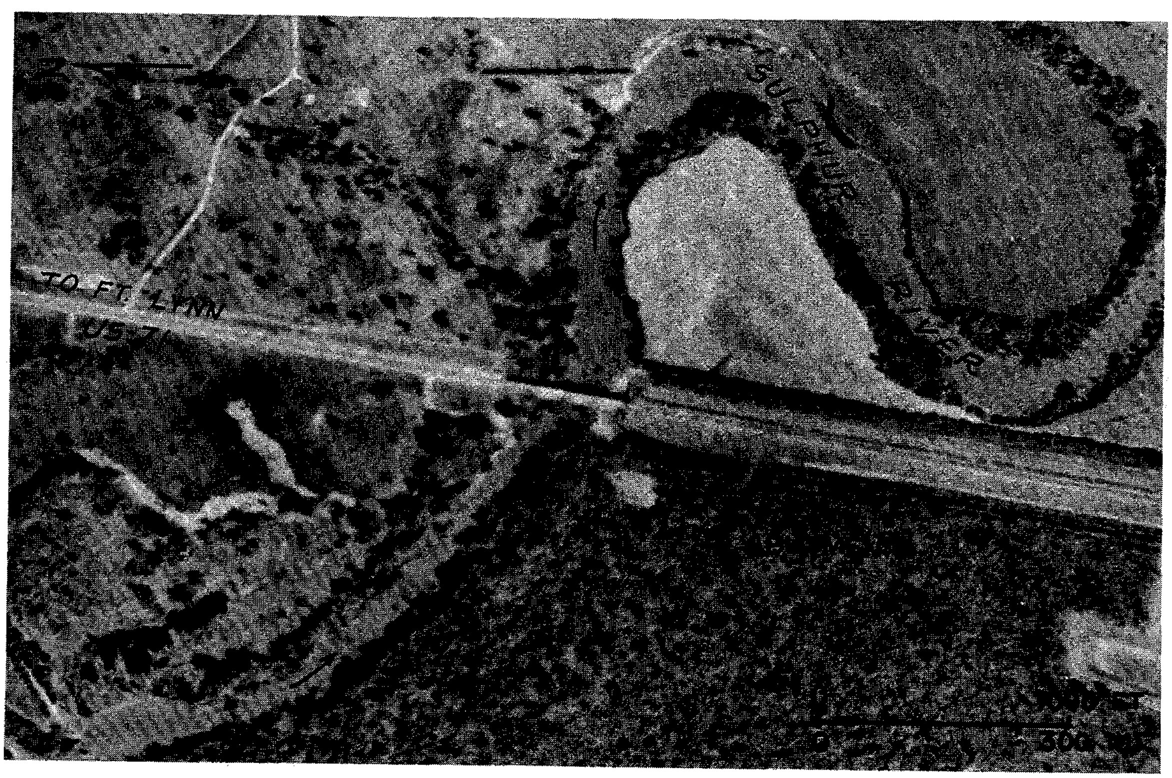


Figure 53. Aerial photograph of Sulphur River at US-71 crossing, on October 16, 1974. (From Arkansas Dept. of Highways.)

appears to have good potential for regions where other types of spurs are subject to damage by ice.

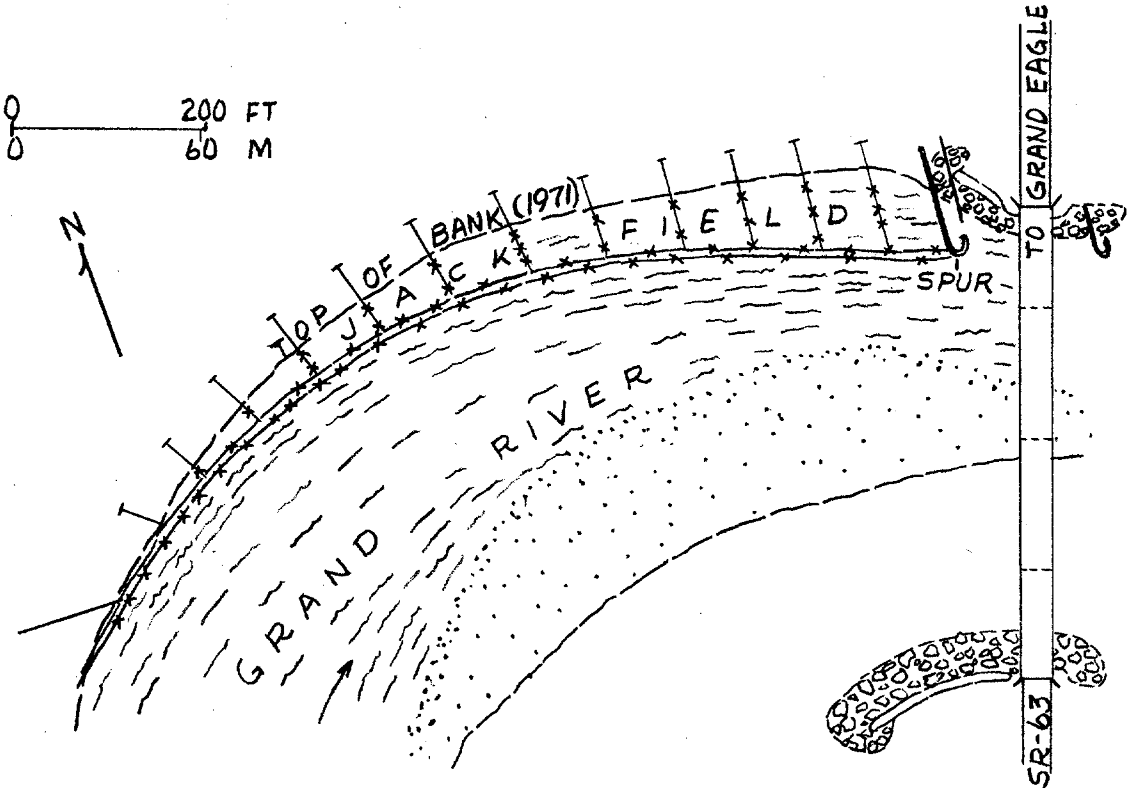


Figure 63. Plan sketch showing countermeasures installed at SR-63 crossing in 1971.

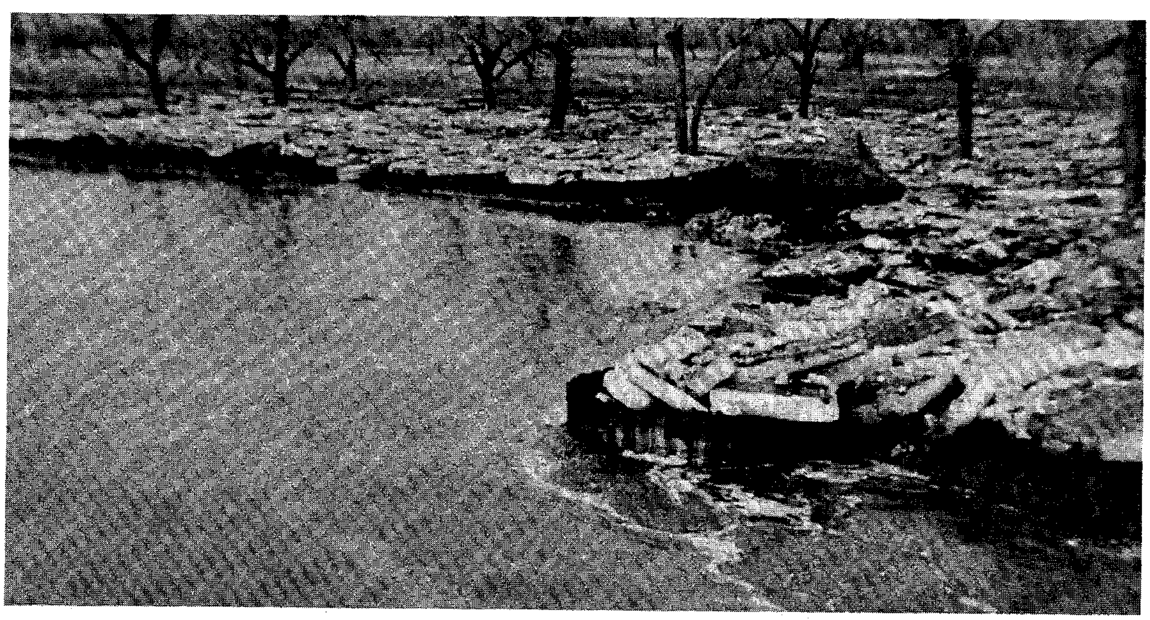


Figure 64. View of left bank upstream from bridge on March 14, 1972, showing flow near bankfull stage and ice blocks on sheet pile spur. (From South Dakota Dept. of Highways.)

SITE 174. HOMOCHITTO RIVER AT SR-33 AT ROSETTA, MISS.

Description of site: Lat 31°19', long 91°06', location as shown in fig. 56. Original bridge, built in 1940, consisted of 60-80-60 ft (18-24-18-m) main spans flanked by several 19-ft (5.8-m) approach spans on timber-pile bents. Four concrete piers in the main channel had concrete footings on timber-pile foundations. In 1963, the bridge was rebuilt to a length of 603 ft (184 m) by the Mississippi State Highway Department and countermeasures were added, as described below. The Illinois Central Railroad bridge, about 700 ft (213 m) upstream from the highway bridge, tends to catch drift at one or another of its 40-ft (12-m) spans and thus to alter the direction of approach flow at the highway bridge.

Drainage area, 750 mi^2 (1,942 km^2); valley slope, 0.0004; channel width in 1971, about 500 ft (152 m). Stream is perennial, alluvial, sand bed, in valley of moderate relief, wide flood plain having distinct natural levees. In 1971, channel (fig. 57) was meandering, locally braided, wider at bends, point bars, cut banks general, silt-sand banks.

In 1906, the channel width of the Homochitto was less than 100 ft (30 m) at the U. S. Geological Survey gage at Rosetta. In 1938, the lower course of the Homochitto (beginning at Doloroso, Miss., about 16 valley miles, or 26 km, downstream from Rosetta) was diverted into the Mississippi River by a new channel, which shortened the course from about 17 mi (27 km) to about 9 mi (14.5 km). In 1940, 14 meander cutoffs were made in the 28-mi (45-km) reach between Doloroso and Rosetta, reducing the river distance between these points by about 4 mi (6.4 km). Channel degradation began at Rosetta in the late 1940's and amounted to about 3 ft (1 m) by 1949, about 8 ft (2.4 m) for the period 1949-66, no measured change for 1966-71, and about 4 ft (1.2 m) for 1973-74 (K. V. Wilson, written communication, May 22, 1974). Total degradation at Rosetta amounted to about 15 ft (4.6 m) in 1974. The degradation has been accompanied by severe bank erosion, channel widening (fig. 58), and shift in position of the main channel.

Hydraulic problems and countermeasures:

1940 Bridge built by U. S. Forest Service.

1953 During series of minor floods (R.I. about 2 yr), pile bents of bridge subsided because of degradation. Mississippi State Highway Department drove steel sheet-pile encasements around main piers to depth of about 20 ft (6 m) below footings and filled them with concrete. Approach spans, supported by steel pile bents, were added, and dumped rock riprap was placed on streambanks and approach embankments. In addition, seven timber-pile spurs were built on the north bank (fig. 59) and several clusters of three timber pile were driven. Each timber-pile spur consisted of three single rows of pile.

1955 Series of minor floods. Scour occurred to a depth of 27 ft (8 m) below top of pier footings. Upstream spur was badly damaged, as was riprap on north bank between highway and railroad. Pier scour attributed to poor configuration of piers (fig. 60), which obstructed flow and caught drift, to skewness of piers to flood flow, to high mean stream velocity (about 7.5 ft/s, or 2.2 m/s), and to progressive degradation of channel.

1956 Right (north) streambank riprapped for distance of 600 ft (180 m) between highway and railroad bridges. Steel pile driven to depth of 90 ft (27 m) beneath footings of main piers. Drift deflectors, consisting of railroad rails driven at 45° to main piers on upstream side of pier, installed.

1957-64 Series of floods, maximum having peak discharge of 141,000 ft^3/s (3,990 m^3/s), R.I. 10 yr, mean velocity in bridge waterway 10.8 ft/s (3.2 m/s). Drift defectors on piers damaged or destroyed. Riprap on right bank eroded and bank recession endangered approach spans. Deflection of current toward right bank probably due to drift accumulation in center spans of railroad bridge.

1965 Revetment made of 160 car bodies anchored with steel cable placed at right bank, to replace eroded riprap.

1973 Flood of March 24 (peak discharge 124,000 ft^3/s or 3,472 m^3/s) destroyed car-body revetment, during the time that three new 100-ft (30-m) spans were being added to north end of bridge. North bank eroded to point 150 ft (45 m) from new abutment.

1974 Flood of April 13 (peak discharge 150,000 ft^3/s (4,245 m^3/s), R.I. in range of 10-25 yr) eroded laterally against pile bents of new spans (which had been driven 60-70 ft, or 18-21 m, below streambed) causing them to fail; and two of the new spans fell into the river.

1975-76 Lateral erosion of right bank has progressed to a point beyond abutment of failed bridge. A new bridge 1500 ft (450 m) in length is under construction; it is to have round concrete piers whose footing tops are 30 ft (9 m) below streambed. Steel foundation pile beneath the footings are driven to a depth of 100 ft (30 m) below streambed. No bank protection is planned.

Discussion: Of the many types of countermeasures tried at the SR-33 bridge, none was adequate against the combination of unfavorable hydraulic factors: recurrent floods, severe degradation, abundant drift, high stream velocity, and rapid lateral migration of a high streambank. Failure of the timber-pile spurs is attributed to the impact of drift during floods, which broke off the piling. Failure of the riprap bank revetment is apparently due to undercutting of the bank and, in view of the depth of scour in the channel (about 40 ft or 12 m below flood plain elevation), the toe of the riprap would be difficult to protect. According to available information, the riprap used was approximately equivalent to class I, and class II riprap was needed.

Approach spans of the Illinois Central Railroad bridge have been lost (one in 1955) and the bridge has been rebuilt and lengthened. In spite of a rather battered appearance (fig. 61) the main piers are intact and the bridge was kept in service. The ability of railroads to move bridge spans from one place to another gives them a decided advantage in coping with a river that migrates laterally.

Although no bank protection is planned for the new SR-33 bridge, this may be needed to hold the river in its present course. By continued erosion of the right bank near the railroad bridge, the river may be diverted from its present course to a new course that intersects SR-33 at the present relief bridge (fig. 58). The probability that this diversion may occur within the next 50 years is estimated at 50 percent (K. V. Wilson, written communication, May 22, 1974).

During excavation for pier construction at the new bridge, several car bodies were found at depths of about 30 ft (9 m) below streambed elevation, and broken concrete riprap was found at a depth of about 45 ft (14 m), at the top of a clay stratum. The car bodies, torn from the bank revetment, may have lodged against the piers of the old bridge, inducing local scour and perhaps failure of the bridge.

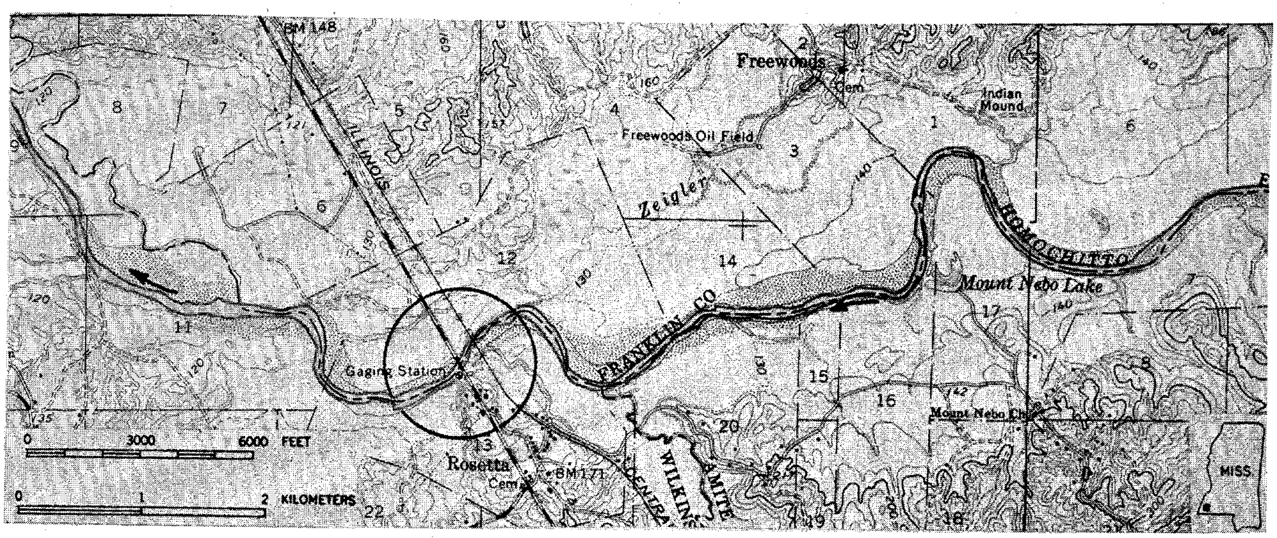


Figure 56. Map showing Homochitto River, SR-33 crossing at Rosetta, Miss. (Base from U.S. Geol. Survey Crosby, Miss., 15' quadrangle, contour interval 20 feet, 1961.)

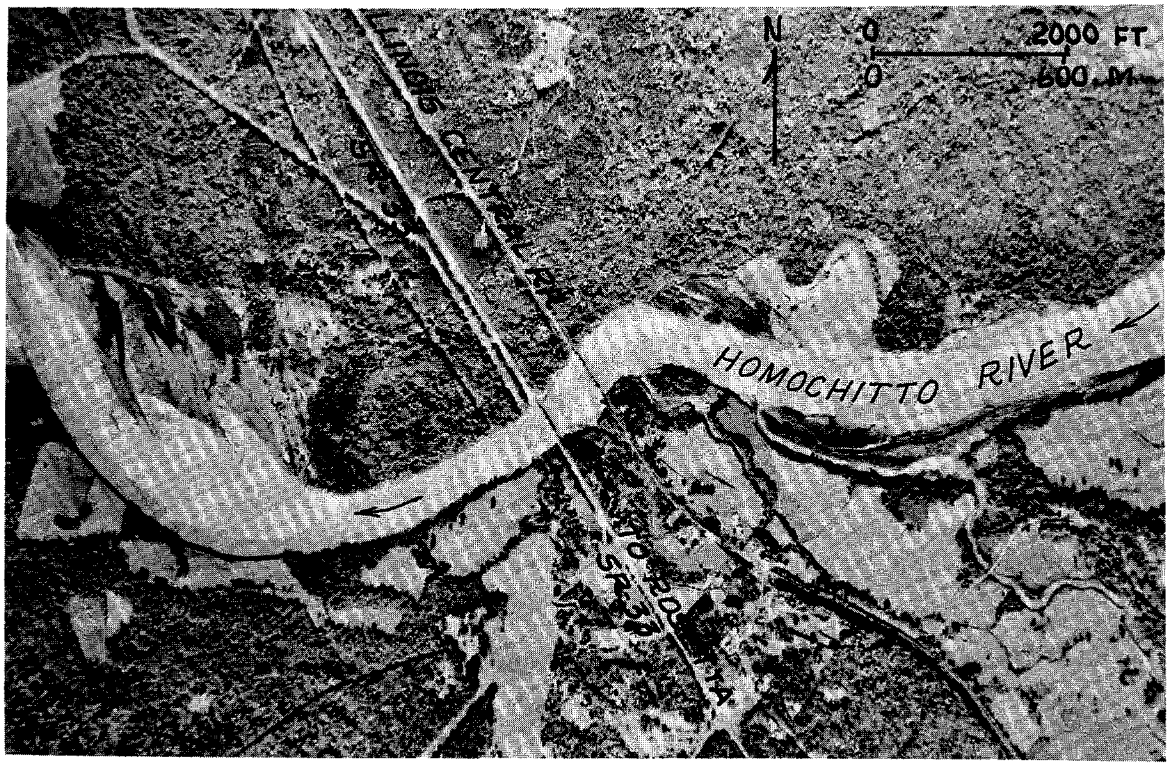


Figure 57. Aerial photograph showing Homochitto River, SR-33 crossing at Rosetta, Miss. (From U.S. Dept. of Agriculture, November 20, 1971.)

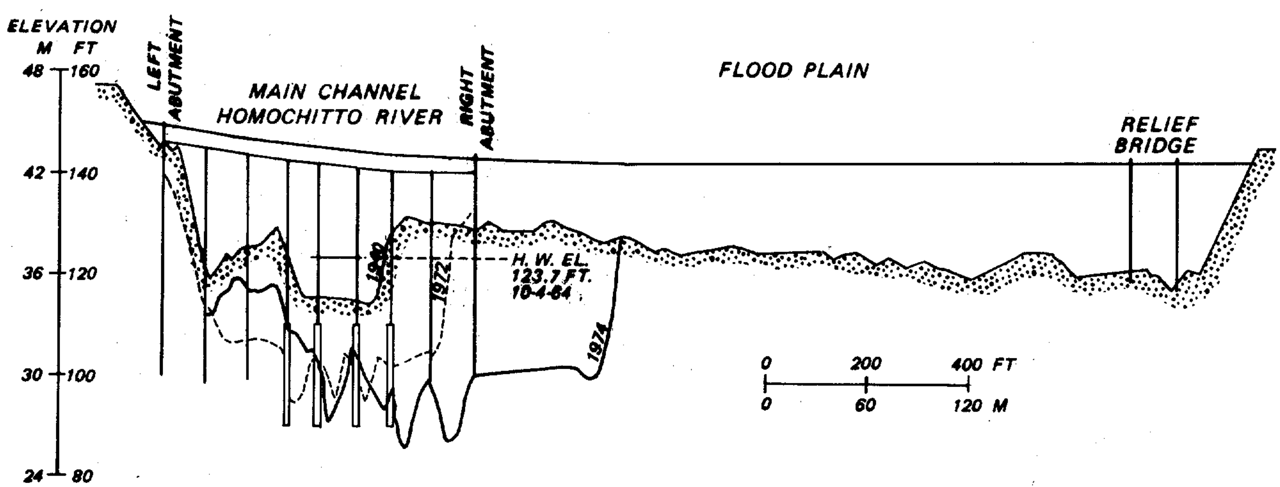


Figure 58. Cross section of Homochitto River, SR-33 crossing at Rosetta, showing degradation and lateral erosion during the period 1940-1974.

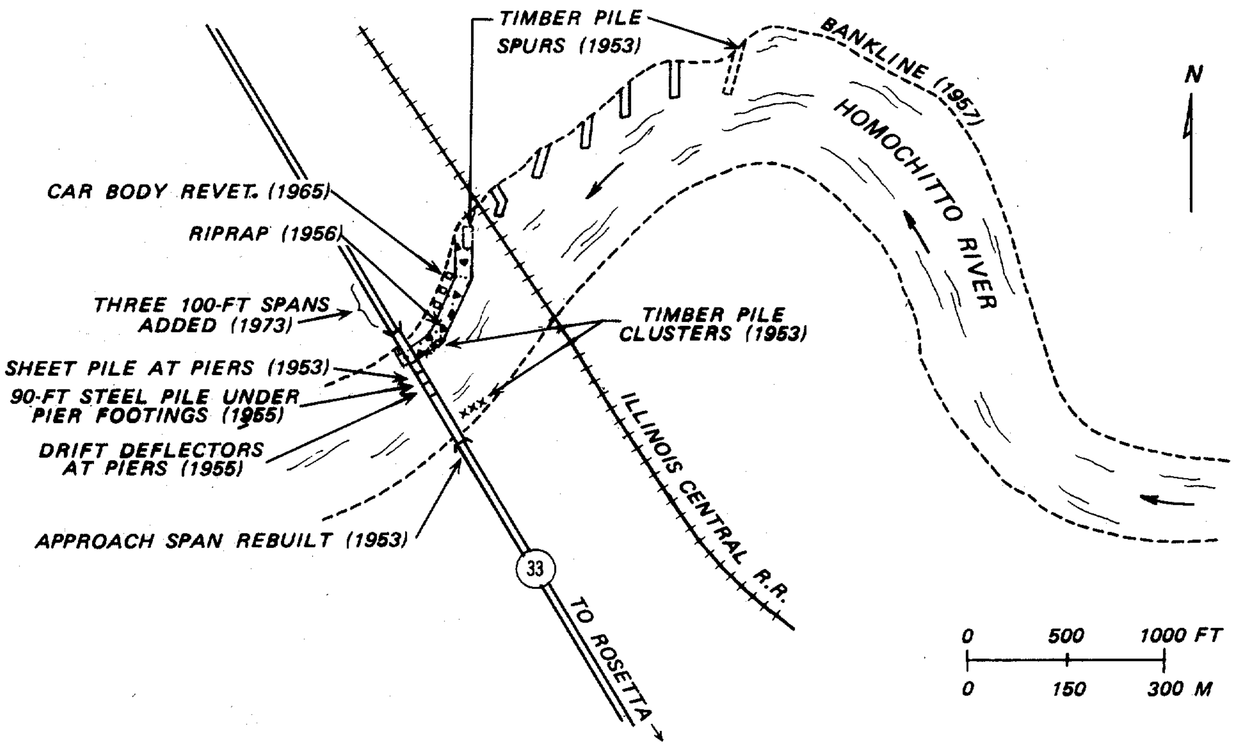


Figure 59. Plan sketch of Homochitto River, SR-33 crossing at Rosetta, showing placement of countermeasures.

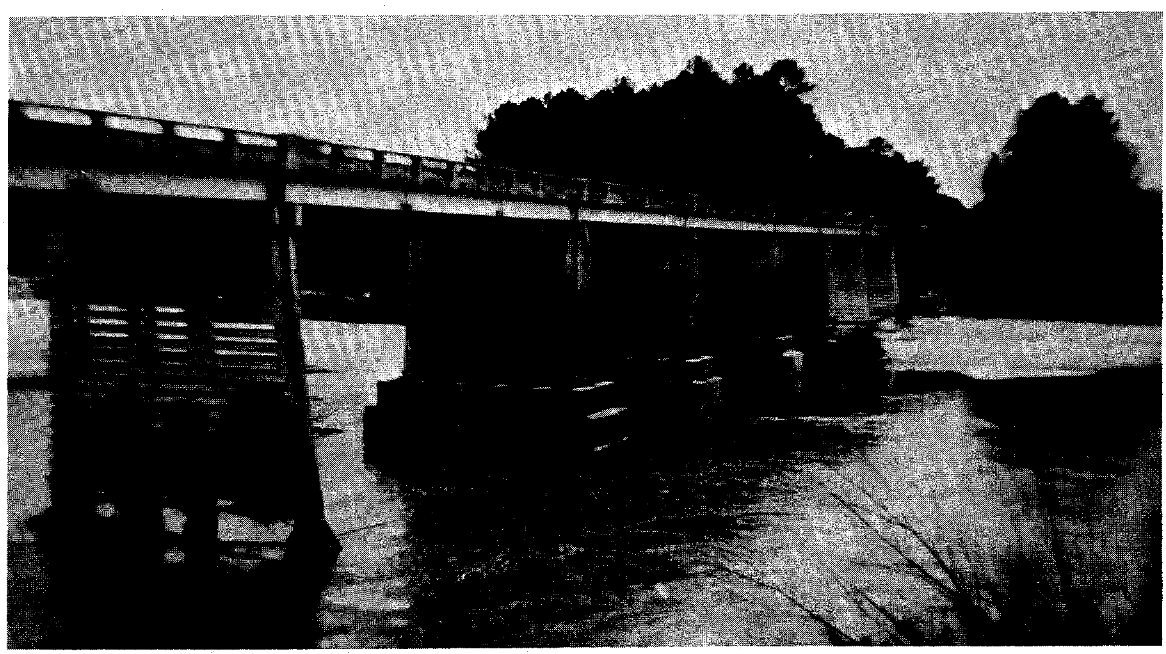


Figure 60. View toward right abutment of SR-33 bridge, about 1970.

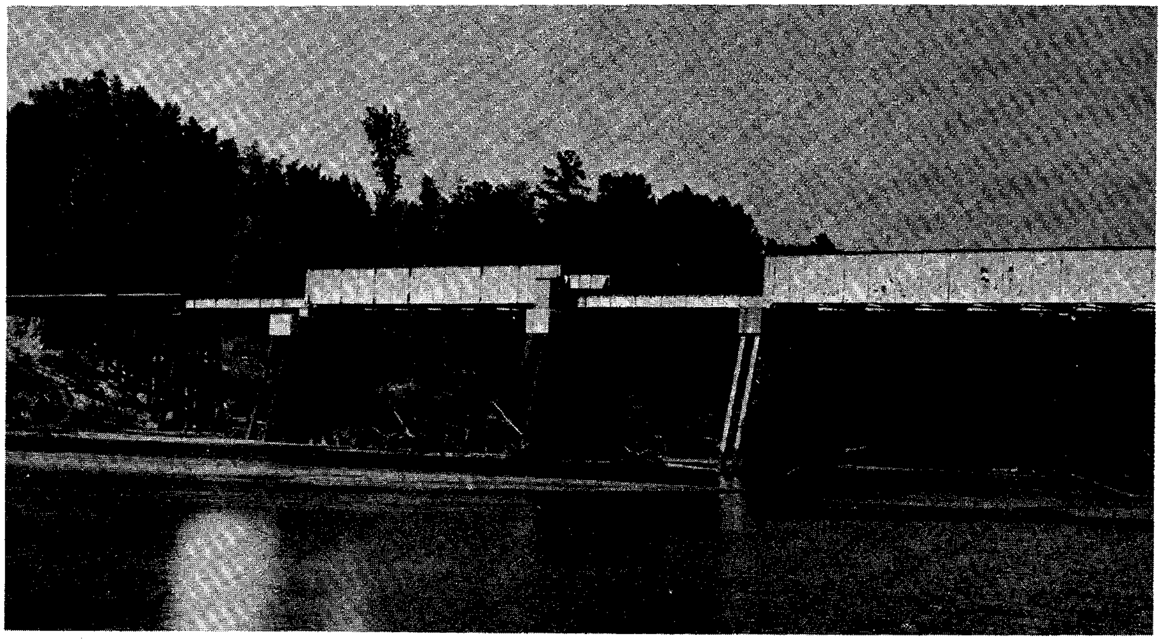


Figure 61. View toward right abutment of Illinois Central RR. bridge in 1976, showing added spans. Remnant of timber-pile spur is visible beneath main span, at right.

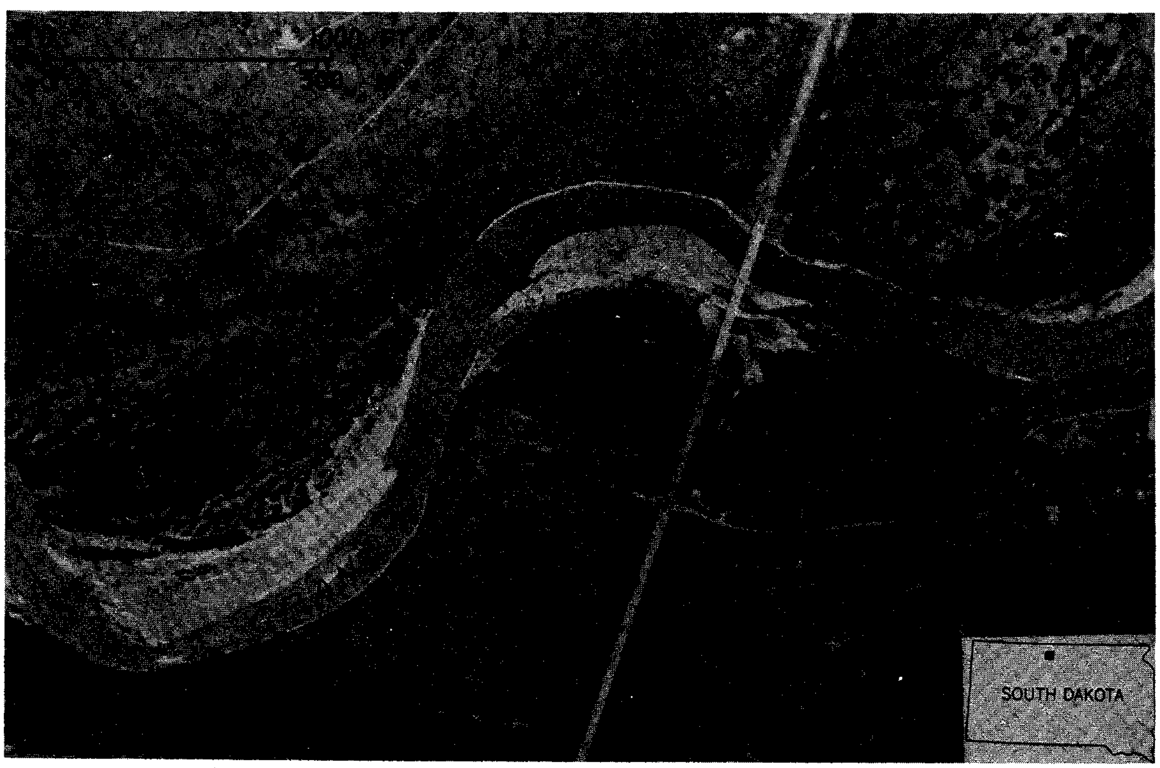


Figure 62. Aerial photograph of Grand River at SR-63 crossing, on June 12, 1971. (From South Dakota Dept. of Highways.)

SITE 175. GRAND RIVER AT SR-63 NEAR LITTLE EAGLE, S. DAK.

Description of site: Lat 45°39', long 100°49', location as shown in fig. 62. Plate girder bridge, 504 ft (154 m) in length, three concrete wall-type piers, spillthrough abutments. Crossing is downstream from bend in stream.

Valley slope, 0.001; channel width, 200 ft (61 m). Stream is perennial, alluvial, sand bed, in valley of moderate relief, narrow flood plain. Channel is meandering, locally braided, wider at bends, point bars, erodible silt-clay banks, tree cover at less than 50 percent of bankline. As indicated by comparison of 1950 and 1971 aerial photographs, the left bank upstream from the bridge migrated laterally about 300 ft (91 m) during the 21-year period.

Hydraulic problem and countermeasures:

1965 Approximate date of bridge construction

1966-71 Fill-slope of left abutment eroded by lateral migration of channel, and undermining of abutment foundation threatened by further migration. As countermeasures (fig. 63), a jack field was built along the left bank upstream from the bridge, two steel sheet-pile spurs were built at the left abutment, and revetted spur dikes were built at both abutments. The main line of jacks consists of two rows of 14-ft (4-m) Kellner-type steel jacks in the channel (total of 205 jacks) parallel with the bankline, and the tie-back lines consist of 10-ft (3-m) jacks. Wire cable of 0.75 in (1.9 cm) diameter was used to tie the jacks along the main line and the tie-back lines. Steel H-pile was driven to anchor the cable, except at the ends, where steel sheet pile was used.

The steel sheet-pile spurs (figs. 63, 64, and 65) are curved at the end and filled with earth.

1972 Flow at overbank stage during spring break-up of ice (fig. 64). Sheet-pile spur was struck by ice blocks up to 2.3 ft (0.7 m) thick, but was not damaged. The jack field was flattened and damaged by ice during this (and subsequent) break-ups.

1974 Inspection showed that bridge abutment is being protected by the sheet-pile spur and that the jack field, although damaged and partly buried, is serving to induce deposition and protect the bankline (fig. 65). Grass has become established on bank behind jack field, and the bank appears to be stabilized.

Discussion: With respect to transport of drift and sediment load by the stream, conditions are favorable for good performance of the jack field; ice, however, is a negative factor. The partial success of the jack field may be only temporary, as experience in Oklahoma (Keeley, 1971, p. 55) has shown that jacks placed in a stream channel (rather than on the banks) usually become ineffective because of underscour and subsequent burial. The steel sheet-pile spur, which in this study was found only in South Dakota, appears to have good potential for regions where other types of spurs are subject to damage by ice.

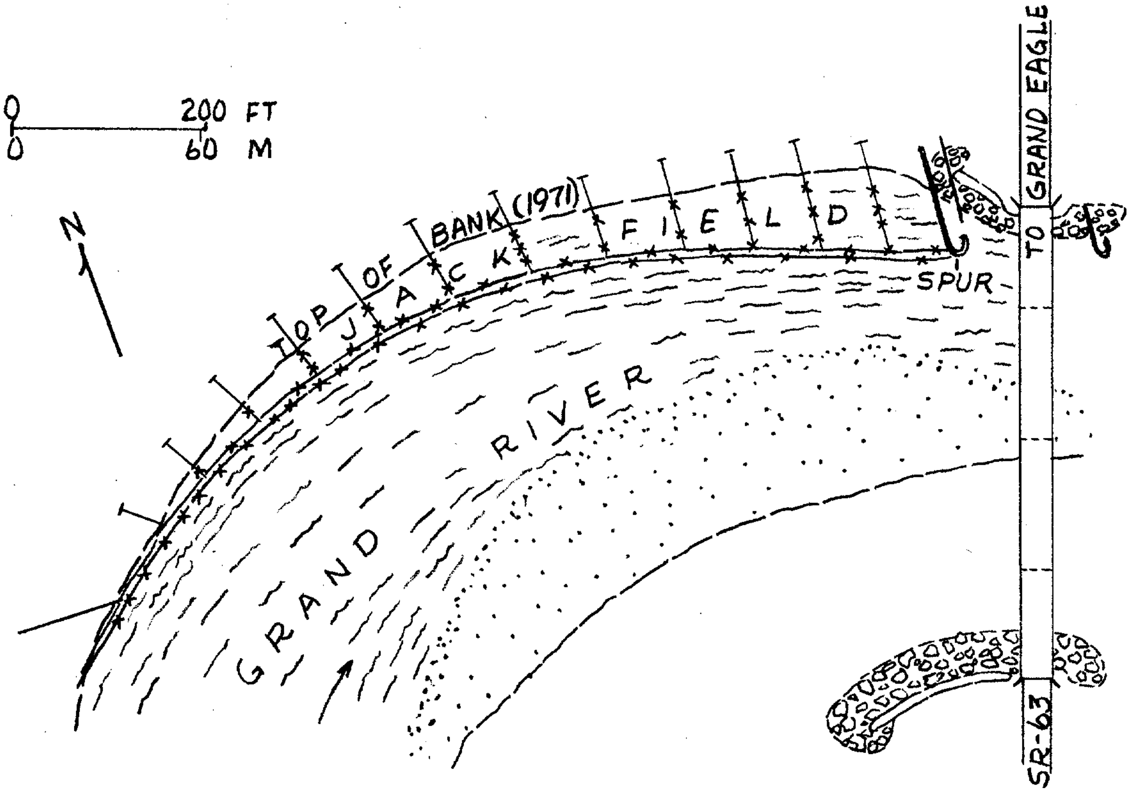


Figure 63. Plan sketch showing countermeasures installed at SR-63 crossing in 1971.

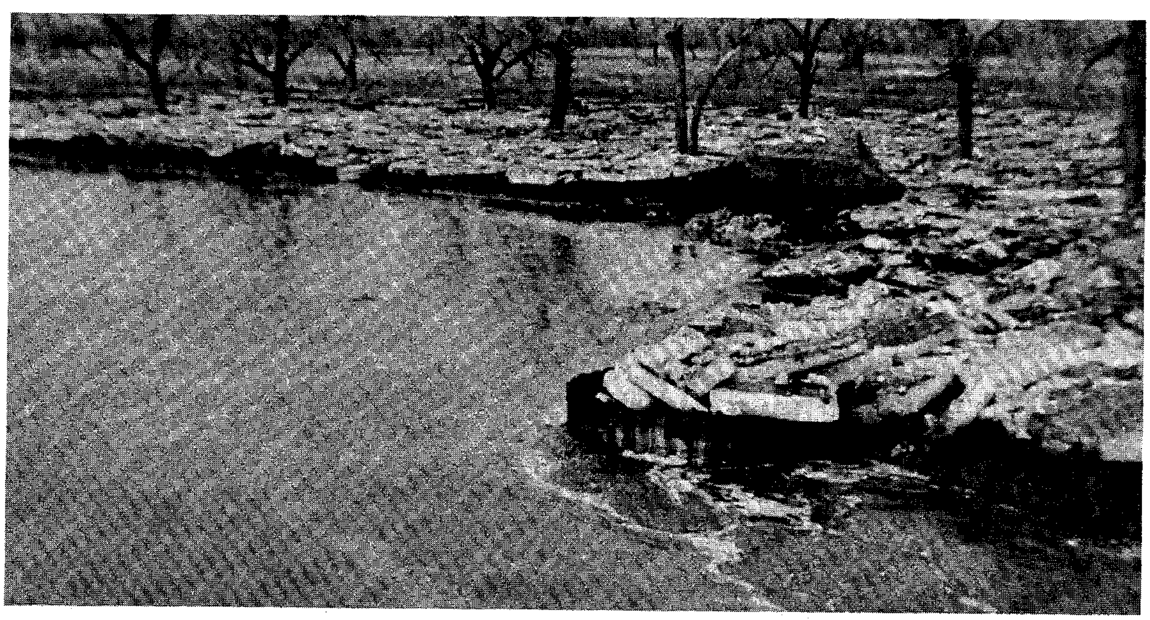


Figure 64. View of left bank upstream from bridge on March 14, 1972, showing flow near bankfull stage and ice blocks on sheet pile spur. (From South Dakota Dept. of Highways.)

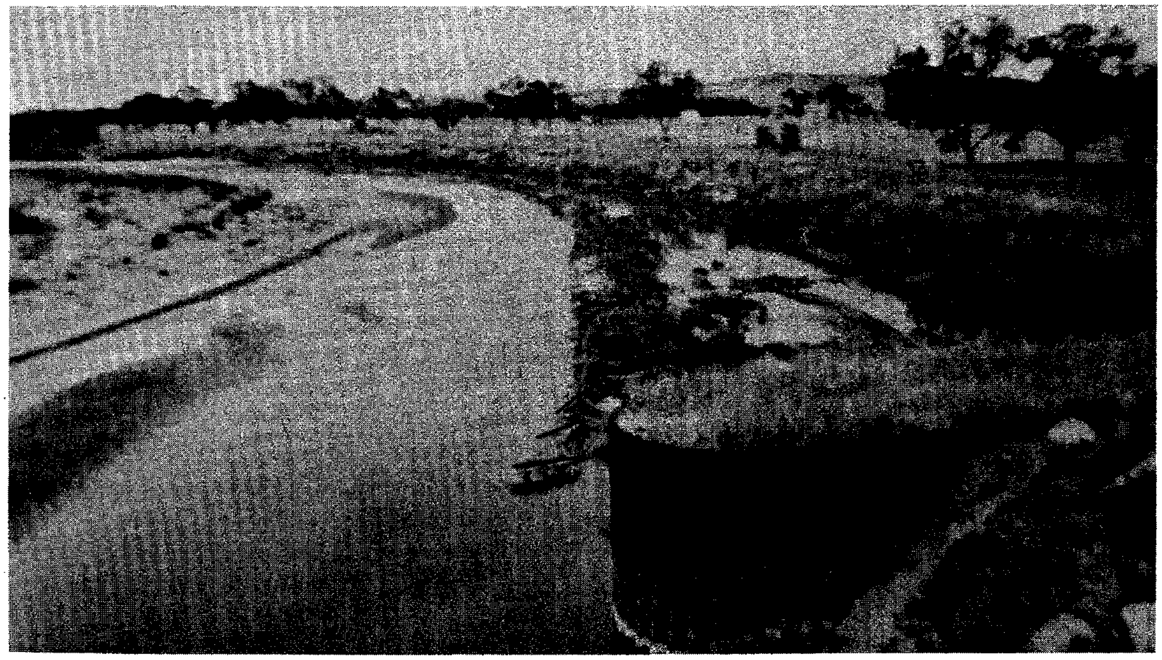


Figure 65. View of left bank upstream from bridge on July 30, 1974, showing sheet pile spur and jack field in background. (From South Dakota Dept. of Highways.)

SITE 176. WEST FORK CROOKED CREEK AT SR-206 NEAR GAITHER, ARK.

Description of site: Lat 36°10', long 93°11', location shown in fig. 66. Bridge is 140 ft (43 m) in length, five reinforced concrete-slab spans, four pile bents each with three square concrete pile, footings on bedrock about 7 ft (2 m) below channel bed, spillthrough abutments. Crossing is strongly skewed (fig. 66) but pile bents are alined with flow.

Drainage area, 5 mi^2 (13 km^2); valley slope, 0.0069; channel width, about 35 ft (11 m), increasing to 48 ft (15 m) at bridge. Stream is perennial, alluvial, gravel bed, in valley of moderate relief, wide flood plain. Channel is artificially straightened, sand-gravel banks.

Hydraulic problem and countermeasure:

1968 Bridge built. Dumped rock riprap placed at abutment fill-slopes, approximately equivalent to Class I, granular filter 0.5 ft (0.15 m) thick, riprap extended at toe to about 1.5 ft (0.45 m) below ground line.

1969-76 During successive moderate floods, the left abutment and left bank upstream from bridge were eroded laterally and some filling occurred at the right abutment. The original riprap was washed away. Large riprap (diameter of largest rocks, about 3 ft or 1 m) was placed at, and upstream from, the left abutment; but most of this that was beneath the bridge has been moved downstream by flood (fig. 67).

Discussion: Position of stream relative to road presented a difficult problem in crossing location and bridge design. The existing alinement results in direct attack of stream on left streambank and abutment, and consequent lateral erosion that has not been controlled with large rock riprap. The large riprap along the left bank upstream from the bridge may have induced turbulence at the left end of the bridge.

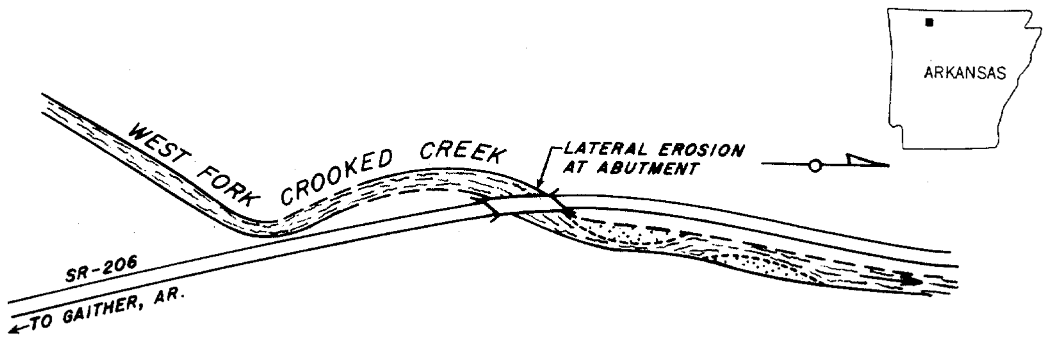


Figure 66. Plan sketch of west fork Crooked Creek at SR-206 crossing.

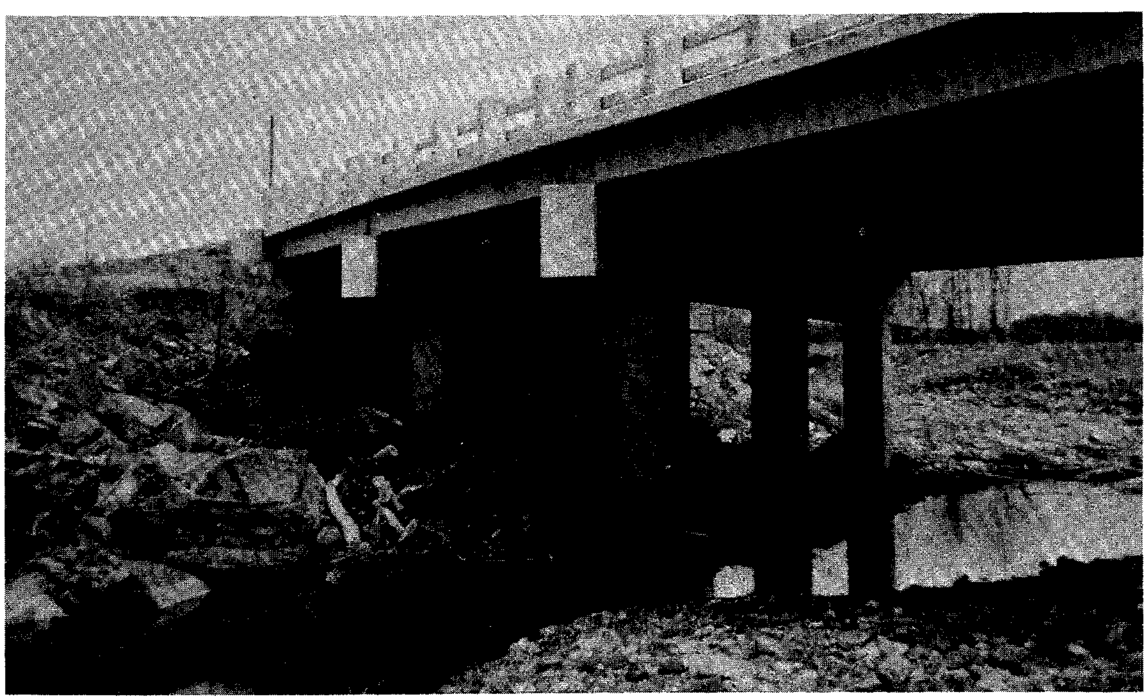


Figure 67. View toward upstream side of left abutment, SR-206 bridge, March 25, 1976.

SITE 177. CADDO RIVER AT I-30 AT CADDO VALLEY, ARK.

Description of site: Lat 34°11', long 93°04', location as shown in fig. 68. Bridge is 1,112 ft (339 m) in length, three webbed concrete piers in channel, 17 pile bents, each having five 16-in (41-cm) octagonal concrete piles, on right-bank flood plain. Crossing is eccentric, most of flood plain is on right bank, where relief openings consist of six 3-ft (1 m) pipe culverts and one triple 10x5-ft (3x1.5-m) barrel box culvert. At the time of bridge construction, a minor tributary entering downstream from the bridge site (figs. 68 and 69) was relocated, such that it now enters just upstream from the I-30 bridge.

Drainage area, 485 mi^2 (1,256 km^2); bankfull discharge, 10,700 ft^3/s (303 m^3/s); channel width, 130 ft (37 m). Stream is perennial but flashy, semi-alluvial, gravel and cobble bed, in valley of moderate relief, wide flood plain. Channel is meandering, locally braided, random width variation, cut banks local, gravel banks, tree cover at 50 to 90 percent of bankline.

Hydraulic problem and countermeasure:

1965 I-30 bridge built, spillthrough abutments riprapped.

1968 During a moderate flood of undetermined frequency, the right bank upstream from the bridge was eroded and local scour occurred at the pier adjacent to the right bank, undermining the footing. Local scour also occurred at three pile bents on the flood plain adjacent to the right bank. Scour is attributed to deflection of flow toward the right bank. Deflection of flow is attributed in part to a gravel bar that formed at the entrance of the relocated tributary on the left bank upstream from the bridge, and in part to a channel on the right-bank flood plain upstream from the bridge, made during commercial removal of gravel (fig. 69).

1970 The gravel bar at the mouth of the relocated tributary was removed and a check dam, consisting of a concrete wall across the tributary and grouted riprap on the banks, was installed. A large area beneath the bridge, around the previously scoured pier and pile bents, was covered to a depth of about 2 ft (0.6 m) with dumped rock riprap (fig. 70A).

1971-76 The gravel bar formed again at the mouth of the relocated tributary (fig. 70B), because of scour below the check dam and channel degradation along the tributary upstream from the dam. The riprap has protected the piers and pile bents from further scour.

Discussion: Scour at the pier and pile bents is attributed to concentration of flow, but influence of the relocated tributary and the gravel mining, in concentrating flow at the right bank, cannot be assessed. The expense of crossing the tributary was avoided by relocating it, but channel degradation in the tributary was probably not anticipated.

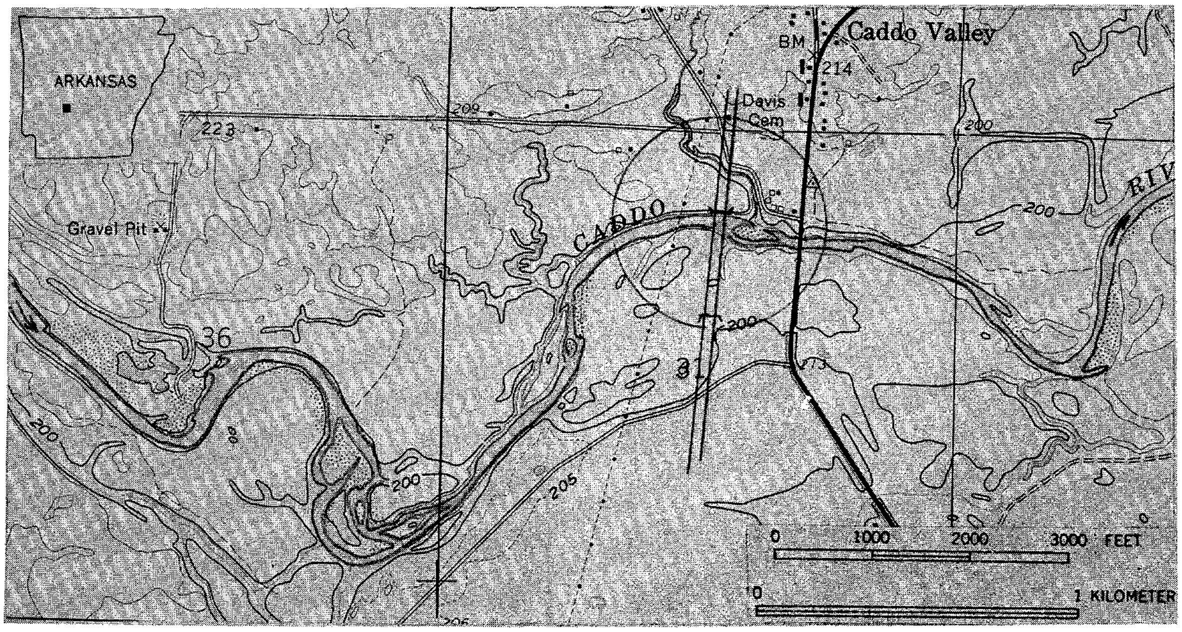


Figure 68. Map showing Caddo River at I-30 crossing (circled). (Base from U.S. Geol. Survey Caddo Valley, Ark., 7.5' quadrangle, contour interval 10 feet, 1959.)

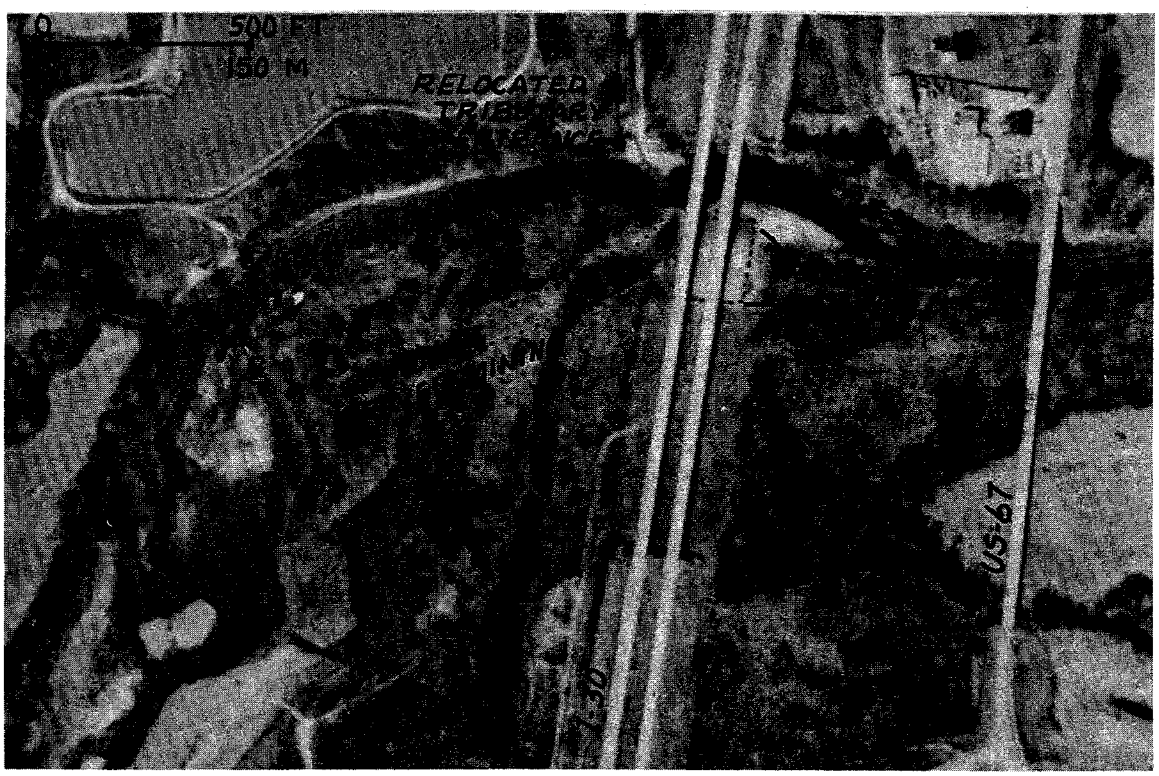


Figure 69. Aerial photograph showing relocated tributary entrance and gravel pits upstream from I-30 bridge, February 12, 1974. (From Arkansas Dept. of Highways.)

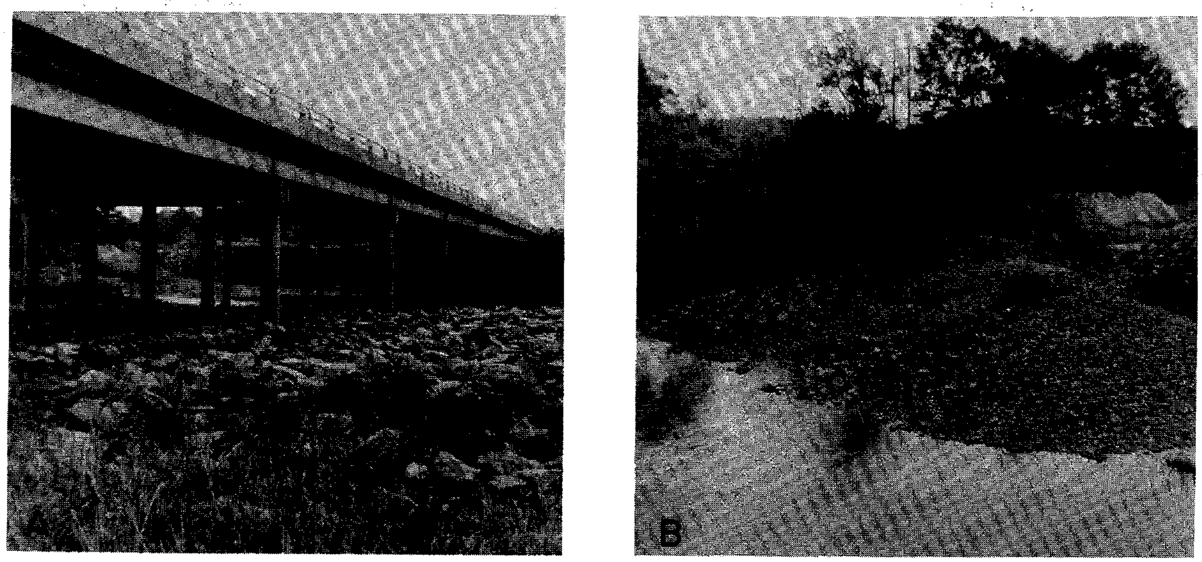


Figure 70. A, upstream view of I-30 bridge, showing riprapped area around pile bents on right bank, in 1976. B, view of gravel bar at mouth of relocated tributary, in 1976. Grouted streambank and check dam in right background.

SITE 178. SOLDIER RIVER AT SR-37 AT SOLDIER, IOWA

Description of site: Lat 41°59', long 95°46', location as shown in fig. 71. Bridge is 243 ft (74 m) in length, continuous I-beam span, open concrete piers on 40-ft (12-m) pile foundations, spillthrough abutments, concrete abutment on 50-ft (15-m) pile foundations. Crossing is downstream from bend in channel.

Valley slope, 0.001; channel width, about 200 ft (60 m). Stream is perennial, alluvial, silt-sand bed, in valley of moderate relief, wide flood plain. Channel is sinuous, channelized before 1930 by local drainage district, somewhat incised, cut banks general, erodible silt-clay banks, tree cover at less than 50 percent of bankline.

Hydraulic problem and countermeasures:

1952 Bridge built, no countermeasures at bankline or abutment.

1953-58 Lateral erosion at left bank upstream from bridge and erosion of fill slope of left abutment, attributed to effects of channelization project.

1959 Single-row timber-pile retard (fig. 72) constructed at left bank, extending about 600 ft (180 m) upstream from bridge and 50 ft (15 m) downstream. The treated timber pile were 30 ft (9 m) in length, driven to a depth of about 25 ft (7.5 m), connected by three galvanized-wire cables, and faced with 4 by 4 in (10 by 10 cm) No. 4 gage welded-wire mesh. The lower edge of the wire mesh was carried streamward for several feet and weighted against the streambed with concrete cubes.

1959-64 Abutment fill-slope entirely removed by lateral erosion, abutment footing and timber-pile foundation exposed (fig. 73).

1965 Four steel H-pile spurs constructed at left bank upstream from bridge (fig. 72), identical in design with spurs at Site 179. In addition, the abutment fill-slope was restored and protected with concrete slope-pavement, 5 in (12.5 cm) in thickness and protected at the toe with a cutoff wall of steel sheet piling driven to a depth of 20 ft (6 m).

1966-77 Sediment and debris accumulated at spurs and willows became established along bankline (figs. 74 and 75B). Inspection in January of 1977 indicated that bank had become stabilized.

Discussion: Although well constructed, the timber-pile retard proved to be ineffective in preventing lateral erosion, probably because it was too low relative to bank height (fig. 72). Although the largest flood that has occurred since the steel spurs were built had a R.I. of only 7 yr, performance of the spurs in promoting the growth of vegetation, and hence in stabilizing the bank, is rated as satisfactory.

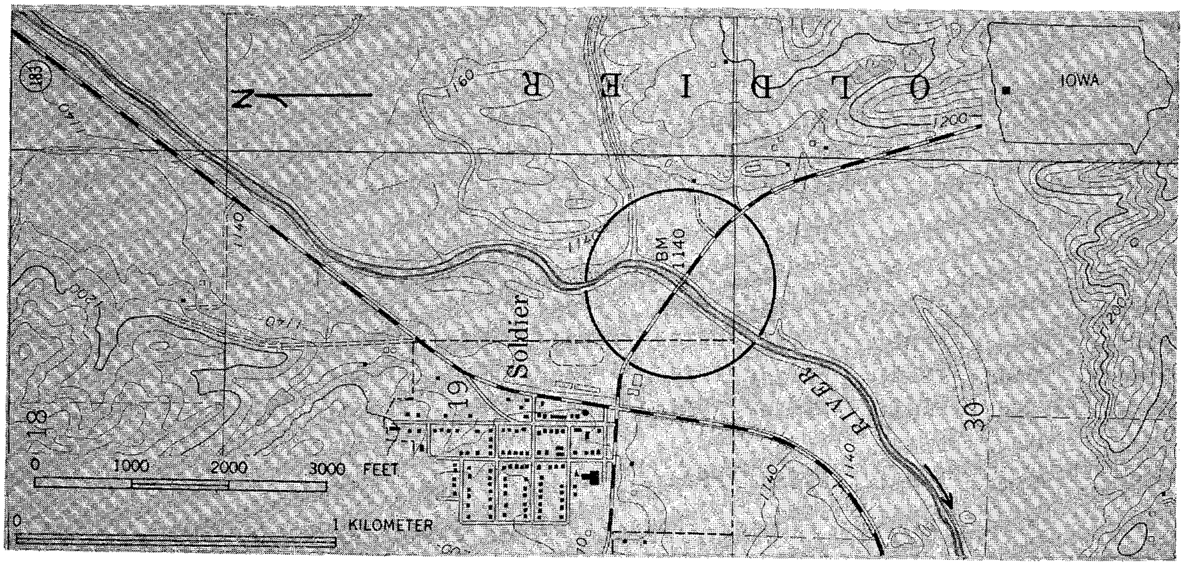


Figure 71. Map showing Soldier River at SR-37 crossing (circled). (Base from U.S. Geol. Survey Moorhead, Iowa, 7.5' quadrangle, contour interval 20 feet, 1971.)

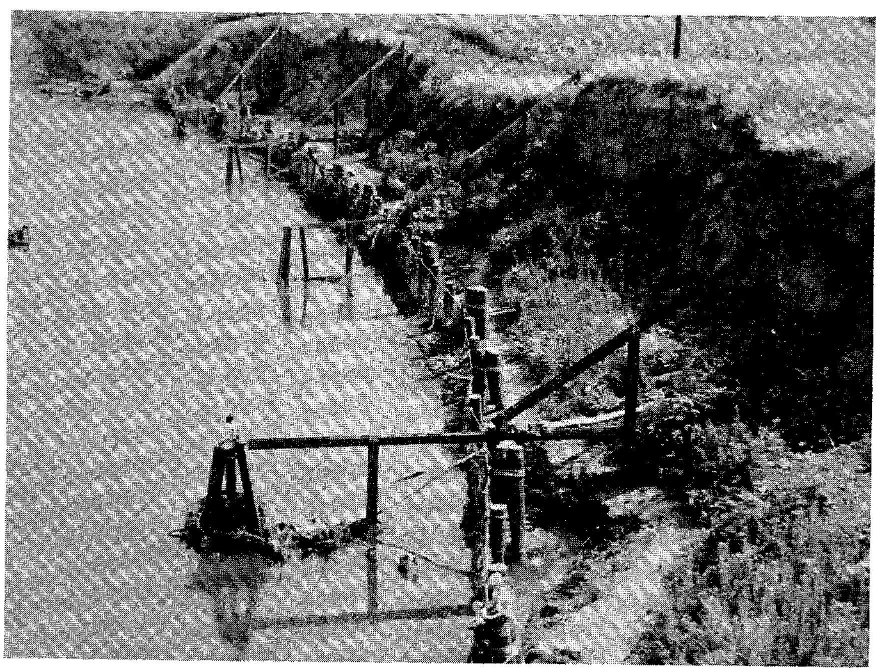


Figure 72. View of left bank looking upstream from bridge, showing steel H-pile spurs and timber-pile retard, July, 1966. (From Iowa Dept. of Transportation.)

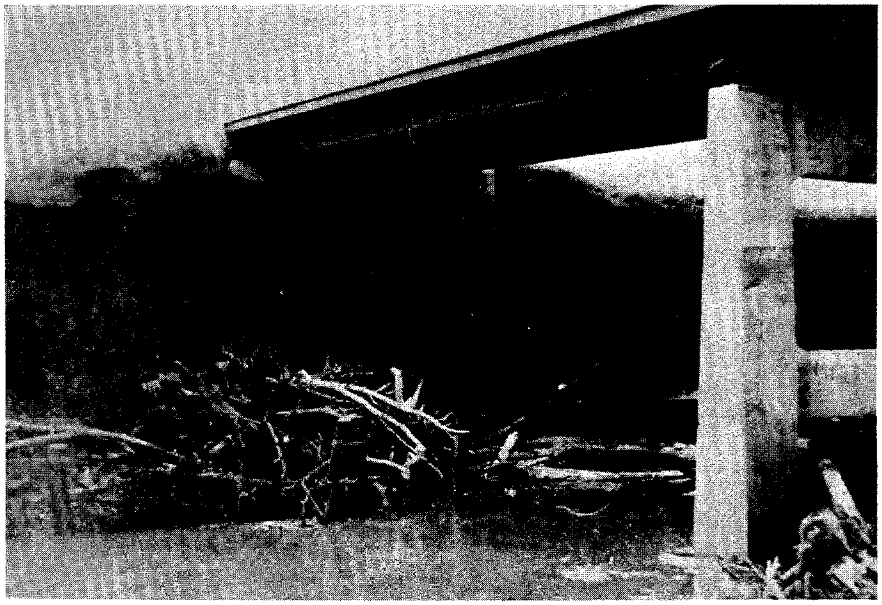


Figure 73. View of erosion at left abutment of SR-37 bridge, 1964. (From Iowa Dept. of Transportation.)

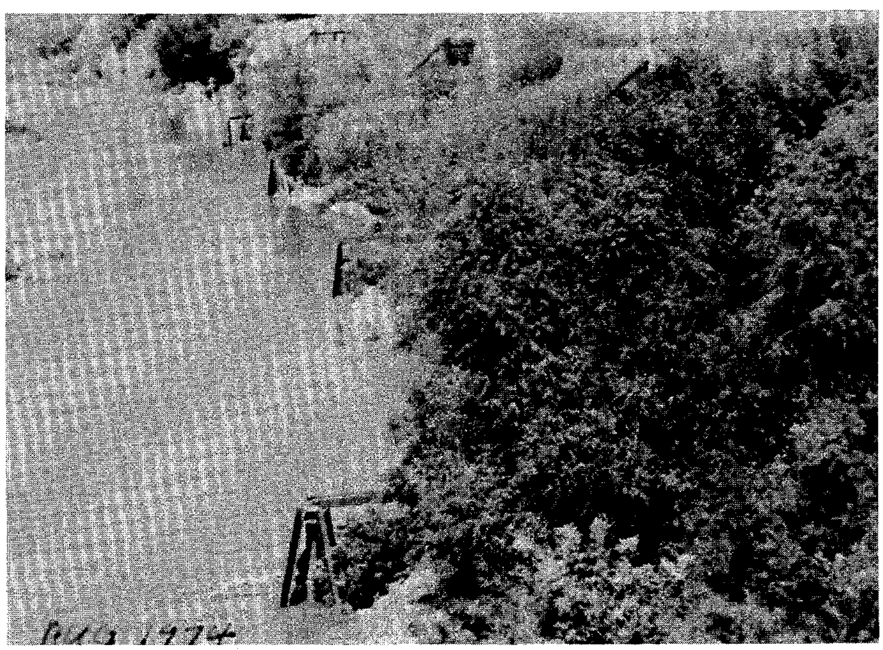


Figure 74. View of left bank upstream from bridge, showing establishment of vegetation, August 1974. (From Iowa Dept. of Transportation.)

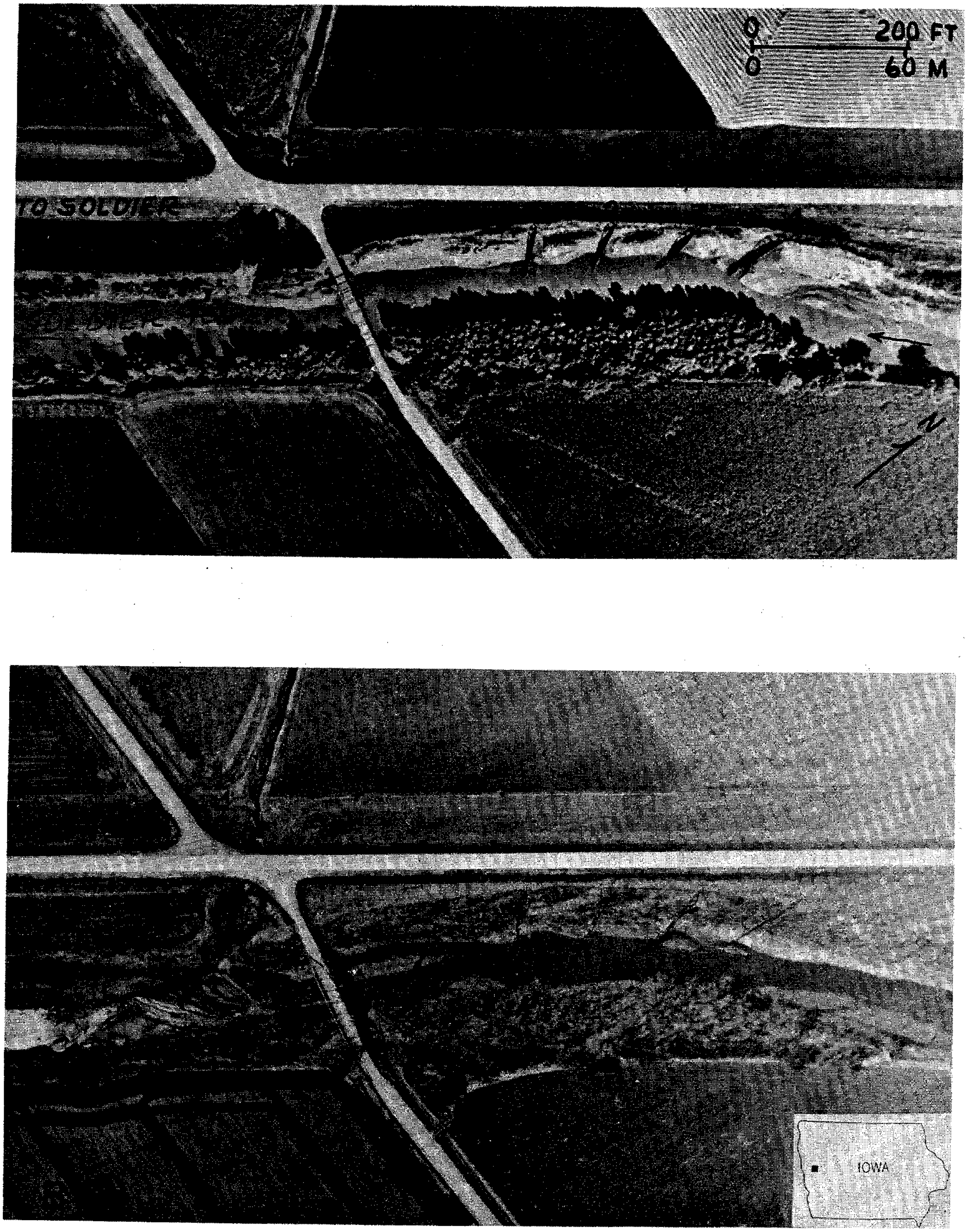


Figure 75. A, aerial photograph of Soldier River at SR-37 crossing, July 29, 1967. B, aerial photograph of same site, April 12, 1976. (From Iowa Dept. of Transportation.)

SITE 179. SOLDIER RIVER AT MONONA COUNTY BRIDGE NEAR SR-183, 1.5 MI (3.4 KM) NORTH OF SOLDIER, IOWA

Description of site: Lat 41°02', long 95°46', location as shown in fig. 76. Bridge is about 200 ft (60 m) in length, single camel-back truss span, spillthrough abutments with wingwalls.

Stream is perennial, alluvial, silt-sand bed, in valley of low relief, wide flood plain. Channel is artificially straightened, somewhat incised, cut banks local, easily erodible silt banks.

Hydraulic problem and countermeasure:

Pre-1920 Bridge built.

1964 Lateral erosion, attributed to channelization, threatened right bridge abutment and roadway of SR-183 (figs. 76A and 77A).

1965 Four steel H-pile spurs constructed upstream from bridge (fig. 76A). Upstream spur 140 ft (42 m) in length, oriented downstream, angle with bankline about 45°. Downstream spur 90 ft (27 m) in length, angle with bankline about 80°. Steel H-pile range in length from 30 to 40 ft (9 to 12 m) welded-wire mesh is attached to upstream side and buried in bank to minimum depth of 3 ft (1 m) (fig. 78).

1966-76 Sediment and debris accumulated at spurs and willows became established along bankline by 1971. By 1975, willows had reached sufficient size to stabilize bankline (fig. 77B).

Discussion: Stabilization by vegetation is attributed to spurs. Position of bankline, relative to SR-183 roadway, did not change significantly between 1965 and 1977. Although no overbank flows occurred during this period, the moderately high flows that did occur (R.I. of 5 and 7 yr, respectively) are considered adequate to test the effectiveness of the countermeasure.

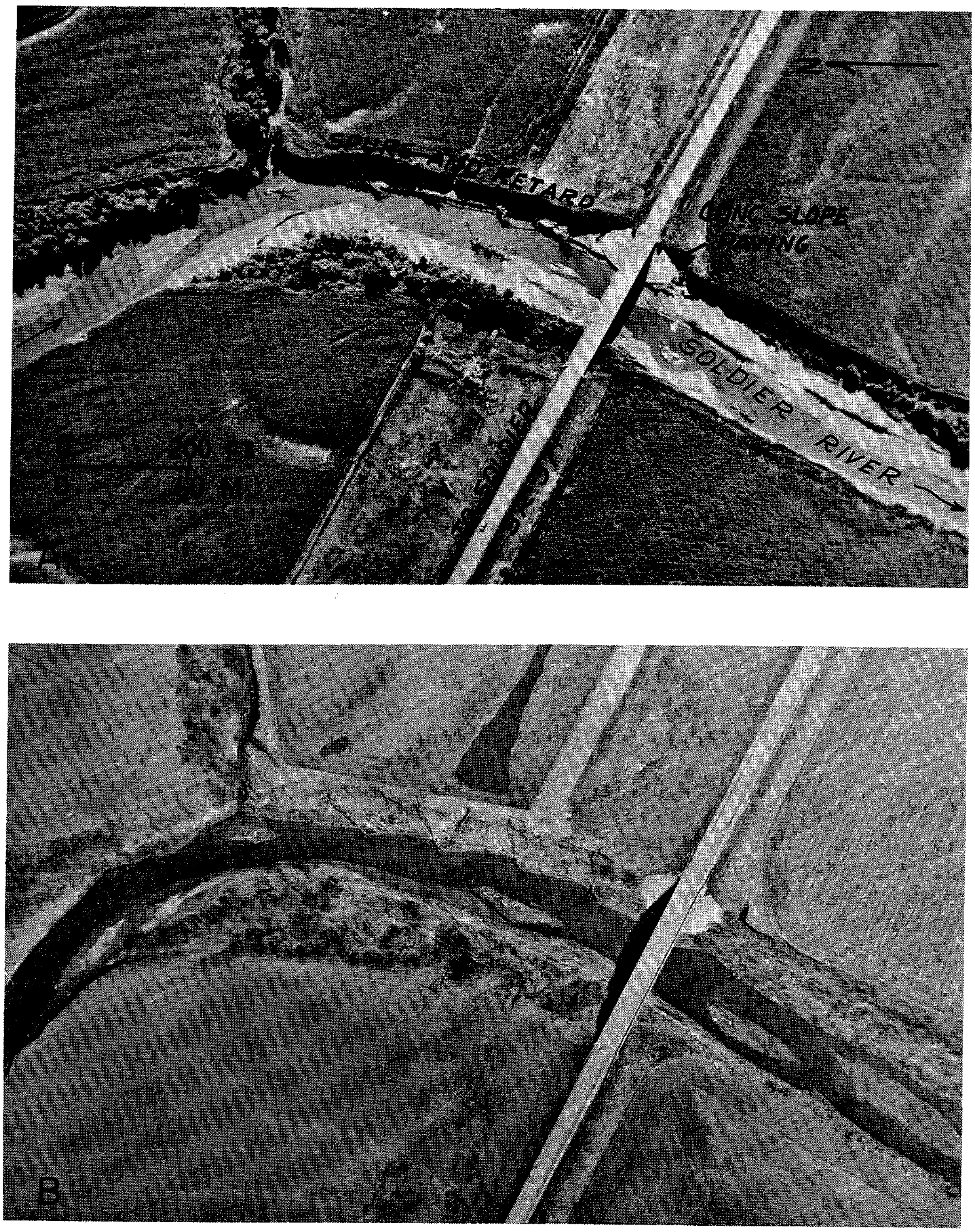


Figure 76. A, aerial photograph showing spurs upstream from Monona County bridge at SR-183, on July 29, 1967. B, aerial photograph of same site on April 12, 1976. (From Iowa Dept. of Transportation.)

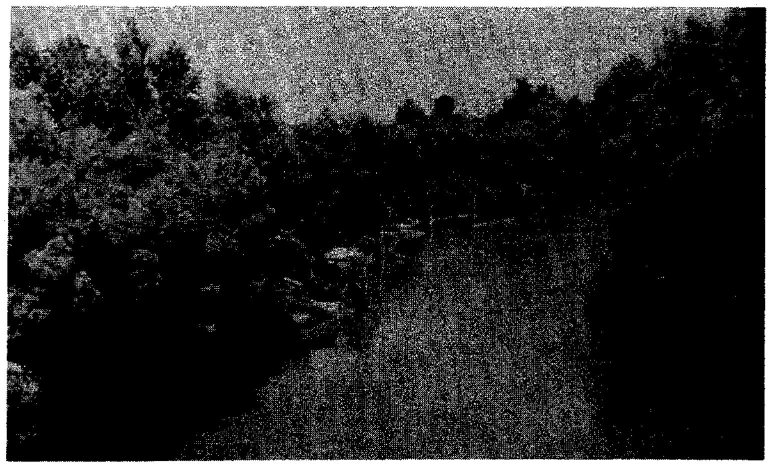


Figure 77. A, downstream view toward bridge, showing steel H-pile spurs shortly after construction, April, 1966. B, upstream view from bridge, showing spurs and establishment of willows along stabilized bankline, August, 1975. (From Iowa Dept. of Transportation.)

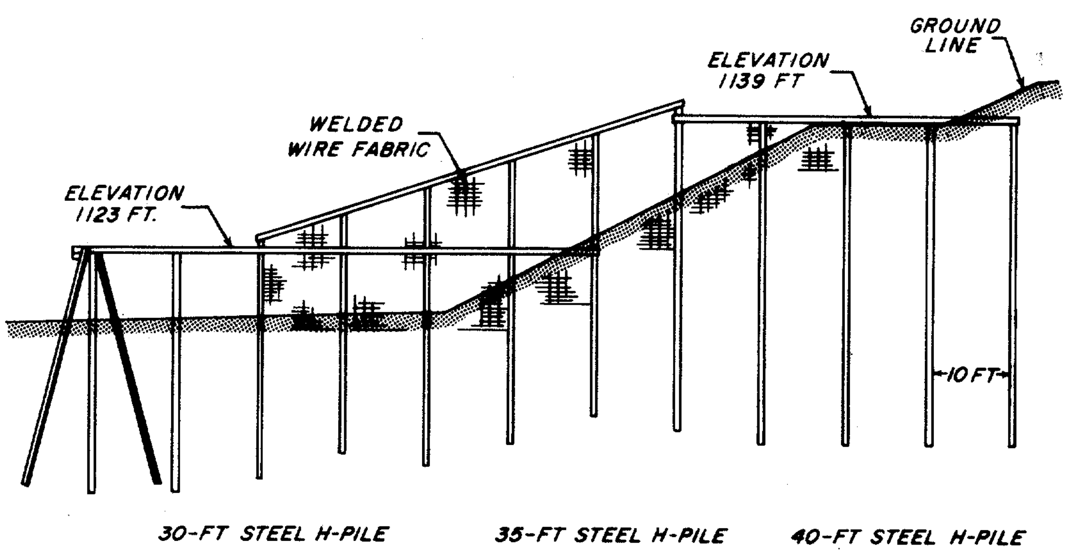


Figure 78. Elevation sketch of spur 3, which is 120 ft in length.

SITE 180. EAST NISHNABOTNA RIVER AT US-34 NEAR RED OAK, IOWA

Description of site: Lat 40°59', long 95°14', location as shown in fig. 79. Bridge is 352 ft (107 m) in length, continuous welded-girder type, concrete wall ("hammerhead") piers in channel, spillthrough abutments.

Drainage area, 890 mi^2 (2,305 km^2); bankfull discharge, about 10,000 ft^3/s (283 m^3s); channel width, about 250 ft (75 m). Stream is perennial, alluvial, sand bed, in valley of moderate relief, wide flood plain. Channel is sinuous, artificially straightened, wider at bends, point bars, erodible silt-sand banks, incised, cut banks general, tree cover at less than 50 percent of bankline. Since it was straightened, the channel has begun to re-establish meanders, and the lateral migration rate is rapid (compare figs 80A and 80B). Bedrock is at shallow depth (about 5 ft, or 2 m) below streambed.

Hydraulic problem and countermeasures:

1964 Bridge built. Two earth embankment spur dikes, of modified elliptical shape, not revetted, placed on upstream side of abutments (fig. 81).

1965 Lateral bank erosion upstream from left abutment was seen as a hazard to the abutment. A steel-pile retard, connected to the bank by two steel-pile spurs, was built upstream from the left abutment (fig. 81). The retard is 520 ft (158 m) in length, and the spurs, about 100 ft (30 m). The retard consists of heavy steel pile (8BP36), spaced at intervals of 10 ft (3 m), and connected at the top by a wide flange beam, and strung together with three steel cables. The retard is faced with welded-wire fabric.

1968 The retard proved to be only partly effective in controlling bank erosion. As a further countermeasure, the abutment fill-slope was extended as a berm, flattened to a 6:1 slope, and revetted with dumped rock riprap (fig. 81). Thickness of the riprap was about 2 ft (0.6 m) at the top of the fill slope, and about 10 ft (3 m) thick at the streamward edge, where the slope was 1.25:1.

1969-75 In 1972, a 60-yr R.I. flood occurred; other high flows during the period were in the range of 3-to-5-yr R.I. The combined effect of the retard and the revetted fill slope has been to halt bank recession and to induce some accumulation of sediment along the bank; however, vegetation has not become established behind the retard (compare figs. 80A and 80B). The spur dikes, although not revetted, were not damaged by the 1972 flood. In Iowa, grass cover is regarded as satisfactory protection for a spur dike, when the risk of damage to the dike is weighed against the cost of revetment.

1976 Progressive migration of the right bank (by about 80 feet during the period 1973-76) was seen as a potential threat to the right approach embankment and abutment (compare figs. 80A and 80B).

As a countermeasure, dumped rock riprap was placed parallel to the embankment (fig. 81). The riprap, which has a D_{50} of 200 lb (91 kg), was placed to a thickness of about 2.5 ft (0.75 m) on a granular filter blanket 1 ft (0.3 m) in thickness. In addition, this riprap was placed behind the retard (fig. 81). Because of the recency of installation, no assessment of these countermeasures can be made.

Discussion: In view of the sharp (nearly 90^0) change in channel direction immediately upstream from the bridge, and the rapid rate of lateral channel migration, maintenance of existing channel alinement will be a severe test.

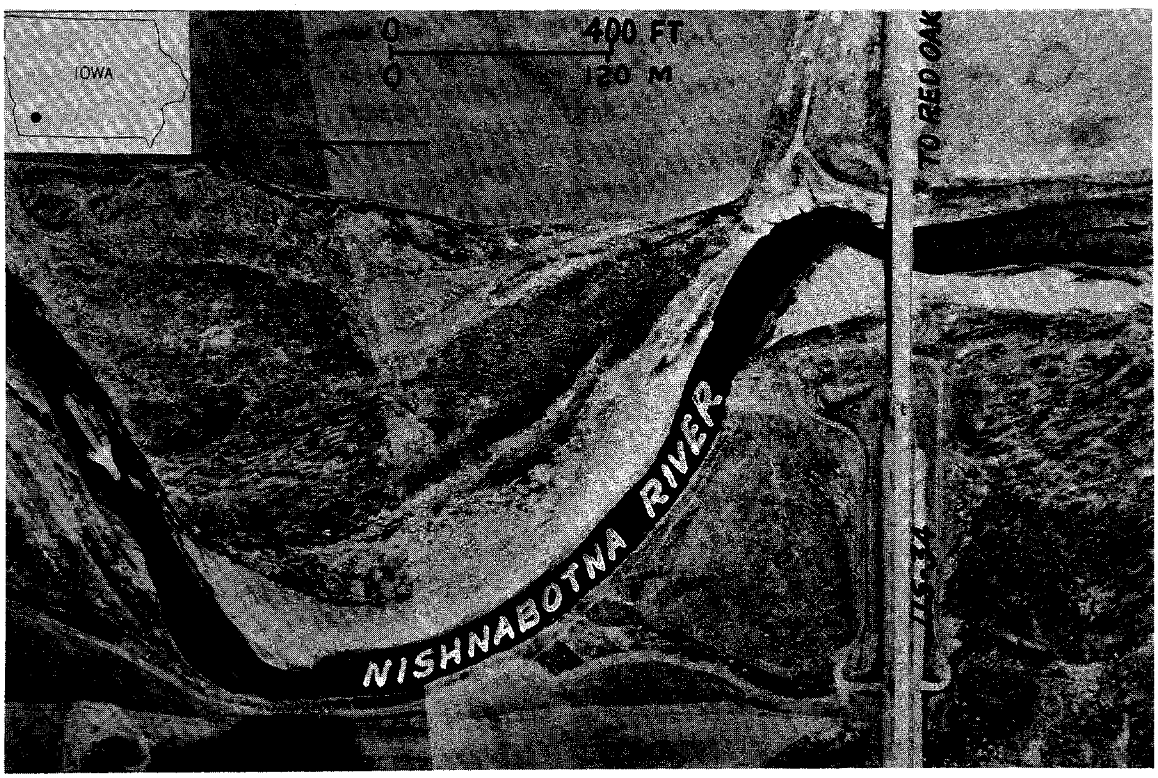


Figure 79. Aerial photograph showing alinement of Nishnabotna River at US-34 crossing, April 13, 1972. (From Iowa Dept. of Transportation.)

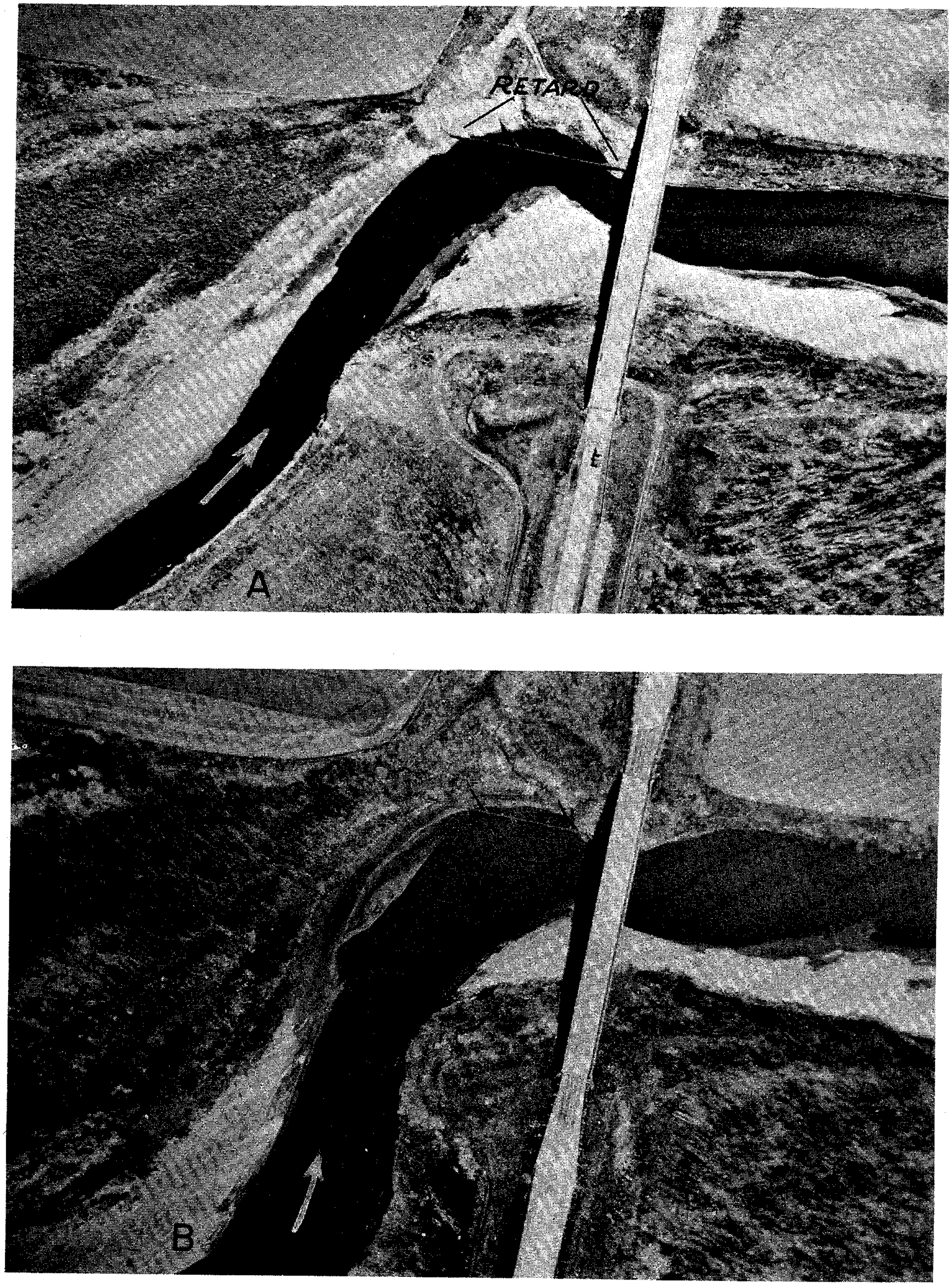


Figure 80. A, aerial photograph showing US-34 crossing of Nishnabotna River, April 13, 1972. B, aerial photograph of same site, April 6, 1976. (From Iowa Dept. of Transportation.)

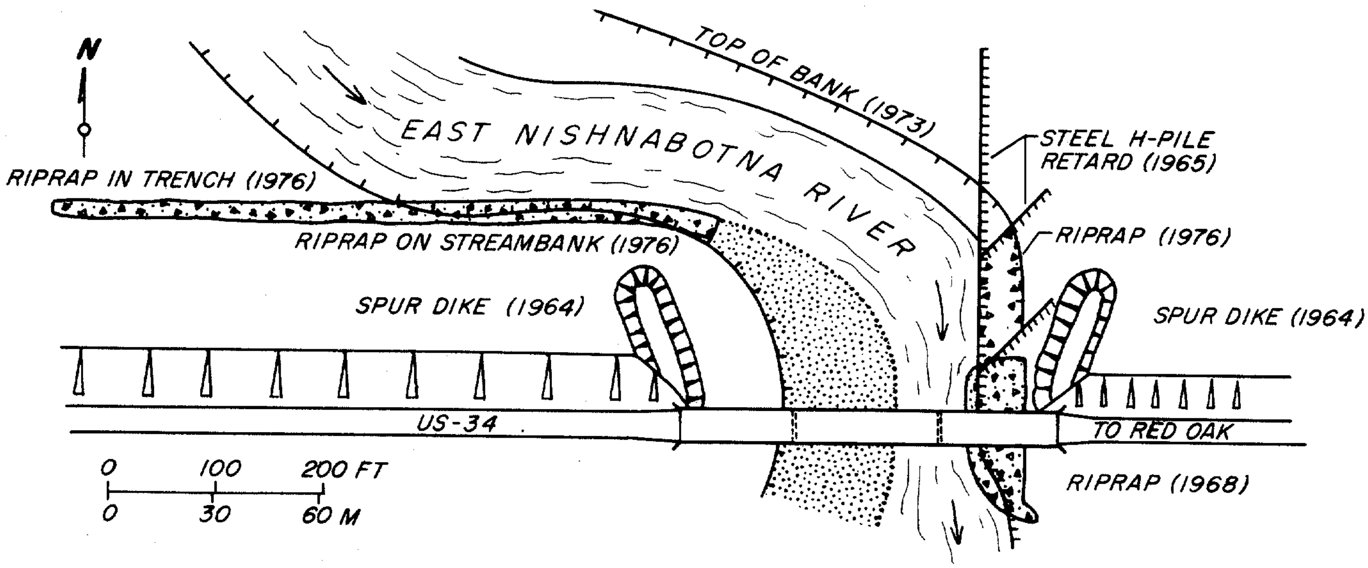


Figure 81. Plan sketch showing countermeasures at US-34 crossing.

SITE 181. EAST NISHNABOTNA RIVER AT SR-48 NEAR RED OAK, IOWA

Description of site: Lat 41°03', long 95°14', location as shown in inset map of fig. 79. Bridge is 350 ft (107 m) in length, welded deck-girder spans, concrete wall-type (hammerhead) piers with spread footings on bedrock at depth of about 5 ft (2 m) below streambed; spillthrough abutments.

Drainage area, about 890 mi^2 (2,300 km^2). Site is about 1.5 mi (2.4 km) upstream from Site 180 and same geomorphic description applies to both sites.

Hydraulic problem and countermeasures:

1973 Bridge built. Because of poor channel alinement at site and various restrictions against channel straightening, several countermeasures were incorporated into the crossing plans to prevent lateral migration of the channel (fig. 82). Nine short spurs or "hard points", built of dumped rock riprap, about 5 ft (1.5 m) in width and barely projecting into the channel, were constructed at the outside of the upstream bend. Spacing between spurs was in the range of 100-150 ft (30-45 m). The riprap was approximately equivalent to Class II: D_{50} specified at 200 lb (91 kg) and D_{90} at 5 lb (2.3 kg). In addition, berms revetted with riprap of the same size were constructed at the abutments and left approach embankment.

1973-75 Flood, R.I. 4 yr, peak discharge 14,500 ft^3/s (411 m^3/s), occurred on May 8, 1973. Other peak discharges during period were in the range of 5,000 to 8,700 ft^3/s (141 to 246 m^3/s) as compared with a bankfull stage of about 10,000 ft^3/s (283 m^3/s). The riprap spurs upstream from the bridge proved to be ineffective in controlling bank erosion. The riprap remained in place, but erosion occurred between and behind the spurs.

1976 The spurs were left in place, but a continuous cover of riprap was placed along the section of bank where the spurs had been built. Class II riprap was placed to a thickness of 2.5 ft (0.75 m), over a granular filter blanket 1 ft (0.3 m) thick, on a slope of 1.5:1. Toe of the riprap revetment was extended for a distance of 8 ft (2.4 m) onto the streambed. This bank protection method has not yet been adequately tested.

Discussion: Although the short riprap spurs were not effective here, the situation is unusually difficult: a high bank of erodible silt, located at the outside of a bend on a channel that has a rapid rate of lateral migration. If used under less extreme circumstances, short spurs may prove to be effective; and they require less riprap than does continuous riprap blanket. The tendency toward lateral erosion at this site may prove to be so troublesome that channel straightening upstream from the bridge may eventually be required.

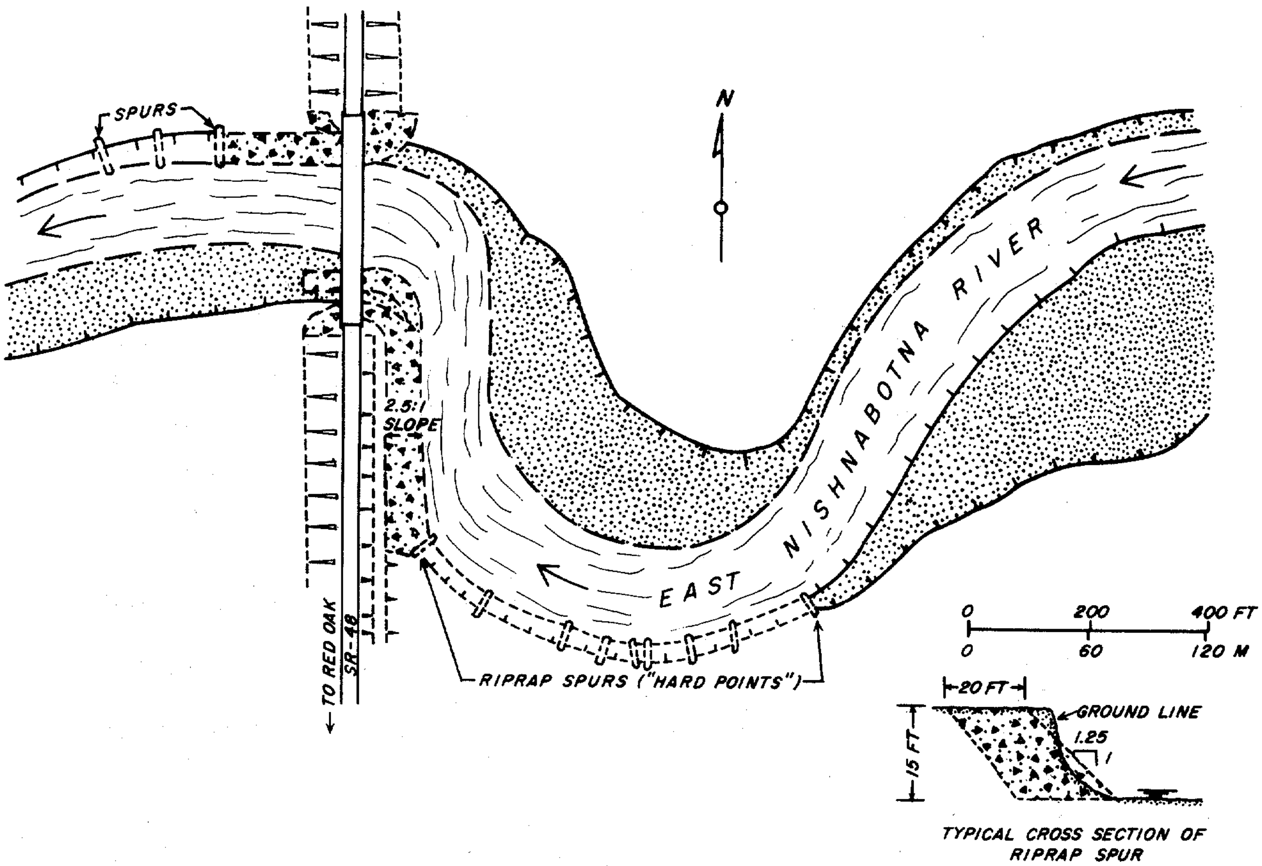


Figure 82. Plan sketch of countermeasures installed at time of bridge construction, East Nishnabotna River at SR-48.

SITE 182. YANKEE FORK SALMON RIVER ALONG FOREST SERVICE ROAD NEAR SUNBEAM, IDAHO

Description of site: Lat 44°21', long 114°43', location as shown in fig. 83. Four bridges, designated according to mileage from Sunbeam, Idaho, as 5.4, 5.6, 6.1, and 6.3, were built at about the same time, are of similar construction, and were damaged during the same flood. The valley floor is narrow and, in order to avoid channel changes, stream was crossed at bends.

Drainage area, about 150 mi^2 (388 km^2); valley slope, 0.008, channel width about 80 ft (24 m). Stream is perennial, semi-alluvial, gravel bed, in narrow valley of high relief, little or no flood plain. Channel has been much disturbed by past dredging for gold, has become sinuous, locally braided, bordered along one or both banks by loose gravel dredge tailings.

Hydraulic problem and countermeasure:

1973 Bridges built, without countermeasures.

1974 Spring flood, R.I. estimated at 100 to 200 yr, peak discharge about 4,900 ft^3/s (139 m^3/s). At bridges 5.4, 5.6, 6.1, and 6.3 (fig. 83), the flood flow at a bend eroded laterally against an abutment, undermining the concrete backwall capping the foundation piling and exposing the upper 102 ft (0.3 - 0.6 m) of the steel H-pile. Bridge wingwalls were of rectangular design, parallel with roadway, rather than flared. The abutment fill caved into the opening eroded beneath the abutment backwall. When sufficient fill had been lost to open a gap behind the wall, the flow impinging on the abutment entered the gap and either partially or wholly breached an approach embankment. At bridge 5.6, a point bar deposit of gravel formed beneath the bridge on the inside of the bend (fig. 84) such that the conveyance beneath one span has been much reduced.

As a countermeasure, the streambanks have been riprapped at the bridge approaches on the outside of bends. Riprap has also been placed at the abutments. The riprap does not appear to conform to any specific design standards, but it includes large stones.

Discussion: The 1974 flood that damaged the bridges was much larger than the flood for which they were designed, but the bridges would likely have withstood the flood with little damage if they had not been built at bends. The crossings were evidently placed under the constraint that the stream location could not be modified, even though the stream had already been grossly changed by dredging for gold (fig. 84). If channel changes had been permissible, the need for bridges 5.4, 5.6, 6.1, and 6.3 could have been eliminated by discontinuous cuts between ponds in piles of dredge tailings, with little change in stream length or slope.

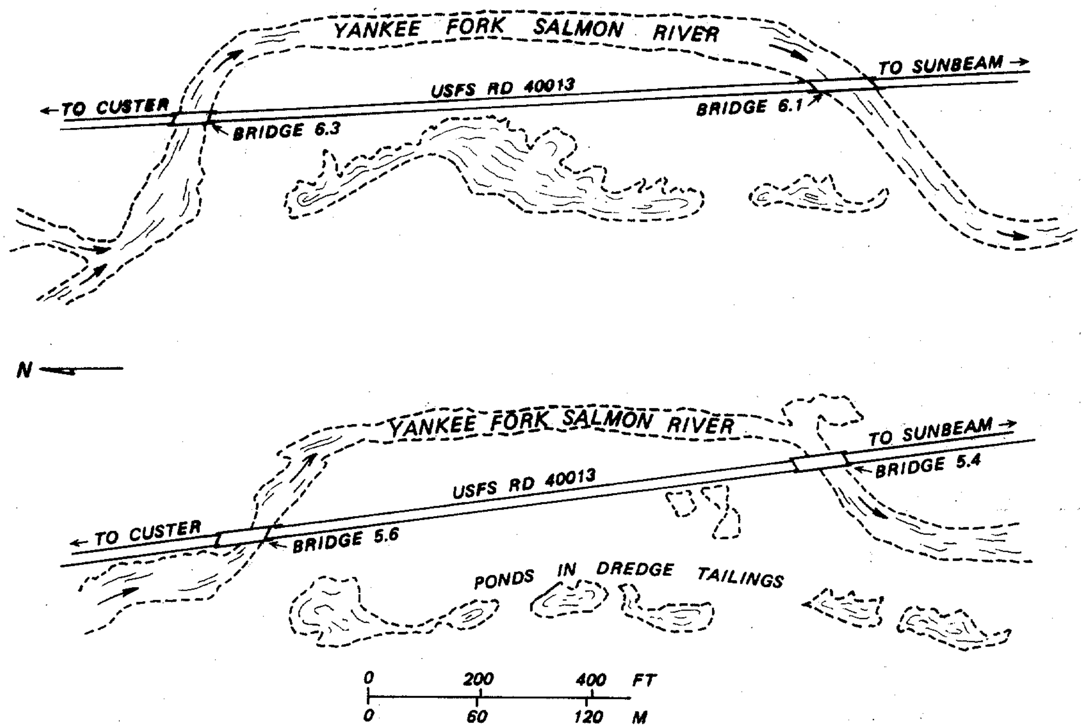


Figure 83. Plan sketch of two reaches of Yankee Fork Salmon River, each crossed by two Forest Service bridges.

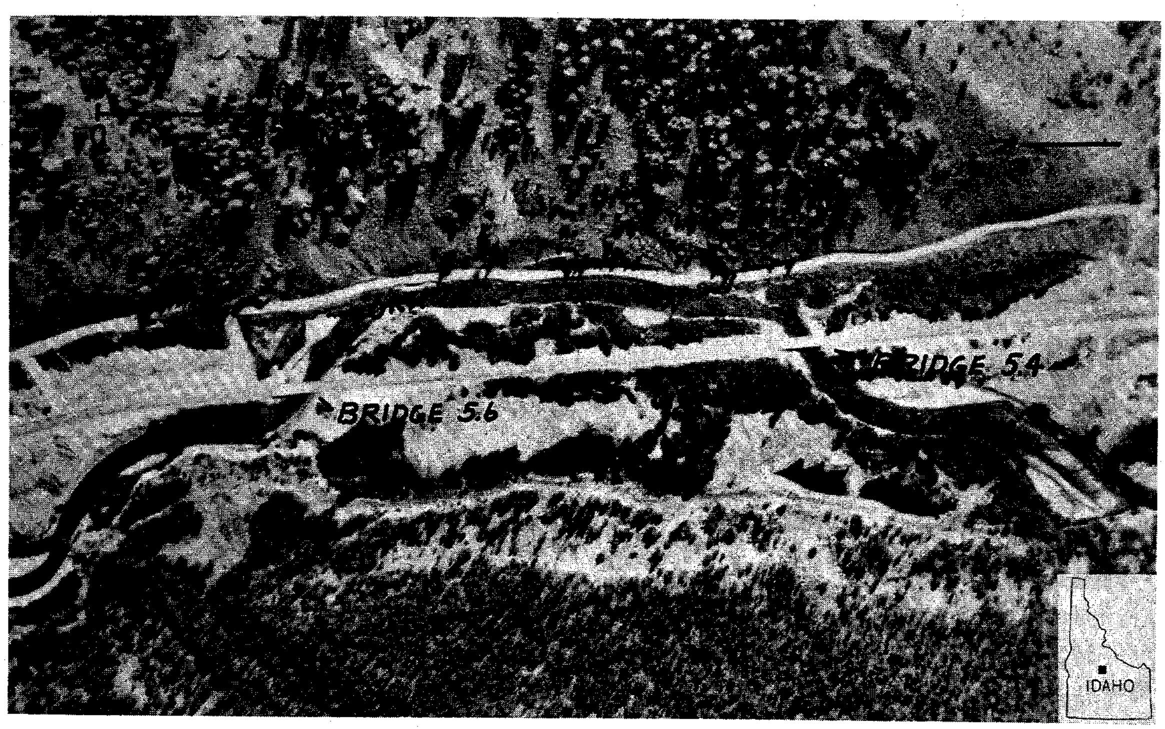


Figure 84. Vertical aerial photograph in 1975 of a reach of Yankee Fork Salmon River, crossed by bridges 5.6 and 5.4. (From U.S. Forest Service.)

SITE 183. EAST BUREAU CREEK AT I-180 NEAR PRINCETON, ILL.

Description of site: Lat 41°20', long 89°23', location at shown in fig. 85. Duel bridges, built about 1970, about 200 ft (61 m) in length, each with two round-nose wall-type concrete piers in channel, spillthrough abutments. Abutment fill-slopes were revetted with concrete slope pavement, 6 in (15 cm) in thickness, anchor wall at center, concrete toe-wall extending 3 ft (1 m) below ground line, wrapped around abutment fill-slope to distance of 10 ft (3 m) behind abutment backwall (fig. 86). Crossing is skewed about 45°, but piers are alined with flow.

Drainage area, 101 mi^2 (262 km^2); valley slope, 0.0026; channel width, about 70 ft (21 m). Stream is perennial, alluvial, sand bed, in valley of moderate relief, wide flood plain. Channel is sinuous, silt-clay banks.

Hydraulic problem and countermeasure: During a flood in 1974, peak discharge about 6,000 ft^3/s (170 m^3/s), R.I. about 50 yr, the toe of the concrete slope pavement at both the left and right bank abutments of the dual bridge was undermined. As a consequence, most of the pavement was lost (fig. 87A). The abutment fill-slope was most severely eroded at the left banks, where the abutment footing and supporting pile were exposed.

Riprap was dumped at the abutments as a temporary measure, and a contract was negotiated with Intrusion-Prepakt Inc. for installation of patented "Fabriform" revetment as a permanent countermeasure. In 1974, about 40,000 ft^2 (3,717 m^2) of 8-in (20-cm) Filter Point style "Fabriform" erosion control mats were installed on regraded slopes along both sides of the stream, beginning with the upstream abutment fill-slopes of the northbound bridge (fig. 87B) and extending about 100 ft (30 m) downstream from the southbound bridge. In the immediate area of the bridges, the "Fabriform" protection began in an anchor trench at the top of each bank and extended continuously across the streambed (fig. 87C). "Fabriform" consists of porous, pre-assembled nylon fabric forms or envelopes which are placed on the slopes and then filled with high-strength mortar by injection. In 1974, the cost of the "Fabriform" installation was in the range of \$12-15 per yd^2, as compared with \$16-19 per yd^2 for concrete slope pavement and \$10-19 per yd^2 for dumped rock riprap, in Illinois. As an additional countermeasure, a check dam built of "Fabriform" (nylon bags filled with mortar) was built downstream from the southbound bridge (fig. 87D). The apparent purpose of this dam is to reduce local and general scour in the bridge waterway, and thereby to protect the toe of the revetment.

Discussion: Although the toe of the concrete slope pavement was keyed in to a depth of about 3 ft (1 m) below streambed by a vertical concrete toe wall, the slope pavement at both banks extended almost to a pier. It seems probable that local scour at the piers contributed to undermining of the slope pavement. In the "Fabriform" installation, this hazard has been avoided by extending the revetment entirely across the streambed and also by construction of the check dam. As yet, the "Fabriform" installation has not been tested by a major flood.

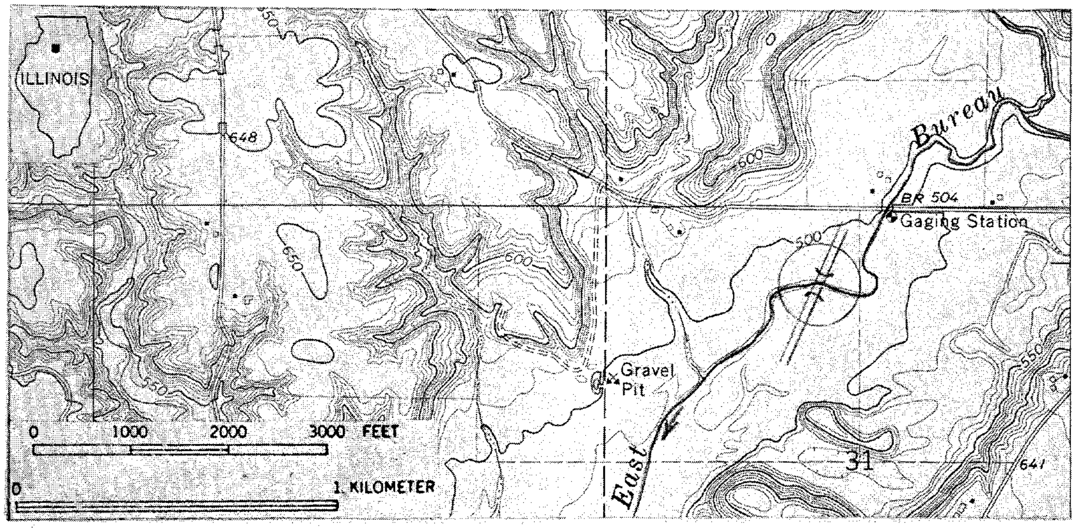


Figure 85. Map showing East Bureau Creek at I-180 (circled). (Base from U.S. Geol. Survey Princeton South, Ill., 7.5' quadrangle, contour interval 10 feet, 1966.)

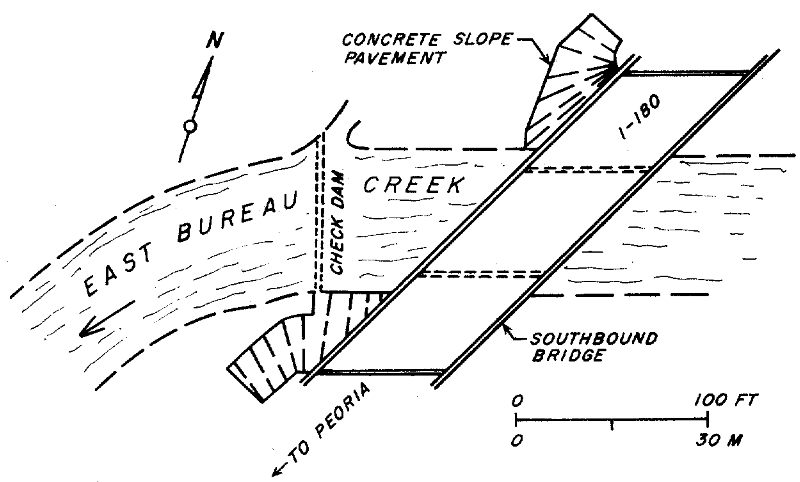


Figure 86. Plan sketch of southbound I-180 bridge showing placement of concrete slope pavement on downstream side of bridge, and position of check dam built in 1974.

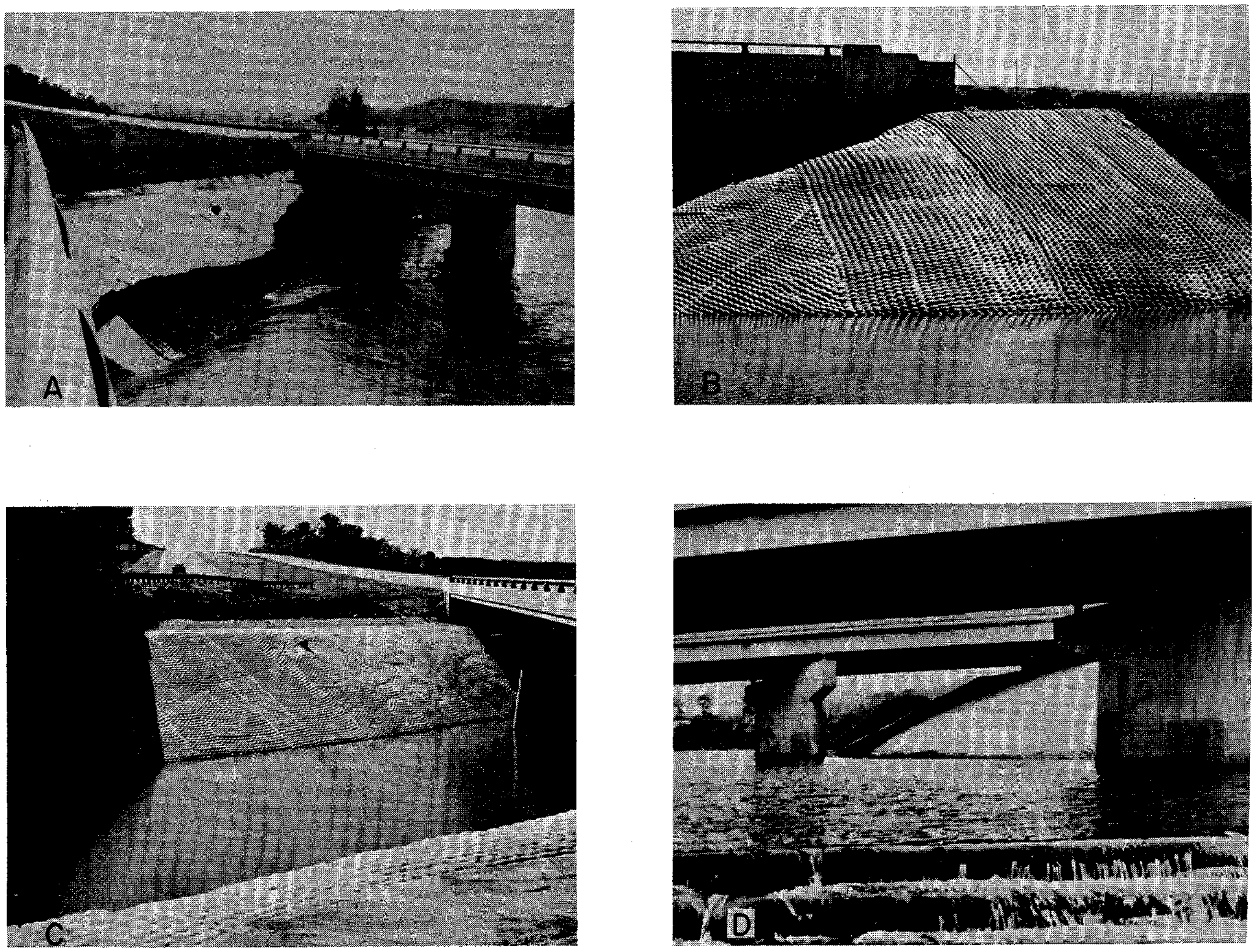


Figure 87. A, view of right (north) abutment fill-slope showing failure of concrete slope pavement at I-180 bridges, in 1974. B, view of "fabriform" revetment on upstream side of south abutment, northbound bridge. C, view of right (north) abutment fill-slope, showing placement of "Fabriform" revetment. D, upstream view of "Fabricast" check dam. (Photographs from Intrusion-Prepakt, Inc.)

SITE 184. WABASH RIVER AT US-460 RELIEF BRIDGE NEAR CROSSVILLE, ILL.

Description of site: Lat 38°08.5', long 87°58', location as shown in fig. 88. Bridge, here designated as relief bridge 3, is 150 ft (45 m) in length, three girder spans, two pile bents in waterway, spillthrough abutments revetted with concrete slope pavement. Drainage area, about 25,000 mi^2 (65,000 km^2).

Hydraulic problem and countermeasures:

1933 Bridge built.

1937 During a major flood, a scour hole developed downstream from the bridge, encroaching within 6 ft (2 m) of the nearest pile bent, and some scour occurred in the bridge waterway. Riprap consisting of concrete rubble and old street pavement was dumped under the bridge and along the north edge of the scour hole for a distance of 16 ft (5 m) from the pile bent.

1938-51 The scour hole continued to grow and deepen during successive floods, was named the "Blue Hole", and reached such diameter and depth as to seriously endanger the bridge and approach embankments. The previously applied riprap is credited with preventing damage to the bridge.

1956 Detailed study of causes for the scour hole was made by Hickenlooper, Guillou, and Chow (1957) and countermeasures were proposed. According to these authors, the hole was 250 ft (75 m) in diameter, the deepest part was 120 ft (36 m) from the nearest pile bent, and the bottom was 35 ft (11 m) lower than the ground beneath the bridge. The scour was attributed to division of flow by a timbered area upstream from the bridge, diversion of one flow segment along the east approach embankment, and resulting turbulence downstream from where the two flow segments met at the bridge. Little scour was observed at the nearby relief bridge 4, where the approach area was not timbered, although the flow conveyed per foot of bridge length during a flood peak was not much less than at relief bridge 3: 89.7 ft^3/s as compared with 112 ft^3/s at bridge 3. As countermeasures, several alternatives were recommended, one of which was straight, permeable spur dikes (double row of steel posts, faced with wire mesh and stone filled) extending 125 ft (38 m) upstream from each abutment. Another alternative was a steel sheet-pile cutoff-wall on the downstream side of the bridge, parallel with the bridge centerline.

1957 Major flood, further enlargement of scour hole. As countermeasures, additional riprap was placed on downstream side of bridge, mainly on the east side, and covered with concrete. The concrete slope pavement at the abutment fill-slope was extended along the upstream embankments (fig. 89) and an earth dike, 4-6 ft (1-2 m) in height, was built upstream from the bridge waterway.

1957-77 The "Blue Hole" persists, but further enlargement is not apparent. Among the factors in control of scour are regulation of the Wabash

River, effects of the countermeasures, and building of a farm levee (restricting flow at the relief bridge) by a local landowner. Inspection of the site in October of 1977 by personnel of the Illinois Department of Transportation showed that the upstream dike is heavily vegetated and that little encroachment of the hole toward the bridge has occurred since 1957.

Discussion: Scour holes at the downstream side of relief bridges are common, but none so large as the "Blue Hole" were found in the present study. Diversion of flow along the approach embankment, here attributed by Hickenlooper, Gillou, and Chow (1957) to the effects of upstream vegetation, usually results in scour beneath the bridge rather than downstream, but no other explanation for the development of the "Blue Hole" is apparent. Because of complicating factors, performance of the countermeasures applied in 1957 cannot be fairly evaluated, but the riprap applied at the downstream side of the bridge (in 1937 and subsequent years) has evidently prevented undermining of the bridge substructure.

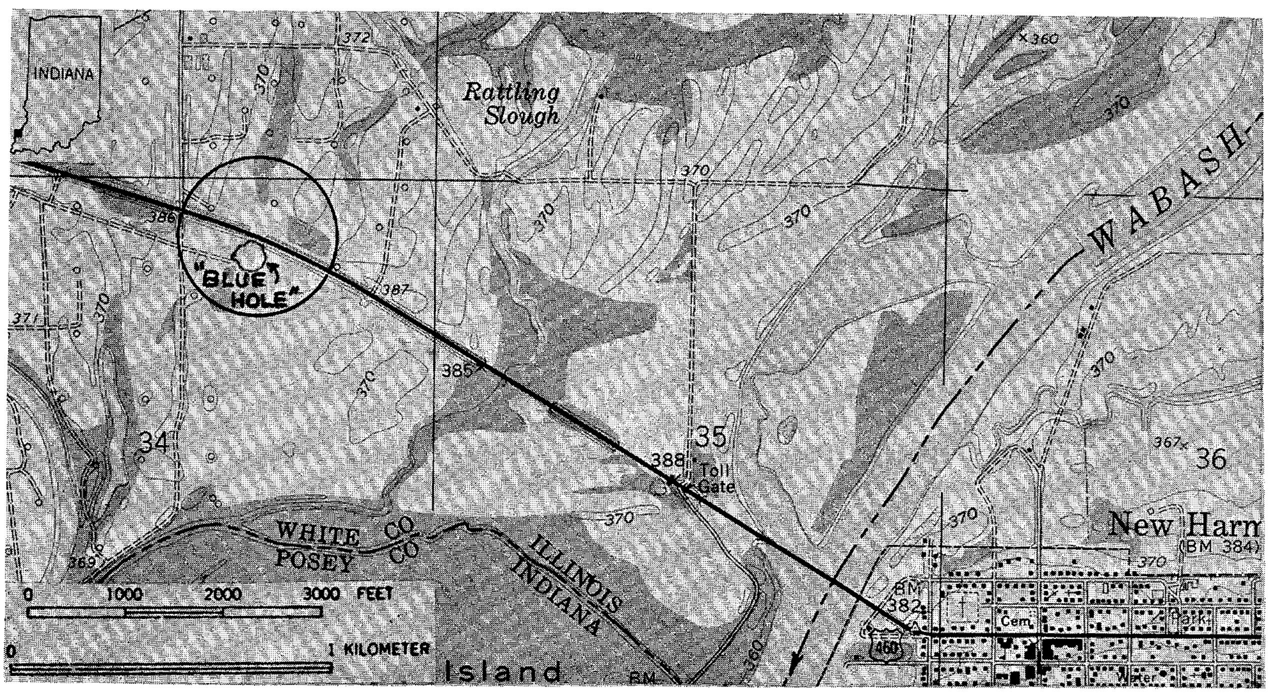


Figure 88. Map showing US-460 crossing of Wabash River and location of relief bridge 3 (circled). (Base from U.S. Geol. Survey New Harmony, Ind.-Ill., 7.5' quadrangle, contour interval 10 feet, 1959.)

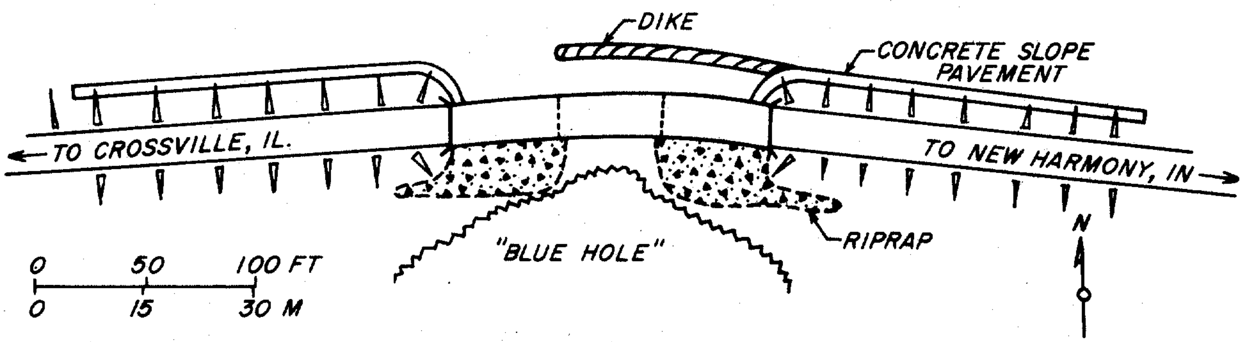


Figure 89. Plan sketch showing countermeasures at relief bridge 3.

SITE 185. WEST PEARL RIVER AT I-59 AT PEARL RIVER, LA.

Description of site: Lat 30°23', long 89°44', location as shown in fig. 90. Dual concrete bridges are 2700 ft (823 m) in length; piers are protected from river traffic by timber-pile fender system (fig. 90).

Drainage area, 8,590 mi^2 (22,250 km^2); valley slope very low, less than 0.0002; channel width, 275 ft (82 m). Stream is perennial, alluvial, sand bed, in valley of low relief, wide flood plain. Channel is sinuous, locally anabranched, equiwidth, not incised, cut banks local, silt-sand banks.

Hydraulic problem and countermeasure: Lateral erosion of left bank at bridge and local scour at piles of fender and foundation piles of bridge pier; effect on bridge use, potential only. Fender piling, which had been driven to depth of about 22 ft (7 m) below river bed, were not deep enough and main problem was probable loss of fender. As a countermeasure, the upstream end of left fender structure was extended to bank, converting it to a kind of spur (fig. 91). Pilings were set deeper and dumped rock riprap was placed around timber piling and bridge piers.

Discussion: Bend in stream seems too slight to have caused lateral erosion, which is possibly caused by location of fender near left bank or by flow deflection from upstream railroad bridge. Countermeasure not yet tested by flood.

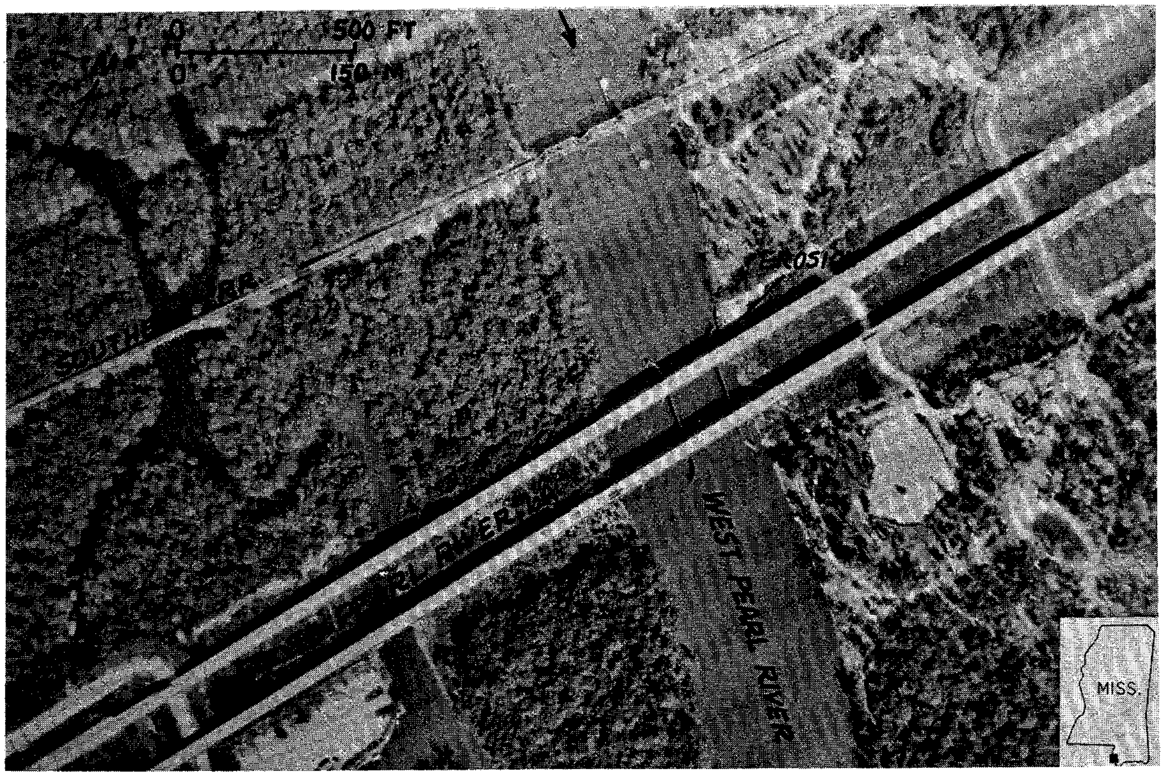


Figure 90. Aerial photograph of West Pearl River at I-59 bridge on September 20, 1974, before countermeasures were installed. (From Louisiana Dept. of Transportation.)

Figure 91. View on March 2, 1977 of upstream end of left timber-pile fender as extended to bank by addition of a double row of timber piles.

SITE 186. WHITE RIVER AT US-83 SOUTH OF MURDO, S. DAK.

Description of site: Lat 43°43', long 100°41', location as shown in fig. 92. Bridge is about 475 ft (145 m) in length, steel-plate girder spans, square-nose webbed concrete piers, spillthrough abutments.

Valley slope, 0.001; channel width, 230 ft (70 m). Stream is perennial but flashy, alluvial, silt-sand bed, in valley of moderate relief, wide flood plain. Channel is highly meandering, wider at bends, point bars, wandering thalweg, cut banks general, silt-clay banks, tree cover at less than 50 percent of bankline. Low-water channel forms a thalweg that tends to meander within a wide, unvegetated high-water channel.

Hydraulic problem and countermeasures:

1960 Approximate date of bridge construction.

1961-67 Wandering thalweg shifted toward left abutment (fig. 93). eroding abutment fill-slope and exposing foundation pile of abutment. Concurrently, sediment accumulated at right end of bridge (fig. 94).

1968 Three spurs constructed of steel H-pile, faced with corrugated sheet metal (fig. 95), were built along left bank upstream from bridge, also two spurs of steel sheet-pile filled with earth (fig. 96). The sheet-pile spurs are ten feet in width, and the opposite rows of sheet pile are tied together with 1.5 in (3.8 cm) steel bars attached to tie plates.

1973 According to a bridge inspection report dated November 19, 1973, the upstream H-pile spur was in good condition. Some erosion was observed at the landward end of the sheet pile spurs, where they were bypassed by high flow. A large amount of debris was reported at one of the piers.

Discussion: The White River at this locality is of a type that commonly causes problems of lateral erosion at bridge abutments. The banks are of erodible silt-clay and the grassland vegetation does not provide much bank protection. The high-water channel may shift rapidly during floods, and the low-water channel, or thalweg, wanders within the banklines, eroding where it impunges laterally against a bank. Lateral erosion at crossing is attributed to a shift of the low-water channel against the bankline.

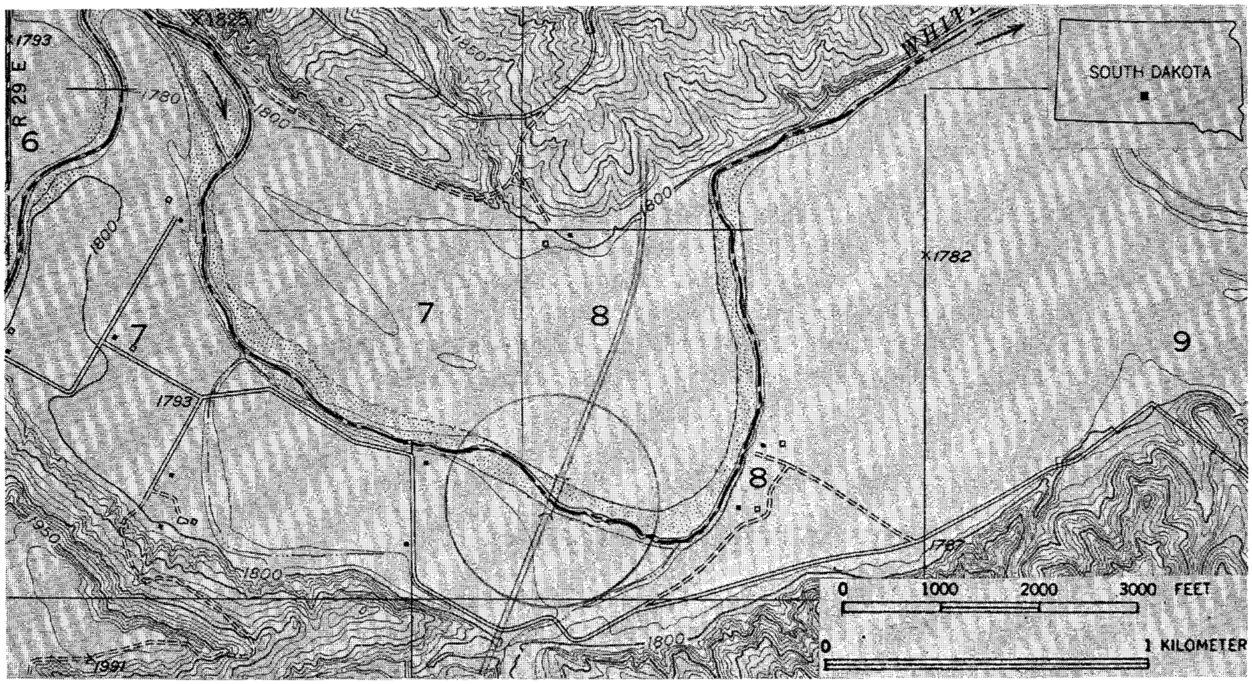


Figure 92. Map showing White River at US-83 crossing (circled). (Base from U.S. Geol. Survey Westover, S. Dak., 7.5' quadrangle, contour interval 10 feet, 1951.)

Figure 93. Aerial photograph of US-83 crossing, White River, on August 27, 1967. (From U.S. Dept. of Agriculture.)

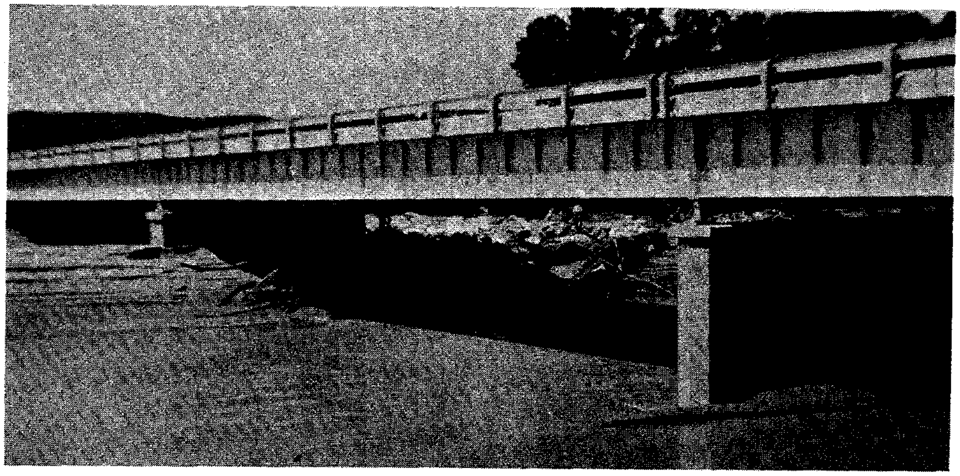


Figure 94. View showing sediment accumulation at right end of US-83 bridge, July 1967. (From South Dakota Div. of Highways.)

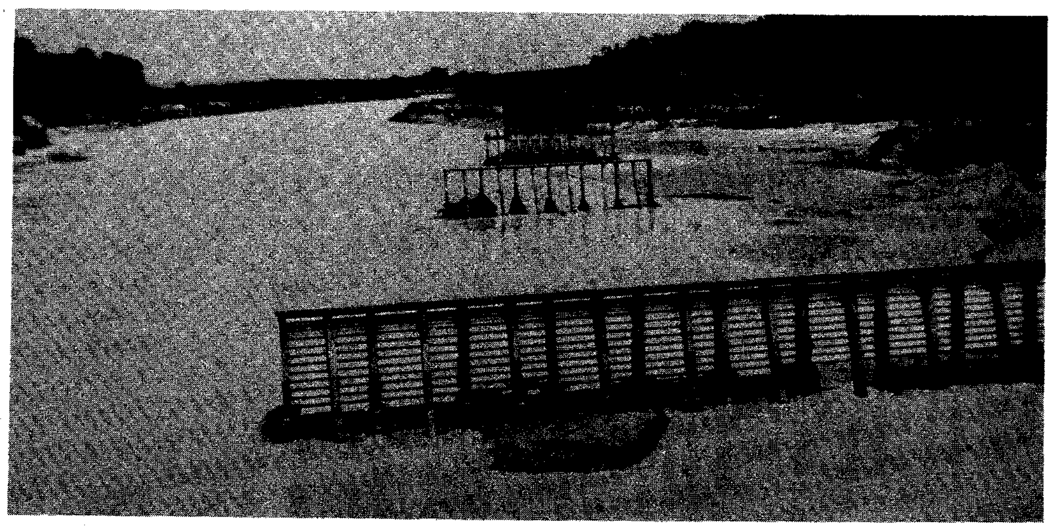


Figure 95. View along right bank upstream from US-83 bridge in June, 1968, showing three steel H-pile spurs (foreground and middle ground) and sheet-pile spur under construction (background). (From South Dakota Div. of Highways.)

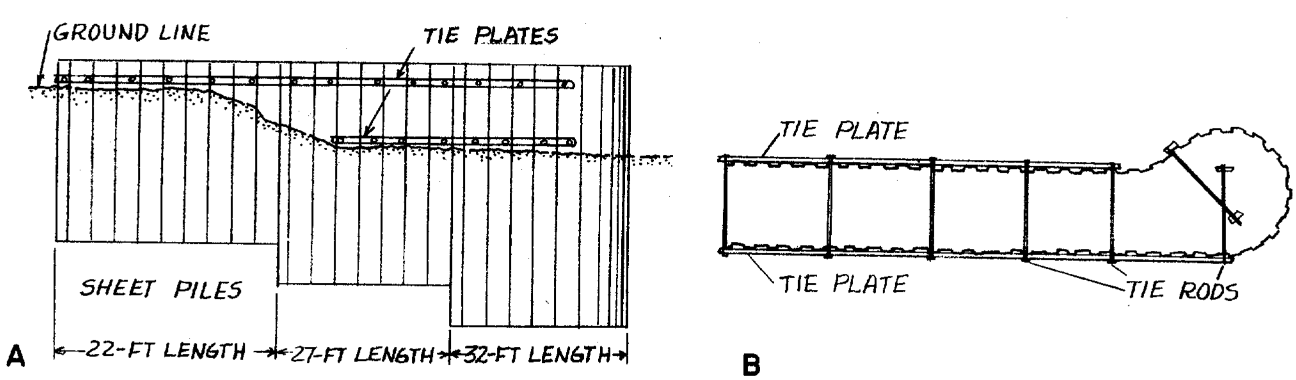


Figure 96. A, elevation and B, plan view, of sheet pile spur.

SITE 187. NORTH SHORE OF LAKE PONCHARTRAIN AT I-10 NEAR SLIDELL, LA.

Description of site: Lat 30°13', long 89°48'. Concrete bridge, pile bents with three round concrete columns, spillthrough abutments.

Hydraulic problems and countermeasures: Problem is wave erosion at lake shore near abutment; effect on bridge use, potential only. Original countermeasure was wall of timber on timber pile, offshore from abutment, and sacked concrete revetment along shore at bridge. In 1972, wave action during a hurricane demolished the timber-pile structure and damaged the sacked-concrete revetment (fig. 97). Riprap of broken concrete slabs was dumped upslope from, and in damaged areas of, the sacked concrete.

Discussion: Timber bulkheads along shore might be more effective than sacked concrete, as these have performed effectively for many years at the nearby abutments of US-11.

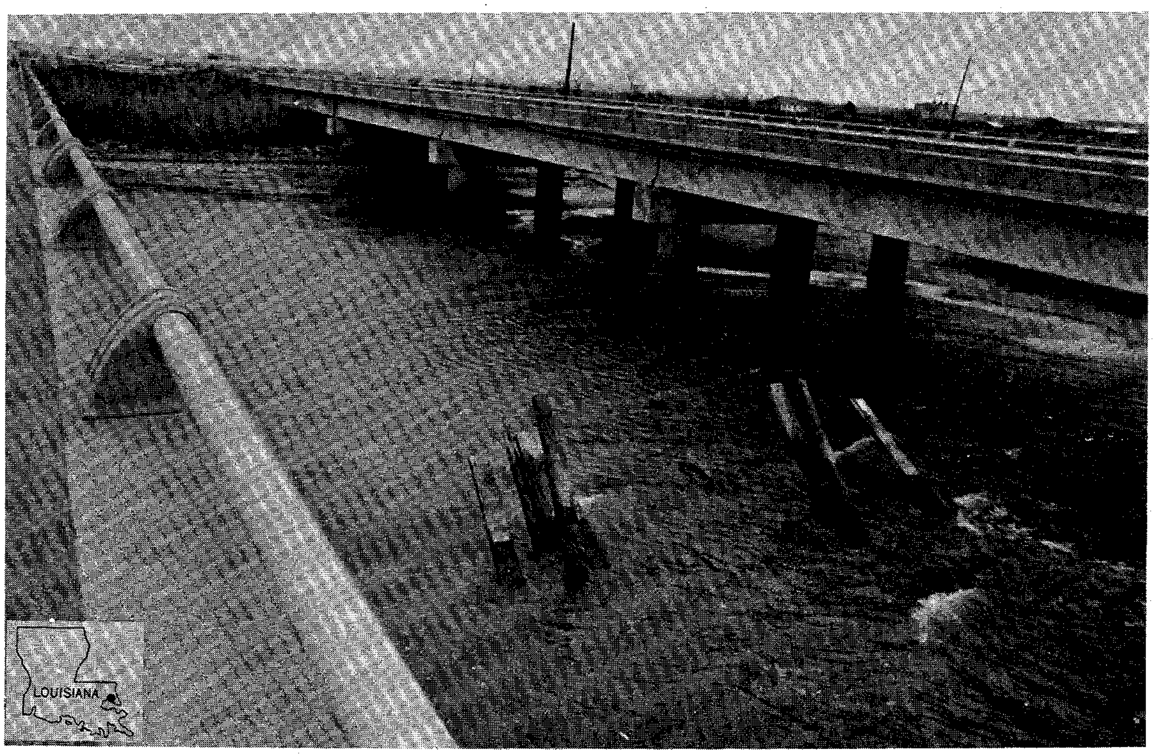


Figure 97. View looking northeast toward abutment of I-10 and shore of Lake Ponchartrain, on March 2, 1977. Remnants of timber-pile wall, foreground, and damaged sacked concrete revetment at abutment, background.

SITE 188. RED RIVER AT US-84 AT COUSHATTA, LA.

Description of site: Lat $32^{\circ}01'$, long $93^{\circ}21'$, location as shown in fig. 98. Bridge is 1900 ft (579 m) in length, steel truss spans, concrete wall-type piers, spillthrough abutments. Crossing is at bend.

Drainage area, 57,426 mi^2 (148,700 km^2); valley slope, 0.0002; channel width, 800 ft (240 m). Stream is perennial, alluvial, silt-sand bed, in valley of low relief, wide flood plain. Channel is meandering, wider at bends, point bars, not incised, cut banks general, silt banks.

Hydraulic problem and countermeasure: Problem is lateral erosion of right bank at, and upstream from, the right bridge abutment (fig. 98). Countermeasure is dumped rock riprap, approximately equivalent to Class I, placed in windrows set back from the bankline (fig. 99) somewhat as described by U.S. Army Engineers (Lindner, 1969, p. VIII-9. The riprap was placed in 1973.

Discussion: By 1976, bank erosion had reached one of the windrows (fig. 100), and the rate of erosion had been reduced. However, not enough time has elapsed for evaluation of the countermeasure.

Figure 98. Aerial photograph of bank erosion at US-84 bridge, January 9, 1976. (From Louisiana Dept. of Transportation.)

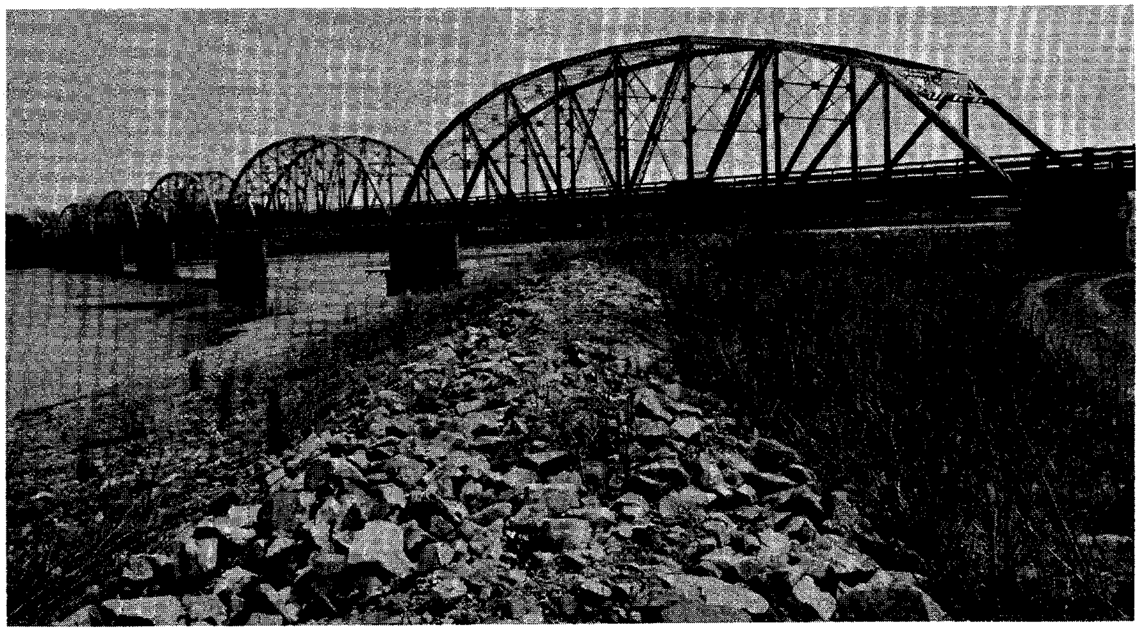


Figure 99. Windrow of rock riprap along right bank upstream from U.S. 84 bridge, March 1977.

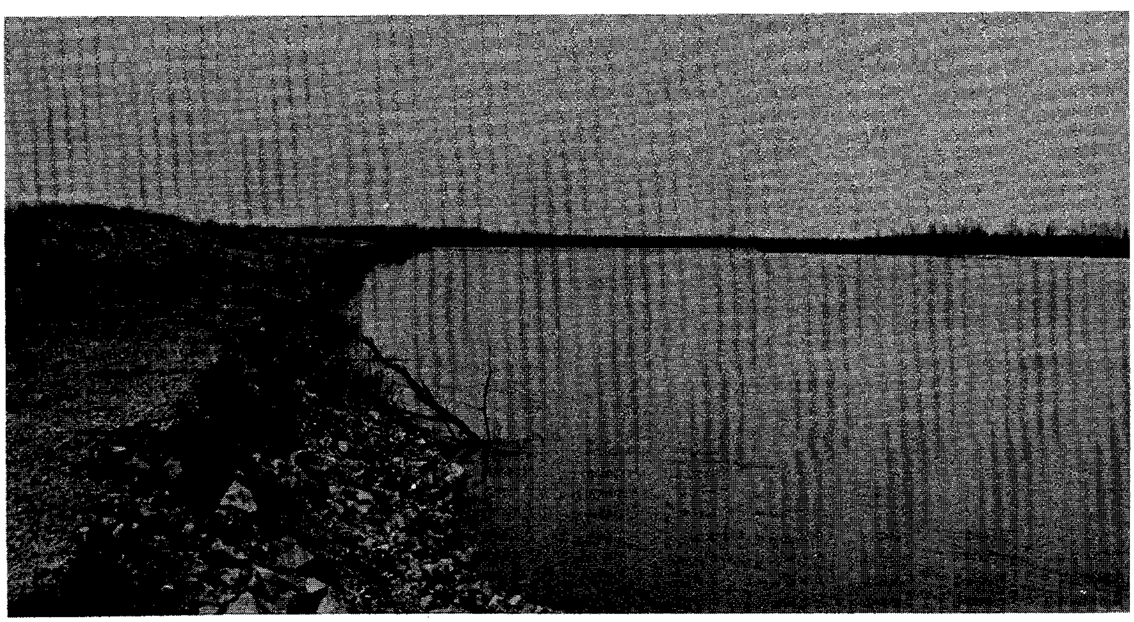


Figure 100. Riprap, emplaced by intersection of a windrow, on eroding bank upstream from US-84 bridge, March, 1977.

SITE 192. THOMPSON CREEK AT US-61 NEAR STARHILL, LA.

Description of site: Lat 30°45', long 91°17', location as shown in fig. 101. Bridge, built in 1960, is 850 ft (259 m) in length, concrete wall-type piers in channel, spillthrough abutments paved with concrete. Crossing at bend in stream (fig. 101).

Drainage area, 249 mi^2 (645 km^2). Stream is perennial, alluvial, sand bed, in valley of low relief, wide flood plain. Channel is sinuous, but disturbed by large-scale mining of sand and gravel (fig. 102).

Hydraulic problem and countermeasure: Lateral erosion of right bank at abutment and left bank upstream from bridge; effect on bridge use, potential only. A new, straight upstream approach channel was dredged (fig. 102), representing a month's work by a contractor, but stream shifted toward previous channel during the next flood. Banks of stream have been riprapped with broken concrete, and a timber retard backed with riprap has been installed at right bank upstream from bridge. On the right side of the channel, the embankment of entering SR-964 has been riprapped, so that it functions somewhat as a spur dike (figs. 102 and 103).

Discussion: Countermeasures have performed satisfactorily, but some of the broken concrete riprap on streambanks and on the embankment of SR-964 is lost during floods. Mining of sand and gravel, and the clearing of natural vegetation along the banks, have contributed to the lateral instability of the channel.

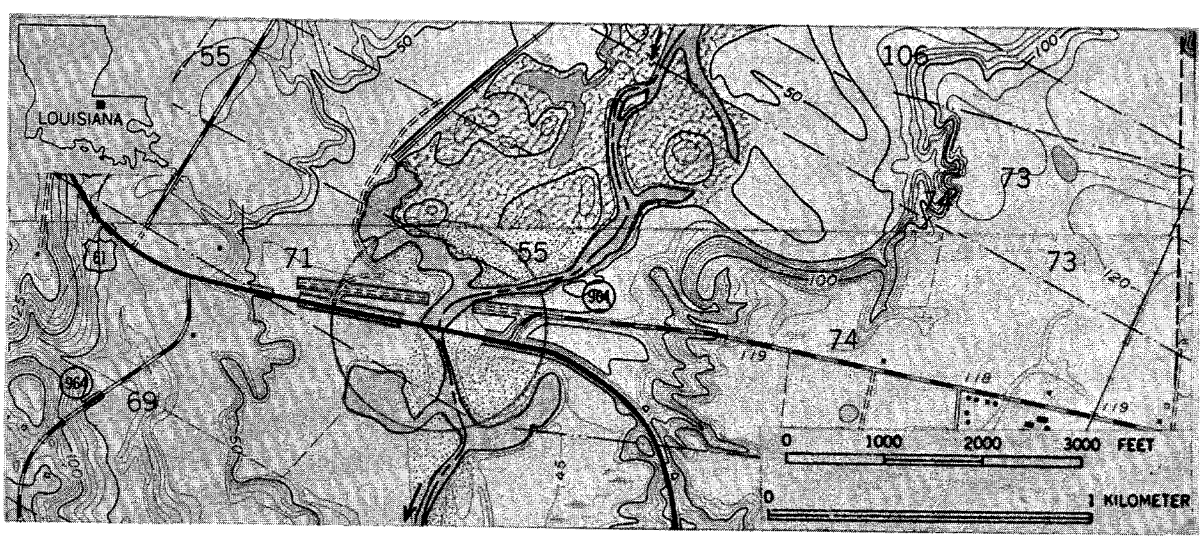


Figure 101. Map showing Thompson Creek at US-61 crossing (circled). (Base from U.S. Geol. Survey Elm Park, La., 7.5' quadrangle, contour interval 10 feet, 1965; and Port Hudson, La., 7.5' quadrangle, contour interval 5 feet, 1963.)

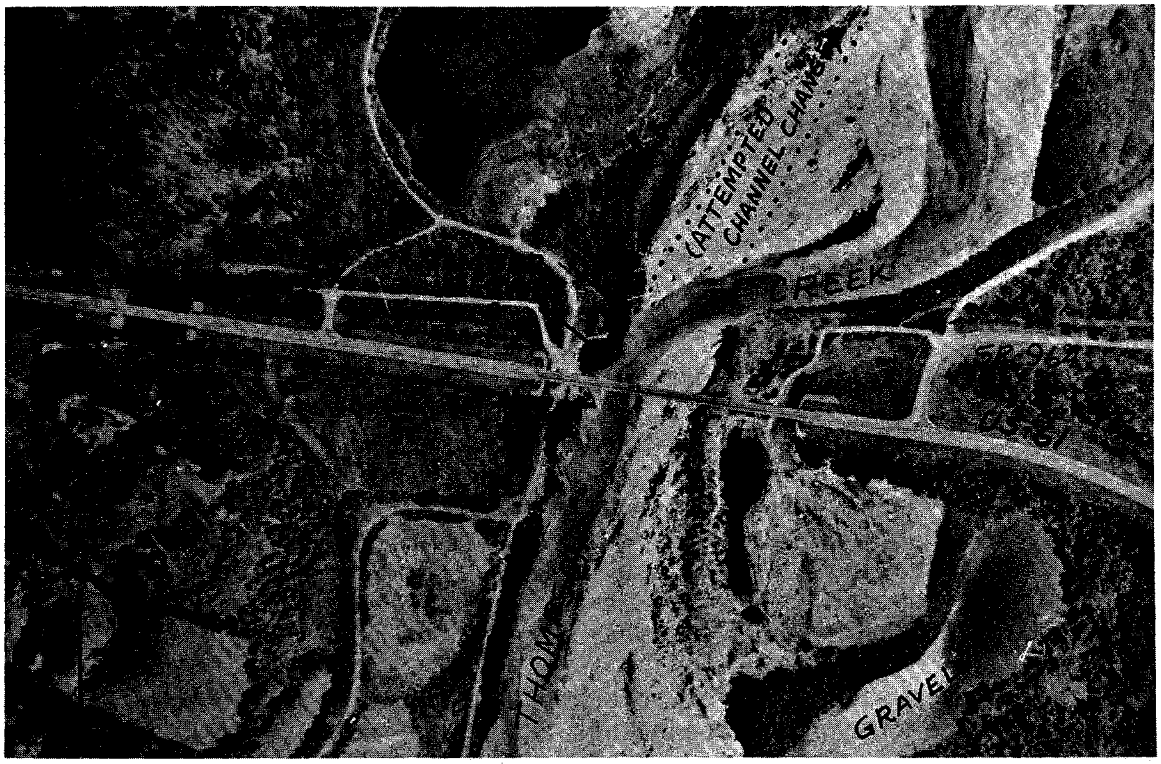


Figure 102. Aerial photograph of Thompson Creek at US-61, on August 9, 1976. (From Louisiana Dept. of Transportation.)

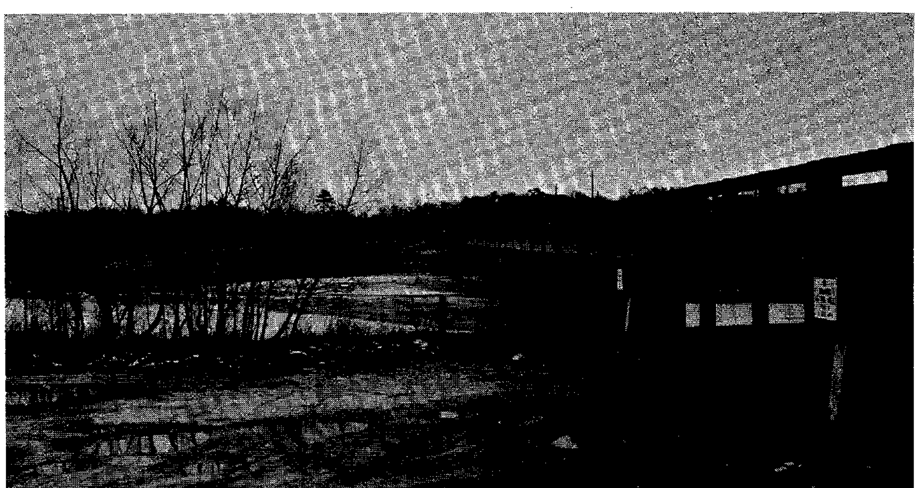


Figure 103. View of upstream side of US-61 bridge, looking toward left bank, on February 15, 1977. Revetted embankment of entering SR-964 is visible beyond bank.

SITE 193. LAWRENCE CREEK AT SR-16 NEAR FRANKLINTON, LA.

Description of site: Lat 30°49', long 90°09', location as shown in fig. 104. Bridge is 450 ft (135 m) in length, pile bents with square concrete pile, spillthrough abutments revetted with sacked concrete. Crossing is at bend.

Drainage area, 50 mi^2 (130 km^2); valley slope, 0.003. Stream is perennial, alluvial, sand bed, in valley of low relief, wide flood plain. Channel is sinuous, not incised, sand-gravel banks.

Hydraulic problem and countermeasure: Gravel mining downstream from bridge caused channel degradation and lateral erosion of banks upstream and downstream from bridge. Lowering of streambed was not regarded as a serious hazard to pile bents in channel. Probability that stream would erode approach embankment upstream from bridge was considered more serious. Effect on bridge use, potential only, except for closure of bridge during addition of span.

As a countermeasure, bridge was lengthened by addition of span to right end in 1967. In 1973, the approach embankment was raised and a curved spur dike of earth, revetted with broken concrete riprap, was built at the right abutment to protect the embankment (fig. 104). Timber cleared during construction of the spur dike was to be placed along the embankment but was placed too far from embankment. During a flood in 1974, flow was diverted against the spur dike, eroding the tip and upstream side of the dike and impinging against the approach embankment.

Discussion: Lateral instability of the channel is directly attributed to gravel mining. Although the spur dike serves to direct flow through the bridge waterway, it does not seem to be an effective measure for protection of the approach embankment, which is subject to erosion or breaching during major floods. The main channel of Thompson Creek is apparently too nearly parallel to the right approach embankment for the spur dike to function properly.

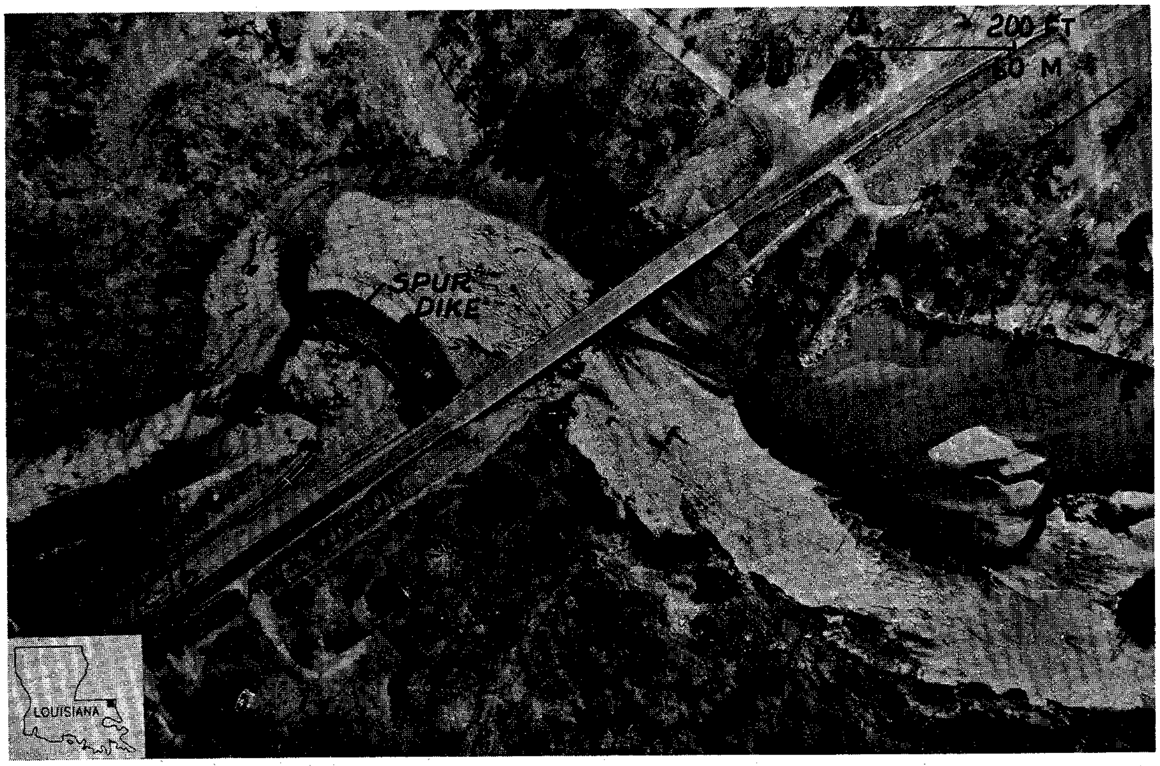


Figure 104. Aerial photograph of Lawrence Creek at SR-16, taken January 21, 1974, during repair of damage to spur dike. (From Louisiana Dept. of Transportation.)

Figure 105. Aerial photograph of Spring Creek at SR-440, on April 5, 1974. (From Louisiana Dept. of Transportation.)

SITE 194. SPRING CREEK AT SR-440 NEAR TANGIPAHOA, LA.

Description of site: Lat 30°53', long 90°30', location as shown in fig. 105. Bridge is 220 ft (66 m) in length, timber pile bents in channel, vertical abutments with timber bulkheads. Crossing is at abrupt bend in channel (fig. 105).

Drainage area, 12.5 mi^2 (32 km^2); valley slope, 0.003; channel width, 25 ft (7.5 m). Stream is perennial but flashy, alluvial, sand bed, in valley of low relief, narrow flood plain. Channel is meandering, silt-sand bed.

Hydraulic problem and countermeasure: Lateral erosion along roadway embankment, where stream flows parallel to embankment, and at right abutment, where stream turns 90° to flow through bridge waterway. Effect on bridge use, potential only. As a countermeasure, roadway embankment is protected by timber bulkhead supported by timber piles, which are reinforced by wire cables passing through the fill to timber piles on the opposite side of the embankment. Abutment is also protected by timber bulkhead (fig. 106). A span has been added to the right-bank end of the bridge.

Discussion: Crossing is located near confluence of Spring Creek with the Tangipahoa River, and the natural course direction of Spring Creek presents a difficult problem in crossing location. Under the circumstances, and without a rather drastic relocation of Spring Creek, the crossing and countermeasures have performed satisfactorily, although there was substantial erosion at the right abutment during floods in 1973.

Figure 106. Upstream side of SR-440 bridge over Spring Creek, looking toward left (east) abutment, in March 1977.

SITE 195. TANGIPAHOA RIVER AT SR-40 NEAR INDEPENDENCE, LA.

Description of site: Lat 30°36', long 90°26'. Bridge, built in 1965, is 600 ft (180 m) in length, concrete wall-type piers in channel, pile bents on flood plain, spillthrough abutments, riprapped.

Drainage area, 489 mi^2 (1,250 km^2); valley slope, 0.0005. Stream is perennial, alluvial, sand bed, in valley of low relief, wide flood plain. Channel is meandering, wider at bends with point bars, not incised, cut banks local, silt-sand banks.

Hydraulic problem and countermeasure: Because of location at bend, stream flows along approach roadway embankment for several hundred feet before entering bridge waterway, and lateral erosion is occurring along the embankment (fig. 107). In addition, the right bank downstream from the bridge is eroding. Effect on bridge use, potential only. As a countermeasure, riprap of broken concrete has been placed along roadway embankment.

Discussion: Although the broken concrete riprap has so far performed satisfactorily, the hazard to the roadway embankment remains. Extension of the left-bank end of the bridge, to replace the endangered embankment, is planned.

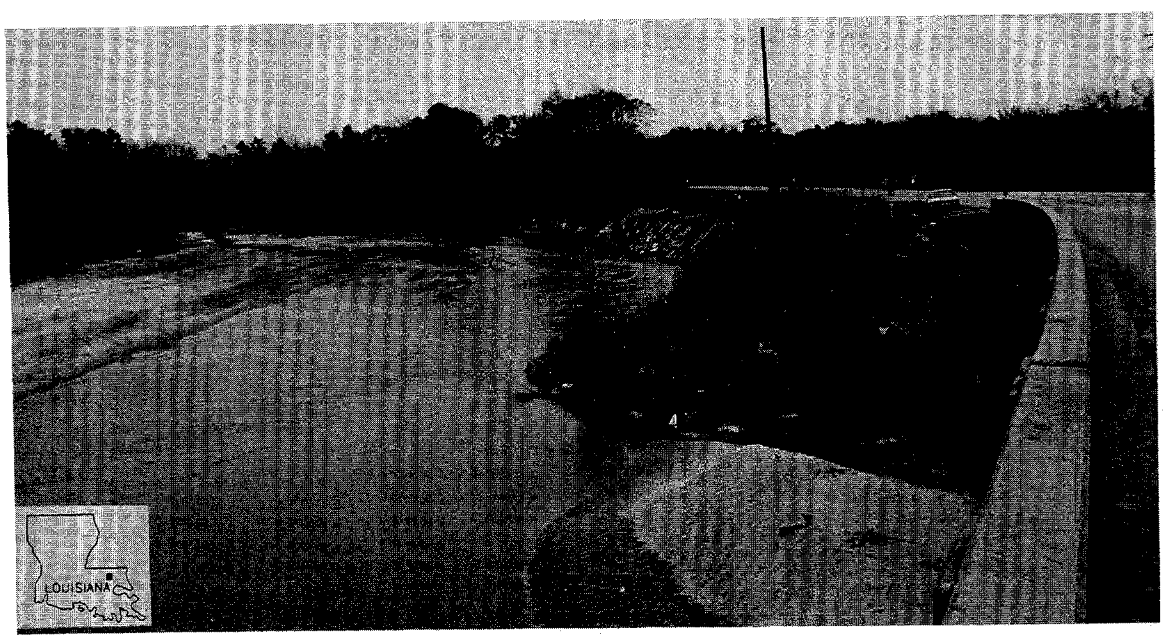


Figure 107. View on March 2, 1977, looking upstream from SR-40 bridge, where stream flows along approach embankment before turning to enter bridge waterway.

SITE 196. TICKFAW RIVER AT SR-441 AT STARNS, LA.

Description of site: Lat 30°34', long 90°40', location as shown in Fig. 108. Bridge, built in 1956, had span added to left end in 1972, present length 209 ft (64 m). Timber-pile bents, spillthrough abutments riprapped with broken concrete slabs. Crossing is at entrance of anabranch.

Drainage area, 236 mi^2 (612 km^2); valley slope, 0.001; channel width 60 ft (18 m). Stream is perennial, alluvial, sand bed, in valley of low relief, wide flood plain. Channel is meandering, locally anabranched, silt-sand banks.

Hydraulic problem and countermeasure: Bypass channel (anabranch) entering from right flows parallel to approach roadway embankment and turns about 90° to flow through bridge waterway (figs. 108 and 109). Impingement of flow at left abutment has caused lateral erosion of abutment fill and of streambanks. Formation of point bar deposit just downstream from bridge has further diverted flow against abutment. Effect of problem on bridge use, potential only, except for closing of bridge during addition of span. Countermeasure consists of addition of span, removal of point bar deposit, and placement of broken concrete riprap on fill-slope of left abutment.

Discussion: Countermeasures have apparently performed satisfactorily, but left abutment is effectively on the outside of an active meander loop and continuing maintenance problems are likely. In addition, the alinement of the bypass channel upstream from the bridge poses a difficult problem in crossing design.

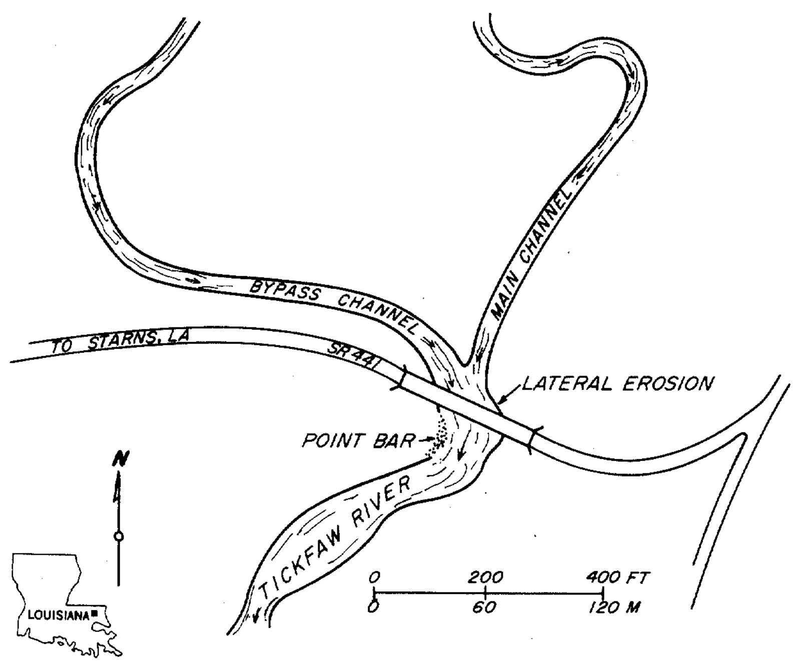


Figure 108. Plan sketch of Tickfaw River at SR-441. (From aerial photograph taken February 13, 1975.)

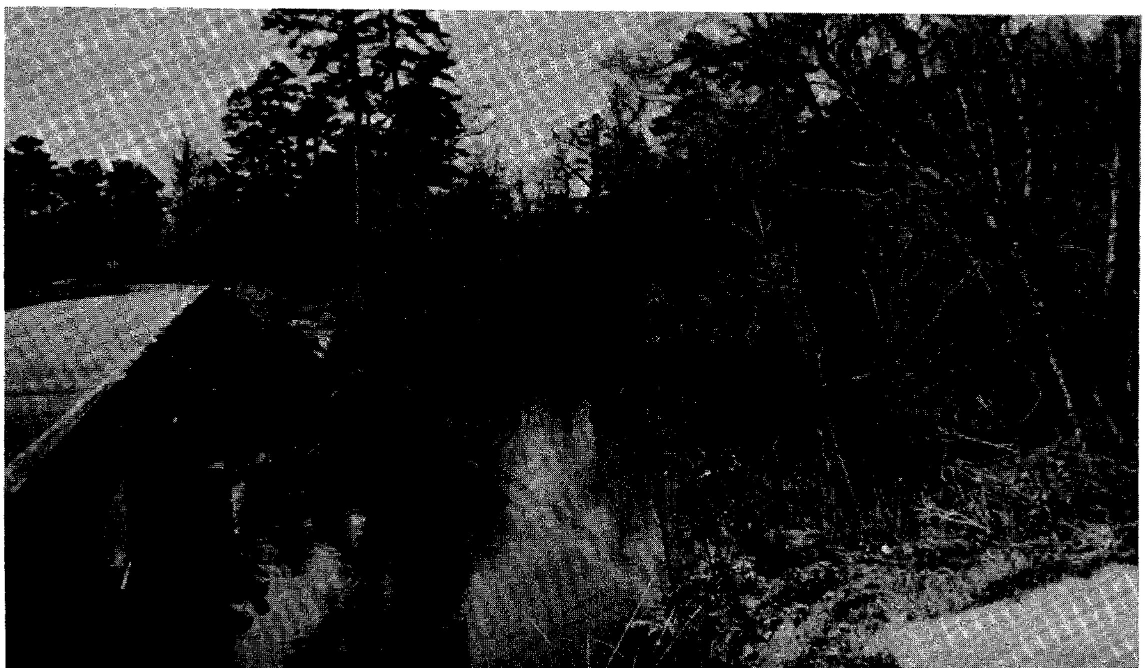


Figure 109. View of bypass channel, center background, which trends almost parallel with approach roadway before entering bridge opening, in March 1977.

SITE 197. AMITE RIVER AT SR-37 AT GRANGEVILLE, LA.

Description of site: Lat 30°44', long 91°51. Bridge is 400 ft (120 m) in length, pile bents with square concrete piles in channel, spillthrough abutments. Crossing is at bend in stream.

Drainage area, 741 mi^2 (1,920 km^2); valley slope, 0.001; channel width, 200 ft (60 m). Stream is perennial, alluvial, gravel bed, in valley of low relief, wide flood plain. Channel is meandering, generally anabranched, sand-gravel banks.

Hydraulic problem and countermeasure: Channel degradation is attributed to gravel mining upstream and downstream from bridge (fig. 110). Pile tips of some bents were found at streambed elevation, with no penetration into ground. Relief bridges were rendered ineffective because of levees at gravel mining operations. As a countermeasure, pile of pile bents are being driven to greater depth.

Discussion: Gravel mining is being continued, and further degradation of streambed may occur.

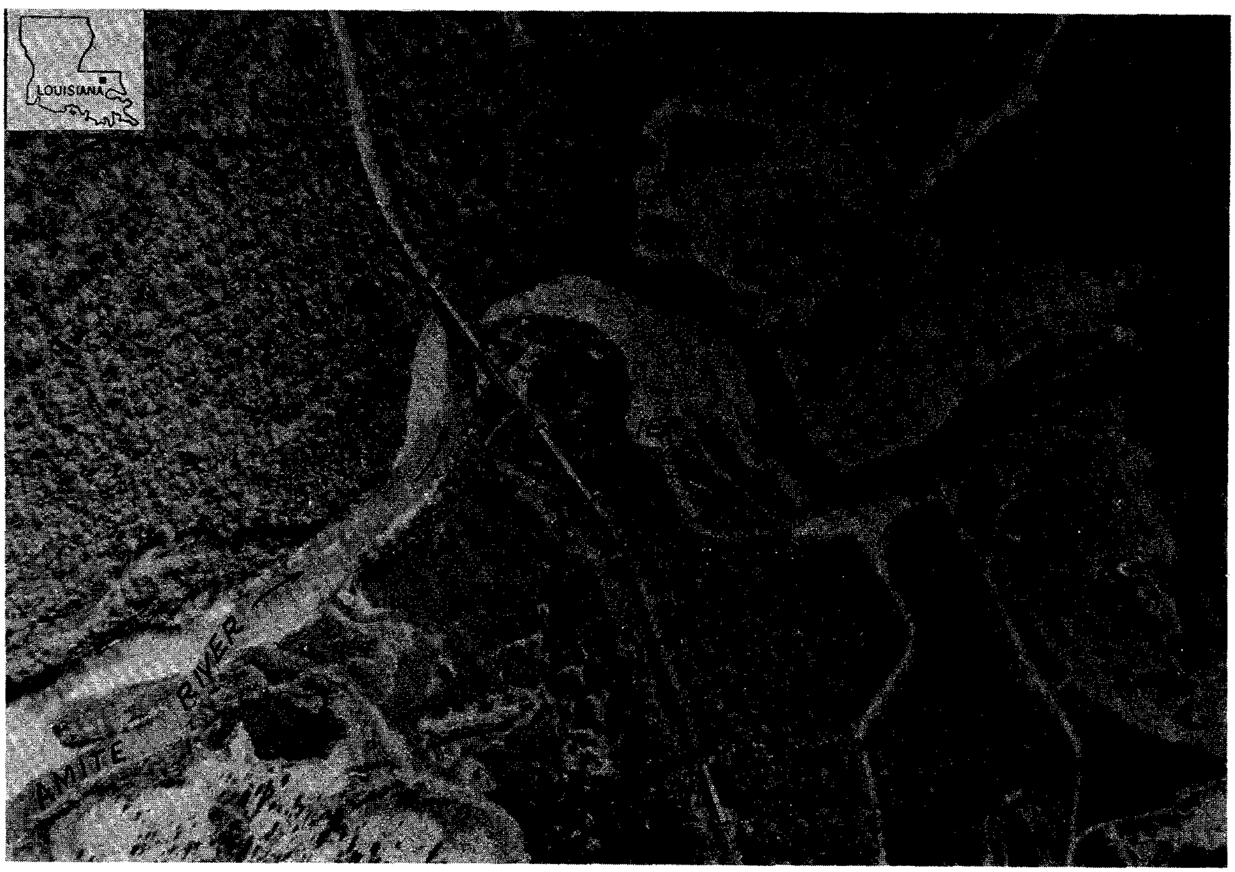


Figure 110. Aerial photograph of Amite River at SR-37, on February 27, 1976. (From Louisiana Dept. of Transportation.)

SITE 198. WHITTEN CREEK AT SR-37 AT BAYWOOD, LA.

Description of site: Lat 30°41', long 90°53', location as shown in fig. 111. Bridge, built in 1956, is 110 ft (33 m) in length, timber-pile bents driven into sand, spillthrough abutments.

Drainage area, 12.4 mi^2 (32 km^2); valley slope, 0.0002. Stream is perennial, alluvial, bed of sand and fine gravel, in valley of low relief, wide flood plain. Natural channel is meandering, not incised, cut banks rare.

Hydraulic problem and countermeasure: Channel degradation is attributed to gravel mining downstream from bridge and possibly to channel dredging upstream from bridge (fig. 111). Channel has been lowered by an estimated 10 ft (3 m) at bridge, timber-pile bents are exposed to hazardous depth and abutment fill-slopes have been eroded (fig. 112). Effect on bridge use, potential only. As a countermeasure, dumped riprap of broken concrete slabs has been placed on abutment fill-slopes, in an estimated amount of 200 yds^3 (152 m^3) and a span 16 ft (5 m) in length has been added to left end of bridge.

Discussion: Dumped riprap has reduced but not halted erosion at abutments, and little riprap remained at abutments on February 18, 1977. Area of bridge opening has increased by a factor of two since gravel mining began, and there were no hydraulic problems at the bridge before gravel mining. Despite addition of span, erosion has continued at left abutment. Partial failure of riprap attributed to sand banks, steepness of banks, and probably to lack of filter blanket and toe protection.

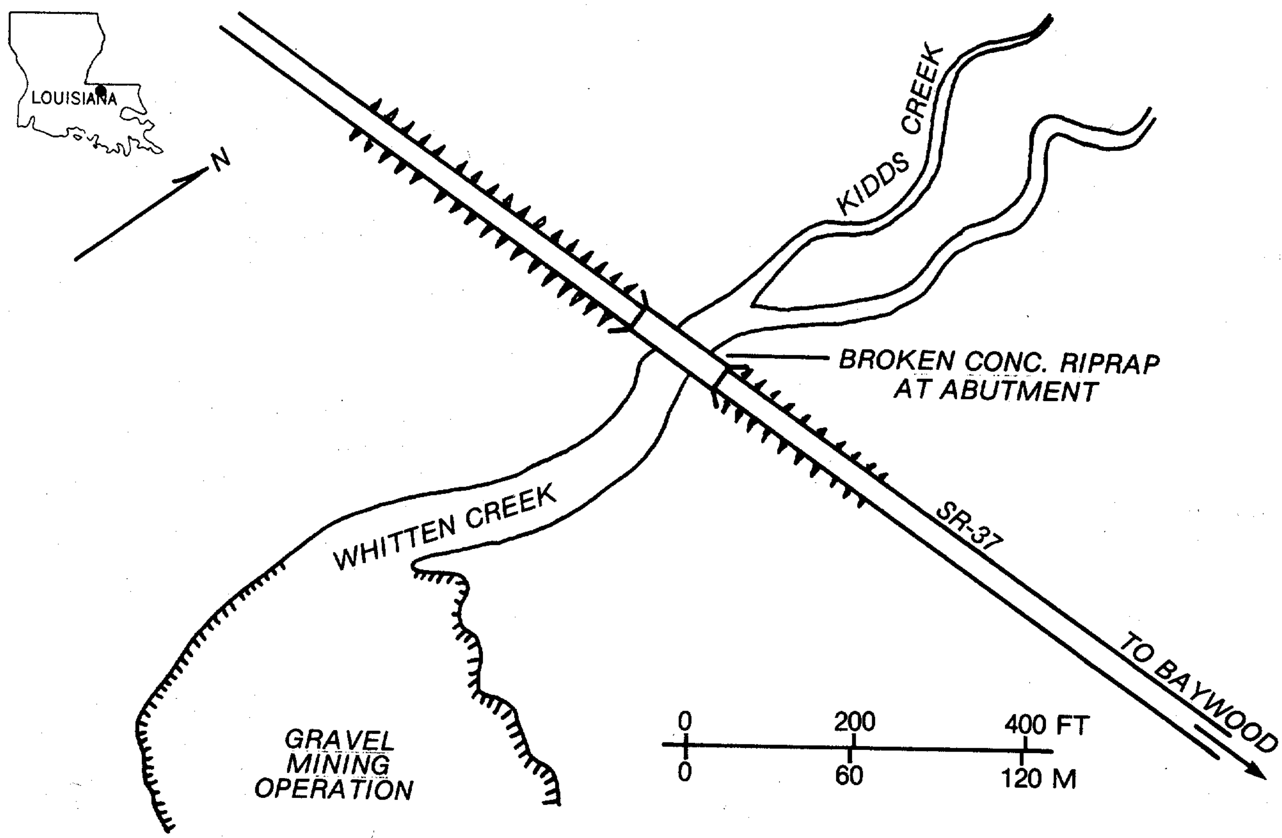


Figure 111. Plan sketch of Whitten Creek at SR-37. (From aerial photograph taken August 31, 1973.)

Figure 112. View of streambed and left abutment of SR-37 bridge at Whitten Creek, on February 18, 1977.

SITE 199. NORTH BRANCH WARD CREEK AT I-12 AT BATON ROUGE, LA.

Description of site: Lat 30°25', long 91°05', location as shown in fig. 113. Dual bridges, built about 1965, are 825 ft (252 m) in length, concrete-pile bents in channel, five square concrete pile to a bent; spillthrough abutment at right bank, paved with concrete at time of construction.

Drainage area, 6.3 mi^2 (16 km^2); valley slope, 0.0008; channel width, 90 ft (27 m). Stream is perennial but flashy, alluvial, silt clay bed, in valley of low relief, wide flood plain. Channel has been straightened, but has some bends; silt-clay banks; receives runoff from urbanized area.

Hydraulic problem and countermeasure: Slumping of channel bank and fill-slope at right abutment; cracking and displacement of concrete paving on abutment fill-slope (figs. 113 and 114). Effect on bridge use, potential only. After failure of the concrete slope-paving, riprap of broken concrete was dumped on the abutment fill-slope, but this proved to be ineffective. In 1974, a bulkhead of steel sheet-piling was driven at the toe of the fill-slope, backfilled with broken (mollusc) shells, and protected at the toe with broken concrete. Performance of this countermeasure is regarded as satisfactory.

Discussion: Slumping is attributed to height of embankment. The nearby bridge on SR-73, built in 1956 when the channel was straightened, has a low abutment fill-slope which has not failed by slumping. Neither the original concrete paving nor the broken concrete riprap that was subsequently applied, would be expected to serve as a countermeasure against slumping.

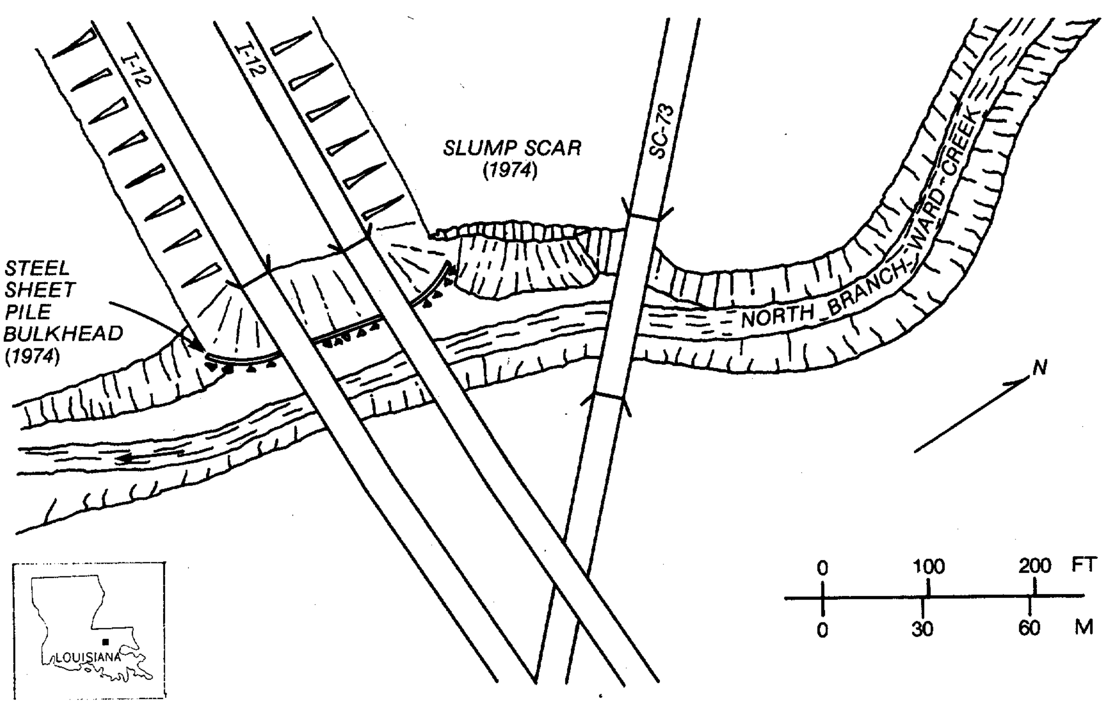


Figure 113. Plan sketch of North Branch Ward Creek at I-12. (From aerial photograph taken February 1974.)

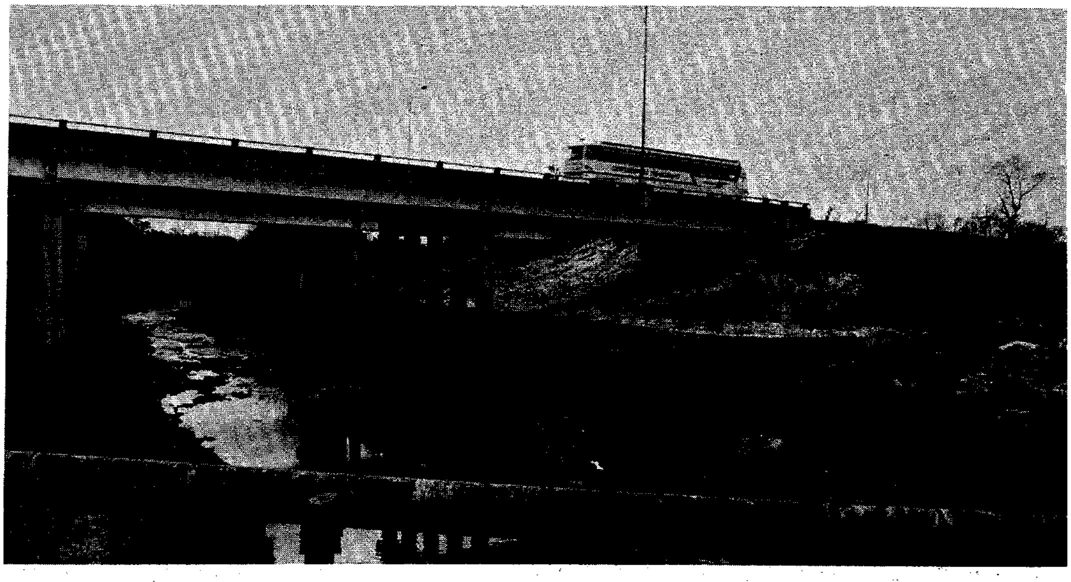


Figure 114. View of right abutment of I-12 bridge on February 18, 1977, showing steel sheet-piling driven to prevent slumping of fill-slope.

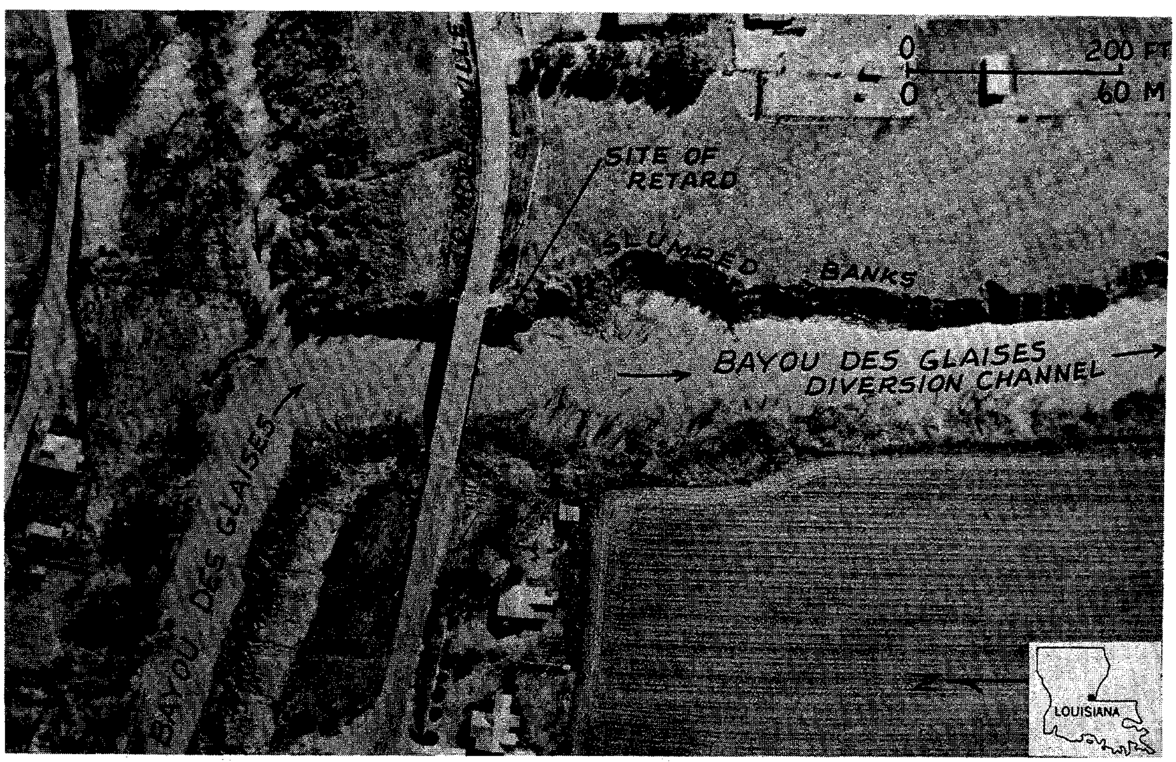


Figure 115. Aerial photograph showing Bayou Des Glaises diversion channel at SR-114 crossing, on March 11, 1971. (From Louisiana Dept. of Transportation.)

SITE 200. BAYOU DES GLAISES DIVERSION CHANNEL AT SR-114 AT MOREAUVILLE, LA.

Description of site: Lat 30°45', long 91°03', location as shown in fig. 115. Bridge, built about 1940, is 165 ft (50 m) in length, concrete deck, timber-pile bents, spillthrough abutments.

Drainage area, 270 mi^2 (699 km^2); valley slope, 0.0014; channel width, 140 ft (43 m). Stream is perennial, alluvial, silt-clay bed, in valley of low relief, wide flood plain. Channel is artificial, straight, right bank is stabilized by willow growth, but left bank is unstable and marked by almost continuous slumps (fig. 115).

Hydraulic problem and countermeasure: Slumping at left abutment and left bank near abutment; erosion progressing toward abutment foundation; effect on bridge use, potential only. Countermeasure is retard (or bulkhead) of timber on timber pile at toe of abutment fill-slope (fig. 116).

Discussion: Cause of left bank slumping not clear, but apparently due to lateral erosion at toe of bank, because internal hydrostatic forces would have affected both banks. As the bank on the downstream side of the left abutment has already been steepened by slumping, further slumping may not be halted by the retard.

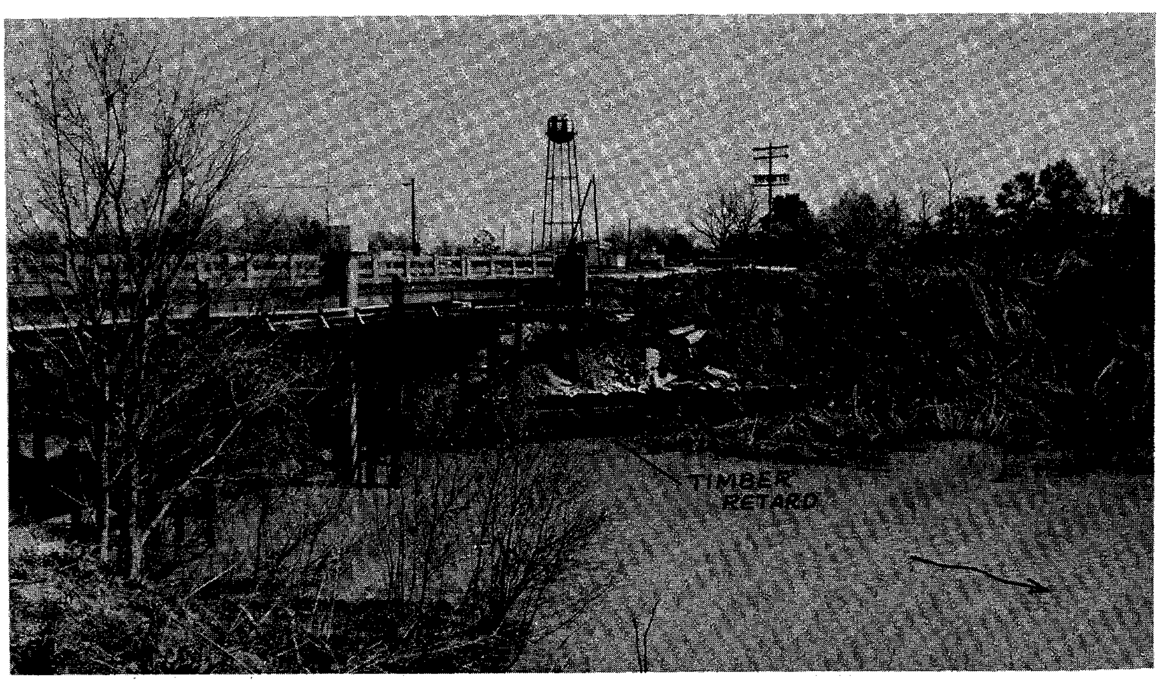


Figure 116. Upstream view of left abutment of SR-114 bridge on March 7, 1977, showing timber retard.

SITE 201. SHIPPEGAN SOUND AT PH-113 NEAR SHIPPEGAN, NEW BRUNSWICK, CANADA (LAMEQUE-SHIPPEGAN BRIDGE)

(All information given here is abstracted from an unpublished report prepared in 1969 by D. L. Sullivan for the Project Committee on Bridge Hydraulics, Roads and Transportation Association of Canada).

Description of site: Lat 47°44', long 64°40' (approximately), location as shown in fig. 117. Crossing is 4,160 ft (1,268 m) in length, of which the bridge is 637 ft (194 m) and the remainder is causeway. Bridge consists of eight spans, including a 65-ft (20 m) lift span. Four piers on the Shippegan (west) side of the bridge are set on sandstone bedrock (fig. 118). The causeway had a sand-gravel core, 1.5:1 side slopes protected by a 4-ft (1.1-m) thickness of sandstone riprap. Flat-faced sandstone blocks, each weighing about 20-50 lb (9-23 kg), placed by hand and machine to give a relatively smooth finish, formed the outside of the riprap blanket. No filter blanket was used, although the core of the causeway was constructed of erodible sand and fine gravel.

The main tidal channel between Shippegan Island the mainland has remained fixed in position at the bridge site, although it wanders at Shippegan Gully (fig. 117). The bridge and causeway reduced the total channel cross section by about 76 percent at mean tide level. The current in the bridge opening is in a southerly direction during flood tide and northerly during ebb tide. The northern side of the causeway is exposed to a fetch of about 30 mi (48 km) of open sea, but breakwaters at Shippegan Gully protect the southern side from severe wave action.

Hydraulic problem and countermeasures:

1958 Bridge construction began, by contractor for Lameque-Shippegan Bridge Corporation. Severe scouring reported by contractor, beginning where fill was being placed for east causeway, amounting to 12-15 ft (3.7-4.6 m) at causeway and about 6 ft (1.8 m) at pier 6.

1961 Settlement of about 0.5 ft (0.15 m) reported at piers 6 and 7 and also at west abutment. Soundings indicated scour to bedrock at points midway between piers 5 and 6, also between piers 6 and 7. Granite riprap was placed around these piers.

1963 Riprap protection on northern side of causeway eroded by wave action; 3,000 tons (2,721 t) of sandstone riprap, in size range of 0.5-1 yd^3 (0.4-0.75 m^3), was placed on causeway slope.

1964 Riprap on northwest side again damaged by waves, road surface was undermined. Additional riprap placed on slopes, but this also was eroded.

1968 Because of continued erosion of causeway, about 10,000 tons (9,070 t) of large sandstone riprap, in size range of 0.75-2 yd^3 (0.6-1.5 m^3), was dumped along the northern side of the causeway and at the abutments, forming a 2:1 slope.

Subsequently, on October 21, a severe storm occurred, accompanied by winds reaching 70 mi/hr (130 km/hr) and waves 8 ft (2.4 m) in height. The waves washed over the causeway on the northern side, at the Shippegan end, where large sandstone riprap had not been placed. The smaller, original riprap was eroded beyond repair and the causeway fill was eroded almost to the road centerline. On the Lameque side of the causeway, where the large riprap had been placed, the waves overtopped the riprap and eroded the causeway fill, also the fill at the western abutment, which was removed to a depth of 12 feet below the road surface.

Immediately after the storm, a contract was let for the dumping of large granite riprap, in the size range of 1 yd^3 (0.75 m^3) and larger, to a slope of about 4:1 along the northern side of the causeway. Soundings made with an echo sounder, after the storm, indicated that the channel bottom along the centerline of the bridge was scoured almost completely to bedrock (fig. 118).

Discussion: Erosion of the causeway is attributed to underestimation of wave action and storm surges, and the use of riprap of inadequate size. Lack of a filter blanket resulted in leaching and piping of causeway fill. Damage to the riprap was increased by inadequate toe protection.

Scour at the bridge waterway is attributed to constriction and consequent increase in velocity of tidal flow. Maximum average velocities, measured during a tidal cycle of slightly larger than normal tides, were 2.8 ft/s (0.9 m/s) between piers 5 and 6, and 3.3 ft/s (1 m/s) between piers 6 and 7. The tractive force on the channel bed in this velocity range was calculated at about 0.07 lb/ft^2 (0.003 kg/m^2), which is greater than the permissible tractive force for sand. Additional factors contributing to scour are: turbulence around the piers, velocity increases during high tides and storm surges, and turbulence created by large waves. Analysis of all these factors would probably have indicated that allowance for scour to bedrock should have been made in the bridge design.

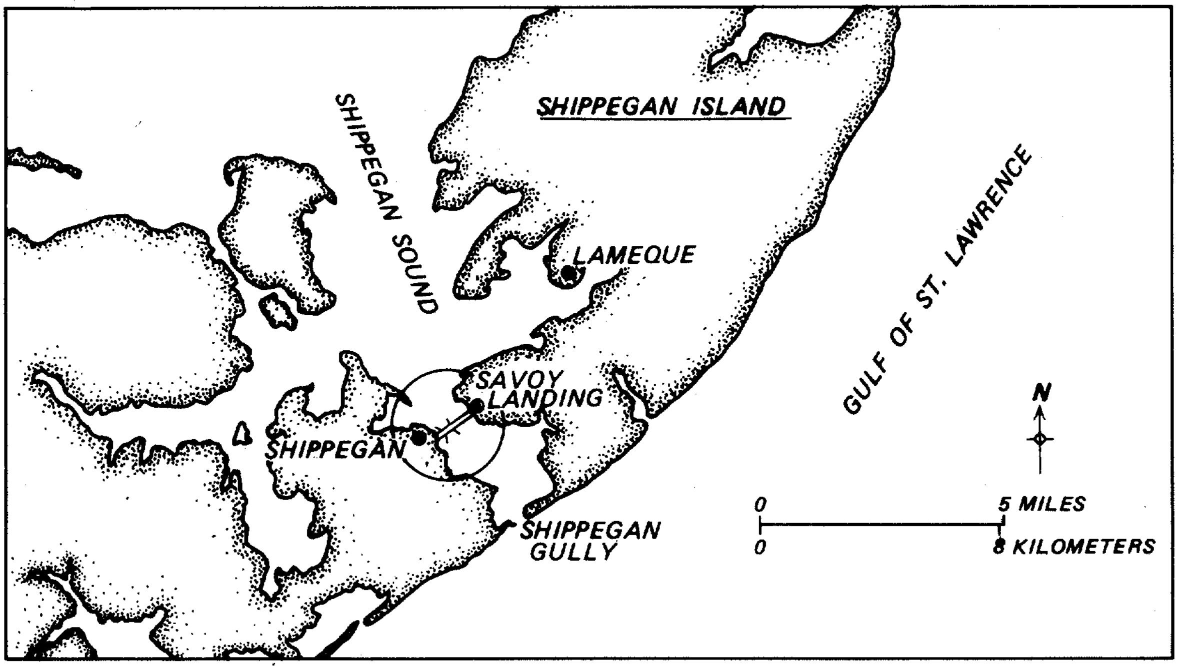


Figure 117. Sketch map showing location of Lameque-Shippegan bridge, New Brunswick, Canada.

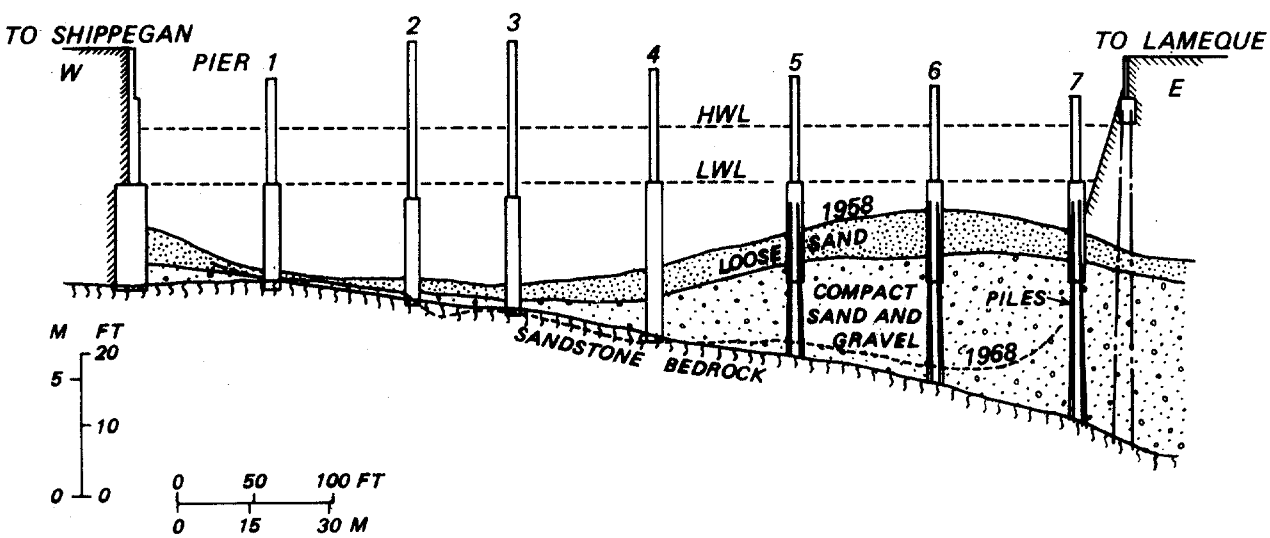


Figure 118. Cross section showing scour at Lameque-Shippegan bridge.

SITE 202. SABINE PASS AT LOUISIANA SR-82 NEAR PORT ARTHUR, TEX.

Description of site: Lat 29°46', long 93°54', location as shown in fig. 119. Bridge is 2000 ft (610 m) in length, vertical (full height) concrete abutments. Crossing at estuary, where subject to wave and tidal action.

Hydraulic problem and countermeasure: Lateral erosion at left (east) abutment and adjacent approach embankment, attributed to wave and tidal action; effect on bridge use, potential only. Countermeasure consists of flat precast concrete blocks stacked at abutment (fig. 120).

Discussion: Erosion at left abutment, on Louisiana side, is attributed to exposed position at tip of a peninsula. Concrete blocks have performed satisfactorily in immediate area of abutment, but additional protection is needed along approach embankment adjacent to abutment.

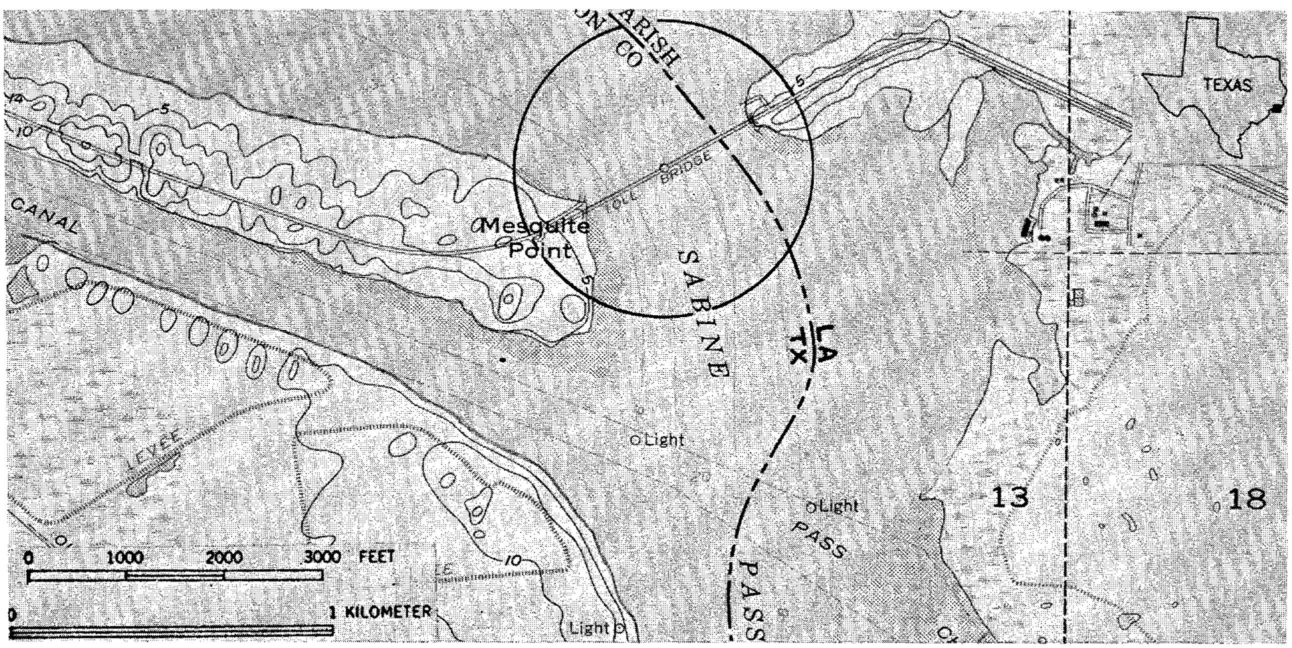


Figure 119. Map showing Sabine Pass at LA SR-82. (Base from U.S. Geol. Survey Port Arthur South, Tex.-La., 7.5' quadrangle, contour interval 5 feet, photorevised 1970.)

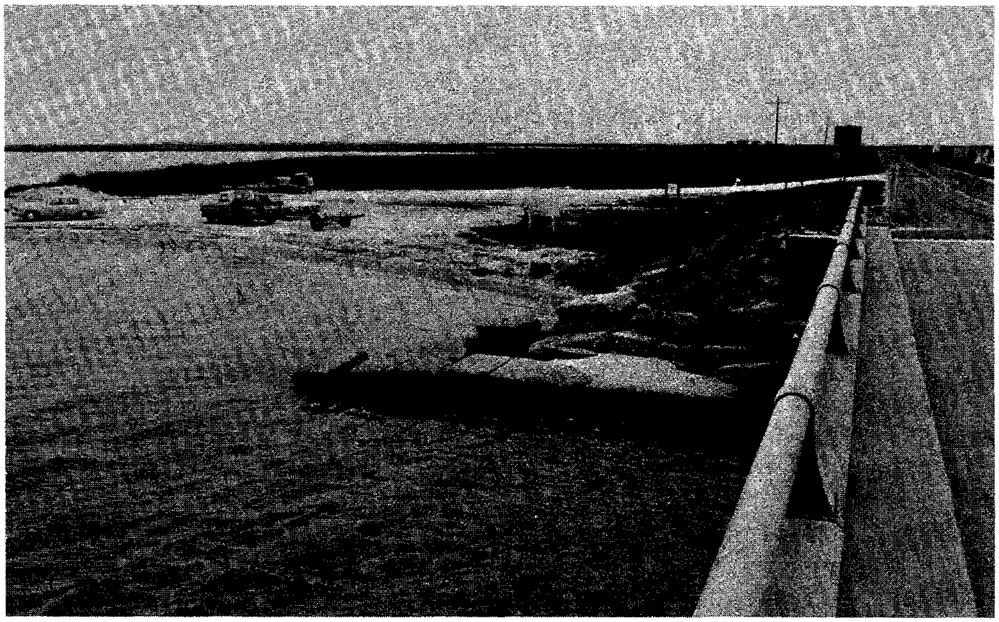


Figure 120. View of upstream side of left bridge abutment, Sabine Pass at SR-82, on March 9, 1977, showing erosion behind concrete blocks.

SITE 203. ATCHAFALAYA RIVER (WHISKY BAY PILOT CHANNEL) AT I-10 NEAR RAMAH, LA.

Description of site: Lat 30°22', long 91°38', location as shown in fig. 121. Prior to bridge construction, Whisky Bay Pilot Channel was cut to divert the flow of the Atchafalaya River into a new and straighter course. Width of channel at time of bridge construction was about 700 ft (210 m) and the thalweg depth was about 85 ft (25 m) below bank elevation. About 1970, rectangular willow mattresses 400 ft (120 m) in length and 200 ft (60 m) in width were sunk to the channel bottom at each of the two main piers (piers 5 and 6) as a countermeasure against scour.

Hydraulic problem: During bridge construction, in 1969, barges were tied up on either side of the channel (fig. 122), and these may have diverted flow toward the center of the channel. For this or some other reason, a deep scour pool formed downstream from the bridge and gradually deepened as it migrated upstream toward the bridge (figs. 123 and 124). In March of 1975, the bottom of the pool was at elevation 156 ft (47 m) below sea level, or slightly below the pile tip of pier 5; but by September, the pool had filled so that its bottom was about 20 ft (6 m) above the pile tip. Subsequently, the upstream migration of the pool appears to have halted, perhaps because of the willow mattress at the main piers.

Discussion: The cause of the scour pool is not known; it may have no connection with bridge construction. Deep pools are known to occur in the Atchafalaya River, and the pool in the Whisky Bay Pilot Channel may represent formation of a pool-and-riffle sequence in the straight artificial channel. The pilot channel has not widened as expected, and a possible countermeasure to inhibit further development of the pool is to widen the channel. No countermeasures were planned (in 1976) by the Louisiana Department of Transportation.

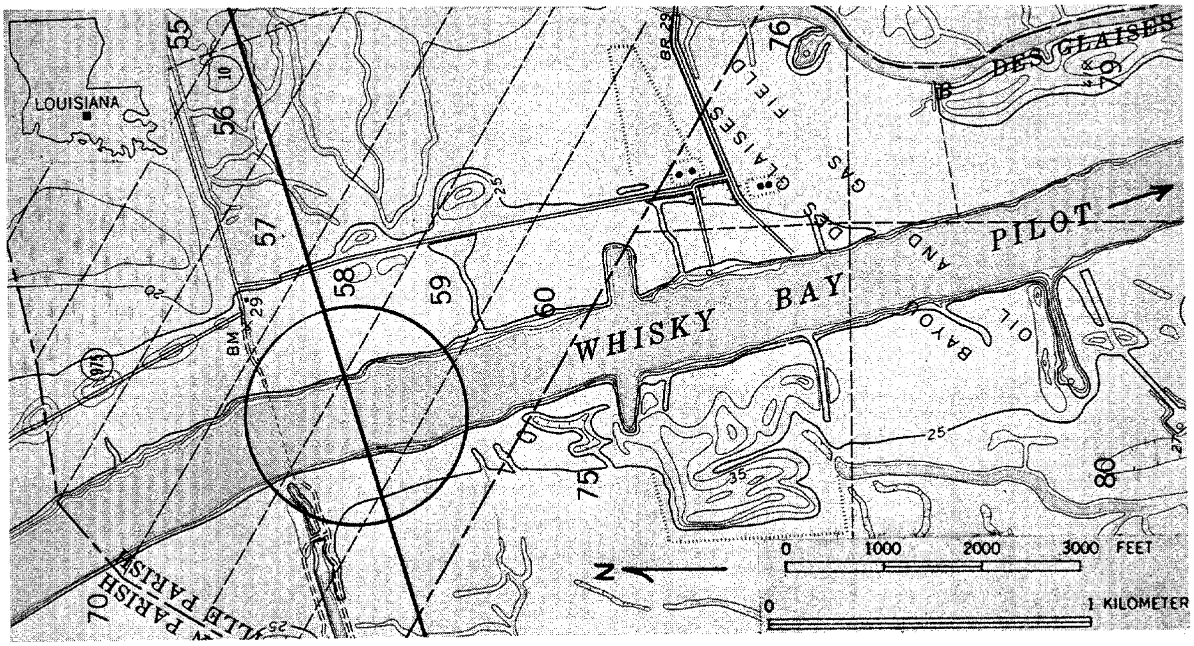


Figure 121. Map showing Whisky Pay pilot channel at I-10 crossing (circled). (Base from U.S. Geol. Survey Butte La Rose, La., 7.5' quadrangle, contour interval 5 feet, 1968.)

Figure 122. Aerial photograph of I-10 crossing site on January 7, 1969, during construction of main piers. (From Louisiana Dept. of Transportation.)

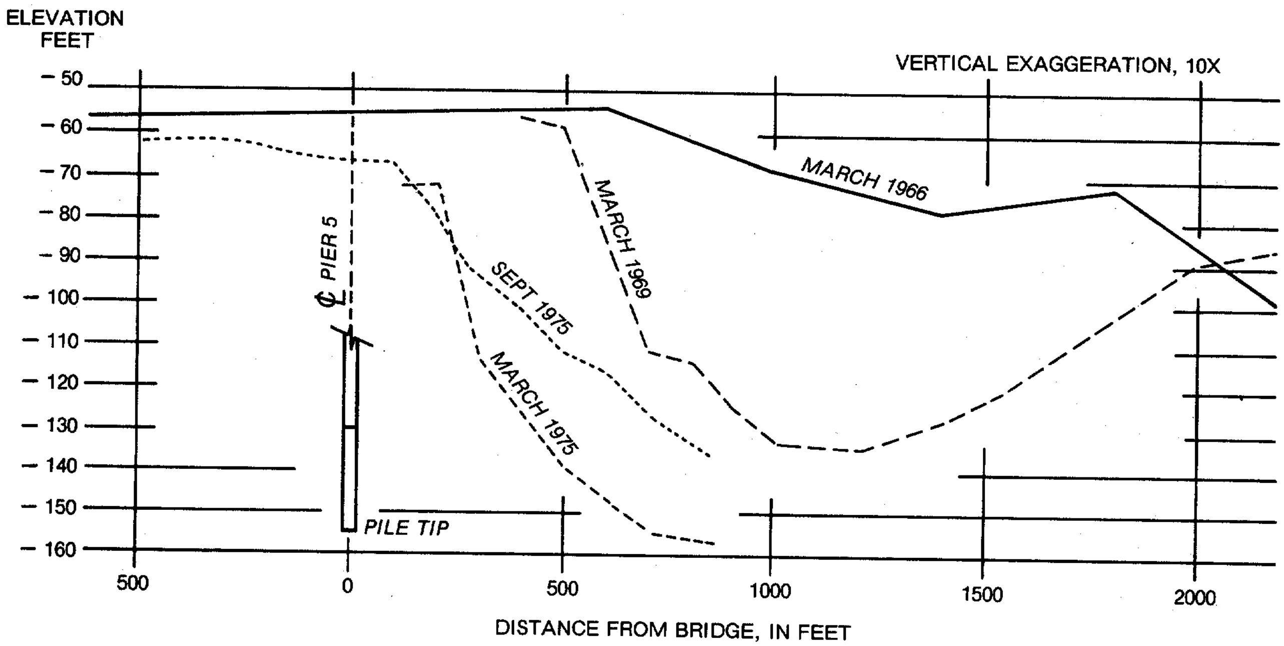


Figure 123. Longitudinal profiles of channel thalweg downstream from I-10 bridge.

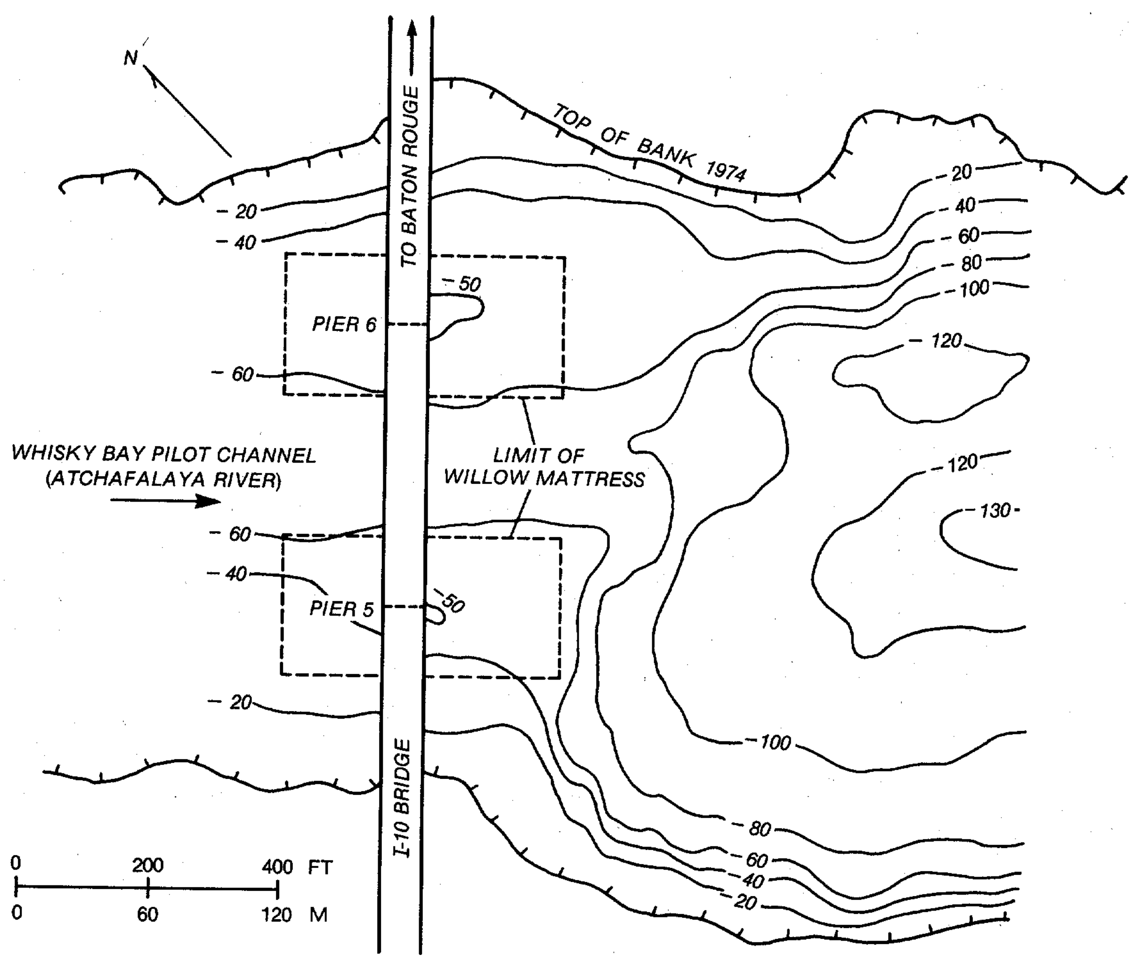


Figure 124. Contour map showing scour pool downstream from I-10 bridge, from soundings made April 23, 1974.

SITE 204. MINNESOTA RIVER AT US-169 AT LE SUEUR, MINN.

Description of site: Lat 44°29', long 93°54' location as shown in fig. 125. Dual bridges 1100 ft (330 m) in length, each of the dual bridges has 9 hammerhead-type concrete piers with single round column, timber-pile foundations. Spillthrough abutments, upper part of abutment fill-slopes protected by hand-placed riprap, lower part by dumped rock riprap. Crossing is skewed relative to flood plain and also eccentric, with river near east side of flood plain.

Drainage area, 15,300 mi^2 (39,625 km^2); bankfull discharge, 20,000 ft^3/s (566 m^3/s); valley slope, 0.00024; width, 250 ft (75 m). Stream is perennial, alluvial, bed of fine gravel, in valley of moderate relief, wide flood plain. Channel is meandering, wider at bends, with point bars, not incised, banks of silt and sand.

Hydraulic problem and countermeasure:

1958 Bridge built, design flood R.I. of 50 yr; extreme high water elevation for design purposes, 739.5 ft; countermeasures as described above.

1965 During a flood of R.I. about 100 yr, peak discharge 106,000 ft^3/s (3,000 m^3/s) overbank flow was diverted along the left (west) approach embankment into the main channel at the bridge, where it produced an eddy that scoured to a depth of 13 ft (4 m) below the footing of pier 2 (fig. 126). High water elevation was 744 ft at the bridge, and mean velocity beneath the bridge was about 6.4 ft/s (2 m/s). As a countermeasure, a revetted earth embankment spur dike, elliptical in shape, was constructed at the left bank late in 1965 (fig. 127). Length of spur dike is 170 ft (51 m), crest width 12 ft (3.6 m), side slope ratio 2:1, revetment of 18-in (46-cm) dumped rock riprap over a 6-in (15-cm) granular filter blanket.

1969 During a flood of R.I. about 50 yr, peak discharge 80,600 ft^3/s (2,281 m^3/s), overbank flow diverted along the left approach embankment eroded the embankment fill and removed the end of the spur dike. Turbulence and eddy currents around the spur dike scoured a hole having a maximum depth of about 22 ft (7 m). As a countermeasure, 5,000 yd^3 (3,800 m^3) of dumped rock riprap was placed along the roadway embankment, and the end of the spur dike was replaced with dumped rock riprap.

1977 Countermeasures applied in 1969 not tested by major flood.

Discussion: Diversion of flow along west approach embankment is increased both by the downstream trend of the embankment and by the eccentricity of the crossing. During the 1969 flood, the spur dike served to transfer the point of serious scour from pier 2 to the tip of the dike, but it did not prevent erosion of the approach embankment.

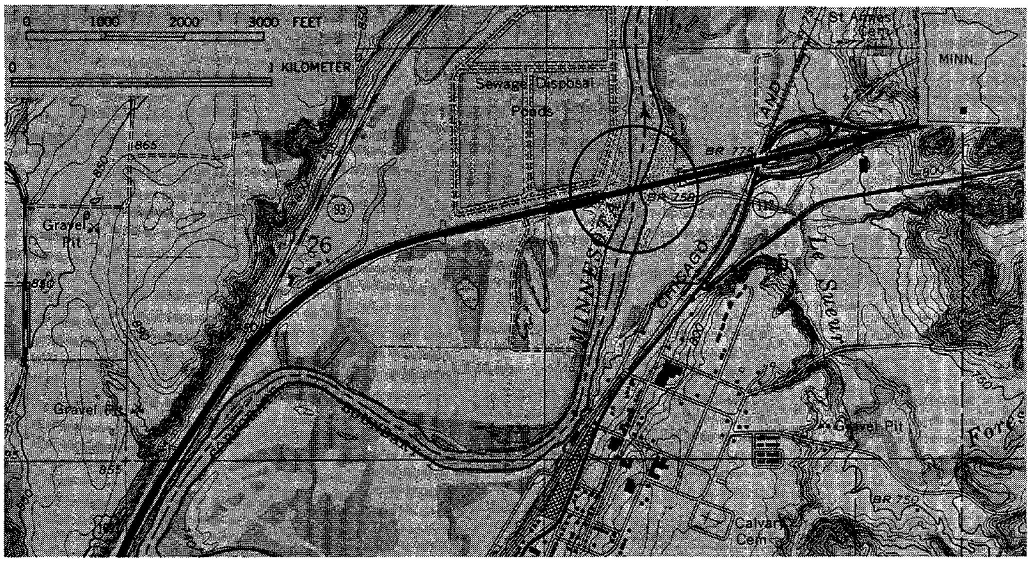


Figure 125. Map showing Minnesota River crossing at US-169 (circled). Base from U.S. Geol. Survey Le Sueur, Minn., 7.5' quadrangle, contour interval 10 feet, 1965.)

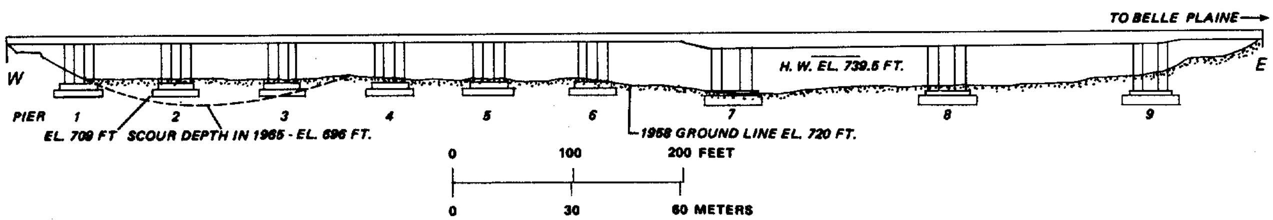


Figure 126. Elevation sketch of US-169 bridge, showing scour depth as measured after the 1965 flood.

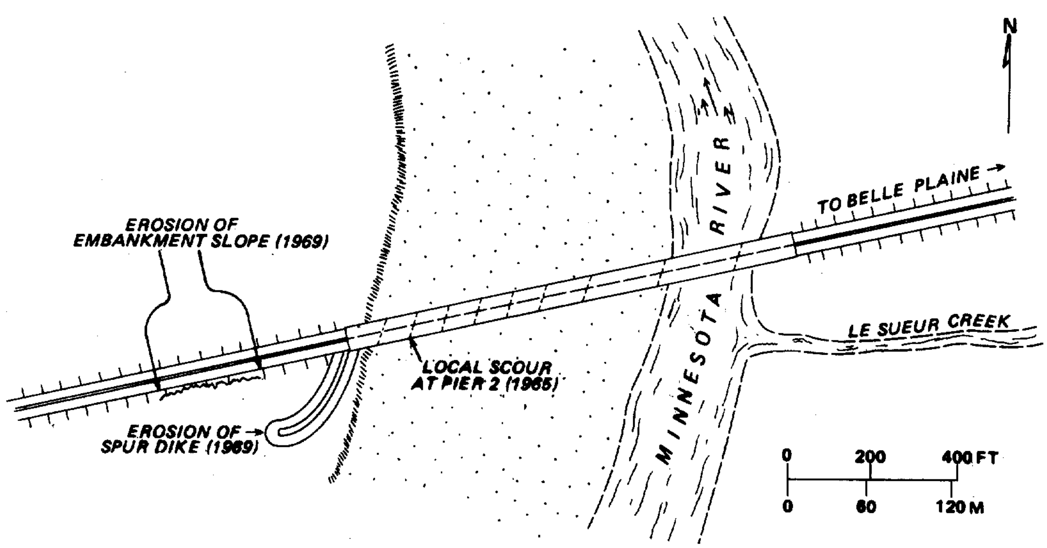


Figure 127. Plan sketch of Minnesota River at US-169.

SITE 205. MISSISSIPPI RIVER AT I-494 AT SOUTH ST. PAUL, MINN.

Description of site: Lat 44°53', long 93°01', location as shown in fig. 128. Bridge is 1,870 ft (570 m) in length, one 421-ft (128-m) overhead-truss arch span, seven girder spans, three beam spans ranging in length from 69 to 231 ft (21 to 70 m). Ten multiple-column concrete piers, each with four square concrete columns (fig. 129). Piers in channel have concrete footings, 87 ft (26.5 m) in length, and timber-pile foundations, pile driven to depth of 40 to 50 ft (12 to 15 m) below streambed elevation at time of construction (fig. 130). Piers 8, 9, and 10 at left bank are founded on bedrock.

Drainage area, 36,800 mi^2 (95,000 km^2); bankfull discharge, about 41,000 ft^3/s (1,160 m^3/s); valley slope, 0.0001; channel width, about 1,000 ft (300 m). Stream is perennial, regulated (bridge site is in pool of downstream dam), semi-alluvial, sand-silt bed, in valley of moderate relief, narrow flood plain. Channel is sinuous, locally braided, silt-sand banks.

Hydraulic problem and countermeasure:

1959 Bridge built, concrete slope pavement placed at abutment fill-slopes.

1960-75 A large volume of material was removed by dredging in the stream channel for a distance of about 3 mi (4.8 km) upstream from the bridge, for the purpose of building flood control dikes and a major barge terminal. In addition, spurs were built along the left bank upstream from the bridge to deflect the flow toward the center of the channel and thus deepen it for barge traffic. The streambed at the bridge was lowered 6-10 ft (2-3 m) by degradation (fig. 130), and local scour reached elevation 646 ft at an upstream (northwest) corner of pier 7, 10 ft (3 m) below the base of the concrete seal. Soil borings indicate that the scoured material was fine-to-medium sand and organic silty clay.

1976 Revetment was placed over an area 205 ft (62.5 m) in length and 162 ft (49 m) in width at pier 7. After the scoured area beneath the concrete seal (fig. 131) was grouted, the scour hole was filled with granular material to serve as a filter blanket, the surface of which was levelled and topped with "Poly-Filter X Cloth". The filter cloth, in overlapping sheets 36 ft (11 m) in width, was sunk and held in place by I-beam sections chained together and secured, on the upstream side, by cables around tubular piling. A second granular filter blanket, about 0.5 ft (0.15 m) thick, was placed over the filter cloth, and dumped rock riprap (Minnesota Class "C", approximately equivalent to Class II) was placed on the filter blanket. Minimum thickness of the riprap blanket was 2.5 ft (0.75 m) at the center, and the thickness was increased around the edges to hold the filter cloth in place.

Discussion: Because of recency of installation, the performance of the revetment at pier 7 cannot be evaluated. The scour damage at pier 7 is clearly assignable to local scour superimposed on channel degradation, which resulted from dredging and the effects of upstream spurs.

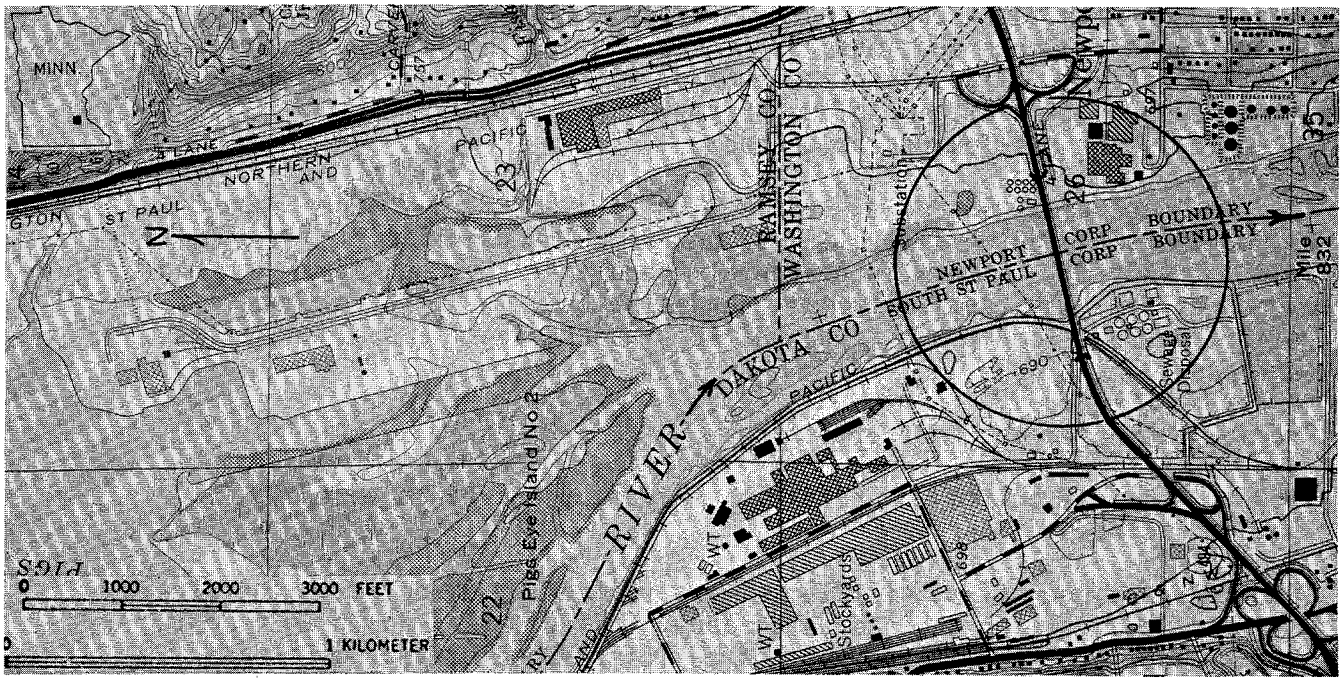


Figure 128. Map showing Mississippi River at I-494 crossing (circled). (Base from U.S. Geol. Survey St. Paul East, Minn., 7.5' quadrangle, contour interval 10 feet, 1967 photorevised 1972.)

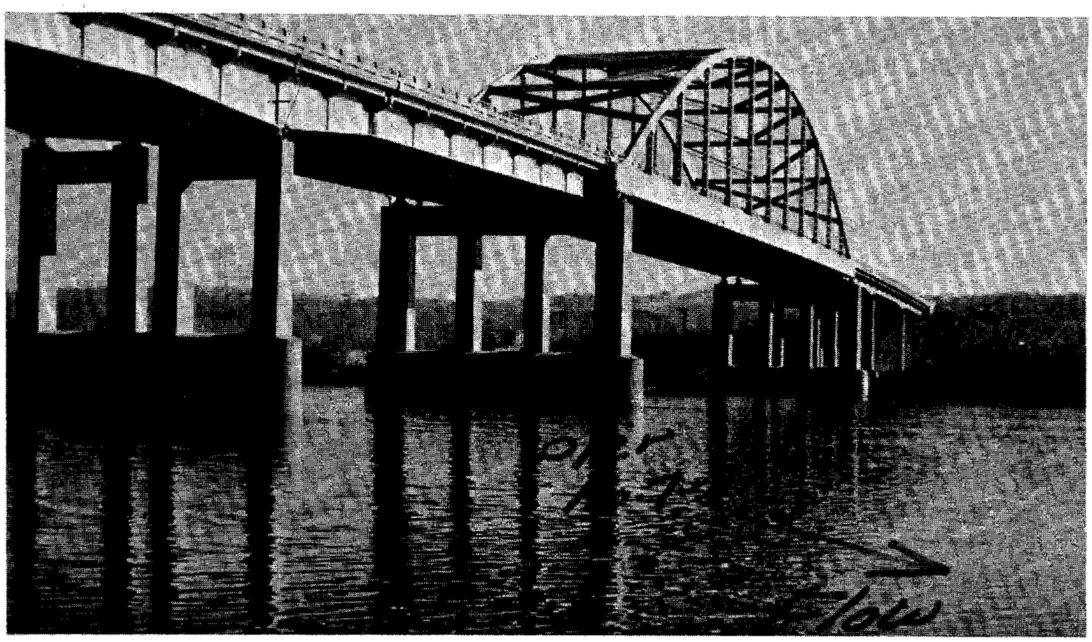


Figure 129. View upstream and toward left bank of I-494 bridge in January 1977. Pier 7 is at center.

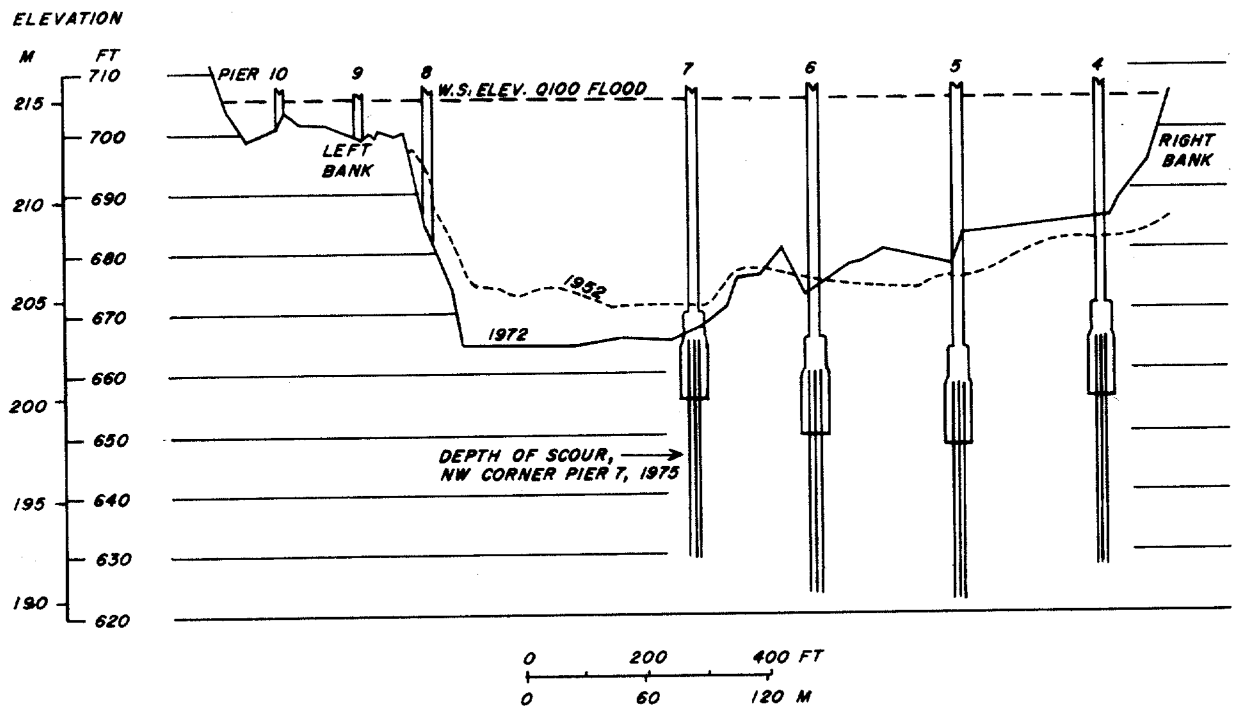


Figure 130. Channel cross section at I-494 bridge, showing channel degradation and depth of scour at pier 7.

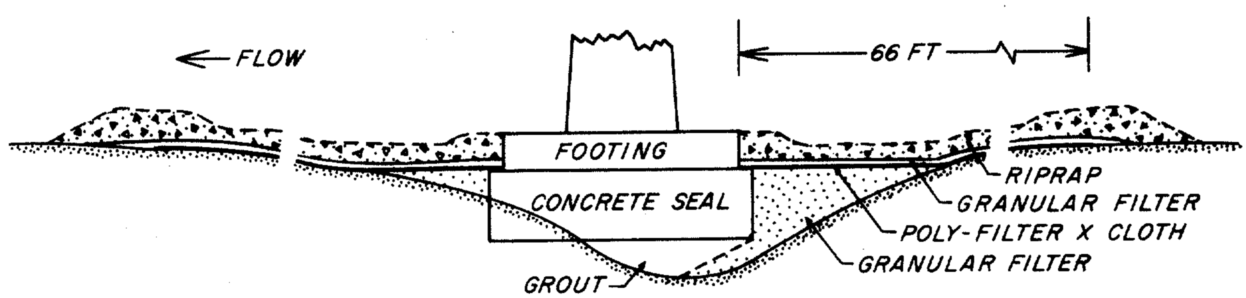


Figure 131. Cross section showing placement of revetment at pier 7.

SITE 206. HOBOLOCHITTO CREEK AT SR-26 NEAR POPULARVILLE, MISS.

Description of site: Lat 30°48', long 89°39'. Bridge is 440 ft (134 m) in length, 80-ft (24-m) steel span over main channel, reinforced-concrete hexagonal piers, precast-concrete pile bents, spillthrough abutments protected with precast-concrete slope paving. Crossing is eccentric, with greater width of flood plain on right side of bridge.

Drainage area, 92 mi^2 (238 km^2); valley slope, 0.00125; channel width, 70 ft (21 m). Stream is perennial, alluvial, wide flood plain. Channel is highly meandering, equiwidth, tree cover on banks greater than 90 percent.

Hydraulic problem and countermeasure:

1947 Bridge built, countermeasures as described above.

1961 Three extreme floods, which scoured the bridge waterway beneath the right approach spans and damaged the abutment slope paving. Velocity measurements showed a large eddy near the right abutment.

1962 Elliptical revetted earth embankment spur dike, 150 ft (45 m) in length, built upstream from right abutment.

1972 Velocity measurements during overbank flows indicate that the eddy has been eliminated by the spur dike, but dike has not yet been tested by extreme floods.

SITE 207. LEAF RIVER AT US-98 AT McLAIN, MISS.

Description of site: Lat 31°06', long 88°, location as shown in fig. 132. Bridge is 2,175 ft (663 m) in length, 400-ft (120-m) cantilevered main span has four wall-type concrete piers on timber-pile foundations. Crossing is at slight bend in stream, but piers are approximately alined with flood flow.

Drainage area, 3,510 mi^2 (9,091 km^2); bankfull discharge, about 21,000 ft^3/s (594 m^3/s); channel width, 350 ft (106 m). Stream is perennial, alluvial, sand bed, in valley of moderate relief, wide flood plain. Channel is meandering, somewhat wider at bends, narrow point bars, cut banks local, silt-clay banks, tree cover along 90-100 percent of bankline (fig. 133).

Hydraulic problem and countermeasures:

1940 Bridge built, main river channel was between center piers (fig. 134).

1941-51 Fourteen floods, largest of which was 5-yr R.I. Main channel shifted to west (toward right bank), such that deepest part was no longer between center piers (figs. 134 and 135). If shift continued, foundations of piers 3 and 4, and adjacent pile bents, were subject to undermining.

1952 Two steel H-pile spurs were constructed at west bank, and dumped rock riprap was placed around piling and at right (west) bank beneath bridge (fig. 135). Steel pile (type 8 BP 36) were used instead of timber because a stratum of hardpan clay prevented sufficient penetration of timber pile. The rows of steel pile were spaced 10 ft (3 m) apart and the pile, which range in length from 40-55 ft (12-16 m) were driven to depths of 20-35 ft (6-10 m). The length of the upstream spur is 170 ft (52 m) and the downstream spur, 100 ft (30 m). Top of the spurs is about 2 ft (0.6 m) above the top of the banks. Horizontal timber sheathing is bolted to the upstream face of the spurs, ending about 2 ft or 0.6 m above low water elevation; and willow poles about 4 in (10 cm) in diameter are attached to the downstream face and the ends are embedded to a depth of about 1 ft (0.3 m) in the streambed.

1953-76 The spurs have successfully returned the main channel to a position between the main piers (fig. 134). When inspected in 1976, the spurs (fig. 136) were found to be in good condition, except that the willow poles originally placed on the downstream face are missing. Flood events during this period include a peak discharge of 128,000 ft^3/s (3,622 m^3/s) in 1961 and 110,000 ft^3/s (3,113 m^3/s) in 1974. The spurs were submerged to a depth of about 8 ft (2.4 m) during the 1961 flood.

Discussion: The banks of the Leaf River in the general vicinity of the site are densely forested, and the absence of wide point bars indicates that the lateral migration rate tends to be slow, although fast enough to cause trouble at the bridge. It is perhaps surprising that a bend so slight as the one at the crossing would cause lateral migration, but the Leaf is not exceptional in this respect. The steel spurs are very well constructed and have performed well.

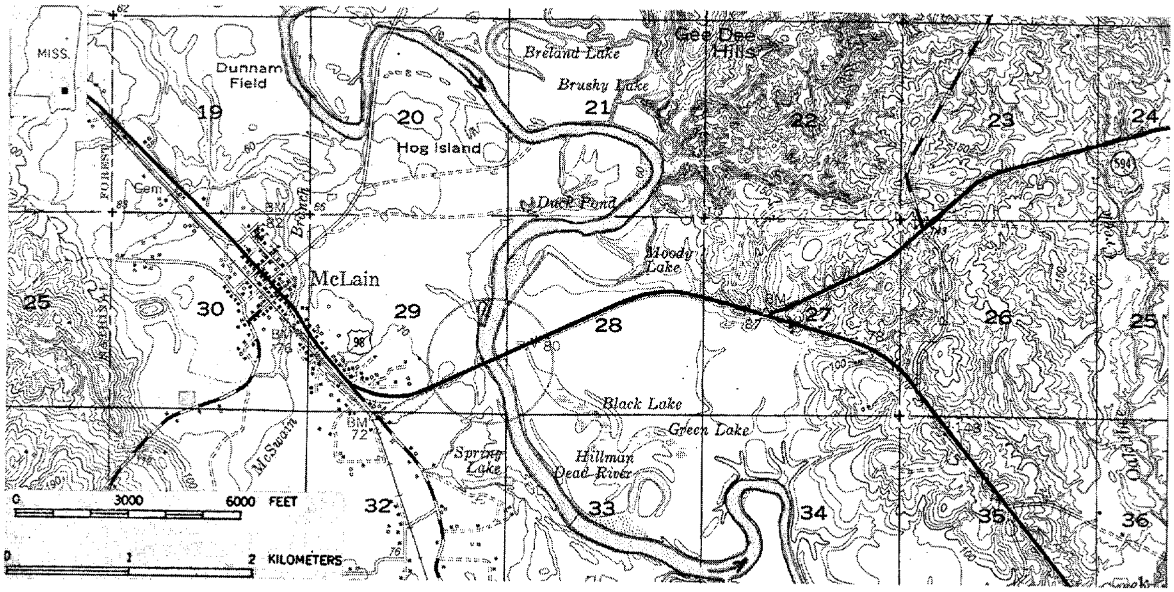


Figure 132. Map showing Leaf River at US-98 crossing (circled). (Base from U.S. Geol. Survey Beaumont, Miss., 15' quadrangle, contour interval 10 feet, 1943.)

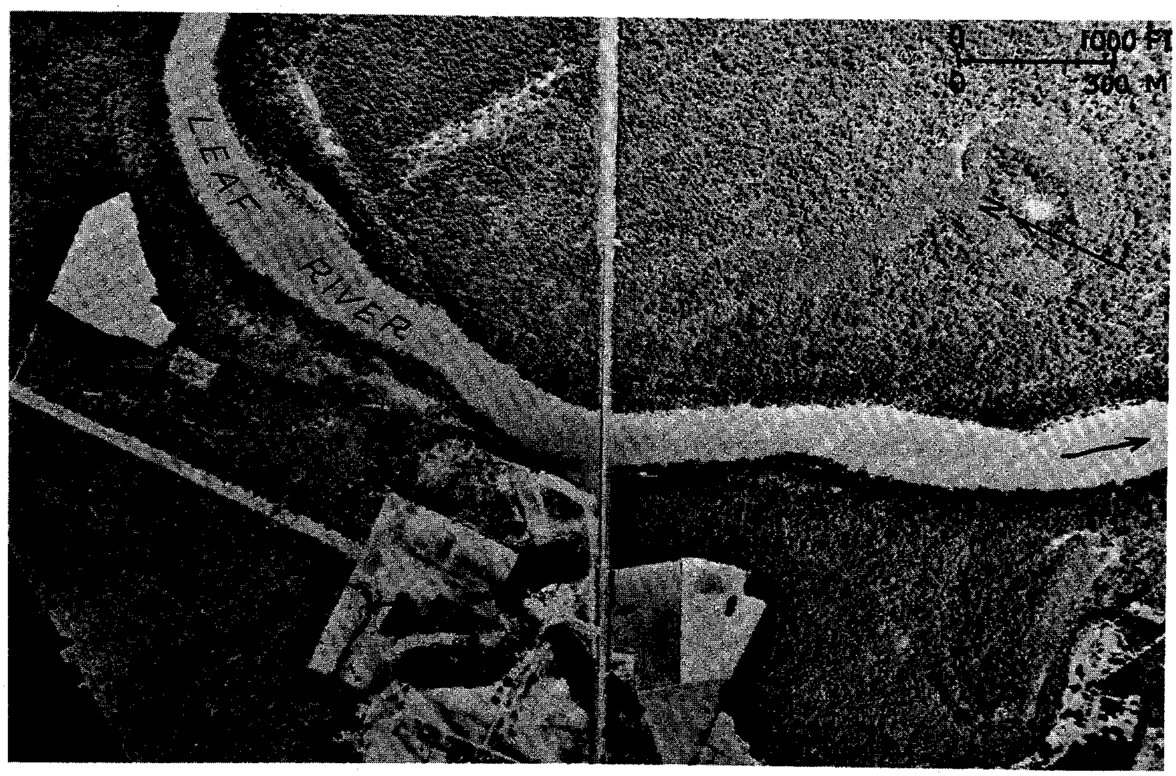


Figure 133. Aerial photograph of Leaf River at US-98 crossing, on April 29, 1973. (From Mississippi Dept. of Highways.)

SITE 245. ROCK CREEK AT US-77 NEAR CERESCO, NEBR.

Description of site: Lat 41°02.5', long 96°38', location as shown in fig. 224. Bridge is 93 ft (28 m) in length, two concrete wall-type piers in channel, timber-pile foundations beneath piers, spillthrough abutments, steel H-pile foundation beneath abutments.

Drainage area, 51 mi^2 (132 km^2); valley slope, 0.0017; channel width, 100 ft (30 m). Stream is perennial but flashy, alluvial, silt-clay bed, in valley of moderate relief, wide flood plain. Channel is meandering, wider at bends, not incised, silt-clay banks, no tree cover along banks in vicinity of bridge.

Hydraulic problem and countermeasure:

1933 Bridge built. As a countermeasure for possible scour or degradation, the abutment headwalls were extended to a depth of 16 ft (5 m) below road elevation and founded on 40-ft (12-m) steel pile (fig. 225). Similarly, the pier foundations are unusually deep for a stream of this size.

1964 Severe flood, R.I. estimated at about 100 yr from flow records on nearby stream. Abutment fill-slopes eroded, piling at abutments exposed. Scour depth not measured, but 24 ft (7 m) of scour was measured at gaging station on nearby stream (Wahoo Creek at Ithaca). As a countermeasure, 6-ft (2-m) lengths of steel sheet-piling were driven at the base of the abutment headwall. Also, the wingwalls were extended with timber-pile retards (or bulkheads), backfilled with broken concrete riprap.

1977 Countermeasures not tested by severe flood. Thalweg has shifted toward right end of bridge (fig. 226), retards have tilted, sheet-pile bulkhead not visible.

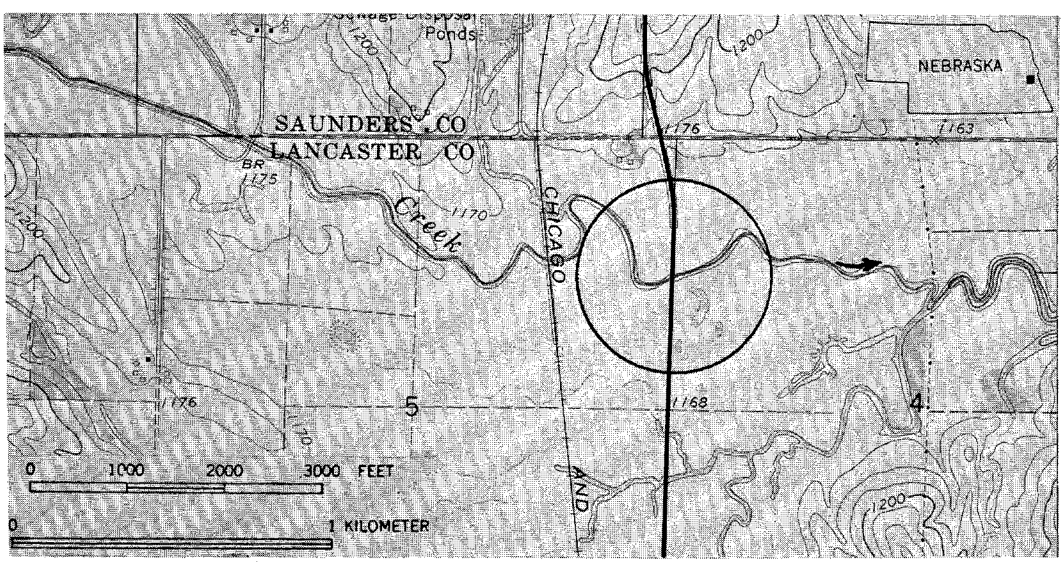


Figure 224. Map showing Rock Creek at US-77 crossing. (Base from U.S. Geol. Survey Ceresco, Nebr., 7.5' quadrangle, contour interval 10 feet, 1968.

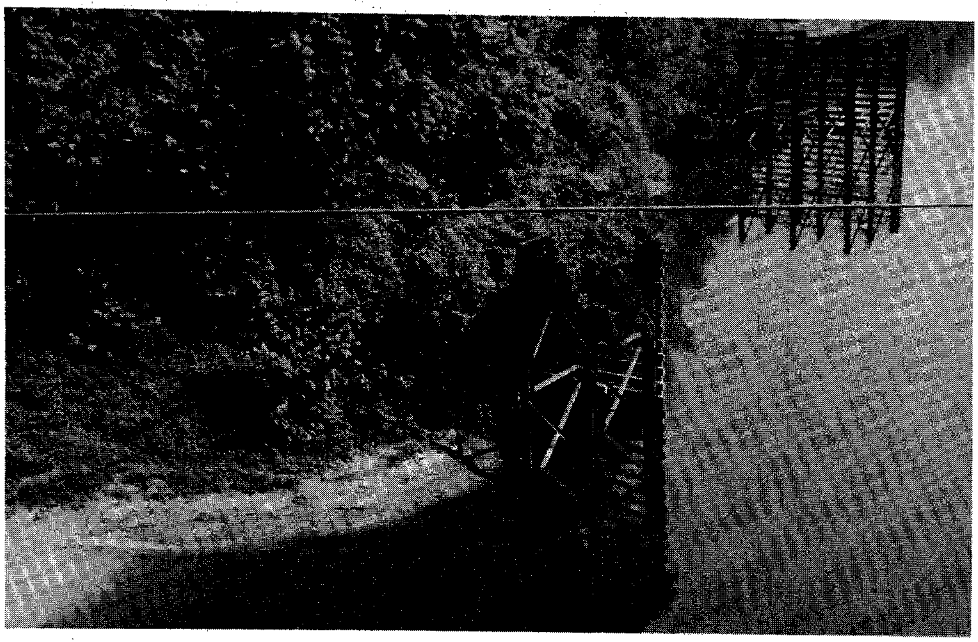


Figure 136. Upstream view of steel spurs, on June 21, 1976.

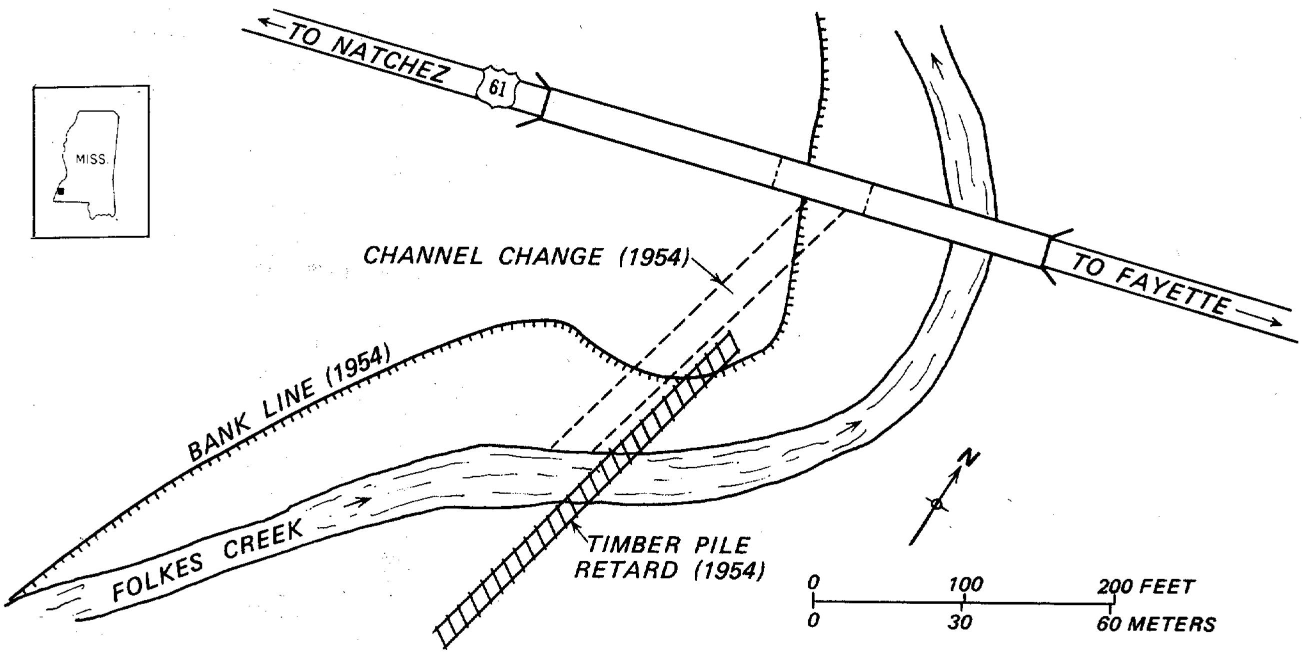


Figure 137. Plan sketch showing channel change and retard as constructed in 1954.

SITE 208. FOLKES CREEK AT US-61 NEAR FAYETTE, MISS.

Description of site: Lat $31^{\circ}39'$, long $91^{\circ}10'$, location as shown in fig. 137. Bridge is 340 ft (102 m) in length, steel I-beam spans on timber-pile bents, spillthrough abutments. Crossing at bend in stream.

Drainage area, about 25 mi^2 (65 km^2); valley slope, 0.0038; channel width, 100 ft (30 m). Stream is perennial, alluvial, in valley of moderate relief, wide flood plain. Channel is sinuous, somewhat wider at bends, point bars of moderate width, probably incised, cut banks local, tree cover on 50-90 percent of bankline.

Hydraulic problem and countermeasure:

1937 Bridge built, no countermeasures on record.

1938-50 Lateral erosion of right bank upstream from bridge reported in 1950, channel migrating toward right abutment.

1954 Channel alinement changed (fig. 137) and retard constructed of double row of timber pile at right bank of new channel.

1976 Field inspection and 1973 aerial photograph (fig. 138) indicate good performance of retard. Although there are no flow records, flood marks at bridge indicate that overbank flows have occurred, which would have overtopped the retard by 2-4 ft (0.6-1.2 m).

Discussion: Good performance of retard is attributed in part to the fact that the channel is incised and the lateral migration rate is slow. If, for example, the large meander loop upstream from the bridge were migrating rapidly, the consequent change in flow alinement at the retard would affect its performance.

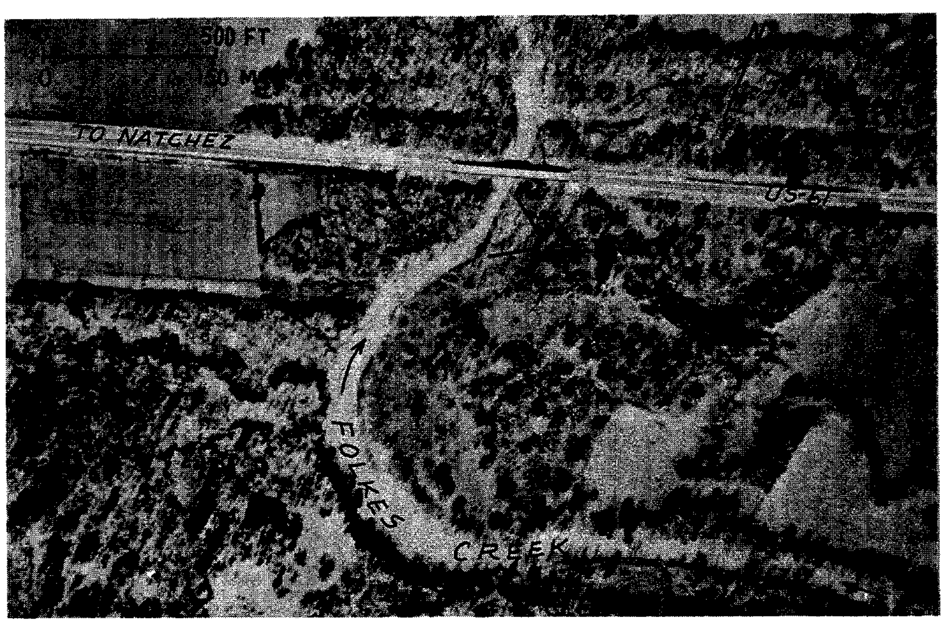


Figure 138. Aerial photograph of Folkes Creek at US-61 on February 10, 1973. (From Mississippi Dept. of Highways.)

SITE 209. LEAF RIVER AT SR-28 AT TAYLORSVILLE, MISS.

Description of site: Lat 31°50', long 89°24'. Bridge replaced in 1970 was 594 ft (181 m) in length, steel truss, round concrete piers in channel, timber-pile bents at approach spans. Bridge built in 1970 is 462 ft (141 m) in length, 2-column concrete piers in channel and pile bents each consisting of ten 18-in (48-cm) square prestressed-concrete pile. Crossing is at bend in channel, but erosion problem is at inside of bend.

Drainage area, 466 mi^2 (1,207 km^2); bankfull discharge, about 13,000 ft^3/s (368 m^3/s); channel width, 275 ft (84 m); valley slope, 0.00066. Stream is perennial, alluvial, sand bed, in valley of moderate relief, wide flood plain. Channel is meandering, somewhat wider at bends, narrow point bars, somewhat incised, cut banks local, silt-sand banks, tree cover on more than 90 percent of bankline (fig. 139).

Hydraulic problem and countermeasure:

1900 Approximate date of construction of old bridge.

1901-60 No specific record of hydraulic problem, but timber-pile spur was built at left bank, evidently because of lateral erosion at pile bents. Erosion is attributed to flood flow that cut across neck of upstream meander loop (fig. 140) and is confined to bridge waterway. Spur was straight, about 100 ft (30 m) in length, and consisted of a double row of timber pile with heavy timber sheathing on upstream face (fig. 141).

1961 Flood, R.I. about 50 yr, which reached an elevation of 256 ft at downstream side of bridge, mean velocity in bridge waterway, about 3 ft/s (0.9 m/s). No erosion problem reported at bridge, therefore performance of spur is considered good.

1970 New bridge built about 50 ft (15 m) upstream from old bridge. Timber spur removed for purposes of construction. Original plan called for new bridge to be built downstream, but analysis by U.S. Geological Survey indicated that left (east) bank at that downstream point was subject to erosion because of eddy action. A spur dike was built at left bank abutment.

1974 Flood, R. I. about 100 yr, high water elevation 257.5 ft. Rapid erosion of bank at bent 2 (fig. 142) was noted at rising stage of flood.

1975 At least 90 ft (27 m) of bank recession noted since 1974 observation. For piles at bent 2, only 8 ft (2.4 m) of penetration remained and bank had receded to position of bent 4 (figs. 142 and 143). Riprap was placed at bent 2, and construction was begun on a curved spur (or retard) of steel H-pile, faced with heavy sheathing, at the east bank (fig. 144).

1976 Flood, R. I. 2 yr, crest elevation 245.5 ft, occurred before completion of steel spur. Piling had been driven but sheathing was not yet in place. Water-discharge measurement made near flood crest showed that water velocities near the left (east) bank were only about 3 ft/s (0.9 m/s) and that flow was not concentrated at the east end of the bridge. Lack of flow concentration there was attributed to effects of the partially completed spur.

Discussion: The Leaf River has evidently become somewhat incised, and is developing a new flood plain at the elevation of the "low bank" in fig. 140, although the older flood plain is still subject to inundation during major floods. At the bridge contraction, the banks of the new flood plain are subject to excessive lateral erosion, although the river banks in most places appear to be stable. Both the old timber spur and the new steel spur have evidently been effective in protecting the projecting left bankline at the bridge. Under most circumstances, a single spur is not likely to be effective in protecting a bank.

Figure 139. Aerial photograph of Leaf River at old SR-28 bridge, on January 16, 1966. (From U.S. Dept. of Agriculture.)

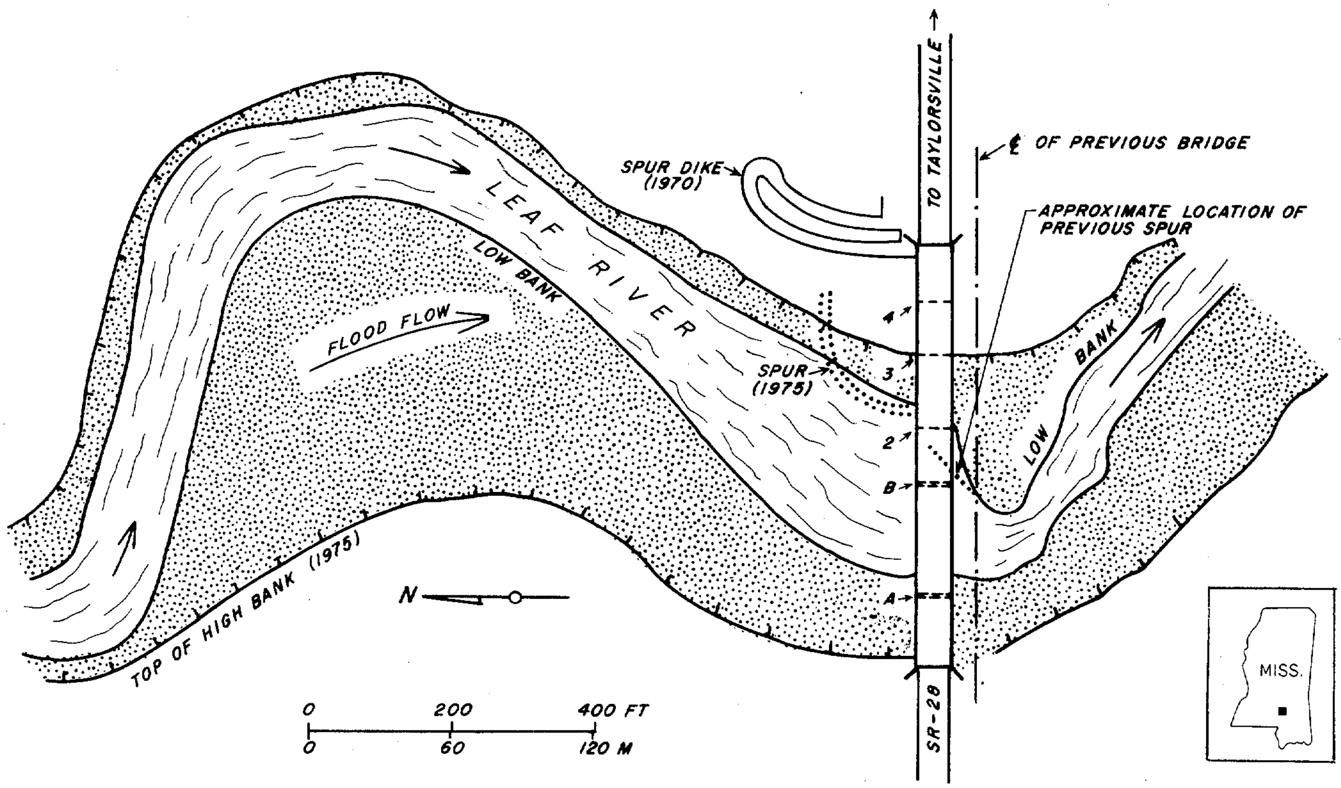


Figure 140. Plan sketch showing countermeasures at SR-28 crossing.

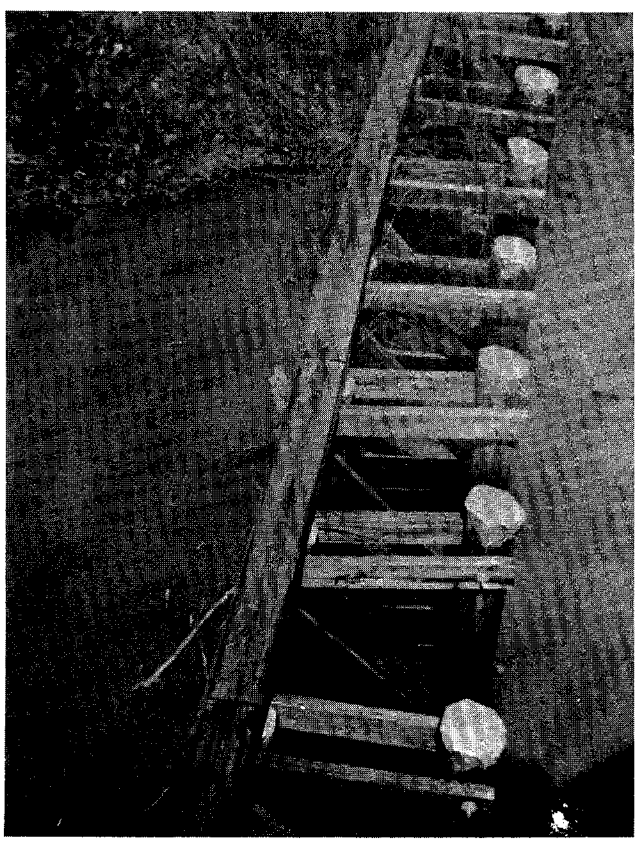


Figure 141. View of timber pile spur at old SR-28 bridge on March 25, 1969, looking toward east bank.

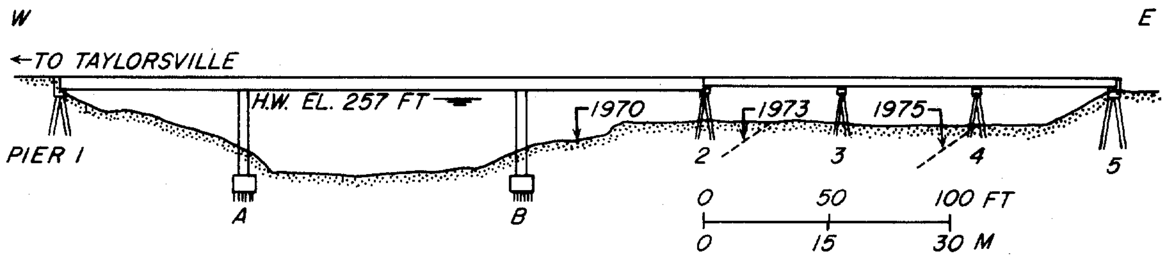


Figure 142. Elevation sketch of new SR-28 bridge, showing bank recession at pile bents.

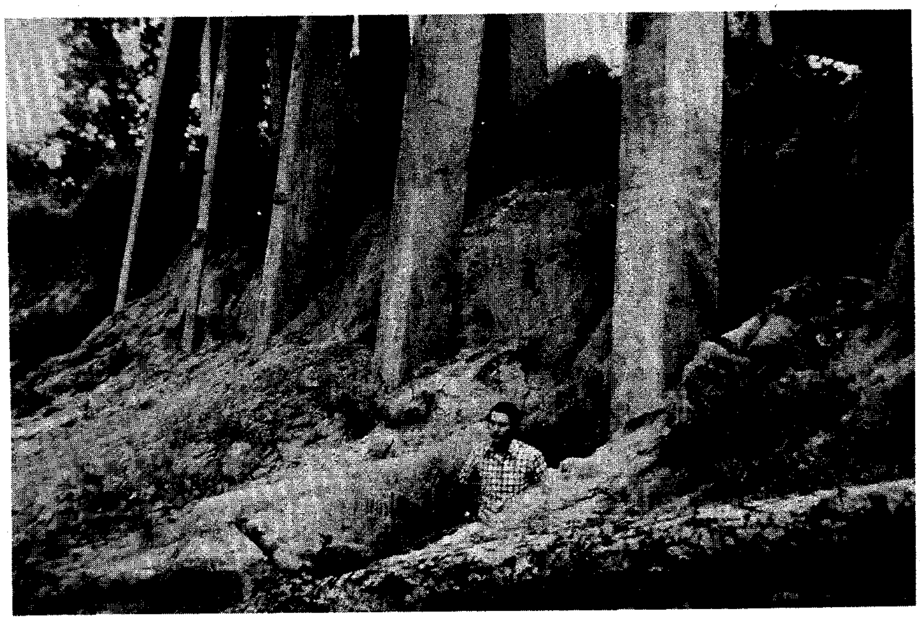


Figure 143. Lateral erosion at bent 4, August 7, 1975.

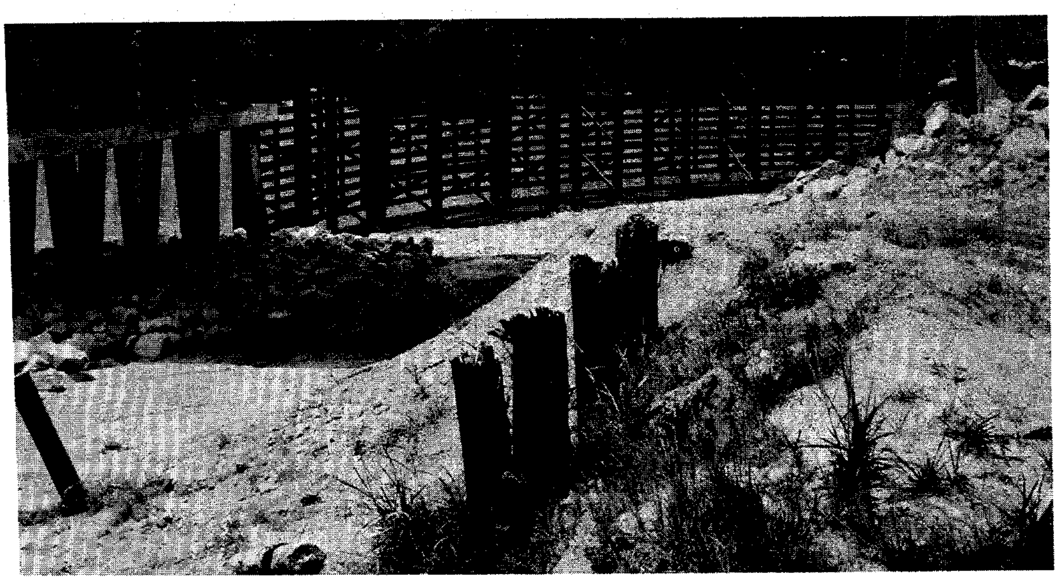


Figure 144. Upstream view showing riprap at bent 2, left, and new steel spur in background, June 29, 1976.

SITE 210. BIG BLACK RIVER AT OLD US-80 NEAR BOVINA, MISS.

Description of site: Lat 32°21', long 90°42.5', location as shown in fig. 145. Bridge is 576 ft (175 m) in length, one 170 ft (52 m) steel truss span, other spans I-beam or concrete. Two concrete, webbed piers in channel, concrete-pile bents, timber-pile foundations, spillthrough abutments. Crossing is at wide bend in channel.

Drainage area, 2,840 mi^2 (7,356 km^2); bankfull discharge, about 7,000 ft^3/s (198 m^3/s); valley slope, 0.00027; channel width, 175 ft (52 m). Stream is perennial, alluvial, sand bed, in valley of moderate relief, wide flood plain. Channel is meandering, somewhat wider at bends, narrow point bars, probably incised, silt-clay banks, tree cover on 50-90 percent of bankline (fig. 146).

Hydraulic problem and countermeasure:

1929 Bridge built, without countermeasures.

1940 Channel cleared and snags removed by U.S. Army Corps of Engineers.

1941-48 Lateral erosion of left bank at bridge exposed pier footings and foundation pile. Bridge subsided and was repaired. Floods during this period included two with peak discharges near 45,000 ft^3/s (1,273 m^3s).

1950 Three timber pile spurs built at left bank, dumped rock riprap placed to thickness of about 1 ft (0.3 m) around spurs and at both piers (fig. 147 and 148). Spurs faced with heavy timber sheathing on upstream side (extending approximately to low water elevation) and with willow poles on downstream side. Minimum penetration of pile, 20 ft (6 m).

1951-73 Numerous floods, including two having peak discharges of 63,500 ft^3/s (1,797 m^3/s). In 1973, spurs were in good condition and vegetation had become established at formerly eroded bankline. Performance of spurs, good.

Discussion: The general aspect of the Big Black River, as shown in fig. 146, indicates moderate stability. Channel width is only slightly greater at bends, exposed point bars are narrow, and bank slumping is mainly restricted to unvegetated banks at the outside of bends. No hydraulic problems were reported at the bridge until the channel was cleared of vegetation and snags. The good performance of the spurs is attributed in part to their role in the re-establishment of vegetation.

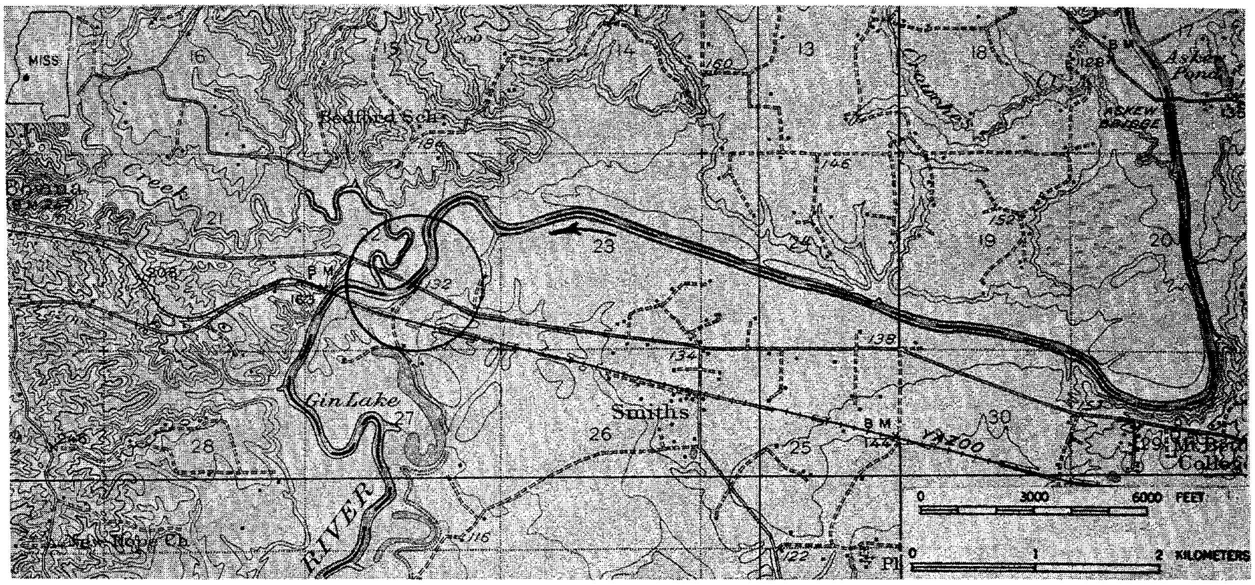


Figure 145. Map showing Big Black River at US-80 crossing (circled). (Base from U.S. Geol. Survey Edwards, Miss., 15' quadrangle, contour interval 20 feet, surveyed 1934-1935.)

Figure 146. Aerial photograph showing Big Black River at US-80 crossing, on November 16, 1973. Comparison with fig. 145 shows that river course changed only moderately during period 1935-1973. (From U.S. Dept. of Agriculture.)

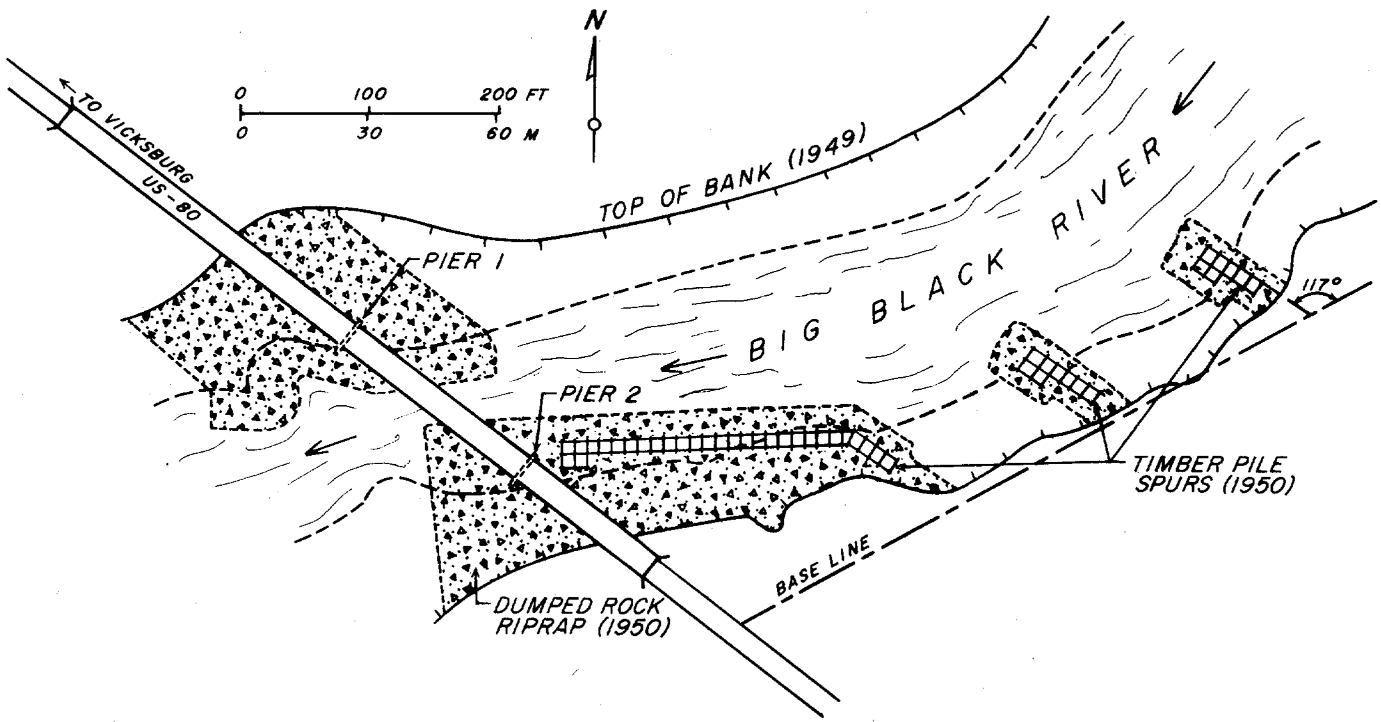


Figure 147. Plan sketch showing countermeasures installed in 1950.

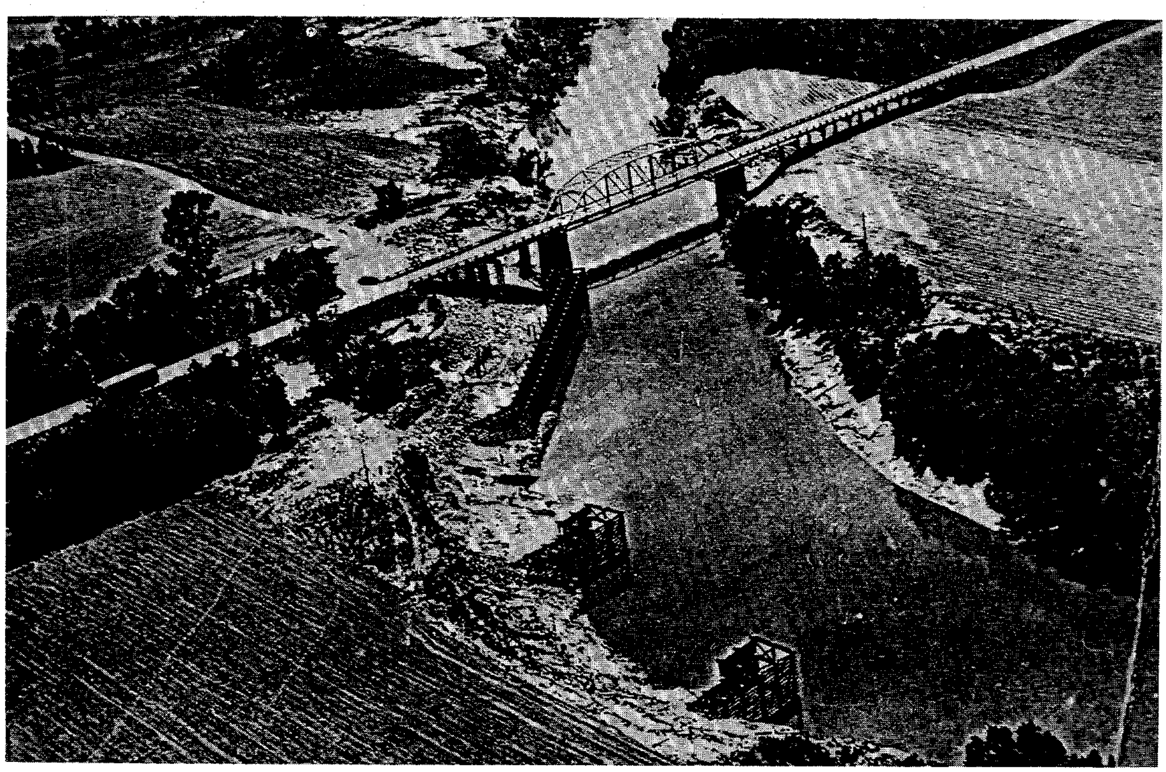


Figure 148. Oblique aerial view, looking downstream, of timber pile spurs. (From U.S. Army Corps of Engineers.)

SITE 211. LOBUTCHA CREEK AT SR-16 NEAR CARTHAGE, MISS.

Description of site: Lat 32°46', long 89°28'. Bridge is 860 ft (262 m) in length, concrete piers in channel, approach spans on timber-pile bents, spillthrough abutments. Channel alinement is nearly straight at main span, but upstream meander loop approaches left abutment and roadway embankment (fig. 149).

Drainage area, 313 mi^2 (810 km^2); bankfull discharge, about 1,300 ft^3/s (37 m^3/s); valley slope, 0.001; channel width, 75 ft (22 m). Stream is perennial, alluvial, in valley of moderate relief, wide food plain. Channel is highly meandering, equiwidth, cut banks local, tree cover on more than 90 percent of bankline, but left bank at bridge approach (where erosion problem occurred) is cleared.

Hydraulic problems and countermeasures:

1938 Bridge built, bankline of creek about 100 ft (30 m) from left approach embankment.

1944 Flood, R.I. about 9 yr, local scour at bents adjacent to left (east) abutment (fig. 149). Very large riprap, approximately equivalent to class III, placed at bents.

1950 Elongated scour hole 10-12 ft (3-3.6 m) in depth was noted at bents west of previously riprapped area. Hole was filled and scoured area riprapped.

1972 Lateral erosion of roadway embankment adjacent to left abutment noted. Retard, 160 ft or 48 m in length, consisting of a double row of timber pile was constructed along bank of creek. Creek bank was graded at 2:1 slope and covered with dumped rock riprap to a depth of 1 ft (0.3 m). Streamward face of retard was covered with 4 in x 8 in (10 cm x 20 cm) horizontal sheathing, back face with vertical willow poles.

1975 Flood, R.I. about 4 yr. Local scour, extending to depth of 10 ft (3 m) below ground line, was noted west of previously riprapped area. This hole was filled and area protected with class III riprap. The timber pile retard had so far been effective in preventing further bank erosion, and willows were becoming established within it.

Discussion: Although Lobutcha Creek is highly meandering, the banks are generally forested and appear stable except where cleared. The banks are cleared at the meander loop adjacent to the approach embankment, and the probability that overbank flow would become concentrated at the left end of the bridge is clearly apparent. The problems of lateral erosion at the approach embankment and scour at pile bents have apparently been solved, despite the difficult crossing location.

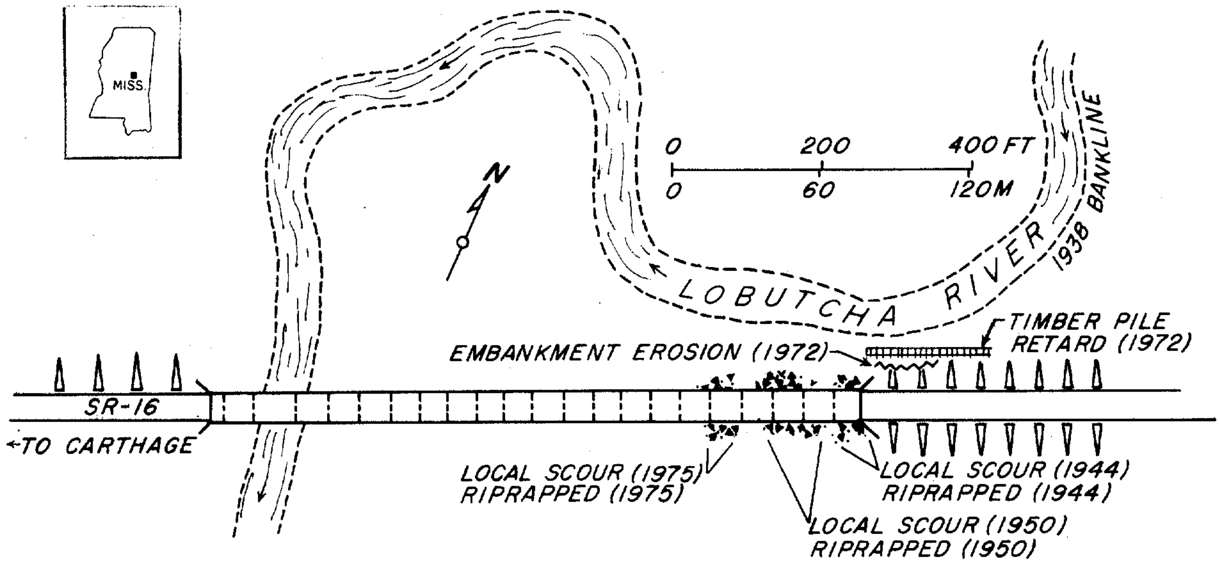


Figure 149. Plan sketch of Lobutcha Creek at SR-16.

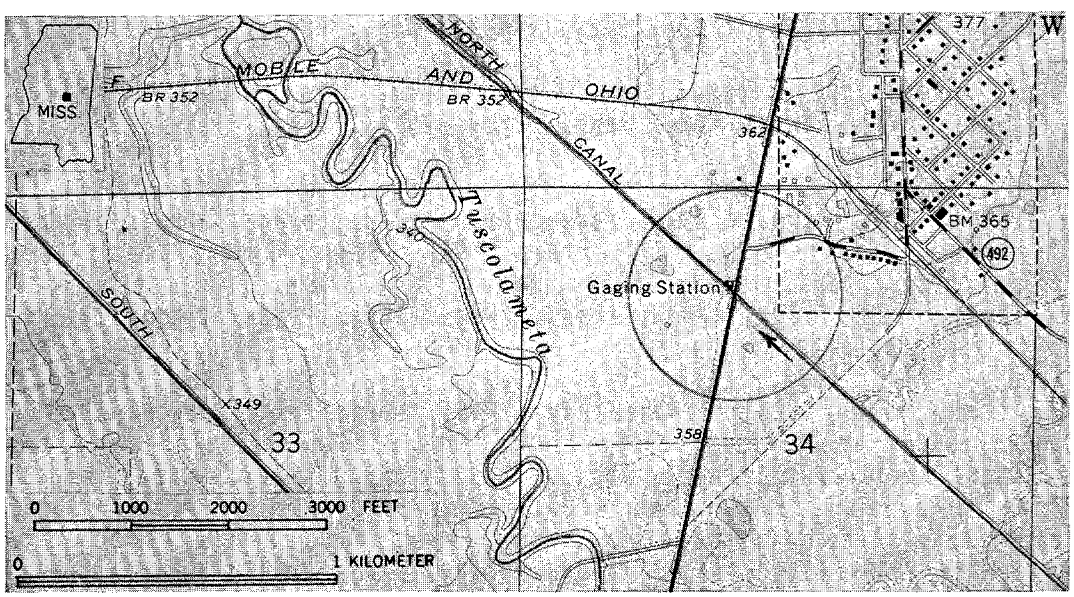


Figure 150. Map showing north canal of Tuscolameta Creek at SR-35 crossing (circled). (Base from U.S. Geol. Survey Walnut Grove, Miss., 7.5' quadrangle, contour interval 20 feet, 1972.)

SITE 212. TUSCOLAMETA CREEK (NORTH CANAL) AT SR-35 AT WALNUT GROVE, MISS.

Description of site: Lat 32°35', long 89°28', location as shown in fig. 150. Bridge is 200 ft (60 m) in length, I-beam spans, precast reinforced-concrete pile bents. Spillthrough abutments, protected with concrete slope paving. Crossing is skewed, and piers and abutments are skewed to bridge centerline.

Drainage area, 411 mi^2 (1,064 km^2); bankfull discharge, about 3,000 ft^3/s (85 m^3/s) valley slope, 0.00047; channel (canal) width, 100 ft (30 m). Stream is perennial, alluvial, sand bed, in valley of low relief, wide flood plain. Channel is straight (artificial), cut banks local, silt sand banks. The natural channel of Tuscolameta Creek is highly meandering and the banks are densely forested. In 1926-28, two drainage canals were constructed on either side of the creek (fig. 150) and this crossing is at the north canal.

Hydraulic problem and countermeasure:

1948 Bridge built, spillthrough abutments paved with concrete. During floods, the bridge functioned as a contraction and overbank flow entered the bridge waterway at almost a right angle to pile bents, creating a large eddy (about 60 ft or 18 m in diameter) at the left bank end of the bridge. Concentration of flow at the left bank end of the bridge was increased by spoil banks 4-6 ft (1-2 m) in height (fig. 151) and by clearing of the forest on the left bank flood plain (fig. 152).

1948-74 Canal degraded about 2 ft (0.6 m). Overbank floods were frequent because of the small conveyance of the canals, and an eddy at the left bank end of the bridge was noted during a discharge measurement in 1971. Two floods (each having an R.I. of about 7 yr) in 1974 caused sufficient erosion at the left bank that the left abutment was considered in jeopardy (fig. 153).

1975 Revetted earth spur dike, of elliptical shape and 50 ft (15 m) in length was built at left abutment (fig. 151) and a 200-ft (60-m) length of the spoil bank was removed upstream from the left abutment. Shortly after completion of the spur dike, a 6-yr R.I. flood occurred and flow remained at overbank stage for 5 days. Inspection after this flood showed that the spur dike had functioned well in directing flow and in controlling erosion at both the right and left banks.

Discussion: Besides the spoil banks, a major factor in flow concentration at the north end of the bridge was the contrasting roughness of the cleared floodplain on the left bank with the vegetated flood plain on the right bank.

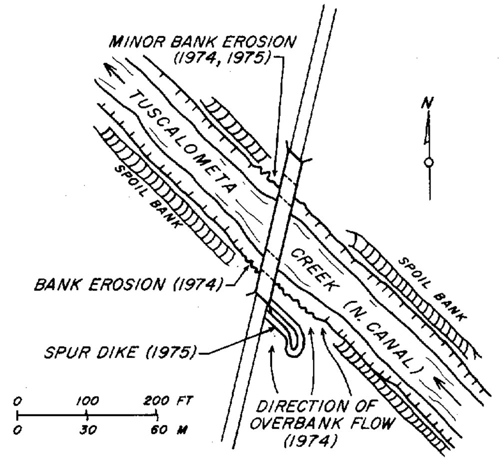


Figure 151. Plan sketch of north canal at SR-35 crossing, showing countermeasures.

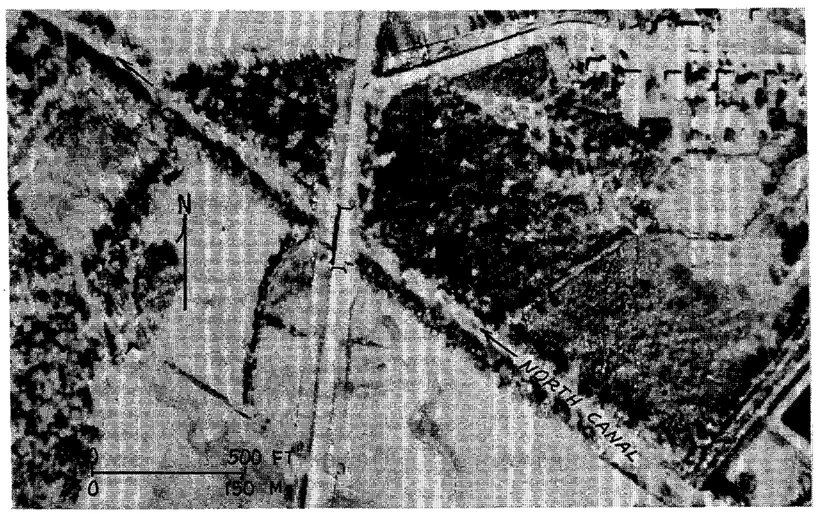


Figure 152. Aerial photograph of north canal Tuscolameta Creek showing cleared flood plain at left (south) bank of canal. (From U.S. Dept. of Agriculture.)

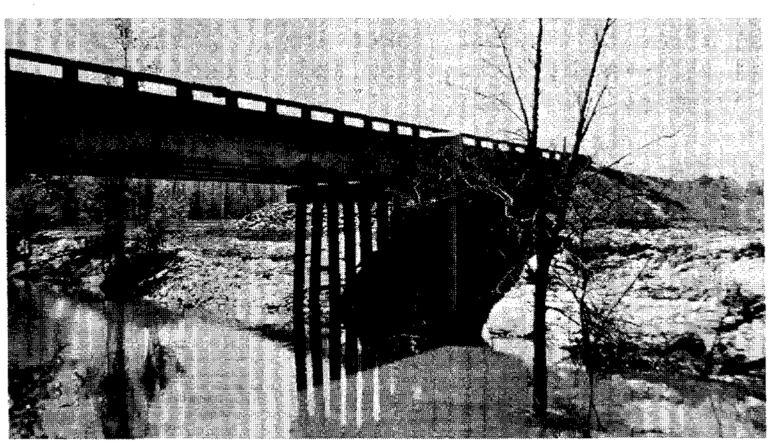


Figure 153. Upstream view of eroded left bank at bridge, on April 6, 1976, spur dike is visible beyond pile bent.

SITE 213. SUGARNOOCHEE CREEK AT US-45 NEAR PORTERVILLE, MISS.

Description of site: Lat 32°42', long 88°29'. Bridge is 680 ft (207 m) in length, 60-ft (18-m) I-beam main span on concrete piers with pile foundations, I-beam approach spans on timber-pile bents. At time of bridge construction, a meander loop (now an oxbow lake on the flood plain north of the bridge, see fig. 154) was cut off and the cutoff channel had a nearly stright alinement normal to the bridge.

Drainage area, about 180 mi^2 (466 km^2); channel width, about 50 ft (15 m). Stream is perennial, alluvial, sand bed, in valley of low relief, wide flood plain. Natural channel is meandering, somewhat wider at bends, point bars, probably incised, cut banks general where banks are cleared, tree cover at less than 50 percent of bankline in vicinity of bridge.

Hydraulic problem and countermeasure:

1936 Bridge built, without countermeasures; meander loop cut off (fig. 154).

1952 Channel cross section surveyed by Highway Department indicates that oxbow lake is filling and that right bank had not receded.

1961 Flood, R. I. 50 yr, peak discharge 11,500 ft^3/s (325 m^3/s).

1971 Meander loop beginning to develop at south bank (fig. 155), pier originally at south bank now in center of channel, sway bracing needed at pile bent adjacent to south bank.

1972 Channel realined and heavy riprap, approximately equivalent to class II, placed for distance of about 450 ft (135 m) on right bank of new channel (fig. 155).

1973-75 One flood of R.I. 20 yr and another of R.I. 6 yr. Site inspection after these floods indicated that banks of new channel are stable, and performance of countermeasure is considered satisfactory.

Discussion: In view of the meandering habit of the stream and the erodibility of the banks where cleared, development of a meander at the bridge (and consequent lateral erosion of the bank) was probable.

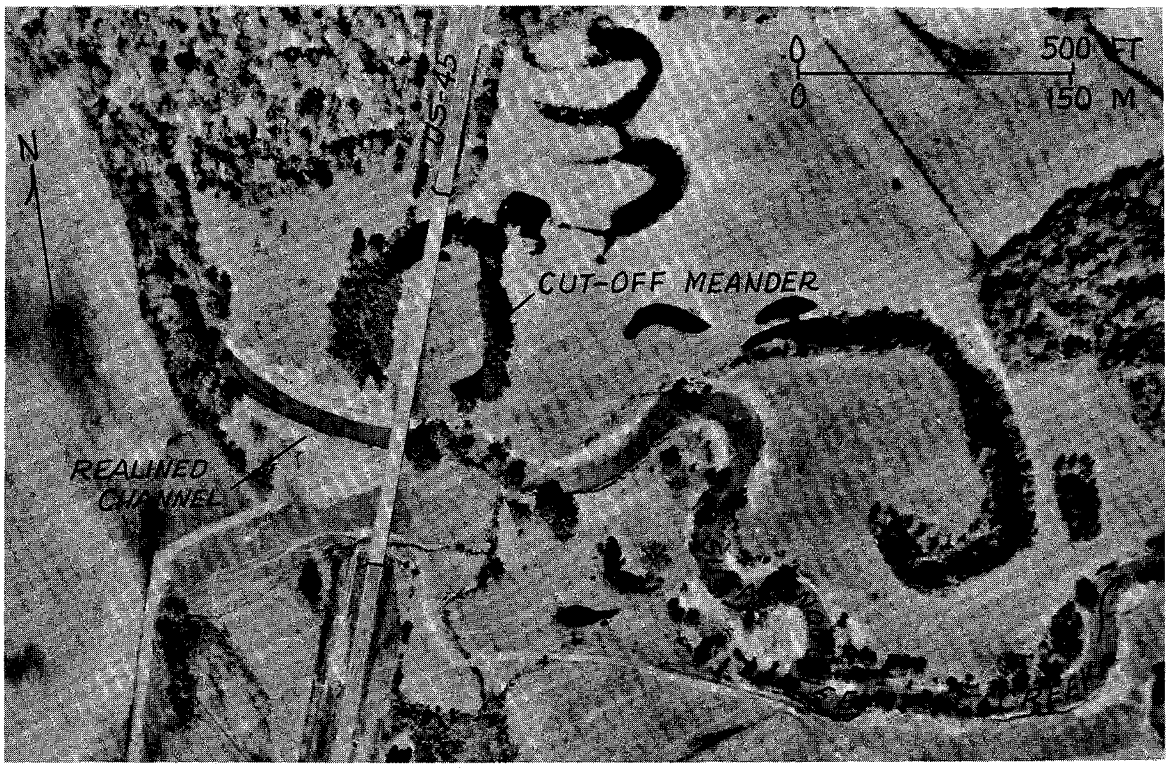


Figure 154. Aerial photograph of Sugarnoochee Creek at US-45 crossing, on March 22, 1973. (From Mississippi Dept. of Highways.)

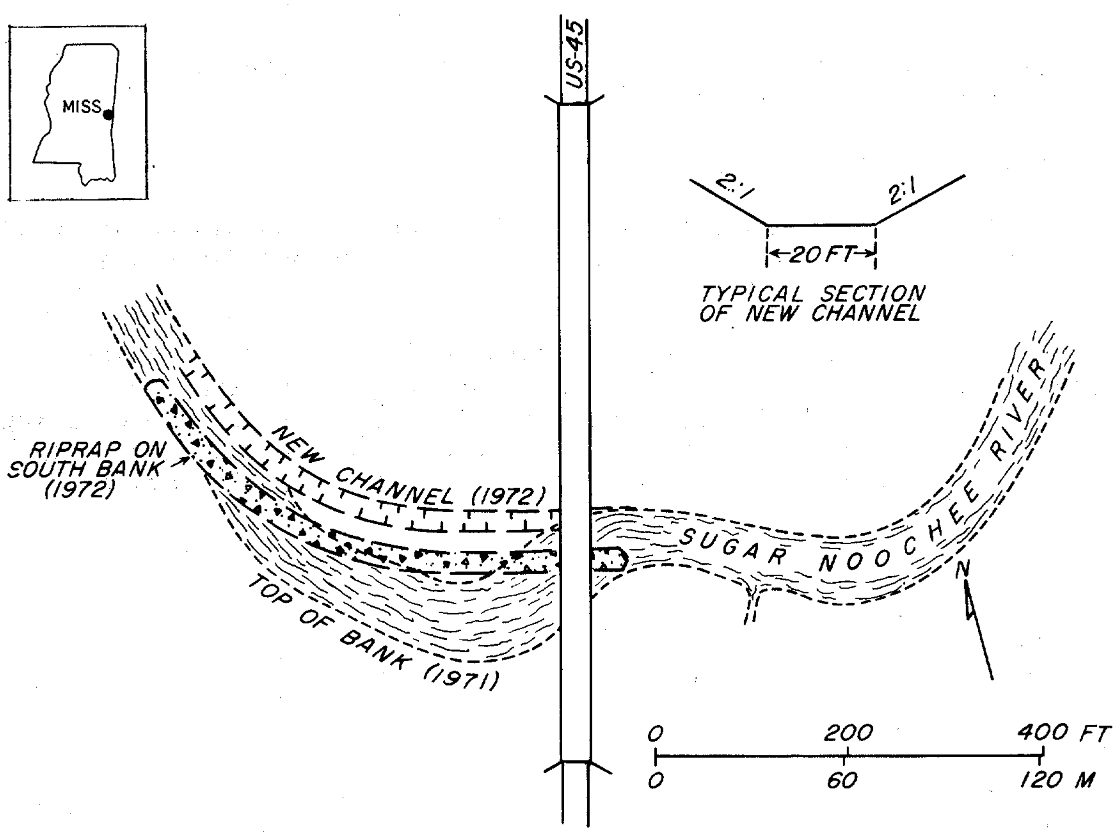


Figure 155. Plan sketch showing countermeasures at US-45 crossing.

SITE 214. BIG BLACK RIVER AT SR-19 AT WEST, MISS.

Description of site: Lat 33°12', long 89°46'. Crossing includes 410-ft (125-m) bridge over main channel, a 280-ft (85-m) relief bridge to the west of the main channel and a 280-ft (85-m) relief bridge east of the channel (fig. 156). Spans are prestressed-concrete beams, supported by precast-concrete pile bents. Flood flow is constricted by roadway embankment, and overbank flow is frequent.

Drainage area, 985 mi^2 (2,551 km^2); channel width, about 80 ft (25 m). Stream is perennial, alluvial, wide flood plain. Channel is meandering, equiwidth, cut banks rare, tree cover at more than 90 percent of bankline (fig. 157).

Hydraulic problem and countermeasure:

1966 Main bridge and relief bridges constructed. Revetted earth embankment spur dikes, of elliptical shape and 140 ft (42 m) in length, constructed on both banks at upstream side of abutments at all three bridges.

1967-68 Two floods during a period of one month, R.I. about 3 yr for one and 4 yr for the other. Spur dikes contracted the flow fairly uniformly into each bridge. Scour of abutment fill noted on downstream side of main bridge and west relief bridge, attributed partly to fact that vegetation had not yet become established on the fill.

1968 Downstream extensions of spur dikes added where erosion of abutment fill had occurred (fig. 156).

1973 Flood, R.I. about 25 yr.

1975 Field inspection indicated that no further erosion has occurred at abutments, therefore downstream extensions of spur dikes are regarded as successful.

Discussion: Erosion of abutment fill on downstream side of main channel bridge is attributed to a side channel that leads toward the abutment (fig. 156). Erosion at west relief bridge is attributed in part to clearing of the flood plain upstream from the bridge. Equiwidth streams such as this reach of the Big Black tend to be laterally stable if the banks remain forested.

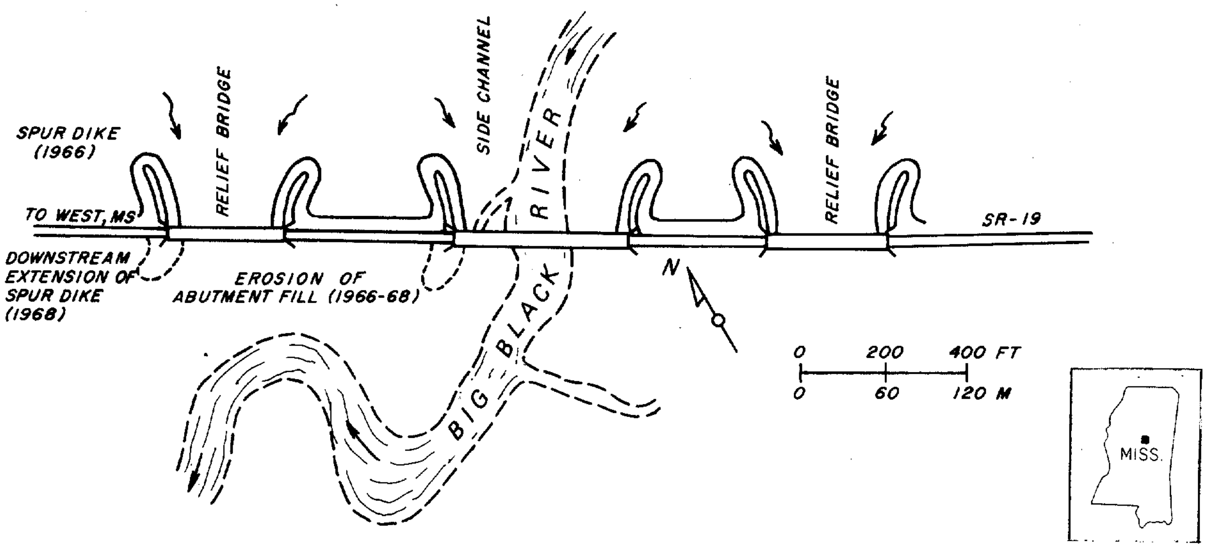


Figure 156. Plan sketch showing SR-19 crossing of Big Black River.

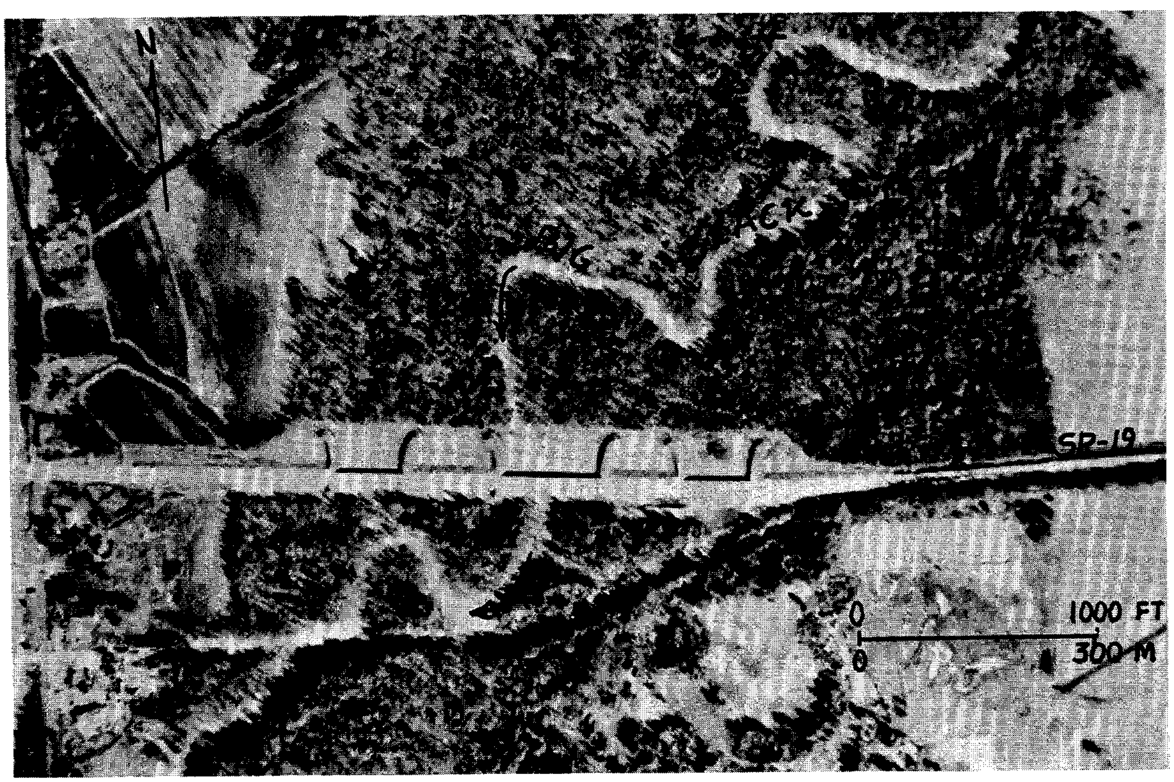


Figure 157. Aerial photograph of SR-19 crossing on December 17, 1972. (From Mississippi Dept. of Highways.)

SITE 215. BIG BLACK RIVER AT SR-12 AT DURANT, MISS.

Description of site: Lat 33°04', long 89°50' location as shown in fig. 158). Bridge is 1,500 ft (450 m) in length, one 205-ft (62-m) cantilever span on concrete piers, 37 I-beam spans on concrete-pile bents. Spillthrough abutments protected by timber-pile bulkheads and riprap. Crossing is at slight bend in stream.

Drainage area, 1,180 mi^2 (3,056 km^2); valley slope, 0.0004; channel width, 150 ft (45 m). Stream is perennial, alluvial, in valley of moderate relief, wide flood plain. Channel is meandering, somewhat wider at bends, point bars vegetated, cut banks local, tree cover on more than 90 percent of bankline.

Hydraulic problem and countermeasure:

1937 Bridge built.

1957-62 Flood of 15-yr R.I. occurred in 1961. Successive cross sections of channel at bridge indicate that channel has migrated to right (at outside of bend) and by 1962 the foundation of pier 2 was exposed and bent "B" had lost lateral support (fig. 159). Slight subsidence of pier 2 was reported.

1964 Three timber-pile spurs, consisting of 3-pile clusters connected by timber stringers, constructed at right bank, each spur about 125 ft (38 m) in length and spaced at 125 ft (38 m) intervals (fig. 160). Top of spurs at bank height, minimum penetration of pile, 20 ft (6 m). Riprap with granular filter placed on channel bottom and banks at spurs.

1965 Performance of spurs observed during flood of 7-yr R.I. at stage above tops of pile. The spurs were deflecting the current toward the left bank, and setting up eddies that eroded the bank.

1966-72 Right bank receded about 15-20 ft (5-6 m) during this period. In June of 1972, channel bed was 5 ft (1.5 m) above bottom of footing at pier 1 and 2 ft (0.6 m), at pier 2.

Discussion: Spurs successfully prevented further erosion of right bank, but caused erosion of left bank by deflection of current. Probably the spurs were too long in relation to channel width at the time of construction (length measured perpendicular to bank, about 90 ft or 27 m; channel width, about 150 ft or 45 m) but now that the channel had widened, erosion of the left bank may not progress further.

Figure 158. Aerial photograph of Big Black River at SR-12 crossing, on January 27, 1976. (From Mississippi Dept. of Highways.)

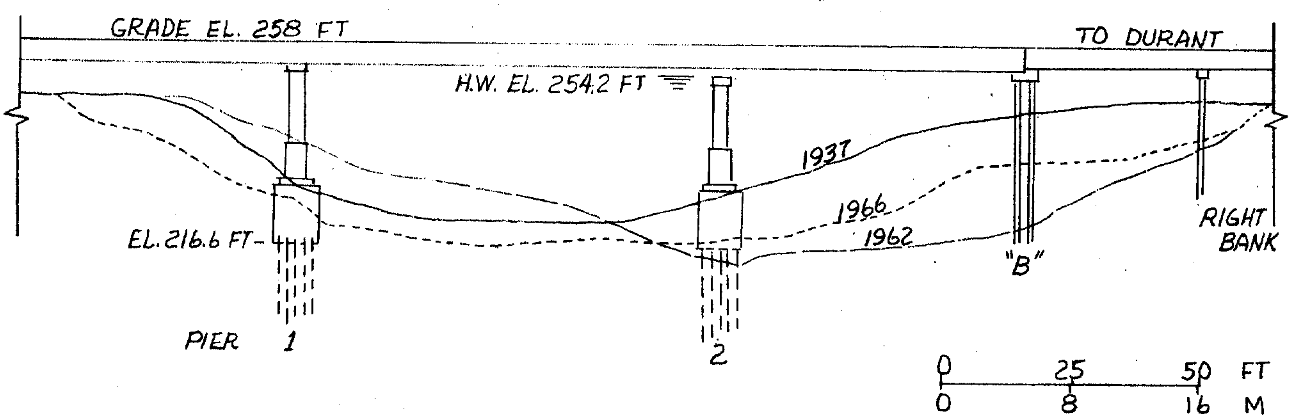


Figure 159. Channel cross sections, as measured on downstream side of SR-12 bridge, in 1937, 1966, and 1962.

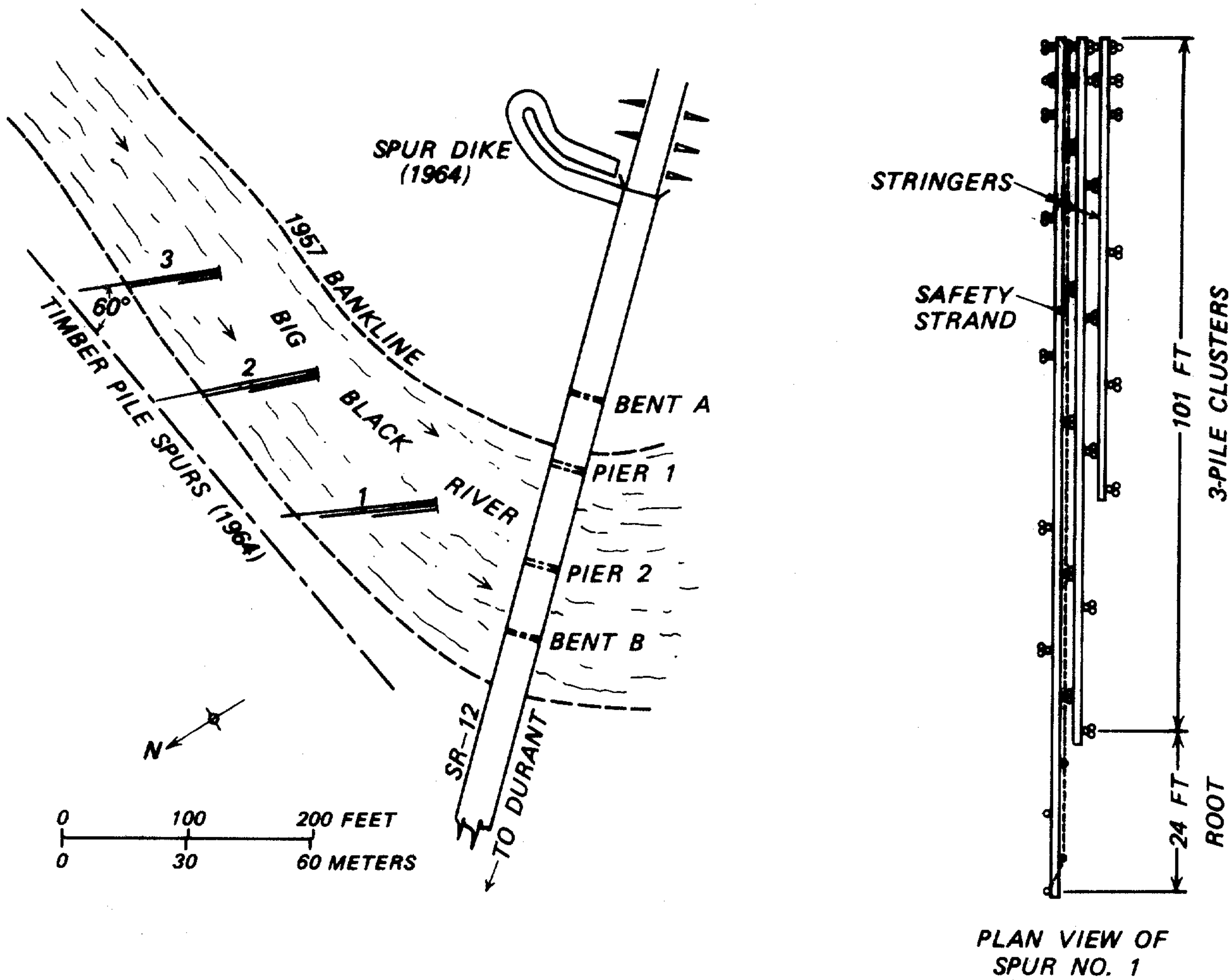


Figure 160. Plan sketch showing spurs installed at SR-12 crossing.

SITE 216. BLACK CREEK AT SR-12 LEXINGTON, MISS.

Description of site; Lat 33°6.5', long 90°02'. Bridge is 220 ft (67 m) in length, reinforced-concrete girder spans, concrete pile bents, spillthrough abutments. Although channel alinement is normal to bridge, there is a large bend in the channel about 300 ft (90 m) upstream from the bridge (fig. 161).

Drainage area, 74 mi^2 (192 km^2); valley slope, about 0.0025; channel width, about 100 ft (30 m). Stream is perennial, alluvial, sand bed, in valley of low relief, wide flood plain. Channel is meandering, wider at bends, point bars, somewhat incised, bank height about 15 ft (5 m), cut banks general, silt-sand banks, tree cover on 50-90 percent of bankline. Rate of meander migration, fairly rapid.

Hydraulic problem and countermeasures:

1956 Bridge built, timber-pile spurs and retards already present on right bank upstream from bridge (fig. 161).

1958 Erosion of left bank caused concern that stream would cut across bend and bypass the bridge. A double-row timber-pile retard was constructed at left bank (fig. 161), a riprap cover about 1 ft (0.3 m) in thickness was placed at the base of the retard, and willows were planted on the bank. The riprap was dumped rock approximately equivalent to class I, the minimum penetration of pile was 20 ft (6 m), and the retard was faced with heavy timber sheathing.

1966 Flood, R. I. estimated at 10 to 20 yr. Right bank upstream from bridge was eroded and timber-pile spur (older than the bridge) was destroyed.

1969 In view of the possibility that erosion of the right bank would lead to outflanking of the bridge, a double-row timber-pile retard, similar in construction to the 1958 retard, was placed at the right bank. In addition, 30 car bodies cabled together and tied to a concrete anchor, were placed behind the retard.

1970 Flood, R. I. estimated at 10 to 20 yr, peak discharge 15,000 ft^3/s (424 m^3/s) average water velocity 6-7 ft/s (2 m/s).

1975 Inspection of site indicates that 1969 retard is in good condition, the car bodies are becoming buried, and vegetation has become established (figs. 162A and B).

Discussion: With the exception of the old timber pile spur that was destroyed by flood in 1966, all of these countermeasures have performed satisfactorily in protecting bank and preserving the channel alinement at the bridge. However, the sharp bend upstream of the bridge is still a potential source of trouble, because it may deflect flow toward the left abutment.

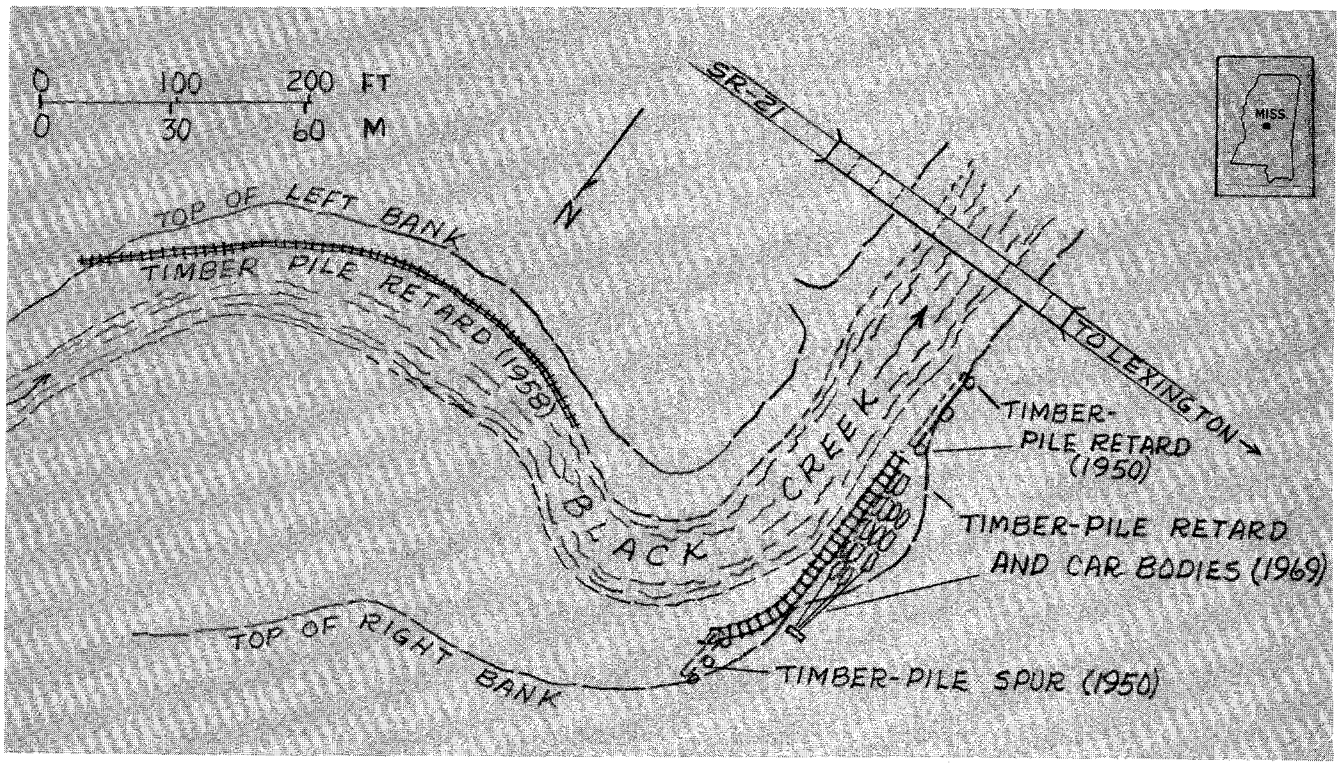


Figure 161. Plan sketch showing countermeasures at SR-12 crossing at Lexington, Miss.

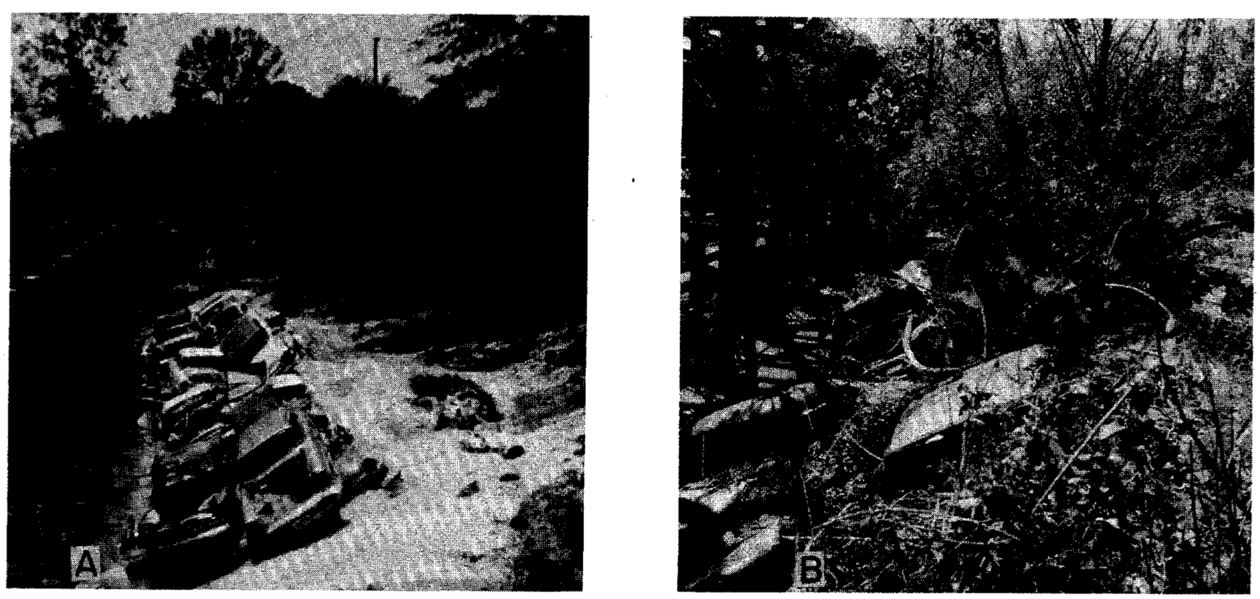


Figure 162. A, downstream view of timber pile retard and car bodies on April 28, 1970, shortly after a flood. B, downstream view of same scene on November 12, 1975, showing partial burial of car bodies and growth of vegetation.

SITE 217. NOXUBEE RIVER AT US-45 AT MACON, MISS.

Description of site: Lat 33°06', long 88°34'. Bridge is 400 ft (120 m) in length, cantilever-girder main span with reinforced-concrete beam approach spans. Concrete piers and pile bents, some with spread footings on bedrock and some with timber-pile foundations. Crossing is skewed 60° to channel (fig. 163).

Drainage area, 812 mi^2 (2,103 m^2); channel width, about 100 ft (30 m). Stream is perennial, semi-alluvial, in valley of low relief, wide flood plain. Channel is meandering, equiwidth, cut banks rare. Bedrock that forms part of channel boundary is firm chalk (porous limestone), resistant to erosion.

Hydraulic problem and countermeasure:

1935 Bridge built, sacked-concrete revetment at abutment fill-slopes.

1949 Flood of 40-yr R.I., no damage reported at crossing.

1951 Flood of 45-yr R.I., no damage reported at crossing.

1969 Flood of 6-yr R.I. Long duration of overbank stage resulted in saturation of earth fill of right roadway approach embankment. Erosion also reported at upstream side of right abutment. Roadway embankment slumped for distance of about 160 ft (48 m), concrete roadway slabs subsided 8-11 ft (2.5-3 m). As a countermeasure, a timber-pile retaining wall was driven at the toe of the approach embankment (fig. 164), and dumped riprap of broken concrete slabs was placed at the upstream side of the right abutment.

1970-76 Many small floods, two largest rated at 11-yr R.I. No further damage reported at crossing.

Discussion: The uniform width, lack of point bars, lack of cut banks, and rather angular meanders of the Noxubee are all indicative of channel stability. Slumping of the roadway embankment is not attributed to lateral erosion by the river, but to slumping of saturated fill that occurred because of the duration, rather than the magnitude, of a flood.

Figure 163. Aerial photograph of Noxubee River at US-45 crossing, on March 11, 1969. Note uniformity of channel downstream from bridge. (From U.S. Dept. of Agriculture.)

Figure 164. View along upstream side of right approach embankment, showing timber pile retaining wall and broken concrete riprap, on November 13, 1975.

SITE 218. TALLAHATCHIE RIVER (FORT PEMBERTON CUTOFF) AT US-82 AT GREENWOOD, MISS.

Description of site: Lat 33°32', long 90°14', location as shown in fig. 165. Dual bridges, 560 ft (170 m) in length, concrete piers and precast-concrete pile bents in channel.

Drainage area, 7,450 mi^2 (19,295 km^2); valley slope, 0.0001. Stream is perennial, sand bed, in valley of low relief, wide flood plain. Channel is highly meandering, equiwidth, silt-clay banks (fig. 166).

Hydraulic problem and countermeasure:

1959-62 Bridge built at site of Fort Pemberton Cutoff, which was constructed by U.S. Army Corps of Engineers, to reduce flood elevations at city of Greenwood.

1964-72 Cutoff plugged.

1973 Extreme flood, cutoff opened to reduce flood elevation.

1973-75 Flow through cutoff increased until it carried most of the flow of the Yazoo River. Flow entering the cutoff from the east, derived mostly from the Yalobusha River (fig. 165), created a large eddy upstream from the bridge. Scour at this eddy penetrated non-coherent silty sand on the bottom of the cutoff and reached a depth of about 30 ft (9 m) below the channel bottom as originally excavated. Only 10 ft (3 m) of pile penetration remained at the first bent left of the left main pier.

1975-76 About 3,000 tons (2,700 t) of dumped rock riprap (somewhat larger than class I) and 1,000 tons (900 t) of broken concrete riprap placed at left side of channel. Sheel piling about 75 ft (23 m) in length were driven at endangered pile bents. Total cost of countermeasures, about $100,000.

1977 Plans made for closure of Fort Pemberton Cutoff. Scour hole in cutoff channel at bridge to be armored with dumped rock riprap, approximately equivalent to class I, in the amount of about 19,000 tons (17,100 t) and at a cost of about $540,000.

Discussion: Construction of the bridge at the cutoff was desirable because the highway was already located at that point. The rivers migrate laterally, as indicated by the conspicuous meander scrolls (fig. 166), but the migration rate is not sufficiently rapid to be much of a hazard during the life of a bridge. A weir was constructed in the cutoff to control the amount of flow, but it failed to function as planned. In addition, the eddy that was generated in the cutoff was not anticipated. It appears that the construction of a bridge across a cutoff such as this one is risky, because the response of the rivers to the cutoff is difficult to predict. No useful judgment can be made about the effectiveness of the riprap placed in 1975-76.

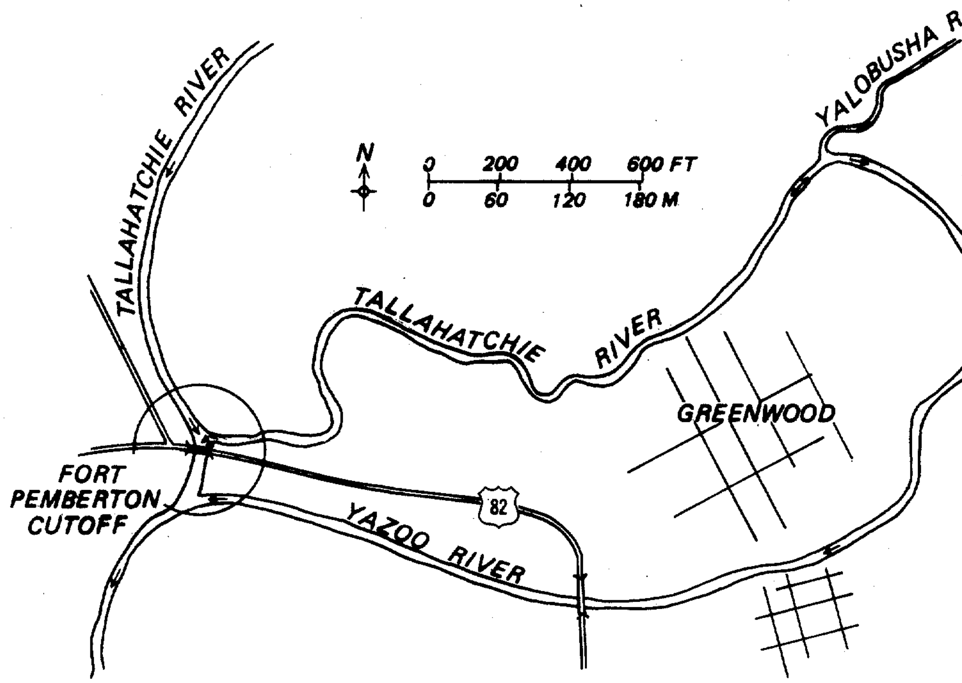


Figure 165. Plan sketch showing location of bridge at Fort Pemberton cutoff, with respect to the Tallahatchie, Yalobusha, and Yazoo Rivers.

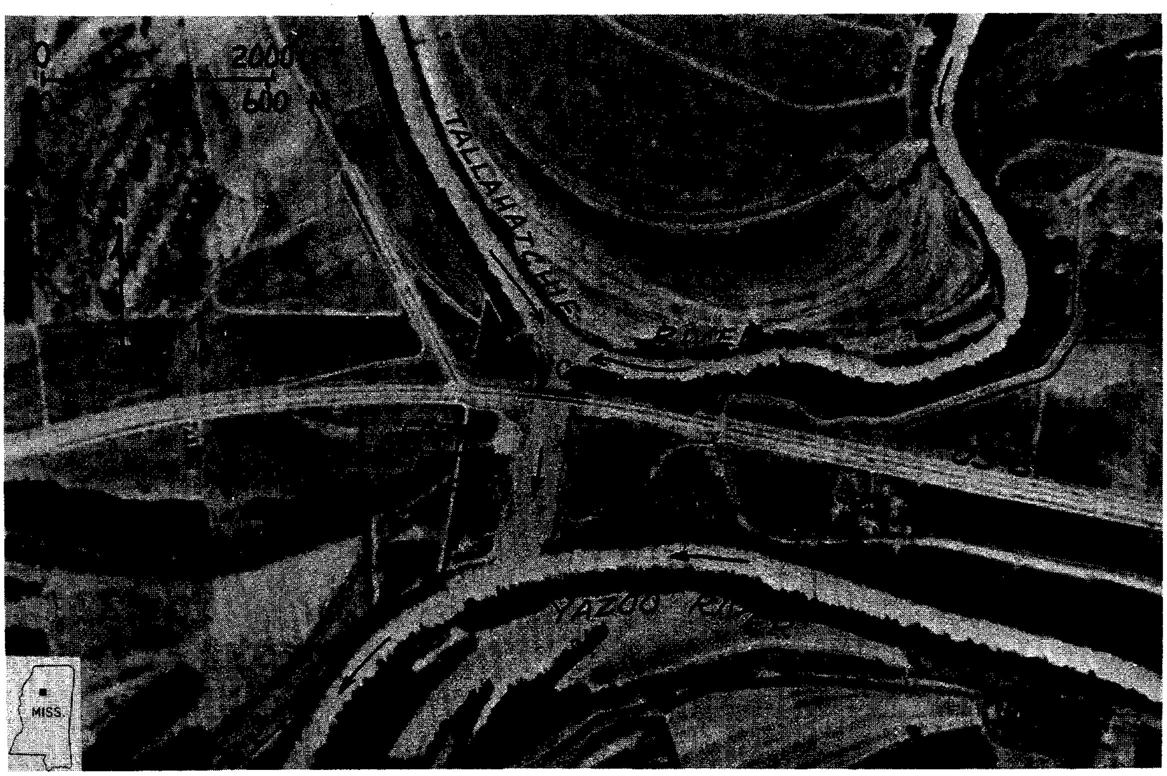


Figure 166. Aerial photograph showing stable aspect of natural rivers by comparison with artificial channel at Fort Pemberton cutoff, on November 21, 1973. (From U.S. Dept. of Agriculture photograph.)

SITE 219. TOMBIGBEE RIVER AT US-82 AT COLUMBUS, MISS.

Description of site: Lat 33°29', long 88°26', location as shown in fig. 167. Bridge is 982 ft (299 m) in length, steel truss with reinforced concrete beam approach spans, two 2-column, concrete, webbed piers having square noses, in channel; precast concrete pile bents. Crossing is skewed about 15°, piers and abutments normal to bridge centerline. Main piers are slightly skewed to flood flow.

Drainage area, 4,490 mi^2 (11,629 km^2); bankfull discharge, about 57,000 ft^3/s (1,613 m^3/s); valley slope, 0.0003; channel width, about 250 ft flood plain. Channel is sinuous, equiwidth, probably incised, cut banks local, 50-90 percent tree cover on bankline (fig. 167).

Hydraulic problem and countermeasure:

1964 Bridge built, no countermeasures installed. Shortly after construction, an eddy was observed at pier II near the left bank, probably resulting from the square nose of the pier and the slight degree of skew to flow. Although the high left bank was resistant to erosion, it began to recede.

1965-72 Three or four car bodies were dumped at the eroding bank by the highway department.

1973 Flood rated at greater than 50-yr R. I., peak discharge 194,000 ft^3/s (5,490 m^3/s). Left bank receded to a point only 6 ft (2 m) from double pile bent 15, where the pile tip elevation was about 136 ft, or about the same elevation as the toe of the receded bank. As countermeasures, the bank was graded to a 1.5:1 slope and protected with dumped rock riprap, approximately equivalent to class I, and a steel H-pile retard about 90 ft (27 m) in length was constructed along the bank upstream from pier II (fig. 168). The H-pile, in a double row, were driven to elevation 105 feet and the retard was faced with 3 in x 8 in (7.5 cm x 20 cm) sheathing.

1975 Flood of 10-yr R. I. Subsequent inspection of the east (left) bank at the bridge showed no erosion, and the performance of the countermeasures was rated as good.

Discussion: The Tombigbee River in this vicinity has fairly stable banks and its general morphology is indicative of stability (fig. 167). A slight bend upstream from the bridge may be responsible for some deflection of flood flow toward the east bank at the bridge, but turbulence resulting from the configuration and slight skewness of the pier may be the main cause of bank erosion. In retrospect, the depth of penetration of pile bent 15 seems inadequate, but the bank material may have been too resistant for deeper penetration of the concrete pile.

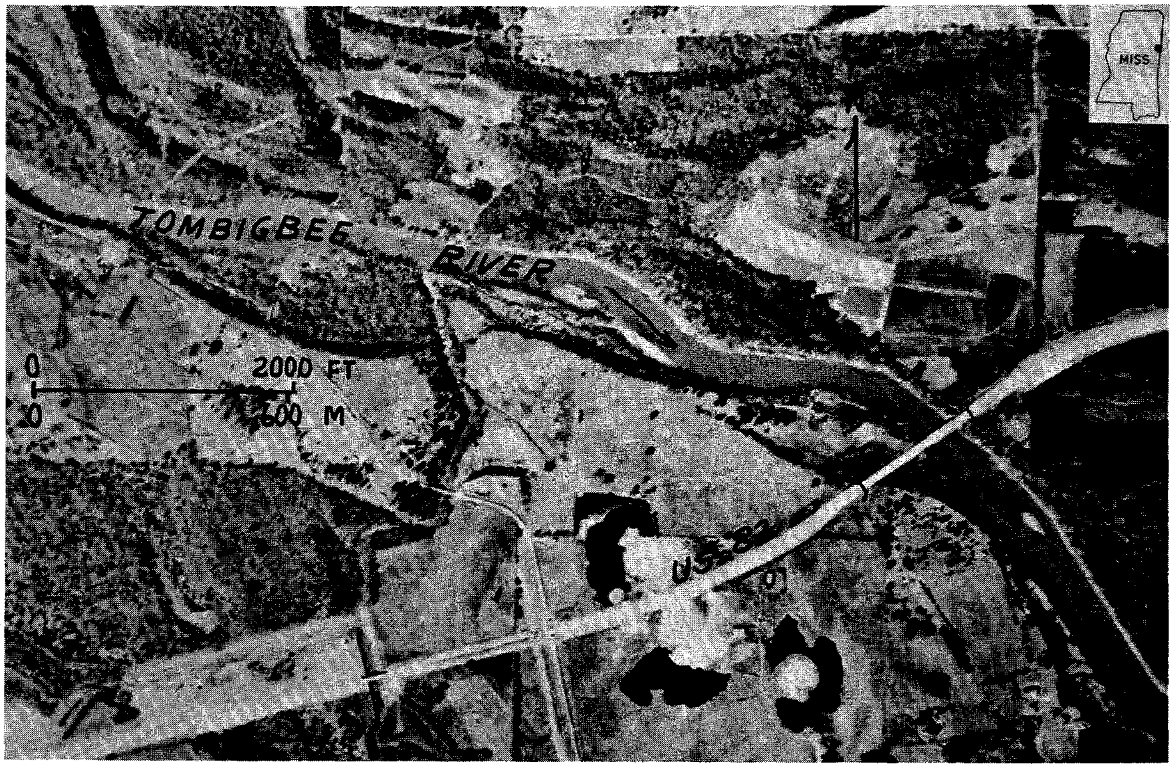


Figure 167. Aerial photograph of Tombigbee River at US-82 crossing, on March 13, 1969. (From U.S. Dept. of Agriculture.)

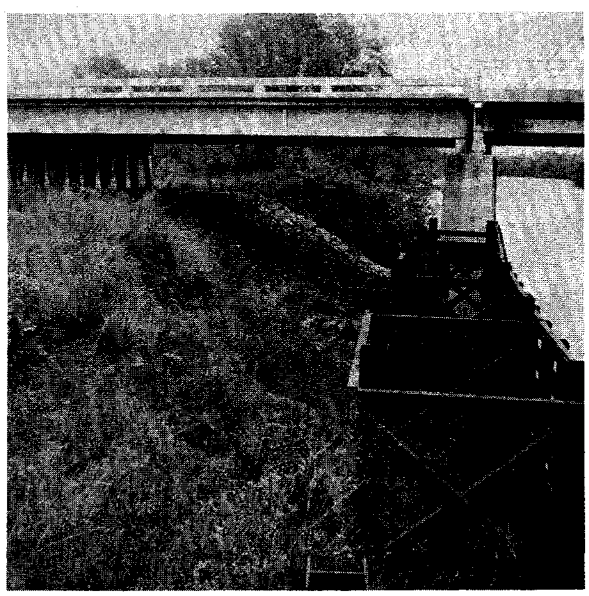


Figure 168. Downstream view of steel-pile retard and riprapped bank (beneath bridge), on November 13, 1975.

SITE 220. TILLATOBA CREEK AT SR-35 AT CHARLESTON, MISS.

Description of site: Lat 34°00', long 90°04', location as shown in fig. 169. Bridge is 240 ft (73 m) in length, I-beam and reinforced-concrete slab spans, steel-pile bents. Channel was straightened at time of bridge construction, both upstream and downstream from bridge.

Drainage area, 52 mi^2 (135 km^2); valley slope, 0.0015; channel width, 150 ft (45 m). Stream is perennial, alluvial, gravel bed, in valley of moderate relief, narrow flood plain. Channel is meandering, probably incised, cut banks general, silt-sand banks. Although some channel instability may be attributed to straightening, there is general instability of channels in this area, probably because of runoff from cultivated areas.

Hydraulic problem and countermeasure:

1956 Bridge built. Previous bridge, located about 250 ft (75 m) upstream, had been outflanked by stream and spans had been added.

1958 Erosion of right bank upstream from bridge was progressing toward abutment. Four timber-pile spurs, ranging in length from 69 to 121 ft (21 to 37 m) and spaced about 100 ft (30 m) apart, were constructed connected by timber stringers and riprap was placed on bed and banks at each spur (fig. 169).

1976 Site was inspected by Geological Survey representative. High water mark indicated that flood greater than design had occurred recently, and performance of countermeasure was rated as good. Riprap had recently been installed as bank protection by the U.S. Army Corps of Engineers (fig. 170). Exposure of the bridge substructure indicates that the channel has degraded 4-6 ft (1-2 m) since 1956.

Discussion: A major source of difficulty in channel straightening is the junction between the straightened reach and the natural channel. In this case, erosion occurred when flow from the natural channel impinged at an angle against the straightened reach. However, general instability and degradation of the channel is not attributed to local straightening at the bridge.

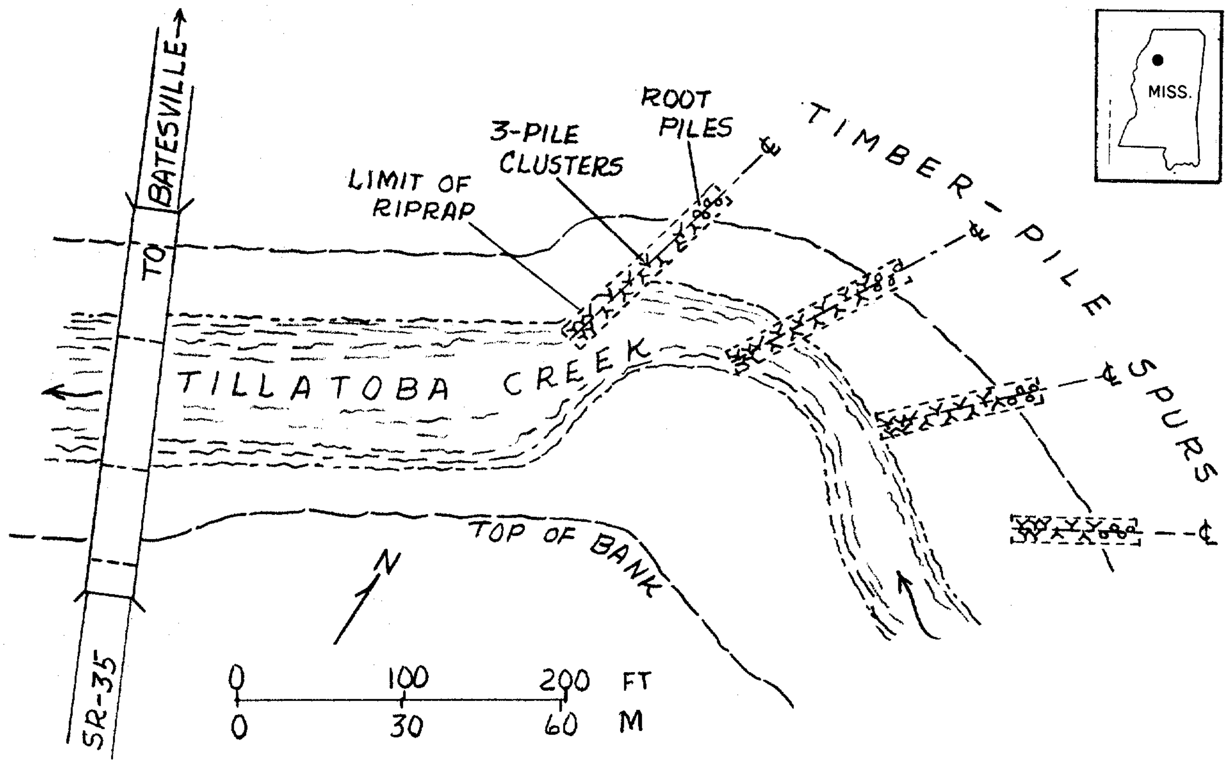


Figure 169. Plan sketch showing location of spurs at SR-35 crossing.

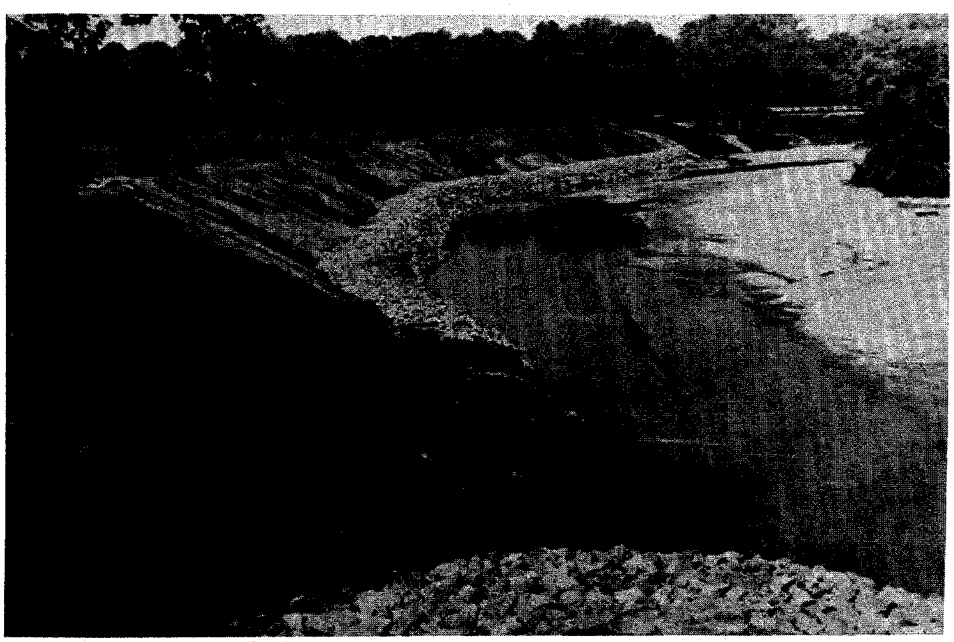


Figure 170. Riprap bank protection installed by Corps of Engineers on Tillatoba Creek. (From Vicksburg District, Corps of Engineers.)

SITE 221. EAST FORK TOMBIGBEE RIVER AT SR-6 AT BIGBEE, MISS.

Description of site: Lat 34°00', long 88°31'. Bridge is 880 ft (268 m) in length, main span continuous concrete box-girder, approach spans reinforced concrete slab, spillthrough abutments. Elliptical revetted earth embankment spur dikes, 150 ft (45 m) in length upstream from abutments. Crossing is at wide flood plain, densely forested.

Hydraulic problem and countermeasure: After the bridge was constructed in 1958, abutment fill was eroded by heavy concentration of flow through a cleared area between the spur dike end and the flood plain forest. In addition, an elongate scour hole about 100 ft (30 m) long, 50 ft (15 m) wide, and 10-15 ft (3-5 m) deep formed at the end of the right spur dike (fig. 171). Willows were planted in the cleared area. An extreme flood occurred in 1973, which widened and deepened the bridge waterway, but no further scour was reported at the abutment. The scour hole remains at the spur dike, but the willows were evidently successful in reducing velocities at the abutment.

Discussion: Cleared areas on generally forested flood plains are a fairly common cause of flow concentration and consequent erosion damage at bridges. Large scour holes at the ends of spur dikes are also commonly reported, although these holes are generally regarded as harmless.

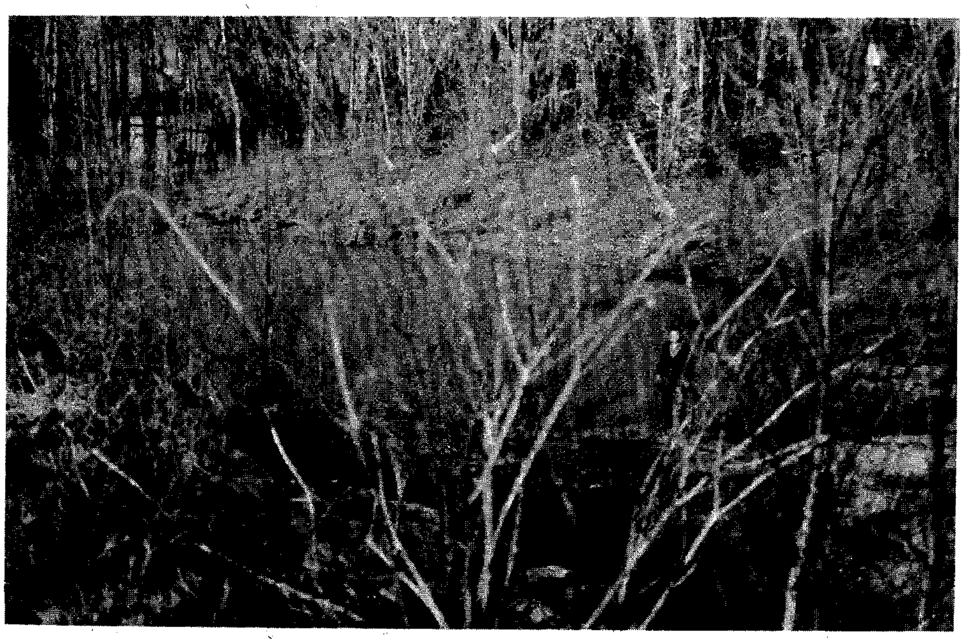


Figure 171. View from end of spur dike (foreground) toward scour hole (background) in March, 1962.

SITE 222. TIPPAH RIVER AT US-78 NEAR POTTS CAMP, MISS.

Description of site: Lat 34°40', long 89°19', location as shown in fig. 172. Bridge is 1,195 ft (364 m) in length, main span 75-ft (23-m) wide flange beam, approach spans steel I-beam, concrete piers in channel, timber-pile bents at approach spans.

Drainage area, 244 mi^2; valley slope, 0.00075; width of natural channel, about 75 ft (23 m), increased to about 225 ft (69 m) following channelization. Stream is perennial, alluvial, sand bed, in valley of low relief, wide flood plain. Natural channel meandering, straightened by channelization, silt-sand banks.

Hydraulic problem and countermeasure:

1937 Bridge built, without countermeasures.

1966 Stream channelized by Soil Conservation Service (fig. 172). According to design plans, new channel had 105-ft (32-m) bottom width, 11-ft (3-m) depth, 1:1 side slopes, and a capacity of 7,700 ft^3/s (218 m^3/s) at an average velocity of 7 ft/s (2 m/s). Banks were riprapped for a distance of 100 ft (30 m), mainly upstream from bridge, and riprap was placed on the channel bottom in the bridge waterway. Within a few months, during which no flows occurred above bankfull, the channel widened downstream from the bridge and the bed was lowered about 8 ft (2.4 m). This erosion is attributed to overfall from the riprap placed on the channel bottom at the bridge.

1968-69 Two minor floods, each of 2- to 3-yr R. I. Riprap at the south main pier failed to prevent local scour, which occurred to a depth of about 7 ft (2 m) below the channel bottom as constructed. Width of channel at eroded area downstream from bridge increased to about 300 ft (90 m).

1971 Concrete jacks placed along bankline at bridge by Soil Conservation Service.

1975 Maximum width of channel immediately downstream from bridge measured at 320 ft (98 m) on aerial photograph, an increase of about 20 ft since 1969 (fig. 172). Concrete jacks apparently controlling bank erosion, although the channel may have reached an equilibrium width.

Discussion: As a result of channelization, the average channel width in the vicinity of the bridge increased from about 75 ft (23 m) to about 225 ft (69 m). Some general degradation probably occurred, but this has not been confirmed. The riprap placed at the bridge at the time of channelization has evidently prevented channel widening immediately upstream from the bridge, but has caused a widening of the channel immediately downstream from the bridge. Performance of the concrete jacks cannot be reliably evaluated with available information.

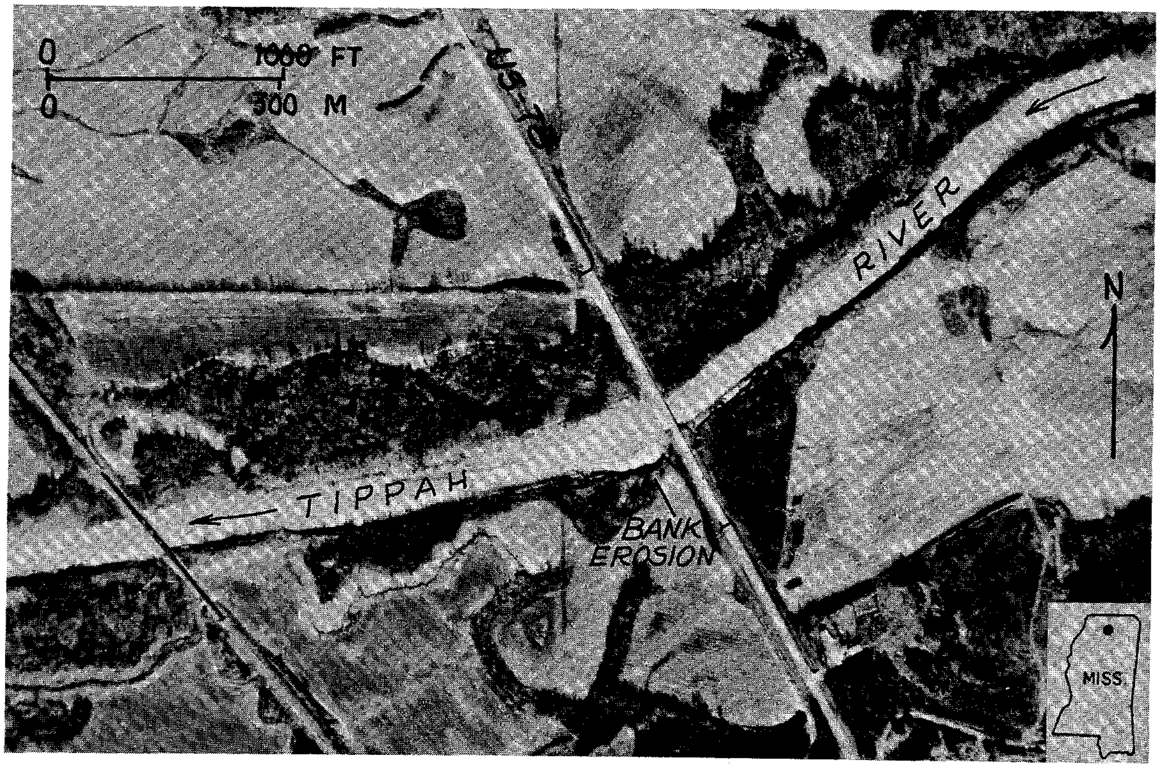


Figure 172. Aerial photograph of channelized Tippah River at US-78 crossing. (From U.S. Dept. of Agriculture.)

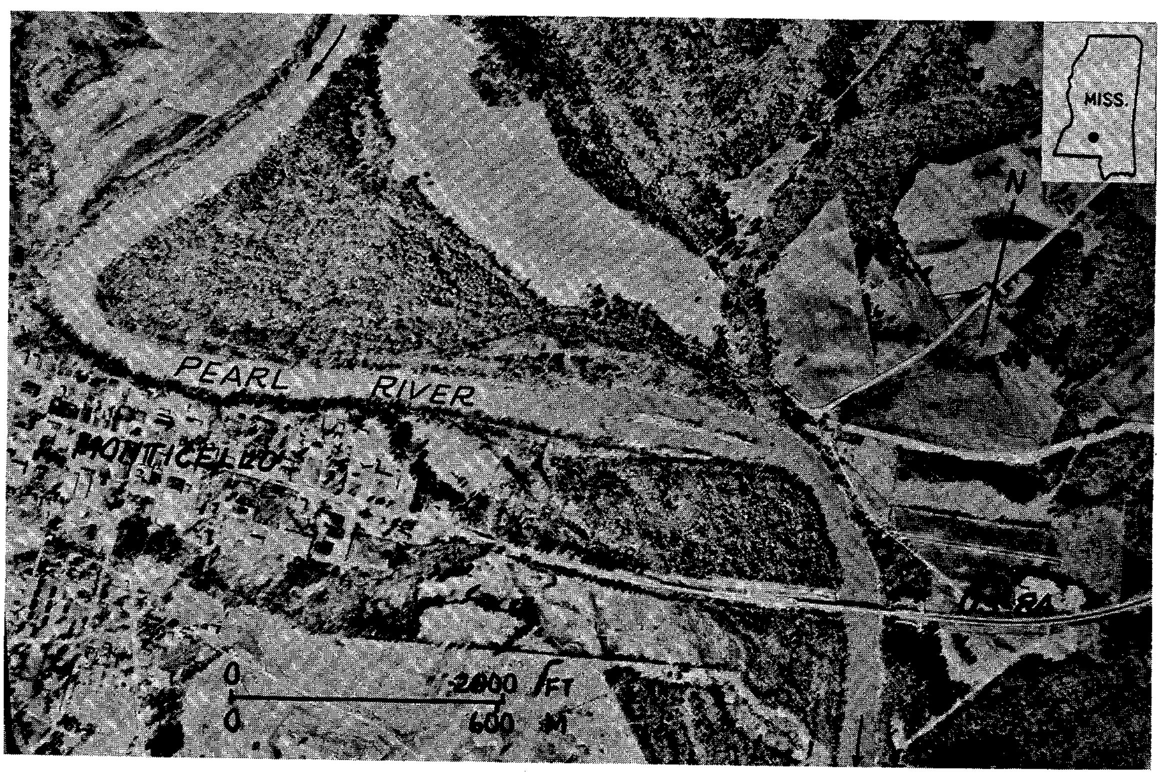


Figure 173. Aerial photograph of Pearl River at US-84 crossing, on February 28, 1973. (From Mississippi Dept. of Highways.)

SITE 223. PEARL RIVER AT US-84 AT MONTICELLO, MISS.

Description of site: Lat 31°33', long 90°05', location as shown in fig. 173. Bridge is 705 ft (214 m) in length, main span is steel girder on webbed concrete piers, approach spans are concrete girder on concrete-pile bents; spillthrough abutments. Crossing is at bend in stream.

Drainage area, 5,050 mi^2 (13,053 km^2); bankfull discharge, about 17,000 ft^3/s (481 m^3/s); valley slope, 0.0003; channel width, 300 ft (90 m). Stream is perennial, alluvial, sand bed, in valley of moderate relief, wide flood plain, well developed natural levees. Channel is meandering, equiwidth with deep narrow cross section (typical of streams with natural levees), cut banks local, bank material erodible silt-clay-sand, tree cover on banks, 50-90 percent (fig. 173).

Hydraulic problem and countermeasure:

1937 Bridge built, broken concrete riprap placed at abutment fill slopes.

1960's Lateral erosion of left bank in vicinity of bridge (fig. 174). Broken concrete riprap and several car bodies placed at bank.

1973-76 Several major floods, including one with peak discharge of 78,200 ft^3/s (2,213 m^3/s). Countermeasures installed in 1960's washed away, and left bank receded for a distance of about 10 ft (3 m), to bent 9 (fig. 175).

1976 Timber-pile retard, about 100 ft (30 m) in length, double row of piles, constructed at left bank, extending upstream from pier IV. Pile had good penetration (according to the district maintenance engineer) and, based on experience with similar retards elsewhere, was adequate for protection of bank.

1977 Four months after installation of retard, during which no flows above bankfull stage occurred, the left bank slumped and broke off the inside row of piles. The outside row was salvaged.

Discussion: Failure of the improvised countermeasures installed in the 1960's is not surprising, but failure of the timber pile retard is perplexing. Evidently, a retard that will withstand major floods is quite vulnerable to bank slumping. Slumping of the banks is attributed by the district highway engineer to a kind of long-term drawdown. The banks had become highly saturated by high river stages during 1973-76, and flows during the late fall and winter of 1976-77 were abnormally low.

The lateral migration rate of the river is judged to be rather less than average, in view of its size and meandering habit, but some lateral erosion is to be expected at the outside of a bend.

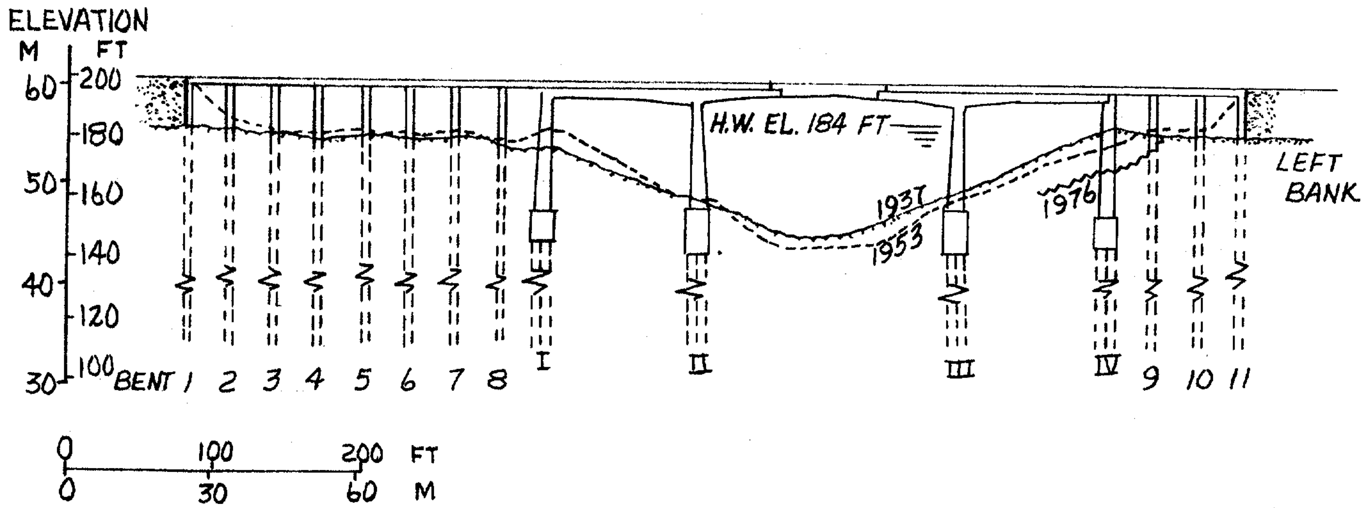


Figure 174. Cross section at US-84 bridge, Pearl River.

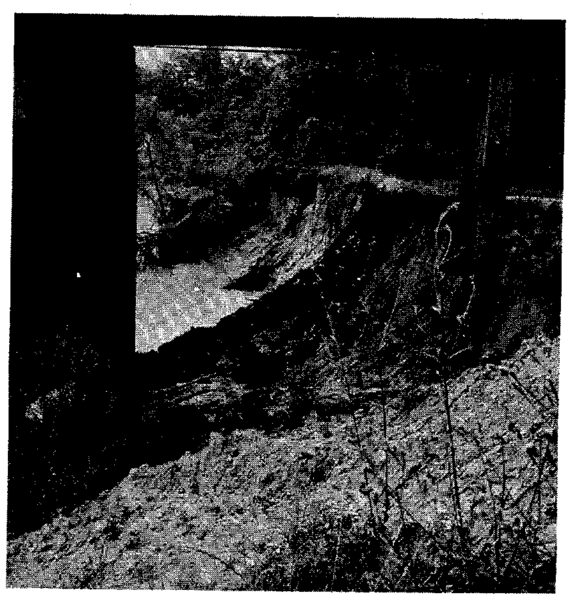


Figure 175. Upstream view of eroded bank at bent 9, US-84 bridge, on June 1, 1976.

SITE 224. HOOKER HOLLOW CREEK AT US-84 NEAR PRENTISS, MISS

Description of site: Lat 31°35', long 89°57'. Bridge is 80 ft (24 m) in length, four 20-ft (6-m) wide-flange steel-stringer spans, timber-pile bents, vertical abutments.

Drainage area, about 12 mi^2 (31 km^2); channel width, about 60 ft (18 m). Stream is perennial, alluvial, sand-gravel bed. Channel is sinuous.

Hydraulic problem and countermeasure:

1937 Bridge built, without countermeasures.

1961 Extreme flood that overtopped roadway, but no damage was reported at bridge.

1973 Channel straightening done by Soil Conservation Service in reach just downstream from bridge.

1974 Extreme flood, left abutment undermined, erosion extended beneath roadway. Timber bulkhead was built by Highway Department on upstream side of left abutment, and left bank was revetted with car bodies (figs. 176 and 177A).

1975 Moderate flood, fill eroded behind right abutment on downstream side. Timber bulkhead constructed on downstream side of right abutment (fig. 177B).

1976 Bulkhead at left abutment leaning streamward, likely to fail during flood. Car-body revetment performing satisfactorily. Vegetation was becoming established along straightened channel downstream from bridge.

Discussion: Timber bulkhead at left abutment proved too weak to withstand turbulence at outside of bend. Inasmuch as there were no hydraulic problems at the bridge until the channel was straightened, the problems that exist are attributed to increased channel slope and consequent widening of the channel.

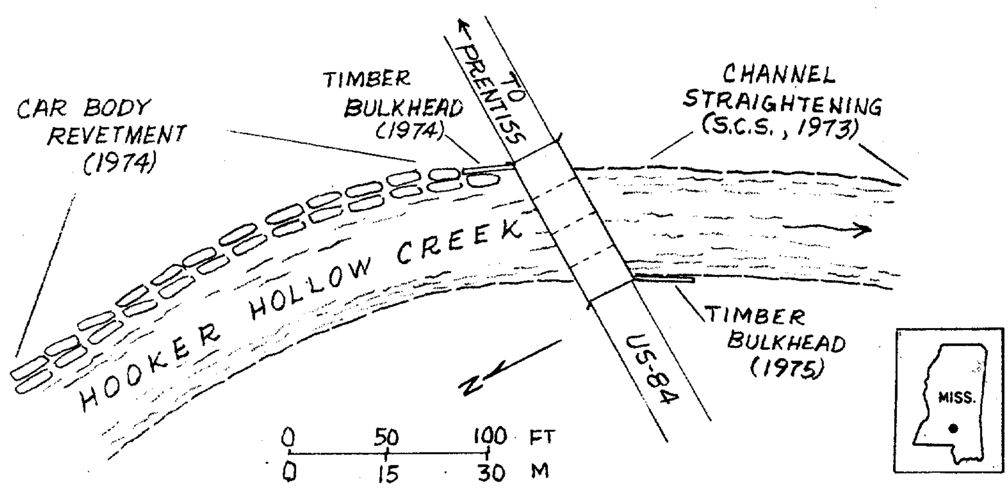


Figure 176. Plan sketch of Hooker Hollow Creek at US-84 crossing.

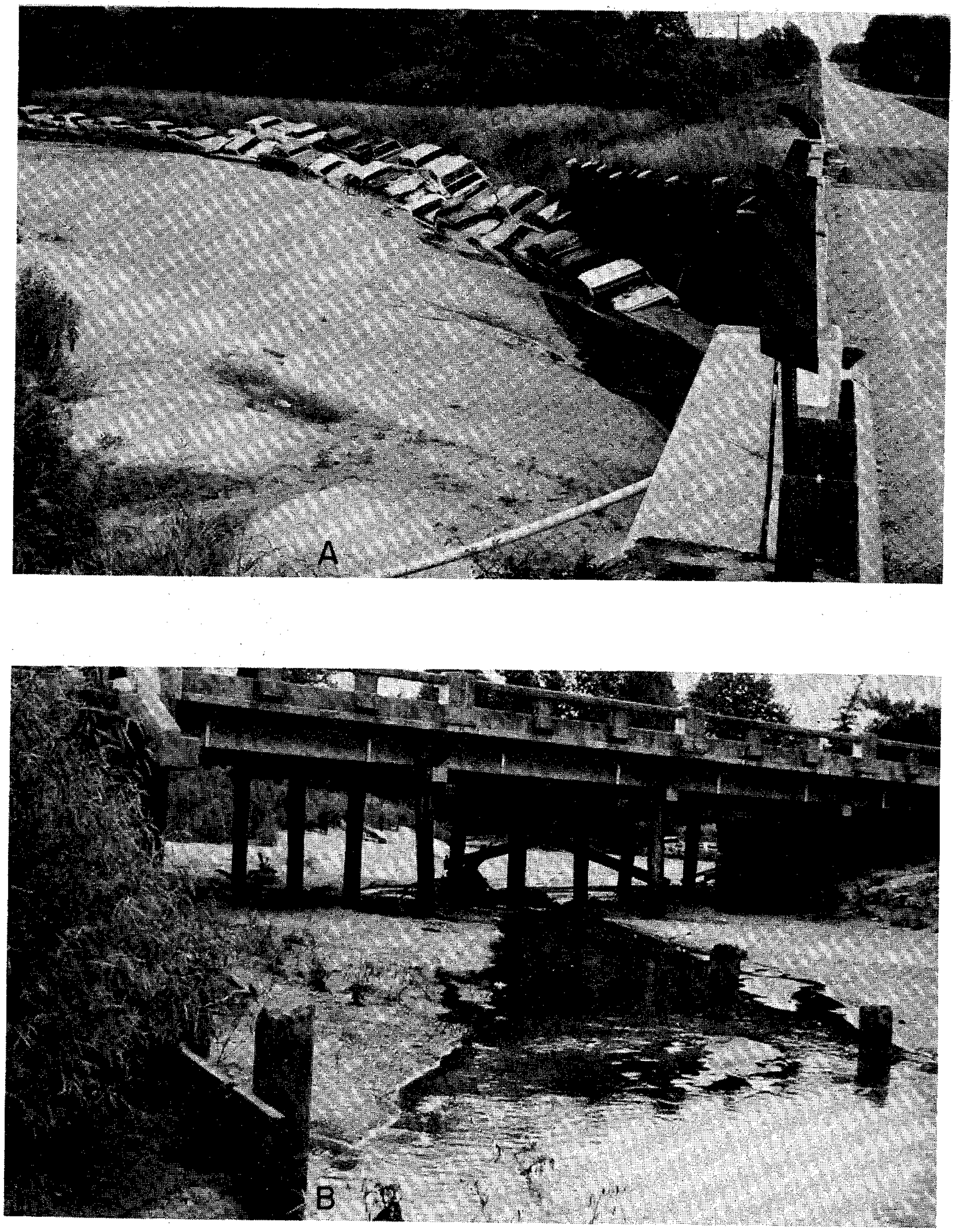


Figure 177. A, upstream view from bridge of timber-pile bulkhead and car-body revetment along left bank of Hooker Hollow Creek, on June 1, 1976. B, upstream view toward bridge showing bank recession at right abutment (left side of photo).

SITE 225. PEARL RIVER AT US-98 AT COLUMBIA, MISS.

Description of site: Lat 31°14', long 89°51', location as shown in fig. 178. Bridge is 697 ft (212 m) in length, main span is 450-ft (137-m) cantilevered plate girder, concrete piers and pile bents on timber-pile foundations, spillthrough abutments. Crossing is at nearly straight reach of channel, but upstream bend deflects flow toward right bank.

Drainage area, 5,690 mi^2 (14,737 km^2); bankfull discharge, about 24,000 ft^3/s (679 m^3/s); valley slope, 0.00015, channel width, about 450 ft (135 m). Stream is perennial, alluvial, sand bed, in valley of moderate relief, wide flood plain. Channel is meandering, random width variation, cut banks general, silt-sand banks. Numerous oxbow lakes on flood plain indicate frequent meander cutoffs, and bank stability has been affected by clearing of forested flood plain.

Hydraulic problem and countermeasure:

1933 Bridge built, riprap placed at abutment fill-slopes.

1948 Natural meander cutoff occurred upstream from bridge (fig. 178). Right bank began to erode rapidly, probably because of increase in local stream slope.

1949 Second cutoff made artificially by Mississippi Highway Department, at site of natural cutoff, in an effort to divert flow away from right bank.

1950 Spur, consisting of a triple row of timber pile, was constructed at an angle of 25° to the right bank about 1,000 ft (300 m) upstream from the bridge.

1953 Continued erosion of right bank (fig. 179). Two timber-pile spurs were constructed at right bank (fig. 180), upstream spur about 600 ft (180 m) from bridge. Riprap was placed around pile (fig. 181), timber sheathing on upstream face of spur, and poles (embedded about 1 ft or 0.3 m into ground) on downstream face. In addition, a small levee was built along the right bank upstream from the bridge; this levee served as a spur dike.

1967 Second bridge built on upstream side of older bridge, for dual highway. Construction required removal of downstream spur (fig. 180). Elliptical spur dike built at right upstream abutment, connected with existing levee.

1968-75 Major floods in 1974 (peak discharge, 94,400 ft^3/s, or 2,671 m^3/s) and in 1975 (peak discharge, 50,200 ft^3/s or 1,420 m^3/s). Vegetation has become established at upstream spur and bank is being protected at the spur (fig. 178A and B). However, bank erosion is continuing at the bridge (fig. 179).

Discussion: The present alinement of the river upstream from the bridge appears to be satisfactory, and it is not immediately apparent why the main channel should be shifting toward the right bank. Closer examination of the aerial photographs shows a minor concavity of the left bank upstream from the bridge, which is apparently deflecting flow toward the right bank at the bridge. The timber-pile spurs have performed well in protecting the right bank.

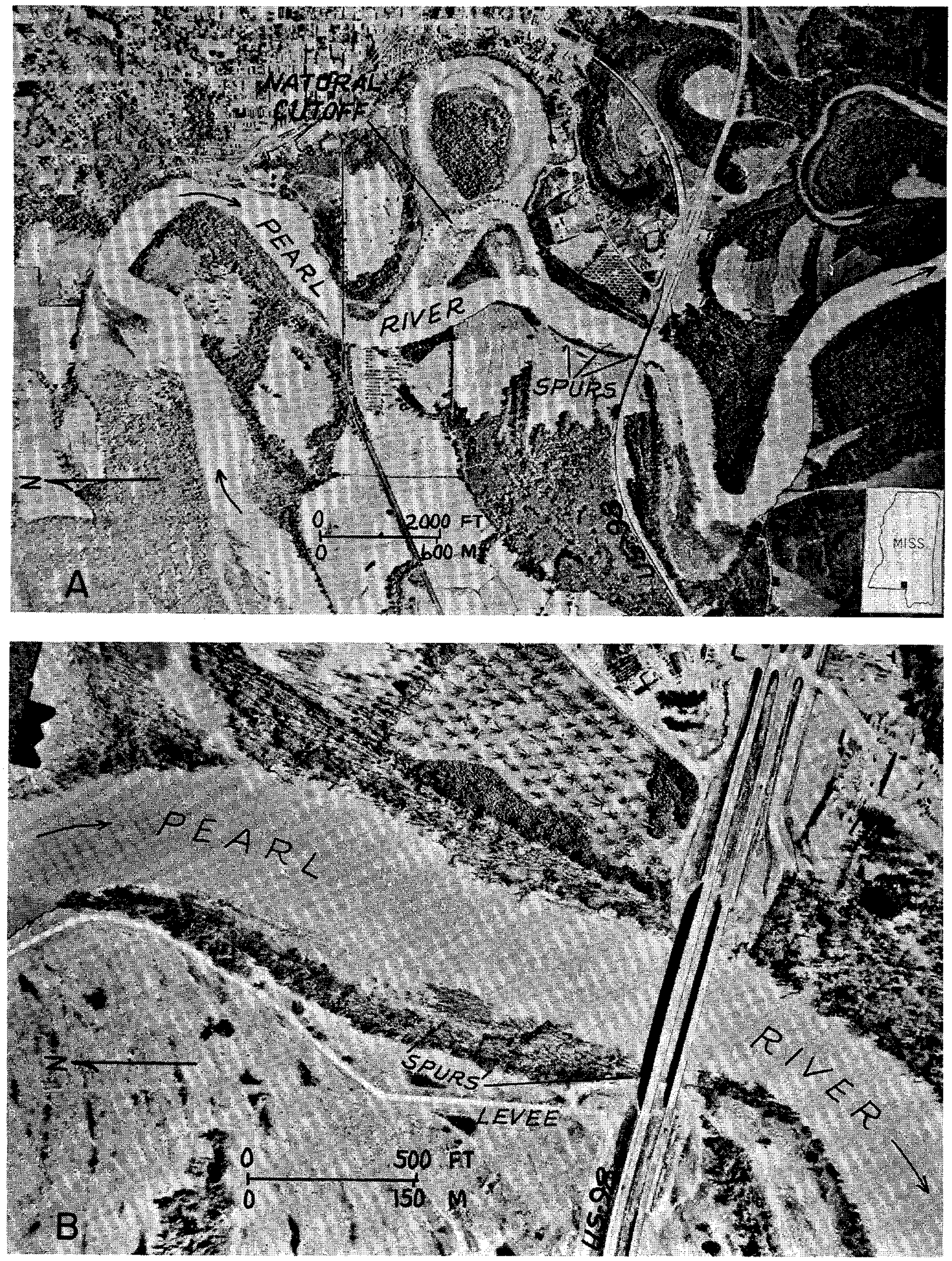


Figure 178. A, aerial photograph of Pearl River at US-98 crossing on December 15, 1964 (From U.S. Dept. of Agriculture). B, aerial photograph of US-98 crossing on February 15, 1973 (From Mississippi Dept. of Highways).

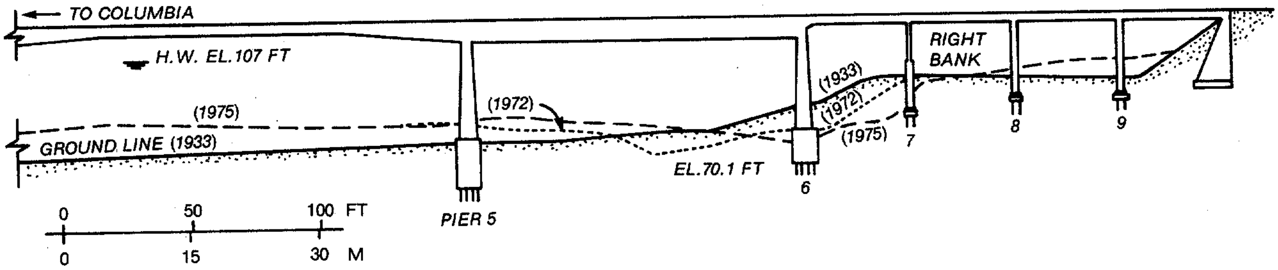


Figure 179. Bridge elevation sketch showing recession of right bank, 1933-75.

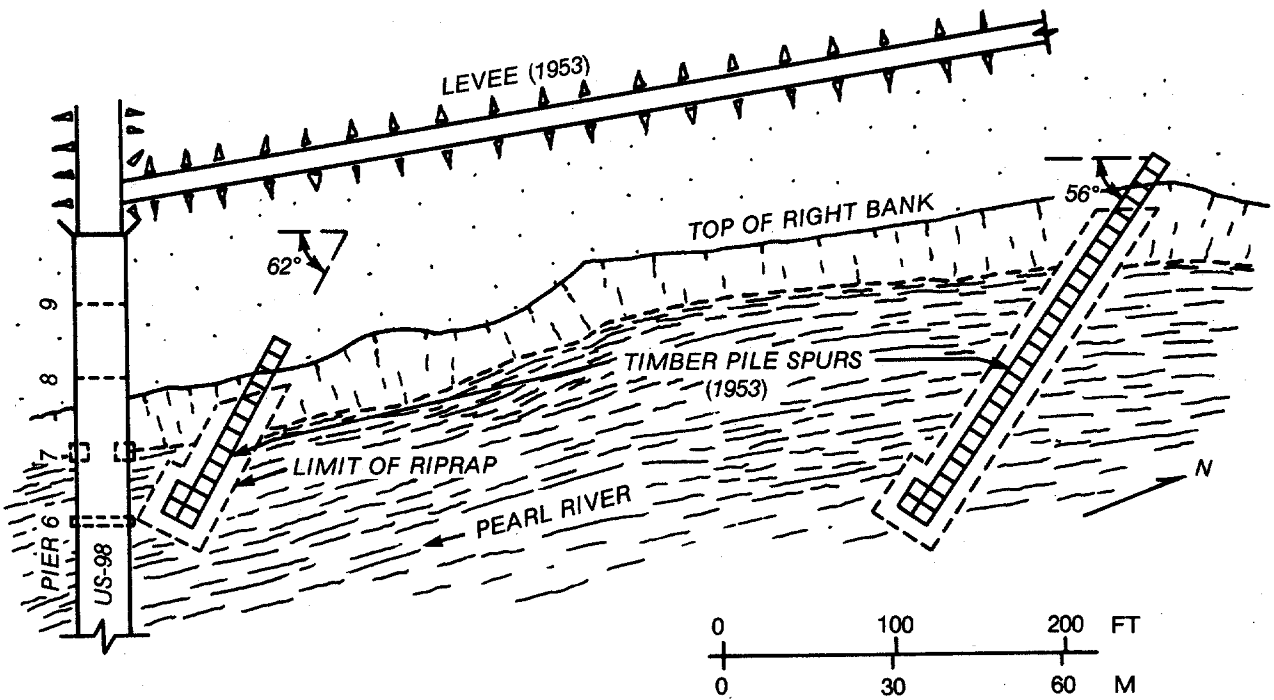


Figure 180. Plan sketch showing spurs and levee constructed in 1953.

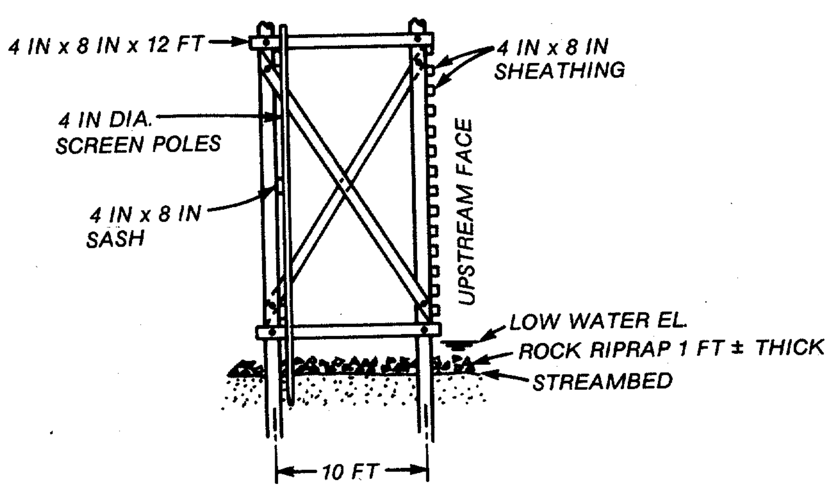


Figure 181. Typical cross section of spur constructed in 1953.

SITE 226. NODAWAY RIVER AT SR-"C" NEAR CLEARMONT, MO.

Description of site: Lat 40°31', long 95°04'. Bridge is 415 ft (126 m) in length, I-beam superstructure, three concrete web-type (dumbell) piers in channel, two concrete-pile bents, spillthrough abutments. Piers and abutments skewed 25° to bridge centerline, were alined with channel at time of construction but are now skewed to flood flow because of rapidly changing channel alinement (compare figs. 182A and B).

Drainage area, about 1,225 mi^2 (3,173 km^2); valley slope, 0.0005; channel width, about 270 ft (82 m). Stream is perennial, alluvial, sand-silt bed, in valley of low relief, wide flood plain. Channel is meandering, wider at bends, point bars, wandering thalweg, cut banks general, coherent but erodible silt-clay banks. The Nodaway River was straightened in 1911 by local drinage districts, but has subsequently widened and begun to meander. The channel is laterally unstable. Trees undercut at the banks, and also those cleared for agriculture, supply abundant amounts of drift to the river.

Hydraulic problem and countermeasure:

1948 Bridge built, without countermeasures.

1950-56 Spurs built to maintain channel alinement, one timber pile with tree trunks attached and the other an earth-and-rock embankment. Recurrent problem with accumulation of drift at bridge.

1959 Pilot channel dredged to realine stream.

1966 Existing rock spur repaired, two new spurs built, consisting of a double row of timber pile, rock filled (fig. 183).

1967-73 Spurs destroyed by lateral erosion, stream channel shifted drastically. Bank upstream from right abutment was severely eroded (fig. 182B), and abutment fill-slope was eroded such that abutment foundation pile are exposed. Failure of spurs is attributed partly to fact that timber pile cannot be driven to suitable depth in hard shale beneath streambed, and partly to poor quality of available riprap, which consists of limestone that tends to disintegrate. Even if the spurs had been effective, it appears that the channel would have shifted away from them.

1973 Rock retard, (called a "trail dike" by Missouri State Highway Commission; similar in position to a spur dike, but not located on flood plain) built along streambank upstream from right abutment (fig. 184). Retard was faced with limestone riprap, blocks up to 2 ft (0.6 m) in diameter. Similar riprap was placed on abutment fill slopes.

1976 Retard is performing satisfactorily, although some bank erosion and slumping is taking place at upstream end of retard, and stream might erode behind it.

Discussion: As a result of straightening carried out about 65 years ago, the Nodaway River now has a wide channel with a wandering thalweg, which is among the most unstable and unpredictable of channel types. Spurs are not likely to be effective in maintaining alinement of a channel of this type, unless they are well built, deployed in substantial numbers, and located on both banks. The rock retard at the Route "C" bridge will probably be effective in protecting the right abutment, but the stream may rapidly shift its point of attack.

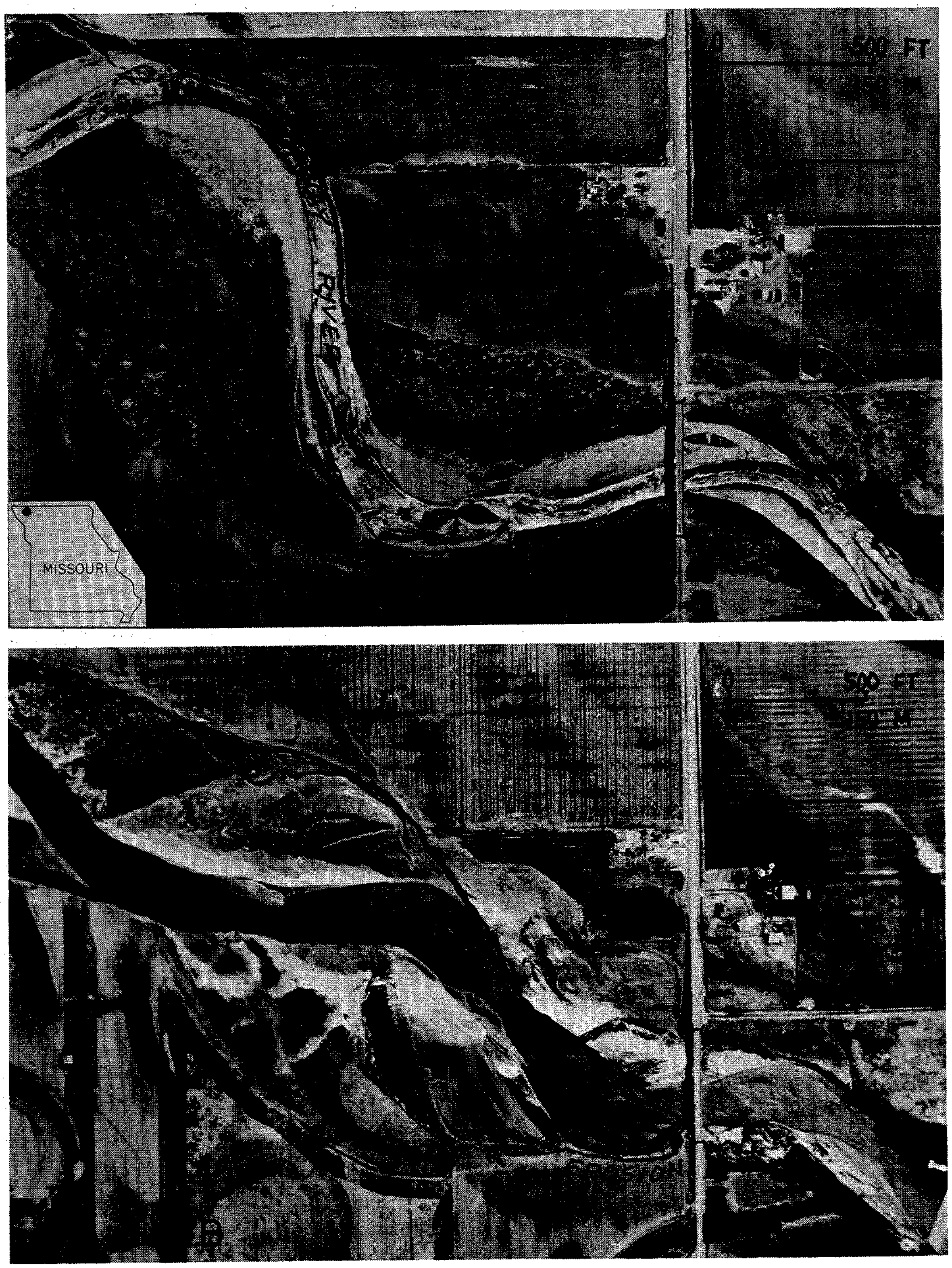


Figure 182. A, vertical aerial photograph of Nodaway River at SR-"C" crossing, February 25, 1967. B, same site on February 28, 1973. (Missouri Highway Commission photographs.)

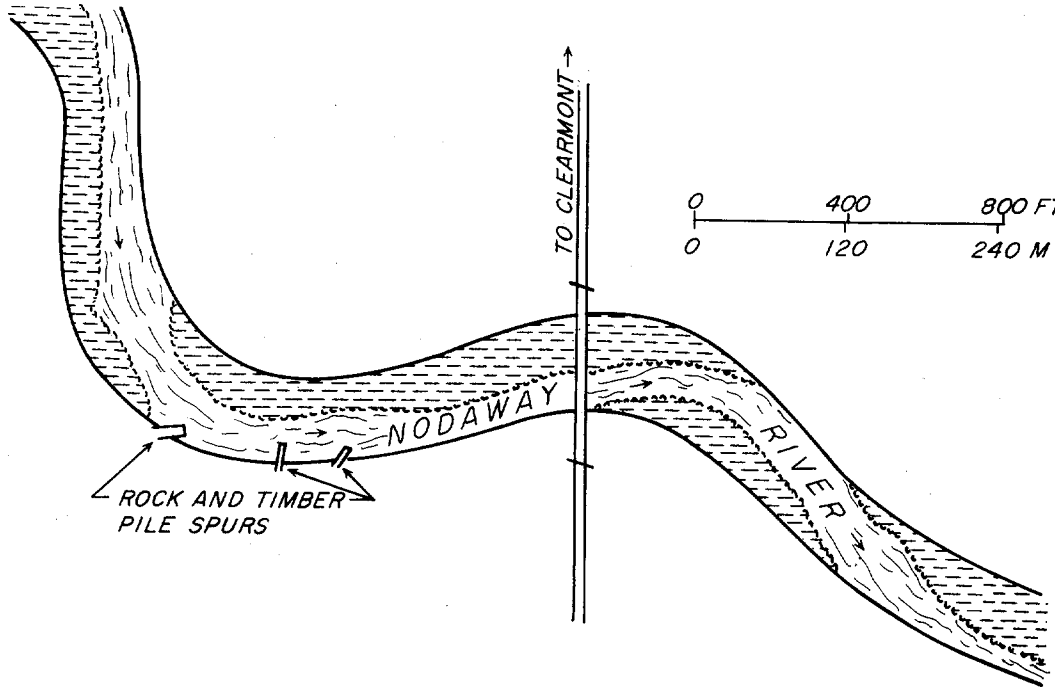


Figure 183. Sketch showing location of spurs at SR-"C" bridge, on February 25, 1967.

Figure 184. Upstream view from right abutment of SR-"C" bridge on June 8, 1976, showing retard (left) and aspect of channel.

SITE 227. MIDDLE FORK GRAND RIVER AT SR-46 NEAR GRANT CITY, MO.

Description of site: Lat 40°29', long 94°23'. Bridge is 293 ft (88 m) in length, one 80-ft (24-m) pony-truss span and six 35-ft (10.5-m) I-beam spans, main span supported by two concrete web type (dumbbell) piers on pile foundations, other spans supported by timber-pile bents. Spillthrough abutments. Crossing is at bend in channel (fig. 185).

Drainage area, about 25 mi^2 (65 km^2); valley slope, 0.0014; channel width, about 230 ft (69 m). Stream is perennial, alluvial, sand-silt bed, in valley of low relief, wide flood plain. Channel is sinuous (previously straightened), incised, cut banks general, silt-clay banks. As a result of the channel straightening, done by local drainage districts probably in the 1920's, degradation and lateral bank erosion has occurred.

Hydraulic problem and countermeasure:

1930 Bridge built. Light rock riprap placed at abutment slopes, no other countermeasures on record.

1967 Channel had degraded and had shifted west of main span. Footings and foundation pile of pier 6 exposed, and channel bed eroded such that only 8 ft (2.5 m) of penetration remained at bents 4 and 5.

1968-72 Two supplementary steel H-pile driven at bent 4 and at bent 5. Retard of dumped rock riprap built at west bank upstream from bent 5 (fig. 186).

1976 Retard has shifted channel to east, such that pier 6 is now in center of low water channel (fig. 187). Channel degradation at pier 6 since 1930 amounts to about 19 ft (6 m).

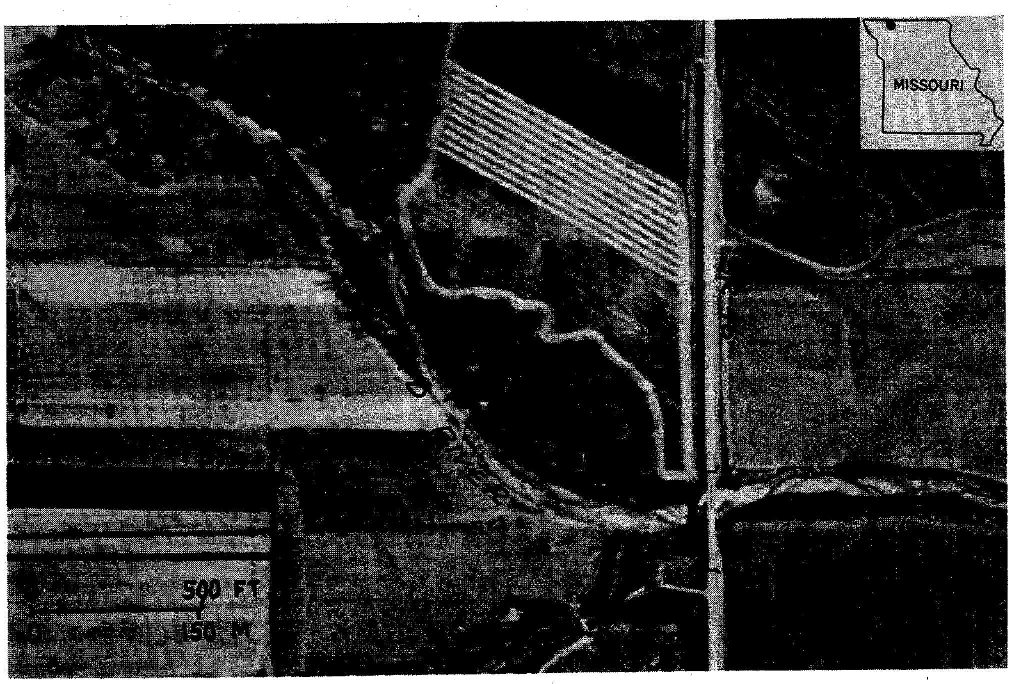


Figure 185. Vertical aerial photograph of Middle Fork Grand River at SR-46, November 19, 1967. (Missouri Highway Commission photograph.)

Discussion: Channel degradation usually occurs most rapidly within a few years after stream equilibrium has been disturbed (as by straightening) and thereafter progresses at a gradually decreasing rate. Degradation at this site may have neared its limit, and the rock retard has apparently been successful in diverting the flow away from the west bank.

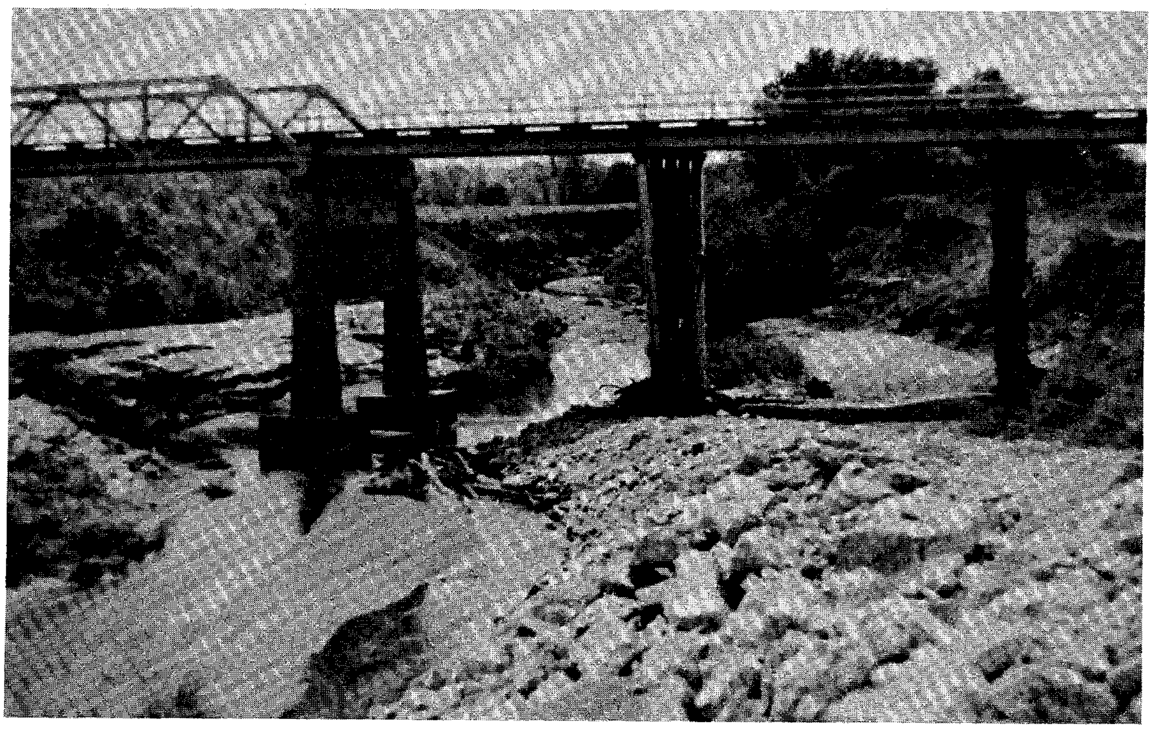


Figure 186. Downstream view of SR-46 bridge on May 26, 1976, showing rock retard (at right foreground), channel degradation at pier 6, and supplementary pile at bent 5. (Missouri Highway Commission photograph.)

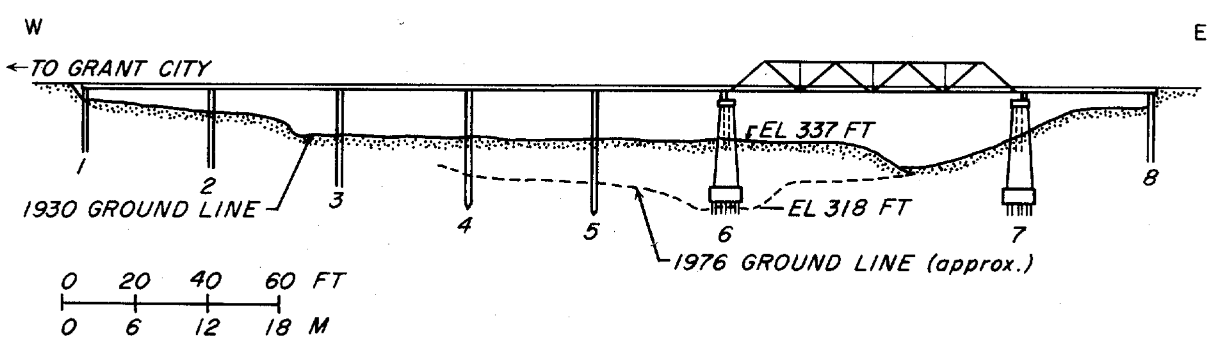


Figure 187. Elevation sketch of SR-46 bridge.

SITE 228. MISSISSIPPI RIVER AT I-155 NEAR CARUTHERSVILLE, MO.

Description of site: Lat 36°07', long 89°36', location as shown in fig. 188. Bridge built 1969-76, length of main spans, 3,590 ft (1,094 m); seven concrete piers in channel, square-column concrete-pile bents on flood plain.

Drainage area, about 925,000 mi^2 (2,395,750 km^2); valley slope, 0.00015; width 4,600 ft (1,400 m); stream is perennial, alluvial, sand bed, in valley of low relief, wide flood plain. Channel is meandering, locally braided, random width variation, cut banks general, silt-sand banks.

Hydraulic problem and countermeasure: Problem is lateral erosion of right bank (while bridge was under construction), which exposed top of footing of bent 14 (figs. 189 and 190); bent was originally on flood plain. According to the U.S. Army Corps of Engineers, Memphis District, the river bank at this location receded about 100 ft (30 m) during period 1964-68 and an additional 120 ft (37 m) during the period 1971-73. A flood (R. I. about 30 yr) which occurred in the spring of 1973 caused much of the lateral erosion. As a countermeasure, a blanket of dumped rock riprap of somewhat smaller size than class II (75-90 percent in the weight range of 60-300 lb or 27-136 kg) was placed at the pile bent (fig. 190). The blanket was rectangular in shape, measuring 225 by 110 ft (69 by 34 m), 3 ft (1 m) in thickness, and keyed in to a depth of 5 ft (1.5 m) at the upstream end.

A measure for protection against pier scour was placed at two main piers during bridge construction. This consisted of a blanket of dumped rock riprap extending from the streambed to a depth of 5 ft (1.5 m). The pier at the left bank is protected by river control measures previously installed by the Corps of Engineers.

Discussion: The lateral erosion at bent 14 occurred on the inside of a bend and was therefore not anticipated. Because of the random width variation of the Mississippi in this reach and its locally braided habit, the locale of probable bank erosion is more difficult to predict than for meandering rivers of more regular pattern.

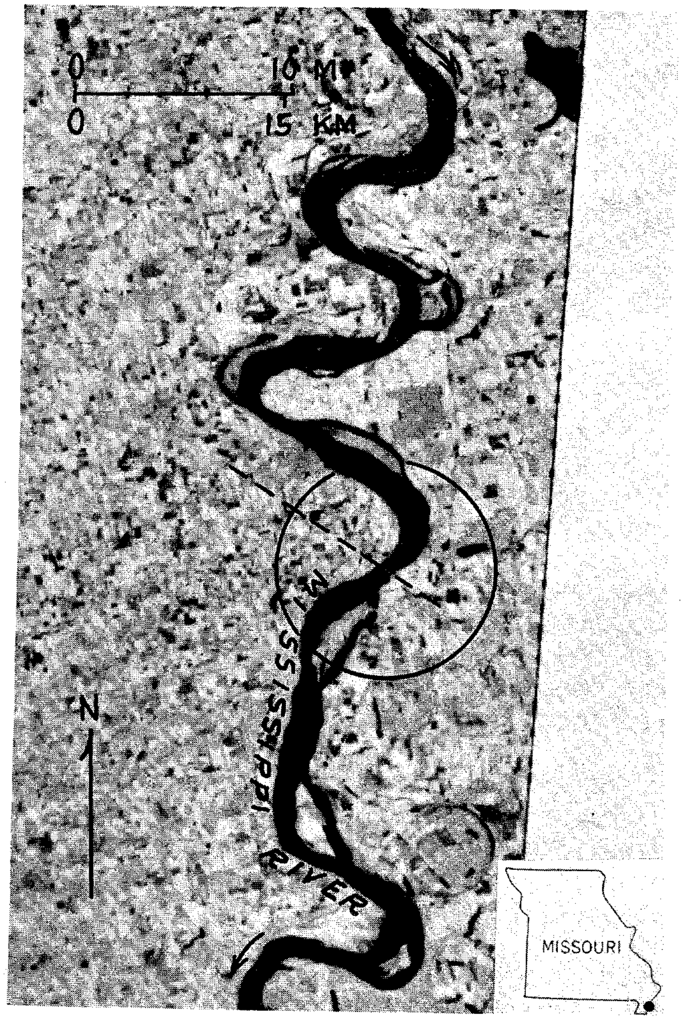


Figure 188. Multispectral scanner image (from landsat I) of Mississippi River at I-155 crossing, October 2, 1972. (National Aeronautics and Space Administration image.)

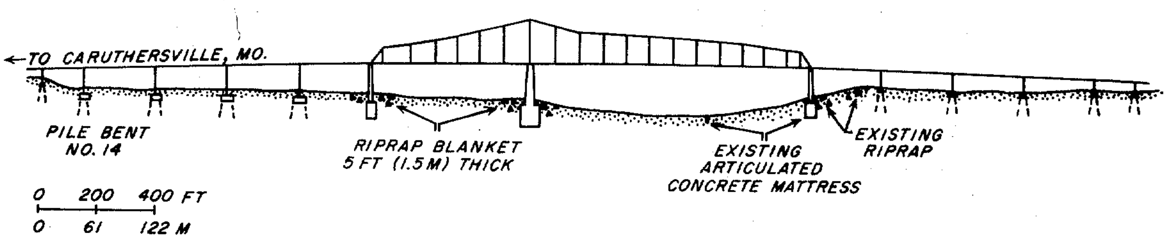


Figure 189. Elevation sketch of I-155 bridge, Mississippi River at Caruthersville, Mo., showing location of bent 14, exposed by lateral erosion, and type of scour countermeasure provided at main piers.

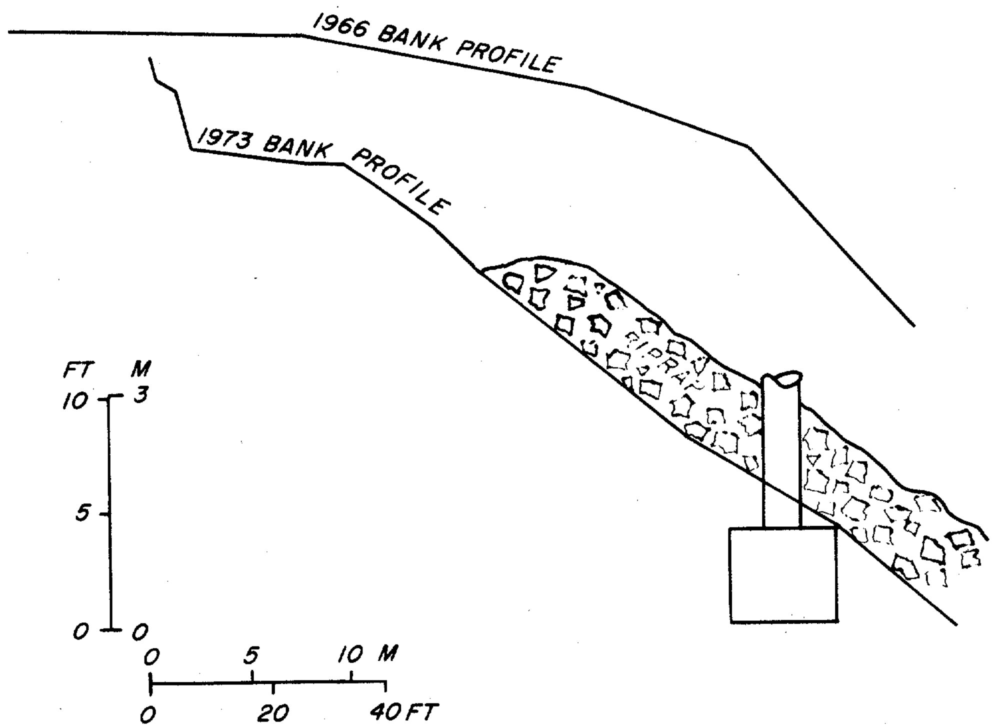


Figure 190. Top, sketch showing exposure of column at bent 14 by lateral bank erosion during the period 1966-73. Bottom, downstream view of bent 14 and riprap at bent. (From Missouri Dept. of Highways.)

SITE 233. KENNEDY CREEK AT US-89 NEAR BABB, MONT.

Description of site: Lat 48°55', long 113°26', location as shown in fig. 191. Bridge is 123 ft (37.5 m) in length, single wall-type pier in channel. Spillthrough abutments have 1.5:1 slope ratio, slope protected with rounded boulder riprap. Pier and abutments skewed to flood flow at about 40°.

Drainage area, 61 mi^2 (158 km^2); valley slope, 0.015. Stream is perennial, alluvial, gravel bed, on alluvial fan at valley side of Saint Mary River (fig. 191). Channel is sinuous, locally braided, not incised, non-coherent gravel banks.

Hydraulic problem and proposed countermeasure:

1961 Bridge built, designed for 50-yr R. I. flood, peak discharge about 5,000 ft^3/s (141 m^3/s, mean velocity in bridge waterway, 11.2 ft/s (3.4 m/s). Countermeasures as described above.

1964 Spring flood, R. I. estimated at 50-100 yr, estimated peak discharge in the range of 8,000-15,000 ft^3/s (226-424 m^3/s). Approach embankment at left abutment washed out, bridge closed for repairs.

1975 Spring flood, peak flow about 5,000 ft^3/s (141 m^3/s) Approach embankment at right abutment washed out, abutment subsided (figs. 192, 193, and 194). Bridge closed, bypass constructed. The pre-construction division of the Montana Department of Highways recommended that the bridge be replaced by a new bridge offset to the south and protected by spur dikes and riprap along the banks (fig. 193). The skewness of pier and abutments at the existing bridge reduces effective waterway area by about 50 percent, and the abutment fill-slopes are too steep for maintenance of riprap protection. Flood flow in Kennedy Creek is considered to be supercritical.

1976 New bridge and spur dikes under construction.

Discussion: Crossing sites on alluvial fans are commonly troublesome because of the tendency of channels on fans to shift laterally. The strategy of crossing the fan at its apex, where the potential for lateral shift is least, is not feasible here because the apex of the fan is inset into the mountain front (fig. 191). Inasmuch as the channel of Kennedy Creek is somewhat incised into the fan at the crossing site, the proposed spur dikes may solve the problem.

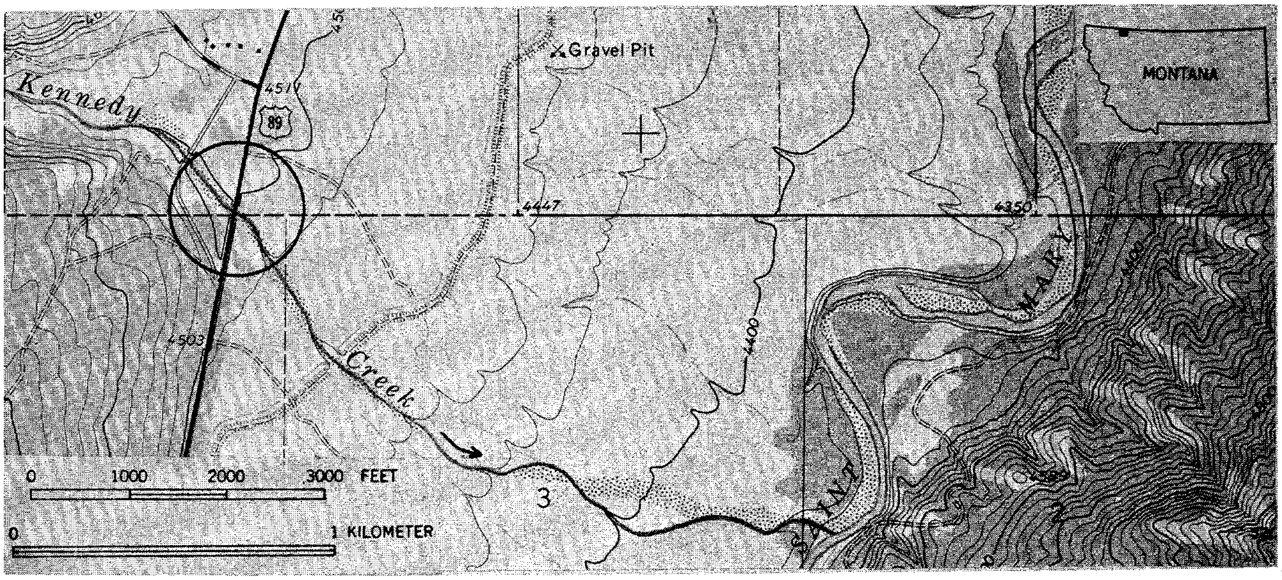


Figure 191. Map showing Kennedy Creek at US-89 crossing (circled). (Base from U.S. Geol. Survey Pike Lake, Mont.-Alta., 7.5' quadrangle, contour interval 20 feet, 1967.)

Figure 192. Aerial photograph of July 8, 1975, of US-89 crossing site at Kennedy Creek. (From Montana Dept. of Highways.)

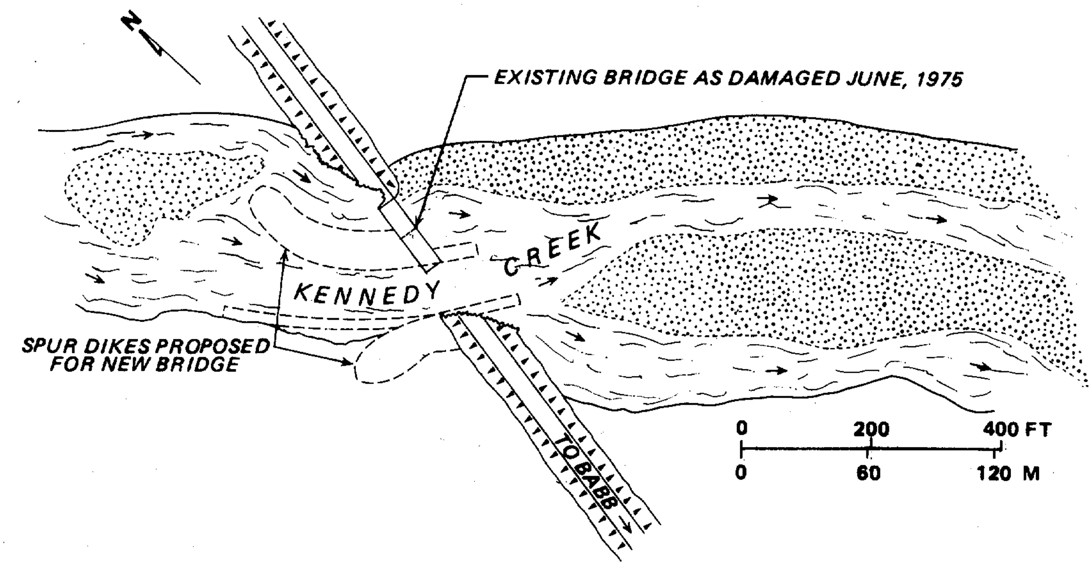


Figure 193. Plan sketch showing 1975 flood damage at US-89 crossing site. Also shown, in dashed lines, are spur dikes proposed for new bridge.

Figure 194. View, looking north, of 1975 flood damage at US-89 crossing site. (From Montana Dept. of Highways.)

SITE 234. MARIAS RIVER AT US-91 NEAR SHELBY, MONT.

Description of site: Lat 48°26', long 111°53', location as shown in fig. 195. Bridge is 547 ft (167 m) in length, 5 wall-type piers in channel, spillthrough abutments. Crossing is at flat apex of meander loop, the limbs of which are migrating rapidly. Piers alined with normal flow, but skewed to flood flow.

Drainage area, 2,610 mi^2 (6,760 km^2); valley slope, 0.006; channel width, about 215 (66 m). Stream is perennial, partially regulated, alluvial, gravel bed, in valley of moderate relief, narrow flood plain. Channel is meandering, locally braided (fig. 196), wider at bends, point bars, not incised, cut banks general, erodible sand-gravel banks.

Hydraulic problem and countermeasure:

1936 Bridge built, no countermeasures on record.

1964 Severe flooding, compounded by failure of two irrigation dams and a community water-supply dam. Peak discharge at bridge, 241,000 ft^3/s (6,820 m^3/s), about six times previously recorded high discharge during 50 years of record. Approach embankments of bridge were breached. During this flood, or a few years previously, a large meander loop was cut off naturally upstream from the bridge (fig. 195), and the consequent increase in channel slope caused rapid erosion of the right bank upstream from the bridge.

1968-74 Progressive erosion of right bank upstream from bridge threatened approach roadway and worsened alinement of flood flow at bridge. Three spurs installed at right bank, intended as temporary measures (fig. 196). Timber-pile structures, alined parallel with piers, installed in stream channel upstream from bridge for improvement of flow alinement.

1975 Flood, R. I. estimated at 100 to 200 yr., peak discharge about 70,000 ft^3/s (1,981 m^3/s). Further erosion of right bank and partial breaching of approach embankment (figs. 197 and 198).

1975-76 Dumped rock riprap, approximately equivalent to class II, placed along right bank for distance of several hundred feet, where approach roadway is threatened.

Discussion: The Marias River has geomorphic characteristics that are associated with rapid lateral migration (meandering, point bars, raw vertical banks) and migration rates for rivers tend to be most rapid downstream from a recent meander cutoff. During floods, a substantial amount of flow cuts across the necks of meander loops, and the bridge has probably escaped damage because the approach roadway embankment has been kept low to permit overspill. A good crossing site is difficult to select, but the present site has one advantage: the left bank is against the bedrock of the valley side. If the right bank can be held with riprap, it will probably be possible to hold the river against the left bank, with which the piers are alined. Local reaches of meandering streams may remain in position along the valley side for substantial periods of time. Little information was obtained about the timber-pile structures in the channel upstream from the bridge, and in any case it would be difficult to evaluate their effectiveness. No scour at piers was reported.

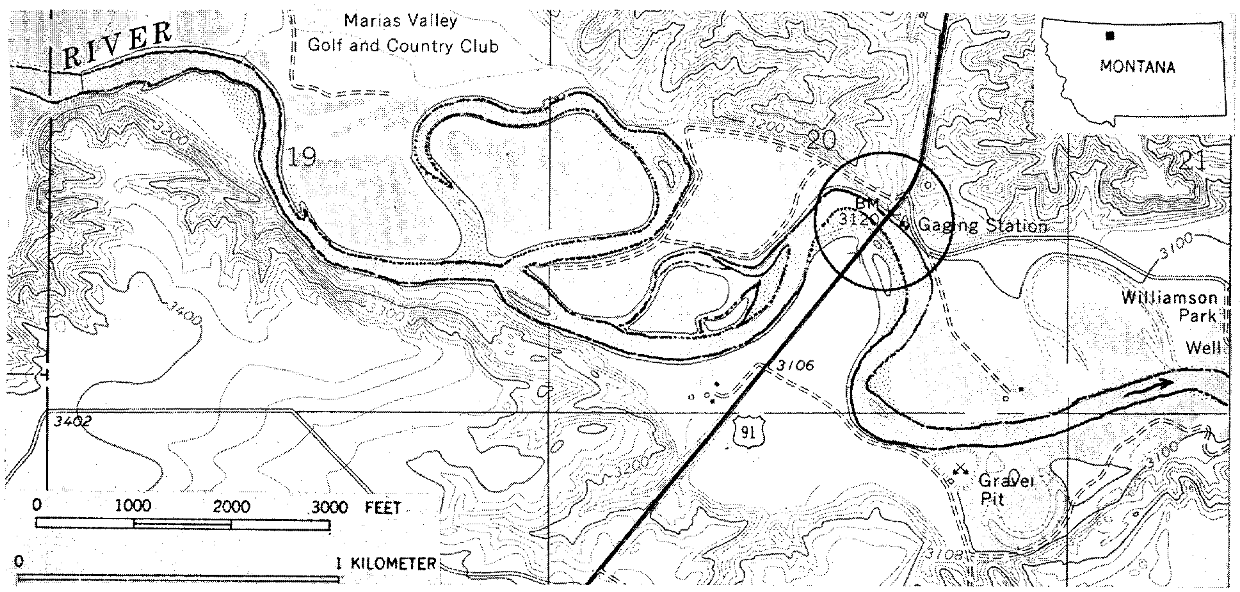


Figure 195. Map showing Marias River at US-91 crossing (circled). (Base from U.S. Geol. Survey Ledger NW, Mont., 7.5' quadrangle, contour interval 20 ft, 1966.)

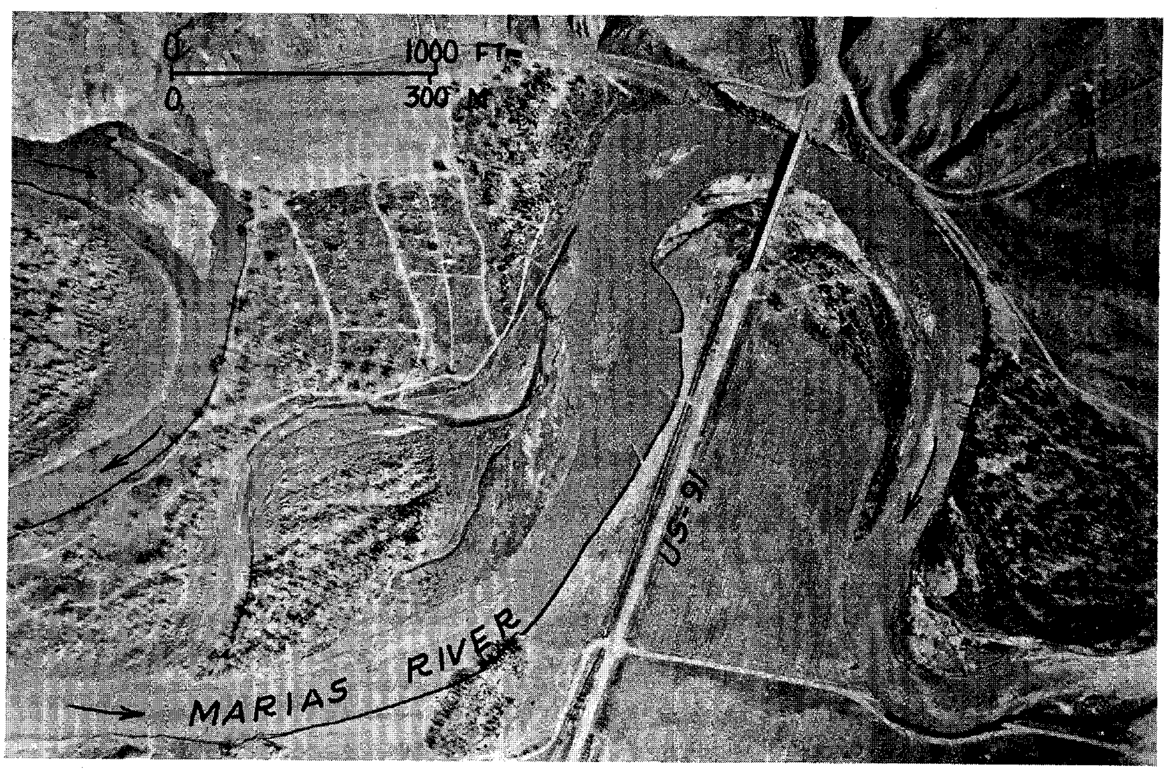


Figure 196. Vertical aerial photograph of Marias River at US-91 crossing on April 12, 1969. (From Montana Dept. of Highways.)

Figure 197. Oblique aerial photograph of Marias River at US-91 crossing, looking downstream, on June 21, 1975. (From Montana Dept. of Highways.)

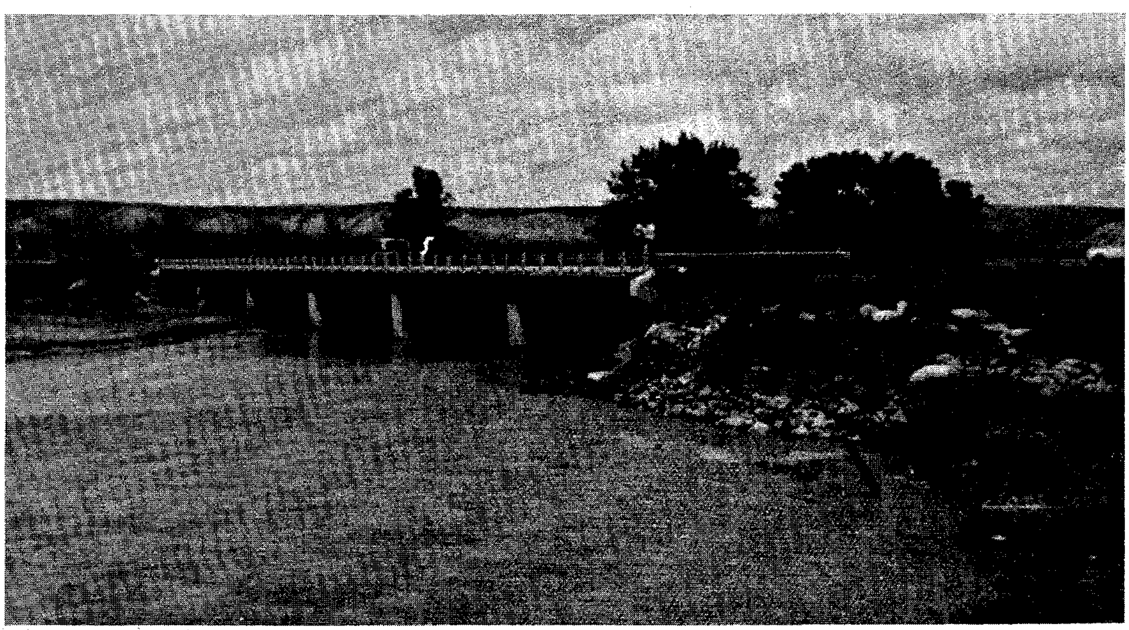


Figure 198. View of eroded right bank upstream from bridge, prior to major installation of riprap. (From Montana Dept. of Highways.)

SITE 235. TETON RIVER AT SR-223 NEAR FORT BENTON, MONT.

Description of site: Lat 47°51', long 110°43', location as shown in fig. 199. Bridge is 299 ft (91 m) in length, two concrete wall-type piers (with pointed nose) in channel, spillthrough abutments.

Drainage area, about 1,800 mi^2 (4,662 km^2); valley slope, 0.0028; channel width, about 100 ft (30 m). Stream is perennial, alluvial, gravel bed, in valley of moderate relief, wide flood plain. Channel is meandering, locally braided, wider at bends, point bars.

Hydraulic problem and countermeasure:

1953 Bridge built, no countermeasures known.

1964 Severe flood, peak discharge 5.7 times the 50-yr R. I. flood. Erosion at abutment fill-slopes and approach embankments, but bridge not damaged.

1965-76 Stream channel has shifted toward right (east) abutment, flows along roadway embankment before entering bridge waterway (fig. 200). Problem is recurrent erosion of abutment fill and exposure of abutment columns. Large riprap (and also car bodies) have been placed at abutment from time to time, but problem continues because of stream alinement.

Discussion: Streams having the geomorphic characteristics of the Teton River (meandering, locally braided, raw vertical banks, wide point bars at bends) tend to shift laterally, either by meander migration or shift of braids. It seems that flow control measures, such as spurs, are needed to improve the alinement of the flow approaching this bridge.

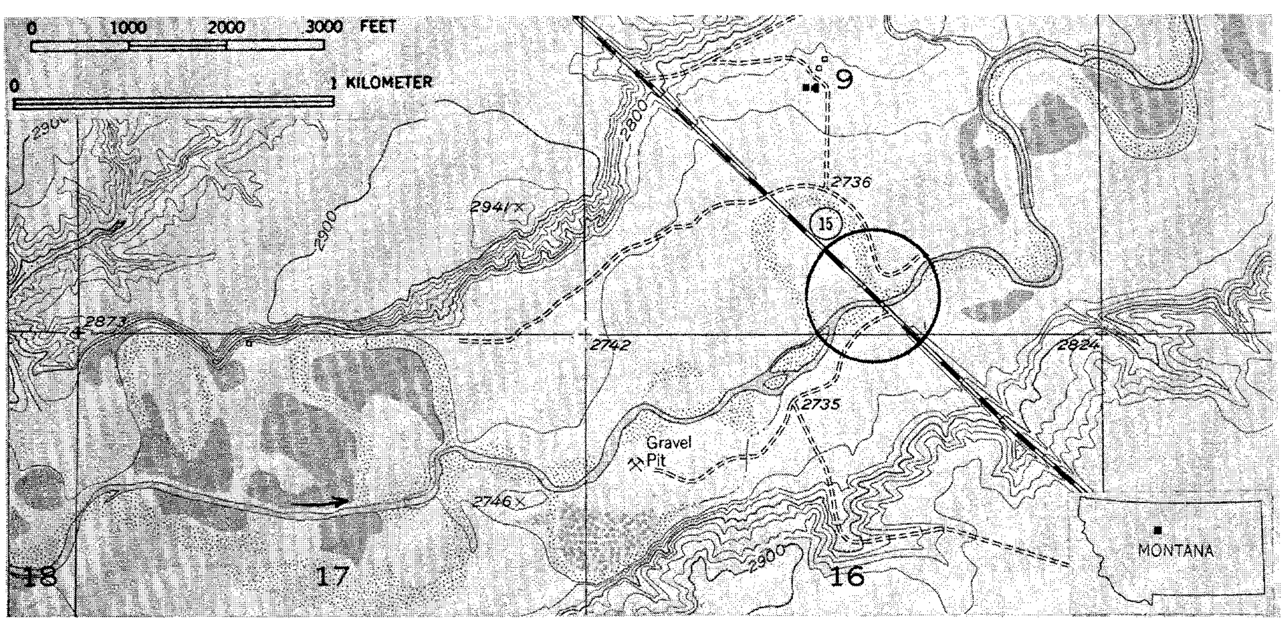


Figure 199. Map showing Teton River at SR-223 crossing (circled). (Base from U.S. Geol. Survey Fort Benton, Mont., 7.5' quadrangle, contour interval 20 feet, 1954.)

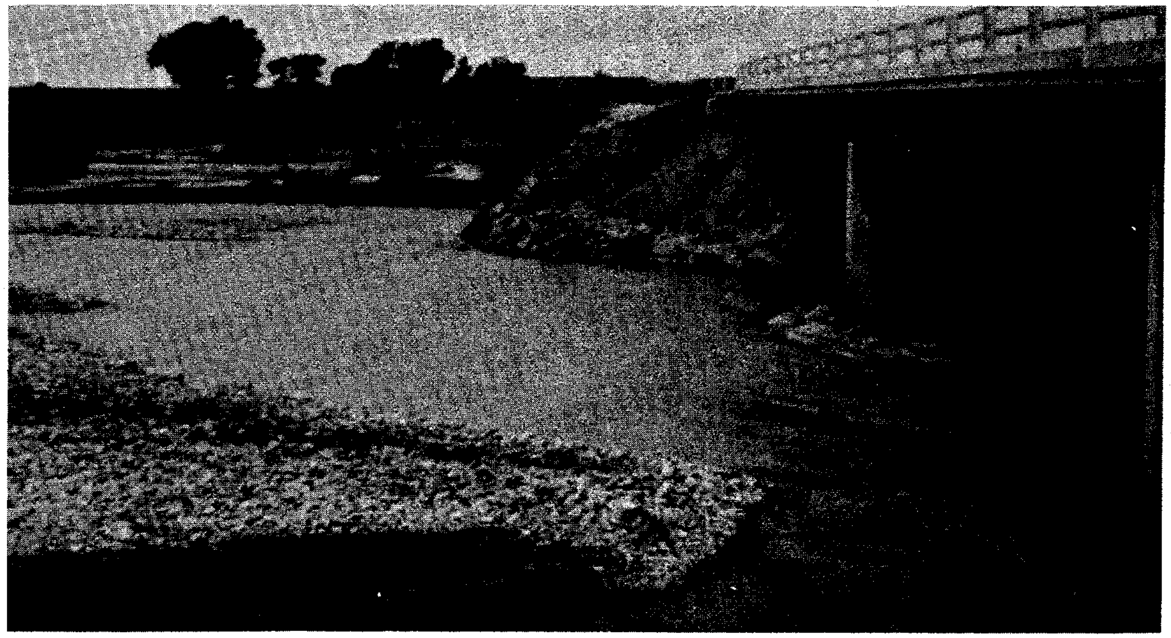


Figure 200. View toward right abutment of SR-223 bridge, showing (at left) alinement of flow entering bridge waterway. (Montana Dept. of Highways photograph, taken about 1975.)

SITE 236. BOULDER RIVER AT I-90 AT BIG TIMBER, MONT.

Description of site: Lat 45°50', long 109°56', location as shown in fig. 201. Bridge is about 325 ft (99 m) in length, with vertical (full-height) concrete abutments, flared concrete wingwalls. Crossing at bend in channel.

Drainage area, 523 mi^2 (1355 km^2); valley slope, 0.006; channel width, 150 ft (46 m). Stream is alluvial, cobble and boulder bed, in valley of low relief (at bridge site), with little or no flood plain. Channel is sinuous, locally braided, locally anabranched, random width variation.

Hydraulic problem and countermeasure:

1938 Bridge built.

1938-75 No hydraulic problems on record. Banks upstream and downstream from right abutment (at outside of bend) protected with riprap consisting of rounded cobbles and gravel faced with rounded stream boulders (fig. 202).

1976 By gradual lateral migration of bankline, vertical abutment had been exposed to lateral stream erosion.

Discussion: Although alluvial streams having sinuous channels and coarse bed material, such as that of Boulder River, tend to be moderately stable, some lateral erosion at bends is to be expected. Riprap of rounded stream boulders failed to prevent migration of channel against abutment.

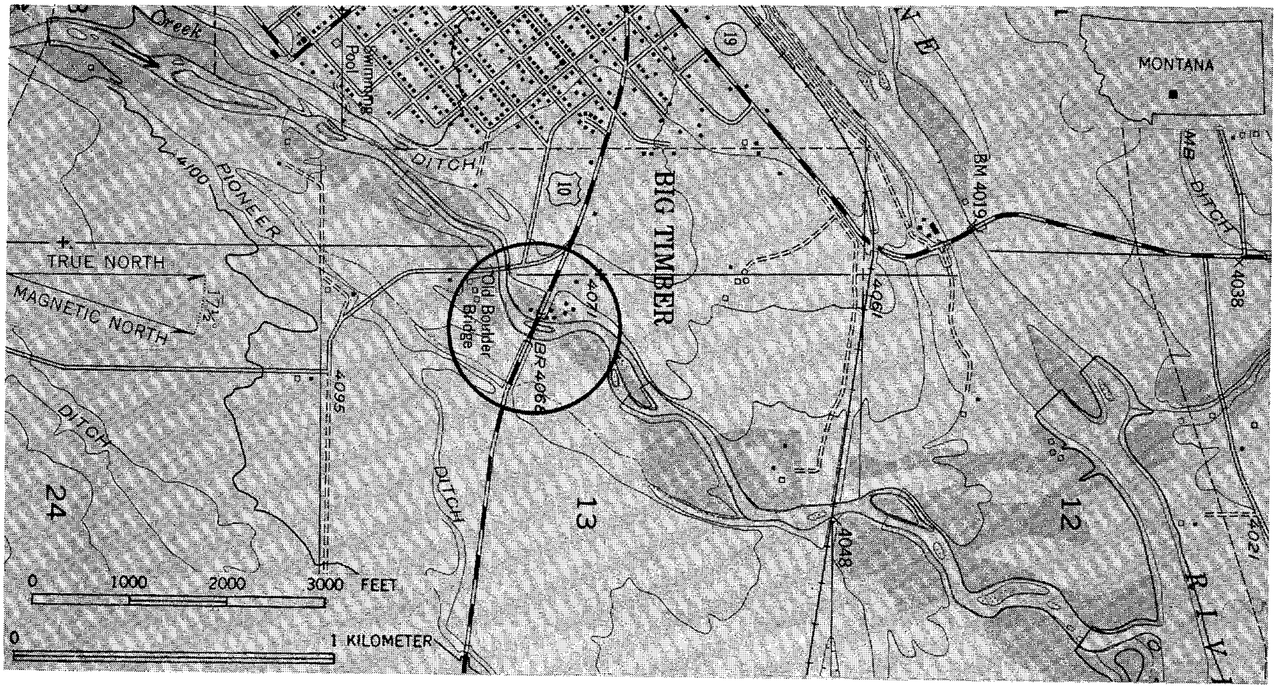


Figure 201. Map showing Boulder River at I-25 (US-10) crossing (circled). (Base from U.S. Geol. Survey Big Timber, Mont., 7.5' quadrangle, contour interval 20 ft, 1953.)

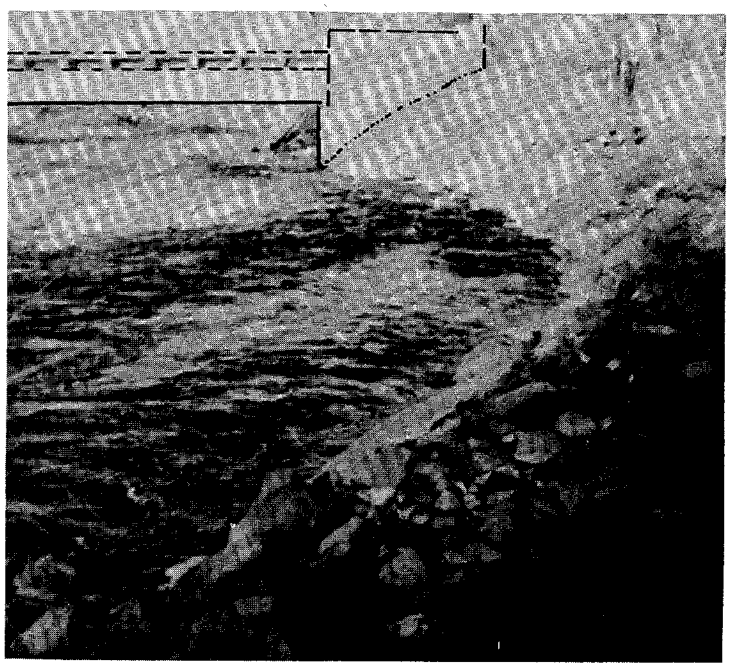


Figure 202. Downstream view of riprapped bank and right abutment, I-25 bridge at Boulder River. (From Montana Dept. of Highways.)

SITE 237. BIG SPRING CREEK AT US-191 NEAR LEWISTOWN, MONT.

Description of site: Lat 47°06', long 108°27'. Bridge is 63 ft (19 m) in length, one wall-type concrete pier in channel, vertical concrete abutments with spread footings. The backwall, wingwall, and footing of the abutments contain no reinforcing steel.

Drainage area, roughly 200 mi^2 (518 km^2); stream is perennial and alluvial.

Hydraulic problem:

1921 Bridge built, without riprap or other countermeasures.

1975 During a severe flood of unknown recurrence interval, general scour occurred in the bridge waterway, to a maximum depth of about 2 ft (0.6 m) below the footing of the right abutment and to the base of the pier footing. The abutment footing subsided, leaving a space of about 3 ft (1 m) between the bottom of the abutment wall and the footing. The abutment subsided also (fig. 203), but the backfill evidently provided enough wedging action to prevent the superstructure and deck from subsiding. Bridge closed for repairs.

Discussion: Although no countermeasures are involved, it is noteworthy that scour would occur 54 years after the bridge was built, from a large but not catastropic flood; and also that the superstructure and deck would be retained in position, even though not supported by the abutment.

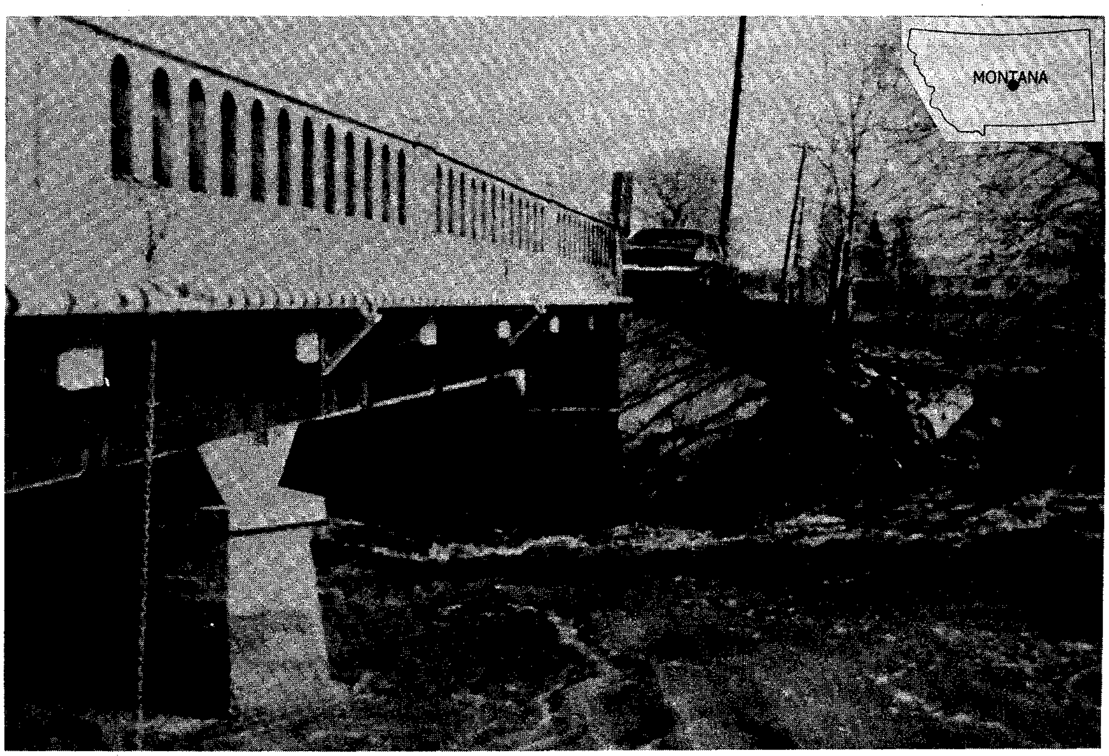


Figure 203. View of right abutment of US-191 bridge in May, 1975, after undermining and subsidence of abutment footing. (From Montana Dept. of Highways.)

SITE 238. MISSOURI RIVER AT SR-16 NEAR CULBERTSON, MONT.

Description of site: Lat 48°07', long 104°28', location as shown in fig. 204. Bridge is 1,170 ft (357 m) in length, has two main steel truss spans, five approach spans and wall-type concrete piers, square nose. Steel sheet-piling construction cofferdam at pier 4 was not cut off, but left in place on three sides of the pier. Crossing is at bend.

Drainage area, 91,557 mi^2 (237,000 km^2); channel width, about 1,000 ft (305 m). Stream is perennial, regulated by Fort Peck Lake and other reservoirs, alluvial, in valley of moderate relief, narrow floodplain. Channel is sinuous, locally braided.

Hydraulic problem:

1934 Bridge built, no riprap or other channel protection measures applied.

1958 Operational level at Fort Peck Lake was reached, streamflow became regulated.

1971-87 Channel degradation of 3-4 ft (1 m) since 1931 was noted; this may be due to the effects of Fort Peck Lake, which is about 100 miles upstream. In addition, downstream migration of the channel bend has occurred and pier 4, which was originally on the flood plain, is now in the low water channel. Progressive scour was noted at the upstream end of pier 4, and contributing factors to scour are sustained flow releases from the lake and the presence of the sheet pile cofferdam (fig. 205), which tends to catch floating drift.

Discussion: Absolute lateral migration rates tend to increase with river size, and bends in large rivers may migrate for large distances during the life span of a bridge. Piers placed originally on the flood plain thus commonly become situated in the river channel and allowance should be made for this in the design of the pier foundations. Projecting-sheet pile cofferdams, if left in place, may contribute to local scour (Neill, 1973, p. 113).

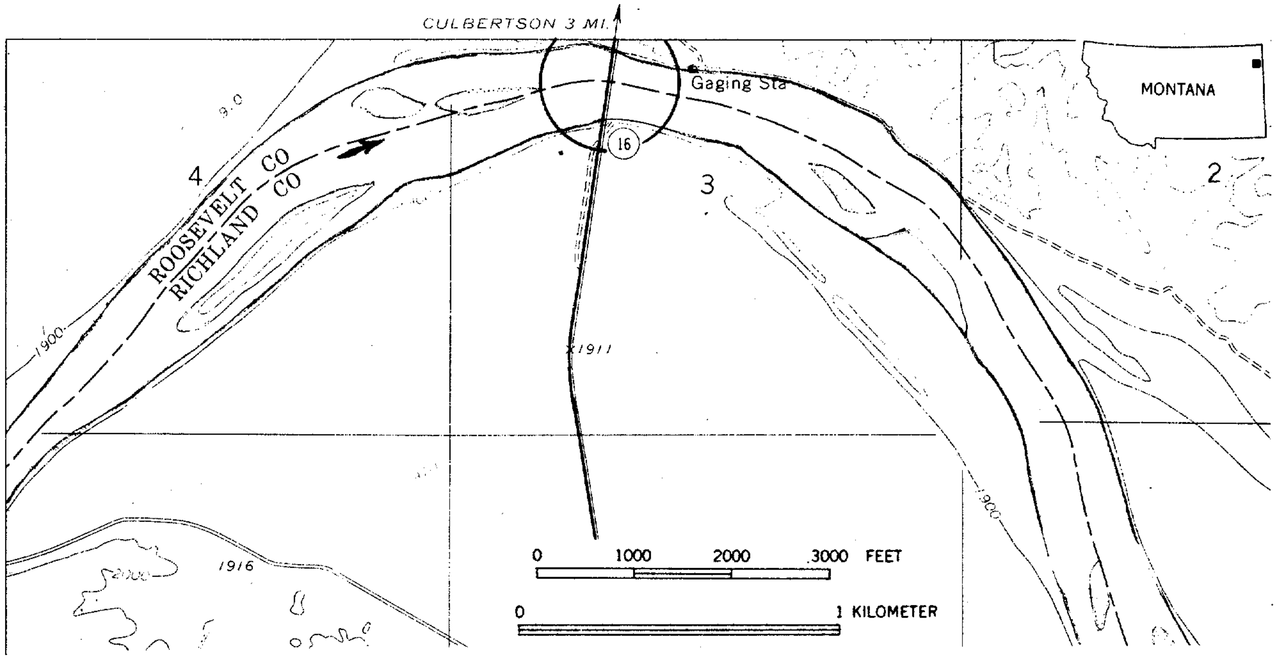


Figure 204. Missouri River at SR-16 crossing (circled). (Base from U.S. Geol. Survey Three Buttes, Mont., 7.5' quadrangle, contour interval 20 ft, 1969.)

Figure 205. View from south bank of Missouri River on September 19, 1971, showing location of pier 4 and sheet pile cofferdam at base of pier. (From Montana Dept. of Highways.)

SITE 239. ELKHORN RIVER AT US-30 AT ARLINGTON, NEBR.

Description of site: Lat 41°27', long 96°22', location as shown in fig. 206. Bridge, replaced by new bridge in 1973, was 382 ft (114 m) in length, four steel pony-truss spans, wall-type concrete piers on steel-pile foundations, vertical concrete abutments on steel-pile foundations.

Drainage area, about 7,000 mi^2 (18,130 km^2); valley slope, 0.0007; channel width, about 250 ft (75 m). Stream is perennial, alluvial, sand bed, in valley of moderate relief, wide flood plain. Channel is meandering, locally braided, wider at bends, point bars, cut banks local, silt-sand banks. Crossing is at slight bend in stream.

Hydraulic problem and countermeasure:

1932 Bridge built. Footing of left abutment was about 21 ft (6 m) below road grade, about 4 ft (1.2 m) below streambed elevation, and about 20 ft (6 m) landward from left bank (fig. 207A). No record of revetment or other countermeasures.

1933-71 Stream gradually migrated laterally toward left abutment. At some time during this period, seven timber-pile spurs were placed at left bank upstream from bridge. Timber pile of each spur were in single row, connected by timber stringers, about 50 ft (15 m) in length, pointing downstream at 60°-70° angle with bank, spaced at about 250 ft (75 m). By 1971, spurs had been substantially damaged by flow, but bank had tree growth and appeared stable (fig. 208).

1971 In February, heavy rains on frozen ground caused moderate flooding and ice break-up in Elkhorn River. Ice collected around piers and left abutment, and failure of abutment is attributed to scour beneath the ice. Spans collapsed after failure of abutment (figs. 208 and 209). Bridge was temporarily repaired and used until new bridge was completed.

1973 New steel girder bridge, 375 ft (112 m) in length, was completed. Abutments were protected by a berm or dike faced with broken concrete riprap (fig. 207B) and extending upstream and downstream from abutment fill slopes. The spurs along the left bank upstream from the bridge were repaired (fig. 210).

Discussion: The Elkhorn River is laterally unstable, as indicated by its meandering habit, wide point bars, and cut banks. Lateral instability is also indicated by comparison of sequential aerial photographs. The timber pile spurs built before 1971 controlled the lateral migration, but the bank seems nevertheless to have receded to the left abutment, exposing the abutment headwall. Sufficient scour occurred beneath the ice jam to cause failure of the abutment. In addition, the channel has degraded several feet, as indicated by exposure of foundation pile at a pier of the Chicago and Northwestern Railroad bridge, about 4,000 ft (1,200 m) downstream. Failure is thus attributed to a combination of lateral erosion, degradation, and scour. Spurs as repaired in 1971 have been subjected to flood of 5-yr R. I., and they appear to be controlling lateral migration.

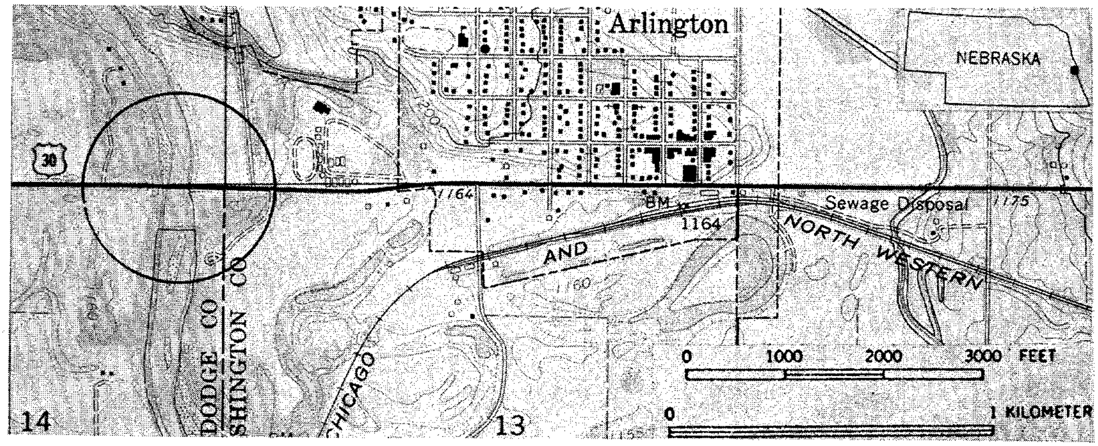


Figure 206. Map showing Elkhorn River at US-30 crossing (circled). (Base from U.S. Geol. Survey Arlington, Nebr., 7.5' quadrangle, contour interval 10 feet, 1968.)

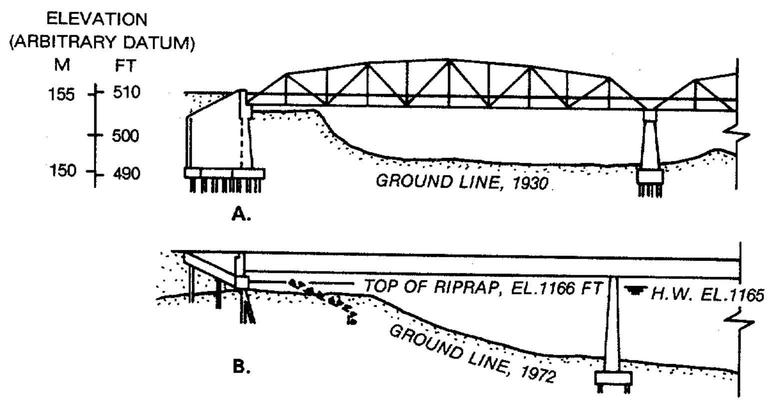


Figure 207. A, elevation sketch showing left abutment of bridge built in 1932. B, left abutment of bridge built in 1973.

Figure 208. Aerial photograph, taken March 29, 1971, showing failure of bridge and location of spurs on left bank upstream from bridge. (Nebraska Dept. of Roads photograph.)

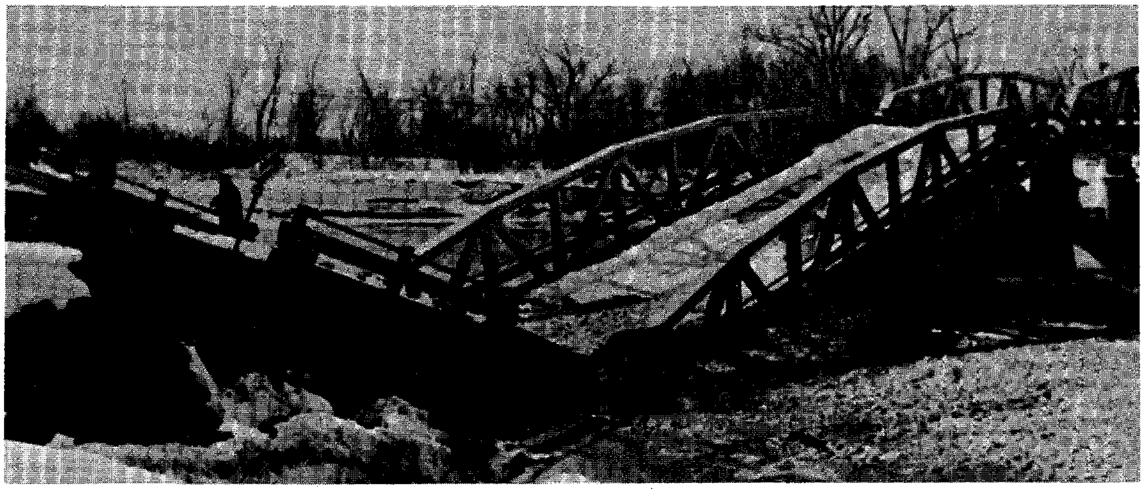


Figure 209. View of left end of failed bridge. (Nebraska Dept. of Roads photograph.)

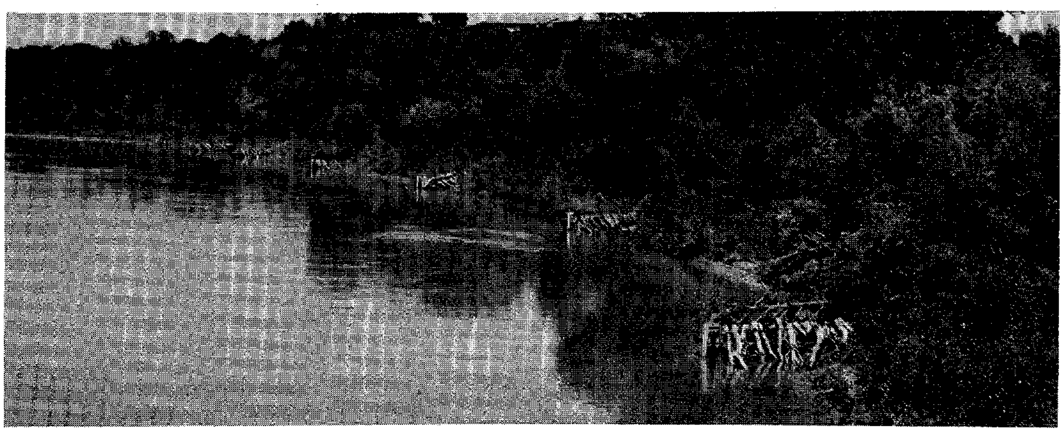


Figure 210. Upstream view, in 1977, of spurs at left bank upstream from bridge.

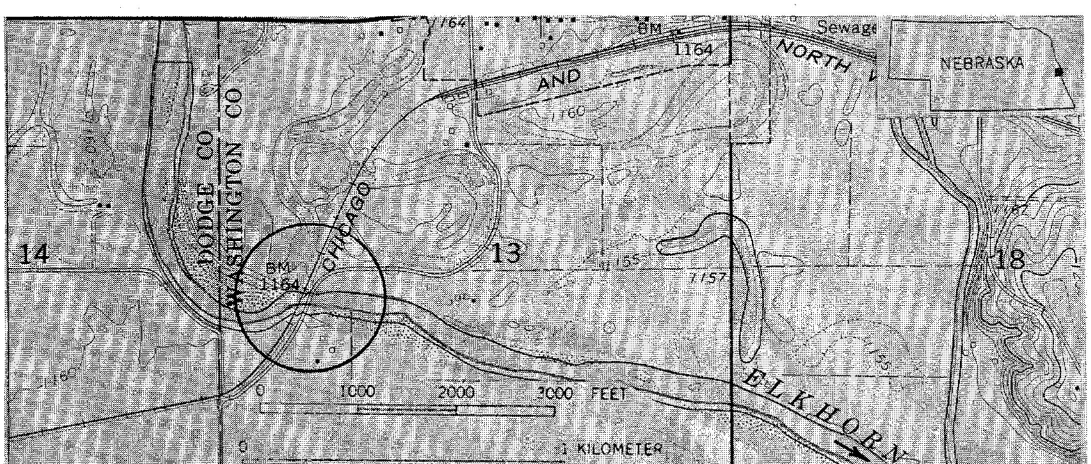


Figure 211. Map showing Elkhorn River at Washington County road crossing (circled). (Base from U.S. Geol. Survey Arlington, Nebr., 7.5' quadrangle, contour interval 10 feet, 1968.)

SITE 240. ELKHORN RIVER AT WASHINGTON COUNTY ROAD NEAR ARLINGTON, NEBR.

Description of site: Lat $41^{\circ}26.5'$, long $96^{\circ}22'$, location as shown in fig. 211. Single-span steel-truss bridge, built before 1920, crossing downstream from bend (fig. 212).

Stream is meandering, sand bed, has a history of rapid lateral migration. See Site 239 for more complete description.

Hydraulic problem and countermeasure: Upstream bend migrated toward right such that, by the late 1960's, the right abutments and approach roadways of both the country bridge and the Chicago and Northwestern Railroad bridge were threatened. About 100 car bodies were placed along the right bank to control bank erosion. Also, a shallow channel was cut across the point bar in an attempt to divert the flow (fig. 212). During a flood in 1971, the car-body revetment was washed out along the section of bank immediately upstream from the bridge, and the bank was severely eroded. The shallow channel across the point bar failed to divert the flow; there was little chance for it to succeed, because the configuration of the bend is unfavorable for cutoff (fig. 211). By 1977, the bank was becoming stabilized by vegetation where the car body revetment was still in place, but was still eroding in the critical area immediately upstream from the bridges.

Discussion: No details are available regarding the method of securing the car bodies to the bank; therefore the countermeasure cannot be fairly evaluated. The case is useful in demonstrating, however, that the point of most severe bank erosion is likely to be downstream from the apex of a bend.

Figure 212. Aerial photograph showing bank erosion at bend upstream from county road and railroad crossing on March 29, 1971. (From Nebraska Dept. of Roads.)

SITE 241. ELKHORN RIVER AT SR-32 AT WEST POINT, NEBR.

Description of site: Lat 41°50', long 96°44', location as shown in fig. 213. Bridge is 350 ft (107 m) in length, steel-girder spans, two wall-type concrete piers in channel, steel-pile foundations. Vertical concrete abutments, concrete wingwalls, steel sheet-pile bulkhead at front and sides of abutment headwall, driven to depth of 29 ft (8.7 m) below base of wingwall.

Drainage area, 4,100 mi^2 (10,600 km^2); valley slope, 0.0008; channel width about 300 ft (90 m). Stream is perennial, alluvial, sand bed, in valley of moderate relief, wide flood plain. Channel is sinuous, artificially straightened, locally braided, not incised, sand-silt banks, tree cover at less than 50 percent of bankline. Since the stream was straightened in 1948, its width has increased from about 225 ft (67 m) for the natural channel to about 300 ft (90 m) for the artificial channel.

Hydraulic problem and countermeasure:

1954 Bridge built, no record of hydraulic problems at old bridge.

1955-72 Channel gradually migrated to right upstream from bridge presenting a potential threat to right abutment. Six steel H-pile spurs, faced with heavy woven wire mesh, were constructed along the right bank in 1971 (figs. 214 and 215). Length of spurs ranged from 100 to 370 ft (30 to 110 m), and angle with base line ranged from 57° to 70°. Length of pile was 30 ft (9 m) and piles were driven to such depth that the top was approximately at bank height.

1973-77 Spurs not tested by major flood, largest flood during period was of 2-3 yr R. I. However, vegetation has become established along bank at spurs 5 and 6, and some sediment has accumulated (fig. 215). At the other spurs, the bank has been built out by dumping of a large quantity of broken concrete from a street repaving project in the town of West Point.

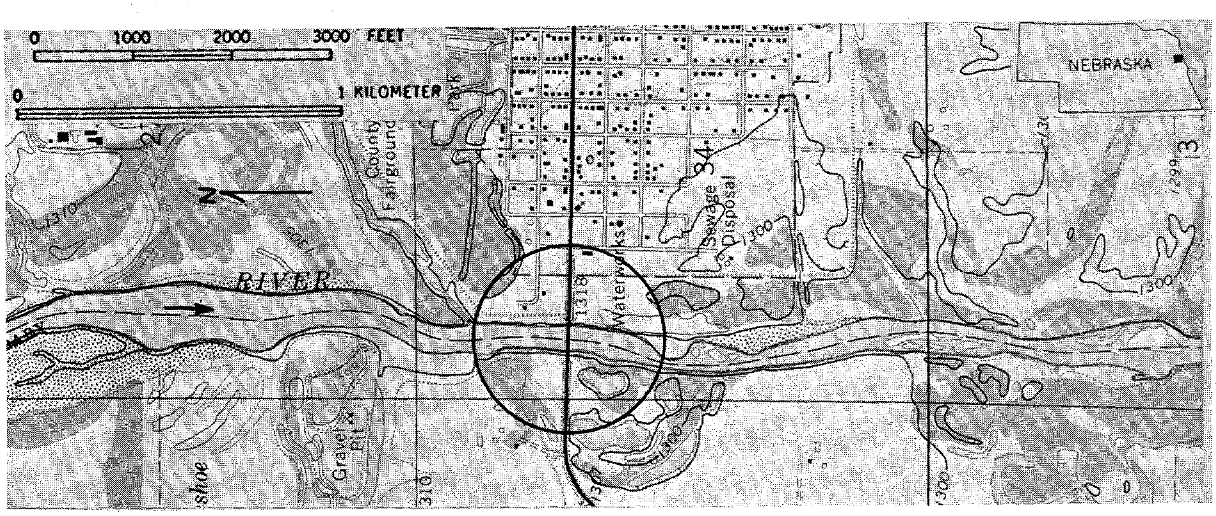


Figure 213. Map showing Elkhorn River at SR-32 crossing (circled). (Base from U.S. Geol. Survey West Point, Nebr., 7.5' quadrangle, contour interval 10 feet, 1966.)

Discussion: The natural channel of the Elkhorn River is meandering, point bars tend to be wide, and the banks are sandy and erodible; the channel has a low degree of lateral stability. When such a stream is channelized, it is very likely to migrate laterally and re-establish its meandering habit. The spurs have not been adequately tested, but the available evidence is favorable.

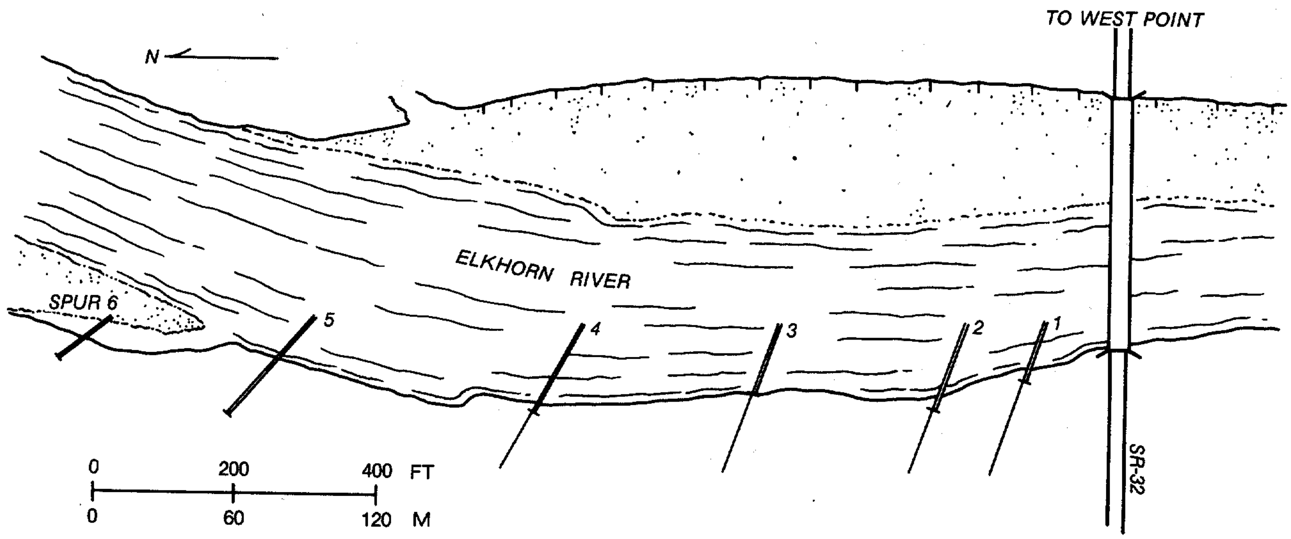


Figure 214. Plan sketch showing layout of spurs at SR-32 crossing.

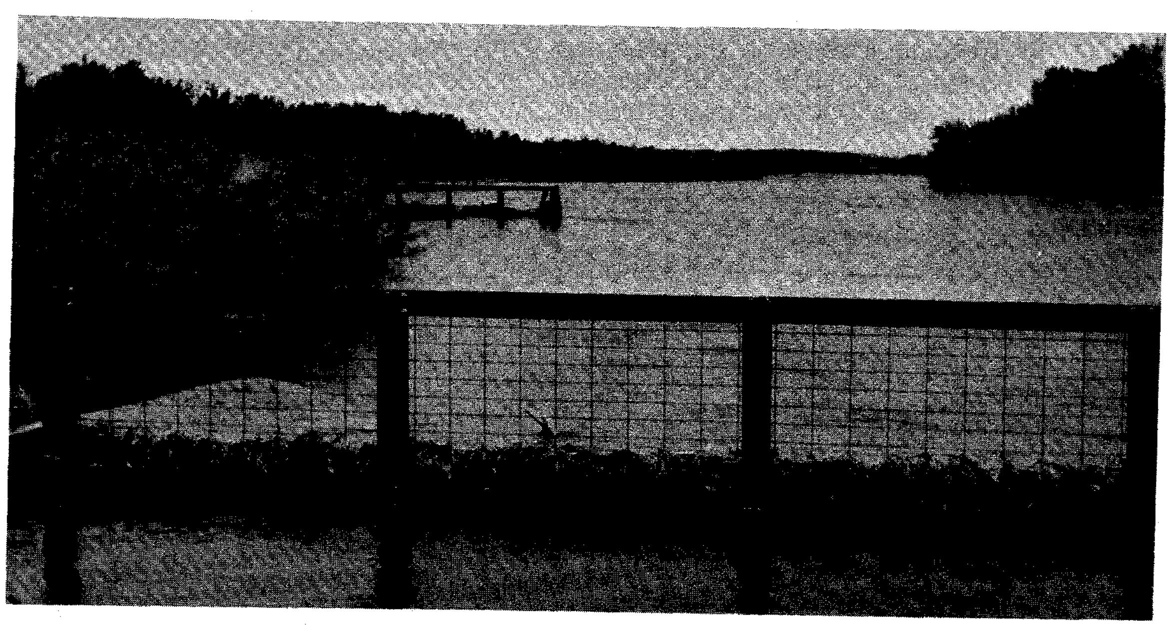


Figure 215. Upstream view of spur 5, foreground, and spur 6, background, in 1977.

SITE 242. LOGAN CREEK AT SR-9 NEAR PENDER, NEBR.

Description of site: Lat 42°08', long 96°44', location as shown in fig. 216. Bridge is 254 ft (76 m) in length, two round concrete-filled steel sheet-pile piers in channel, spillthrough abutments. Sheet pile at piers driven to depth of 55 ft (16.5 m) below cutoff elevation, or about 35 ft (10.5 m) below streambed elevation at time of construction. Steel foundation pile at abutments driven to depth of 45 ft (13.5 m) below cutoff elevation.

Drainage area, about 725 mi^2 (1875 km^2); valley slope, 0.0006; channel width in 1977, about 250 ft (75 m). Stream is perennial but flashy, alluvial, sand bed, in valley of moderate relief, wide flood plain. Channel is straight, artificial, cut banks general, silt banks. An entirely new channel has less than half the length (and about twice the slope) of the old channel.

Hydraulic problems and countermeasures:

1947 Bridge built. No revetment or flow control structures were installed, but foundations were driven to substantial depth for a stream of this size, as described above.

1948-70 Channel degraded about 7 ft (2 m) and increased in width from about 150 ft (45 m) to about 250 ft (75 m). At some time during this period, probably in the mid-1960's, steel H-pile spurs were installed both upstream and downstream from the bridge, to control the bank recession (fig. 217).

1971 Flood, peak discharge about 32,000 ft^3/s (905 m^3/s). Individual point velocities as high as 14.5 ft/s (4 m/s) were measured at the gaging station at Pender, 2 mi (3.2 km) downstream.

1977 Footings and foundation pile of left abutment are exposed, and sheet pile at piers is exposed for a vertical distance of about 7 ft (2 m) (fig. 218). Streamward end of spur just upstream from birdge has been pushed over by ice or debris.

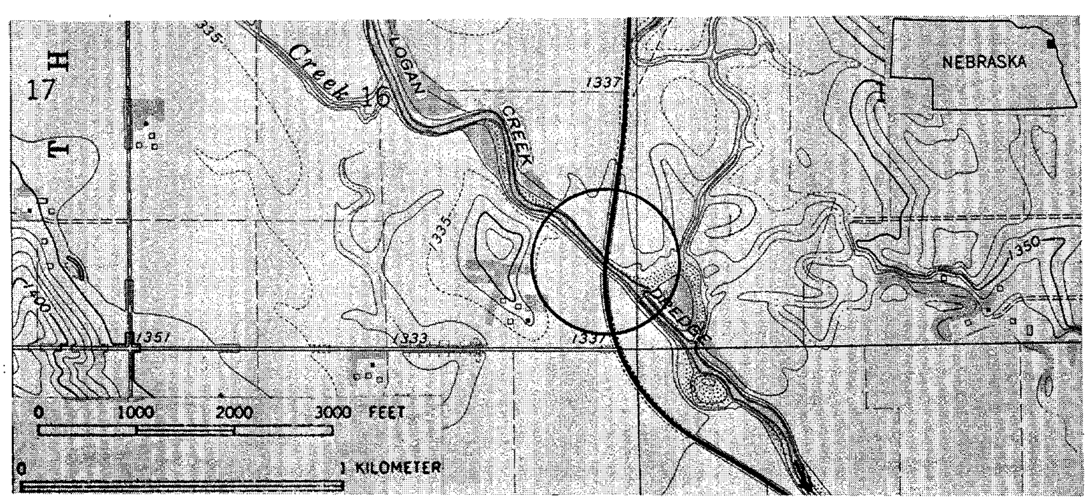


Figure 216. Map showing Logan Creek at SR-9 crossing (circled). (Base from U.S. Geol. Survey Thurston, Nebr., 7.5' quadrangle, contour interval 10 feet, 1966.)

Discussion: Although steel H-pile spurs have been effective in controlling bank erosion on streams similar to this one in Nebraska and Iowa, they cannot control channel degradation. Also, their effectiveness in controlling bank erosion depends on accumulation of debris and sediment at the spur, which is inhibited by progressive degradation. The channel has probably approached an equilibrium adjustment of depth and slope, and the spurs may now serve to maintain channel alignment and control further widening. Maintenance of the spurs has been prevented by lack of access over privately owned land. Depth of foundations at piers and abutments has so far prevented damage to bridge.

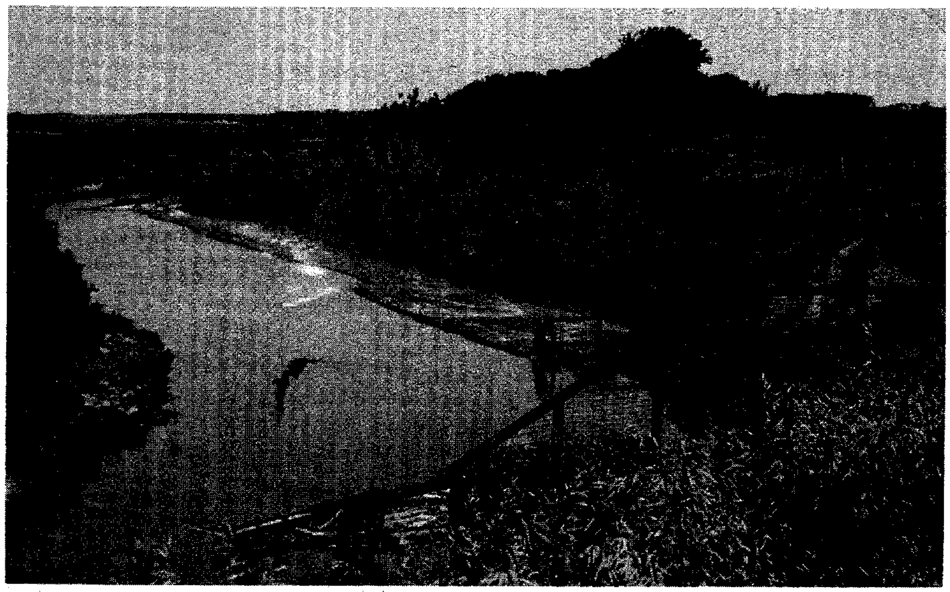


Figure 217. View of steel-and-wire mesh spurs on left bank upstream from bridge, in 1977.

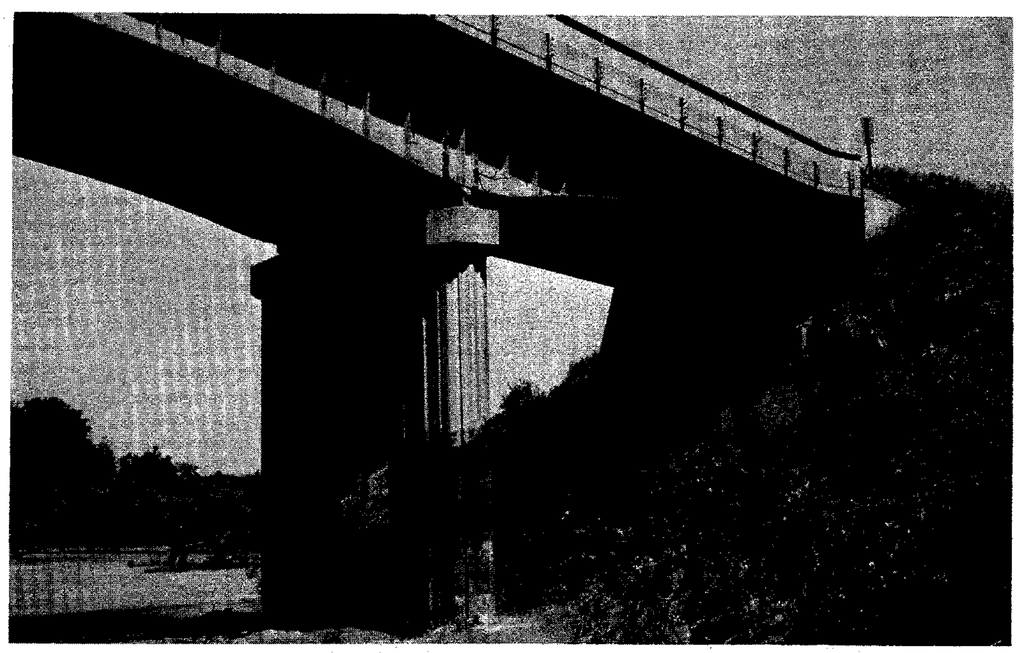


Figure 218. Upstream view of exposed piers (unpainted part), and footings of left abutment, in 1977.

SITE 243. NIOBRARA RIVER AT US-183 NEAR SPRINGVIEW, NEBR.

Description of site: Lat 42°40', long 99°45', location as shown in fig. 219. Bridge is 371 ft (113 m) in length, multiple-span steel girder, four wall-type concrete piers in channel, right abutment of spillthrough type, left abutment vertical concrete with wingwalls.

Drainage area, 8,000 mi^2 (20,700 km^2); valley slope, 0.0015; channel width, about 1,000 ft (300 m). Stream is perennial, alluvial, sand bed, nearly straight, generally braided (fig. 243.2), cut banks local, sand banks.

Hydraulic problem and countermeasure:

1955 Bridge built. Riprap of broken concrete slabs placed at right abutment and along approach embankment, which projects into bed of channel (fig. 220). Sheet-pile bulkhead driven into streambed along front of right embankment, for prevention of scour.

1957-58 Stream thalweg, which was near left side of channel, shifted to right and caused some erosion of roadway embankment that extends into channel. An earth dike, similar in function to spur dike, was built upstream from right abutment (fig. 220). Spurs were built along the dike at a 30° angle, pointing downstream. The spurs were originally permeable timber-pile structures, connected by a timber stringer, and with a cluster of steel H-pile at the end. Tree trunks were attached laterally to the spurs. Spur length is about 100 ft (30 m) as measured along the crest, and the interval between spurs is about 160 ft (49 m).

1959-77 Riprap of broken concrete slabs has been added to the spurs from time to time, building them up and making them impermeable (fig. 221). Area behind dike has become vegetated. Performance of dike and spurs is considered good by district highway engineer.

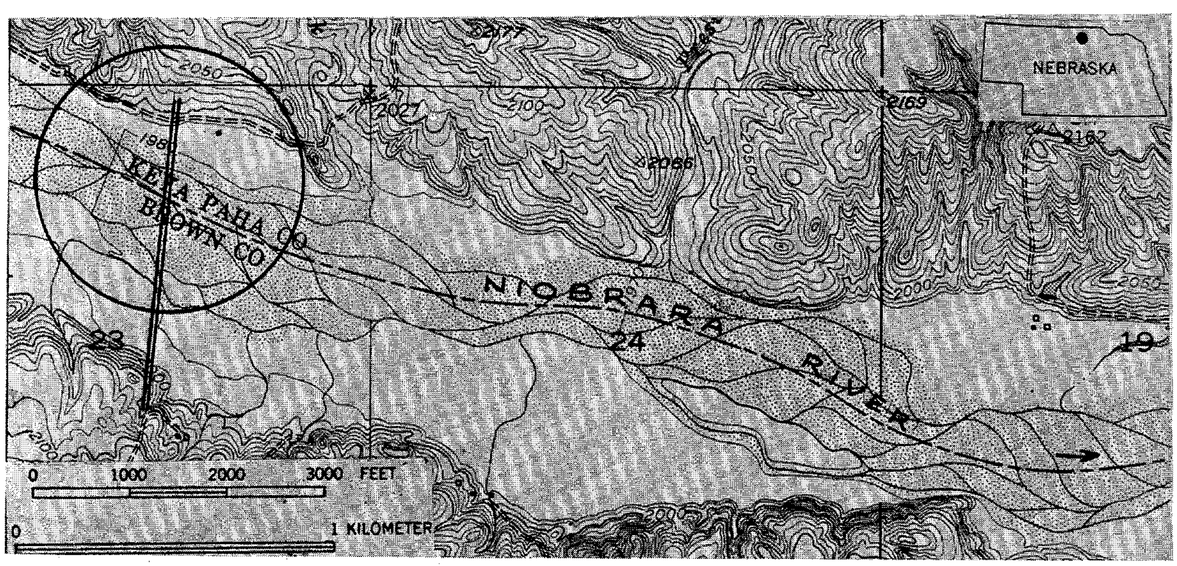


Figure 219. Map showing Niobrara River at US-183 crossing (circled). (Base from U.S. Geol. Survey Bassett NW, Nebr., 7.5' quadrangle, contour interval 10 feet, 1950.)

Discussion: By means of the riprapped road embankment and the dike with projecting spurs, the channel has been successfully constricted to about one-third its natural width.

Figure 220. Aerial photograph of Niobrara River at US-183 crossing, in 1956. Layout of dike with spurs, built in 1958, has been added. (From Nebraska Dept. of Roads.)

Figure 221. Upstream view of spur projecting from dike, in 1977.

SITE 244. NORTH PLATTE RIVER AT SR-11 AT NORTH PLATTE, NEBR.

Description of site: Lat 41°08', long 100°43', location as shown in fig. 222. Bridge is 570 ft (174 m) in length, multiple-span welded-plate girders, concrete wall-type piers in channel, steel-pile foundations, vertical concrete abutments with wingwalls.

Drainage area, 26,300 mi^2 (68,100 km^2); valley slope, 0.0009; channel width, indefinite but about 500 ft (150 m). Stream is perennial, highly regulated, alluvial, sand bed, in valley of moderate relief, wide flood plain. Channel is straight, generally braided (fig. 223), sand banks, tree cover at more than 90 percent of bankline.

Countermeasures:

1960 Bridge built, replacing older bridge at same location and with similar alinement, except that right abutment of new bridge is about 150 ft (45 m) downstream from older bridge. Permeable spurs, built of single row of timber pile from which attached tree trunks extend downstream, placed at left bank to close off old river channel and direct overbank flow through bridge (fig. 223). These were built at old bridge and left in place for new bridge. New bridge has steel sheet-pile bulkheads at abutments and broken concrete riprap at right bank upstream from right abutment.

1977 Countermeasures functioning satisfactorily, but have not been tested by severe flood; stream is highly regulated by upstream dam.

Discussion: This case provides an example of countermeasures used on a wide, braided sandbed stream. At such crossings, it may be feasible to shorten the bridge length by the use of measures (such as spurs) to constrict the channel.

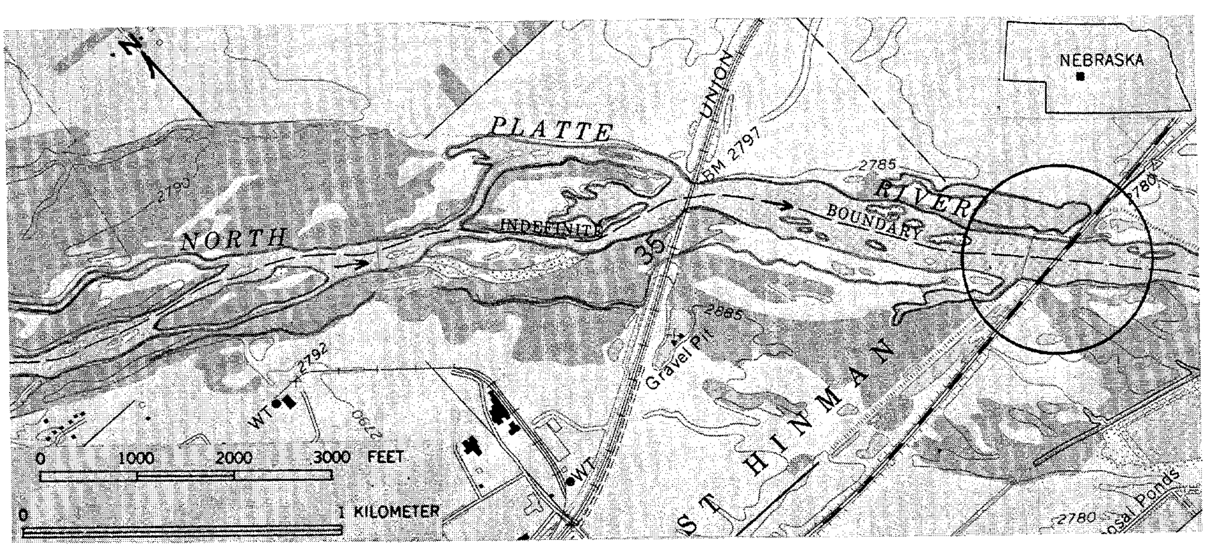


Figure 222. Map showing North Platte River at SR-11 crossing (circled). (Base from U.S. Geol. Survey North Platte east, Nebr., 7.5' quadrangle, contour interval 20 feet, 1970.)

Figure 223. Aerial photograph of SR-11 crossing in 1959, showing bridge (replaced in 1960) and permeable spurs. (From Nebraska Dept. of Roads.)

SITE 245. ROCK CREEK AT US-77 NEAR CERESCO, NEBR.

Description of site: Lat 41°02.5', long 96°38', location as shown in fig. 224. Bridge is 93 ft (28 m) in length, two concrete wall-type piers in channel, timber-pile foundations beneath piers, spillthrough abutments, steel H-pile foundation beneath abutments.

Drainage area, 51 mi^2 (132 km^2); valley slope, 0.0017; channel width, 100 ft (30 m). Stream is perennial but flashy, alluvial, silt-clay bed, in valley of moderate relief, wide flood plain. Channel is meandering, wider at bends, not incised, silt-clay banks, no tree cover along banks in vicinity of bridge.

Hydraulic problem and countermeasure:

1933 Bridge built. As a countermeasure for possible scour or degradation, the abutment headwalls were extended to a depth of 16 ft (5 m) below road elevation and founded on 40-ft (12-m) steel pile (fig. 225). Similarly, the pier foundations are unusually deep for a stream of this size.

1964 Severe flood, R.I. estimated at about 100 yr from flow records on nearby stream. Abutment fill-slopes eroded, piling at abutments exposed. Scour depth not measured, but 24 ft (7 m) of scour was measured at gaging station on nearby stream (Wahoo Creek at Ithaca). As a countermeasure, 6-ft (2-m) lengths of steel sheet-piling were driven at the base of the abutment headwall. Also, the wingwalls were extended with timber-pile retards (or bulkheads), backfilled with broken concrete riprap.

1977 Countermeasures not tested by severe flood. Thalweg has shifted toward right end of bridge (fig. 226), retards have tilted, sheet-pile bulkhead not visible.

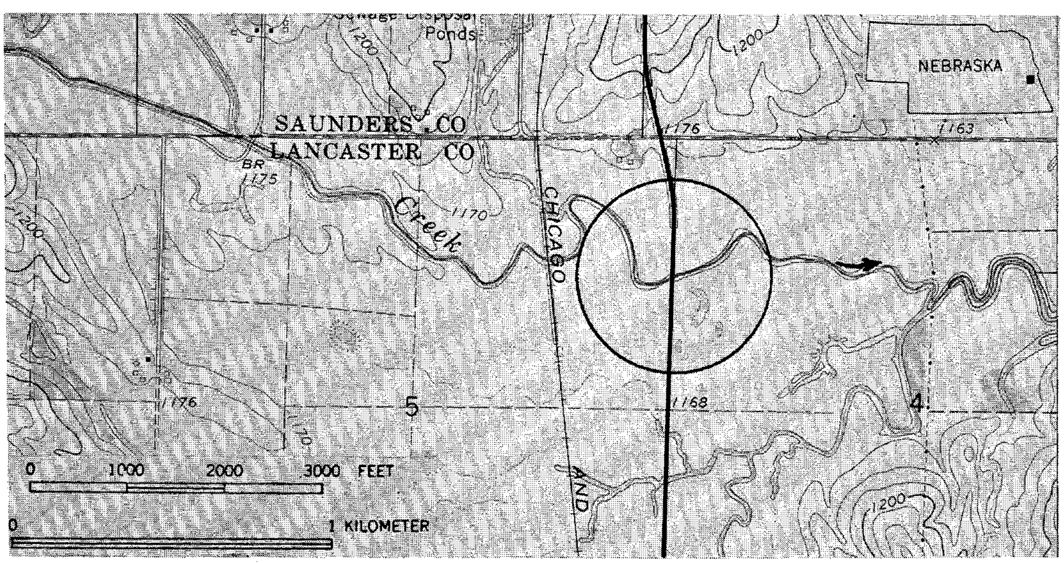


Figure 224. Map showing Rock Creek at US-77 crossing. (Base from U.S. Geol. Survey Ceresco, Nebr., 7.5' quadrangle, contour interval 10 feet, 1968.

Discussion: The channel boundary materials in this region are dominantly silt, and are suceptible to rapid lateral erosion and vertical incision. The piers and abutments were designed to withstand vertical scour or incision, and wingwalls were provided against lateral erosion. However, the wingwalls and timber-pile retards may prove inadequate to protect the bridge from lateral erosion in another severe flood, particularly in view of the fact that the thalweg has shifted against the right abutment.

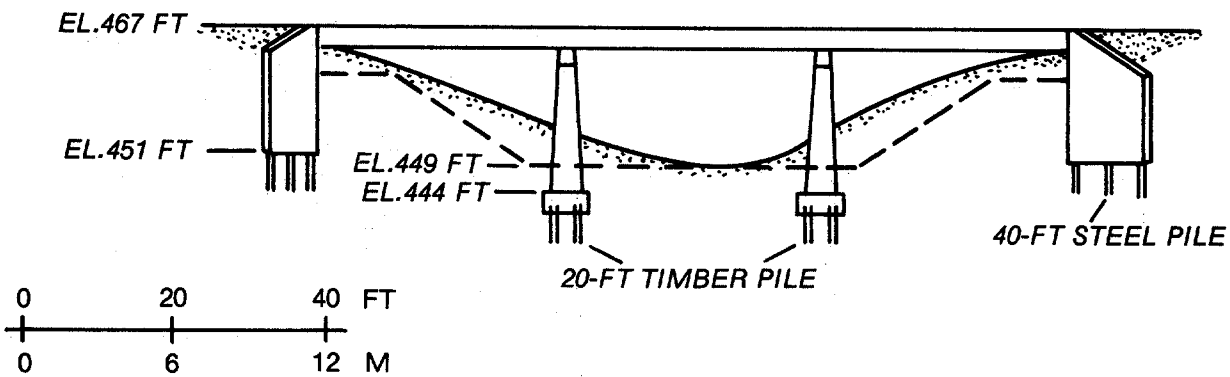


Figure 225. Elevation sketch of US-77 bridge at Rock Creek.

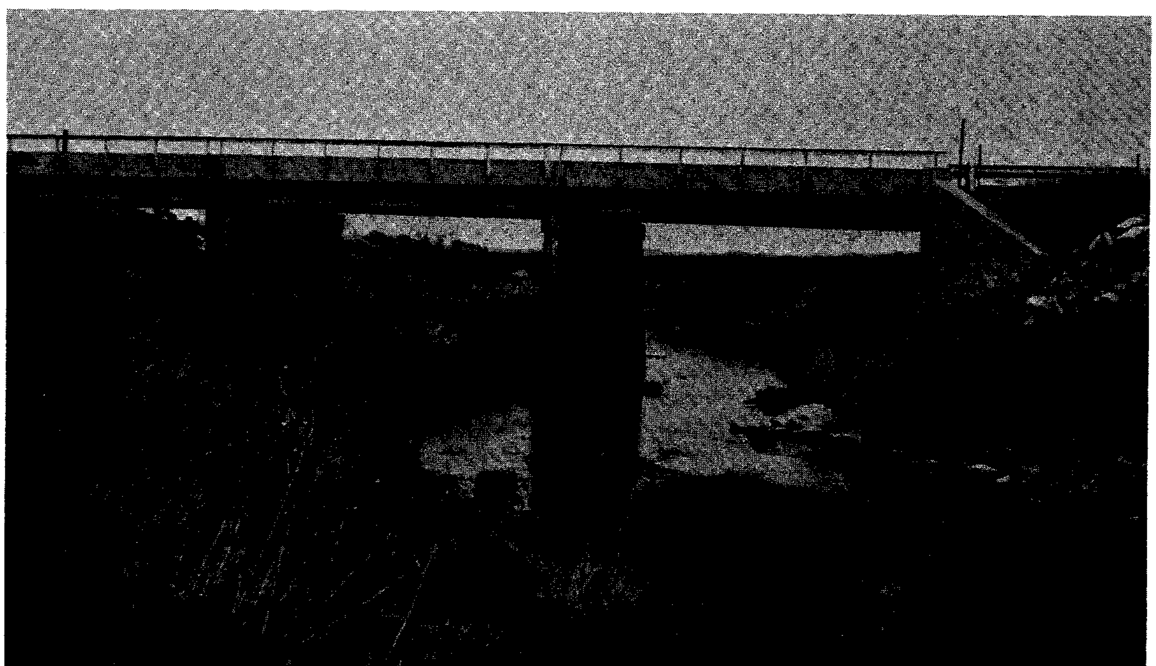


Figure 226. Downstream view of US-77 bridge in 1977.

SITE 246. SOUTH FORK LITTLE NEMAHA RIVER AT SR-50 NEAR COOK, NEBR.

Description of site: Lat 40°30', long 97°11'. Bridge is 181 ft (55 m) in length, concrete wall-type piers in channel, steel-pile foundations, spillthrough abutments, concrete wingwalls. Minimum penetration of foundation pile below cut-off elevation, 50 ft (15 m) at abutments and 24 ft (7 m) at piers; bottom of seal at piers about 6 ft (2 m) below streambed elevation, channel width, 180 ft (55 m). Stream is perennial but flashy, alluvial, silt bed, in valley of low relief, wide flood plain. Channel is sinuous, artifically straightened, incised by degradation, cut banks general, silt-clay banks.

Hydraulic problem: Since bridge was built in 1950, channel has degraded about 8 ft (2.4 m) exposing pier footings and about 2 ft (0.6 m) of foundation pile. Lateral erosion has removed most of abutment fill slopes. Slump cracks in soil behind right abutment were noted in 1971 bridge inspection report. No countermeasures have been applied, but lengthening of bridge will probably be required.

Discussion: Channel degradation, resulting from channel straightening, is general throughout the Nemaha River system. Few bridge failures have resulted from degradation because of the depth to which bridge foundations have been placed in Nebraska. The lateral erosion accompanying channel degradation has been the more serious problem.

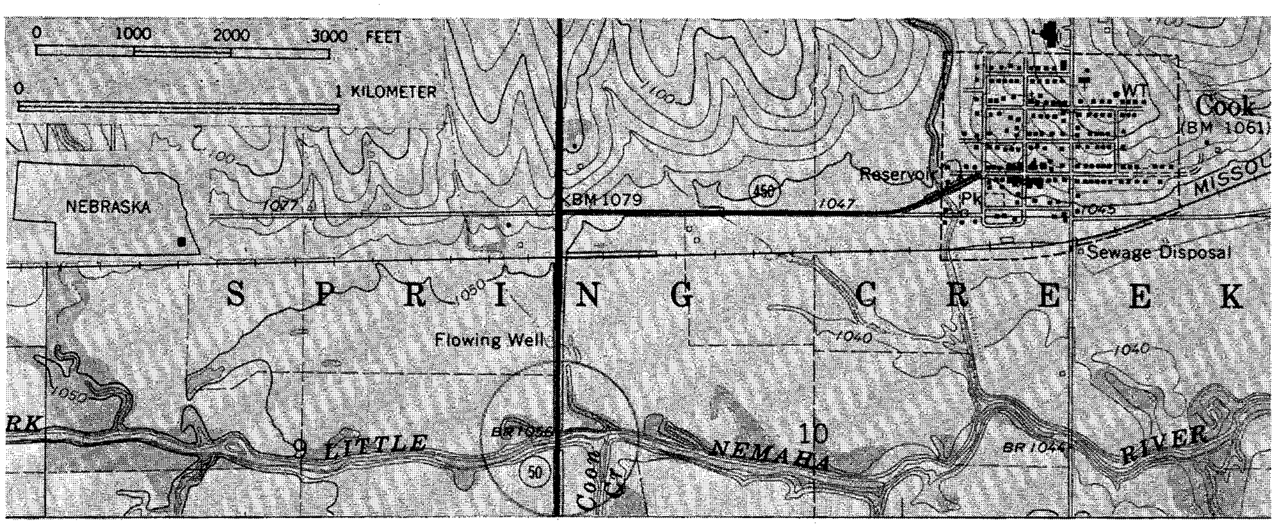


Figure 227. Map showing South Fork Little Nemaha River at US-50 crossing (circled). (Base from U.S. Geol. Survey Cook, Nebr., 7.5' quadrangle, contour interval 10 feet, 1966.)

SITE 247. SOUTH PLATTE RIVER AT US-83 AT NORTH PLATTE, NEBR.

Description of site: Lat 41°07', long 100°46', location as shown in fig. 228. Dual bridges, of which only the downstream bridge is described. Bridge is 640 ft (195 m) in length, multiple-span prestressed-concrete girders, wall-type concrete piers, pile foundations, vertical abutments.

Drainage area, 24,300 mi^2 (62,900 km^2); valley slope, 0.0013; channel width, about 600 ft (183 m). Stream is perennial, regulated, alluvial, sand-gravel bed, in valley of moderate relief, wide flood plain. Channel is sinuous, generally braided, not incised, cut banks local, sand banks, tree cover at less than 50 percent of bankline.

Hydraulic problem and countermeasure:

1965 Bridge built, steel sheet-pile bulkheads driven at sides and front of abutments (fig. 229).

1966-73 Footings and supporting pile at first and third piers from right bank were exposed because of channel degradation. As a countermeasure, concrete bases were poured at these piers to protect exposed foundation pile.

Discussion: Although the channel boundaries of the South Platte River are rather indefinite owing to its braided habit, the bridge spans most or all of the average channel width. Degradation apparently began before the bridge was constructed, inasmuch as the datum of the U. S. Geological Survey gage at this locality was lowered by 3 ft (1 m) during the period 1945-56. A similar amount of degradation during the period 1965-1973 exposed the pier footings. The cause of the degradation is not apparent. The sheet-pile bulkheads at the abutments serve as a protection against scour of the easily erodible sandy banks.

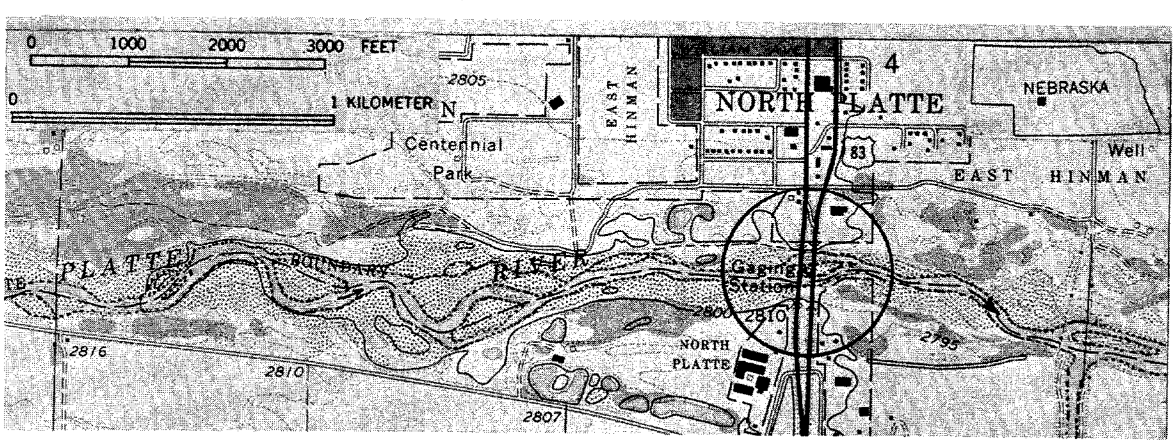


Figure 228. Map showing South Platte River at US-83 crossing. (Base from U.S. Geol. Survey Lake Maloney, Nebr., 7.5' quadrangle, contour interval 20 feet, 1969.)

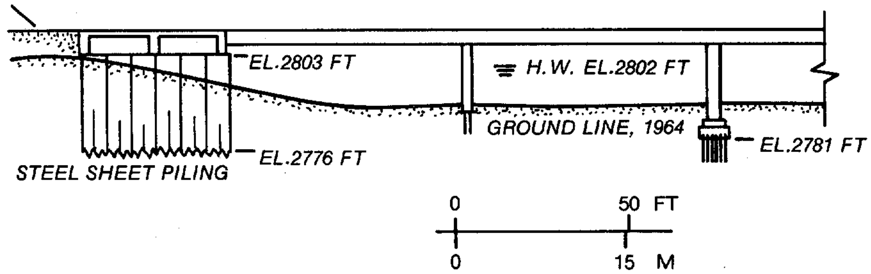


Figure 229. Elevation sketch of left abutment of US-83 bridge, showing steel sheet-pile bulkheads.

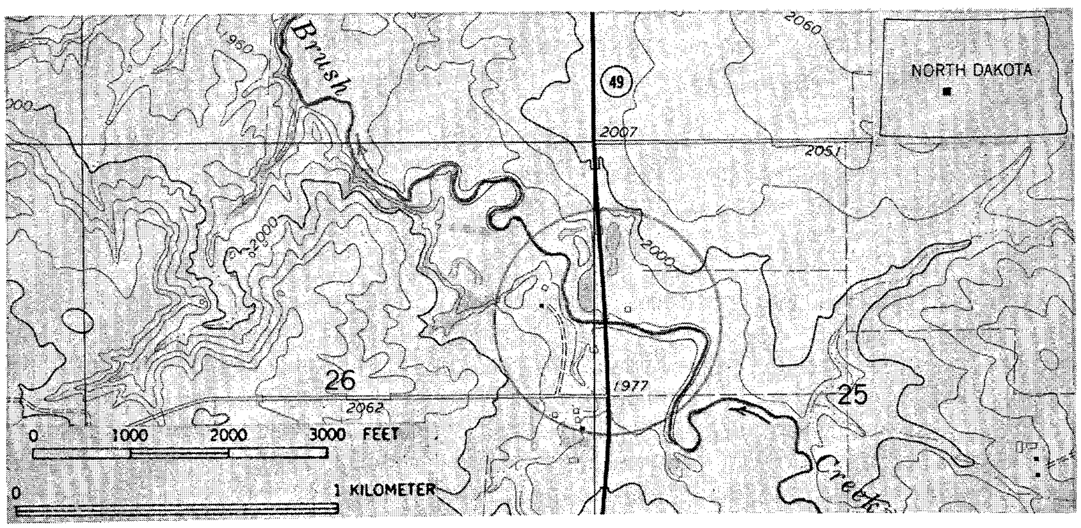


Figure 230. Map showing Brush Creek at SR-40 crossing (circled). (Base from U.S. Geol. Survey Medicine Butte NE, N. Dak., 7.5' quadrangle, contour interval 20 ft, 1968.)

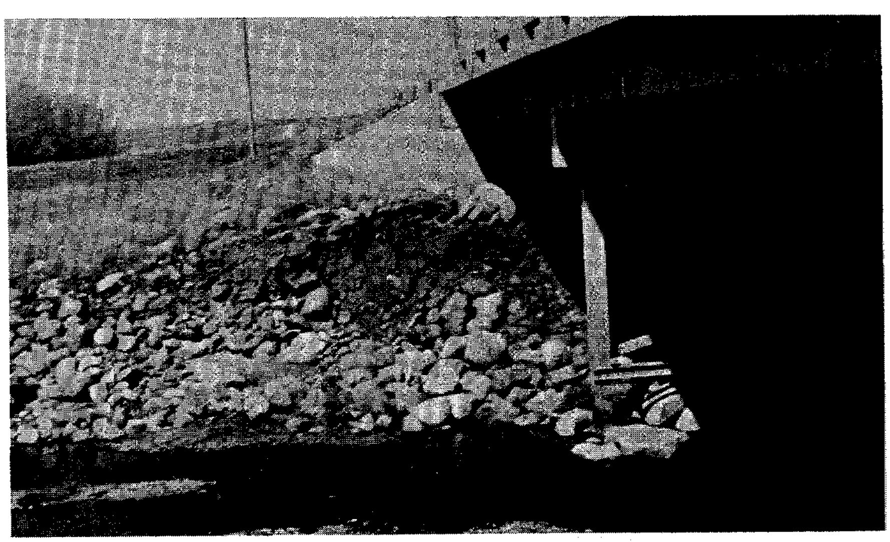


Figure 231. View of downstream side of north abutment on April 24, 1969. (From North Dakota Dept. of Highways.)

SITE 248. BRUSH CREEK AT SR-49 NEAR BEULAH, N. DAK.

Description of site: Lat 47°11', long 101°47', location as shown in fig. 230. Bridge is 100 ft (30 m) in length, 3-span continuous slab, steel H-pile bents, driven into shale to a depth of 20 ft (6 m) below channel bed. Spillthrough abutments.

Drainage area, about 50 mi^2 (130 km^2); valley slope, 0.007. Stream is perennial, alluvial, gravel bed, in valley of moderate relief, wide flood plain. Channel is meandering, wider at bends, point bars, cut banks general, erodible silt-clay banks. Stream is in grassland, tree cover sparse along banks.

Hydraulic problems and countermeasures:

1964 Severe flood of undetermined recurrence interval. Five inches (12.5 cm) of rain fell in four hours in drainage basin of creek. Stream eroded laterally against right (north) abutment, exposing foundation piling and eroding adjacent approach embankment fill. In addition, erosion of left bank at bend downstream from bridge (fig. 230) was damaging to farmstead. As a countermeasure, the channel was filled and shaped to smooth alinement and to a cross section of uniform trapezoidal shape, and the sideslopes were revetted with "fieldstone" riprap. This channel work extended from the eroded bank at the farmstead, upstream through the bridge opening.

1969 Stream flowed at bankfull stage during spring. Riprap at the north abutment was damaged, probably by ice (fig. 231).

Discussion: Although the bridge appears to be on a straight reach of channel, (fig. 230) erosion of the right (north) bank is attributed to the effects of a minor curvature along this reach. The riprap damaged at the bridge abutment in 1969 was of rounded glacial boulders, which may be more susceptible to damage than angular, quarried riprap.

SITE 250. CANNONBALL RIVER AT SR-49 NEAR NEW LEPZIG, N. DAK.

Description of site: Lat 46°21', long 101°57', location as shown in fig. 232. Bridge, built in 1930's, was 145 ft (44 m) in length, 3-span I-beam superstructure, spillthrough abutments.

Drainage area, roughly 1,200 mi^2 (3,100 km^2); valley slope, 0.0017; channel width, 150 ft (46 m). Stream is perennial, alluvial, in valley of moderate relief, narrow flood plain. Channel is highly meandering, definitely incised.

Hydraulic problem and countermeasure:

1950 Severe flood, bridge deck submerged to depth of about 6 ft (2 m), flow carried both ice and floating debris. Bridge superstructure was displaced as much as 6 in (15 cm), and other structural damage was reported. In addition, abutment fill-slopes were eroded. As a countermeasure, riprap was placed at both abutment slopes.

1966 Riprap proved effective as protection for abutments, but streambank erosion was reported. Because of structural deficiencies and inadequate waterway opening, bridge replacement was recommended.

1970 New bridge constructed, of much greater length (325 ft, or 98 m) and larger waterway opening. Trapezoidal channel is typical of bridge waterways in North Dakota (fig. 233). Riprap is glacial boulders and cobbles.

1970-76 New bridge subjected to 25-yr R.I. flood, no damage reported.

Discussion: Inadequacy of waterway opening for old bridge is apparent from its submergence and from damage to superstructure by ice and debris. Abutment of old bridge, however, was effectively protected by fieldstone riprap. Trapezoidal waterway openings, as used for the new bridge, are feasible on North Dakota streams because the channels tend to be incised.

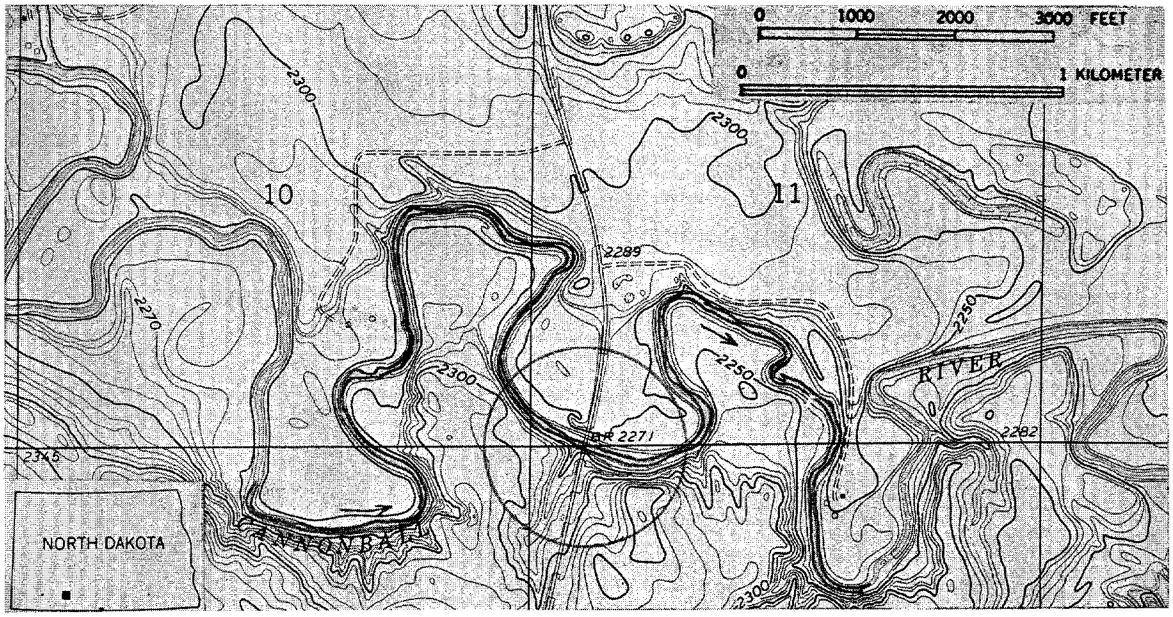


Figure 232. Map showing Cannonball River at SR-49 crossing (circled). (Base from U.S. Geol. Survey New Leipzig south, N. Dak., 7.5' quadrangle, 1972.)

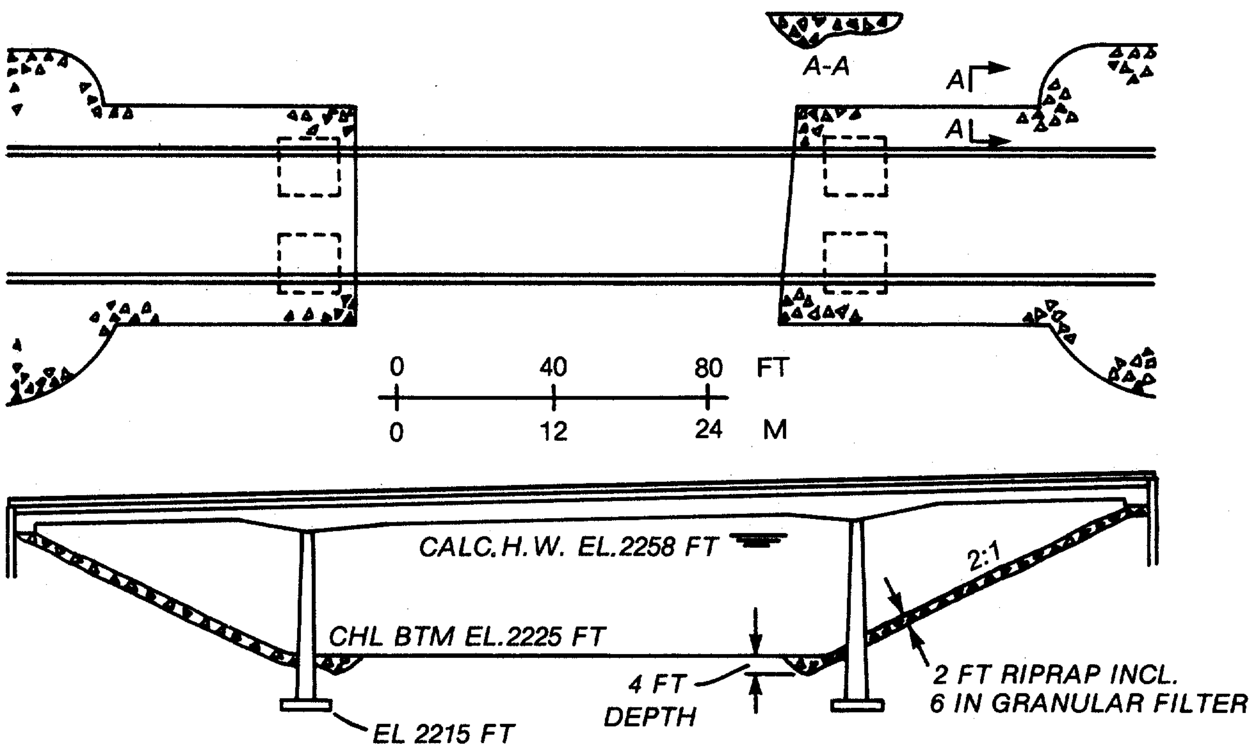


Figure 233. Plan and elevation sketch showing design features and countermeasures at new SR-49 bridge, built about 1970.

SITE 252. TIOGA RIVER AT CONNECTING ROAD TO US-15 AT BLOSSBURG, PA.

Description of site: Lat 41°41', long 77°03', location as shown in fig. 234. Bridge is 125 ft (38 m) in length, single-span steel pony truss, full-height vertical concrete abutments, concrete wingwalls. Right streambank was protected by a concrete retaining wall that extended upstream and downstream from bridge. Crossing is at channel bend of very large radius, but curvature is enough to deflect flow toward right bank. Skewness of crossing, 27°.

Drainage area 83 mi^2 (215 km^2); bankfull discharge 10,000 ft^3/s (283 m^3/s); channel width, about 150 ft (45 m). Stream is perennial, alluvial, gravel and cobble bed, in valley of moderate relief, narrow flood plain. Channel is sinuous, wider at bends, point bars, banks of non-coherent gravel, cobbles, and coarse sand.

Hydraulic problem and countermeasure:

1933 Bridge built. Concrete retaining wall, 12 ft (3.6 m) in height, built at right bank in 1933 or at some date long before 1972.

1972 Flood, R.I. 90 yr, peak discharge 21,000 ft^3/s (594 m^3/s), mean velocity in bridge waterway about 8 ft/s (2.4 m/s). Right streambank eroded, sections of concrete retaining wall undermined and damaged, slight scour at right bridge abutment and wingwall. No countermeasures installed.

1975 Flood, R.I. 18 yr, peak discharge 11,200 ft^3/s (317 m^3/s), mean velocity in bridge waterway estimated at 11 ft/s (3.3 m/s). Right streambanks severely eroded, remnants of retaining wall damaged beyond repair, right bridge abutment and wingwalls undermined.

1975-76 Gabion retaining wall built at bridge, extending upstream (fig. 235A), built at right bank downstream from bridge. Gabions made of PVC-coated hexagonal woven wire, filled with cobbles and coarse gravel. Retaining wall was protected at toe by a thin flexible gabion apron, and revetment was protected at toe by keying a gabion into the streambed and supporting it with riprap (fig. 235B). These countermeasures have not yet been tested by high flow.

Discussion: The coarse gravel and cobble banks of Tioga Creek proved not to be resistant to lateral erosion during high flow. Failure of the concrete retaining wall is attributed to inadequate keying at toe and sides. The flood of 1975, although much smaller than the 1972 flood, did more damage because it encountered banks that were already unstable and a retaining wall that was already damaged.

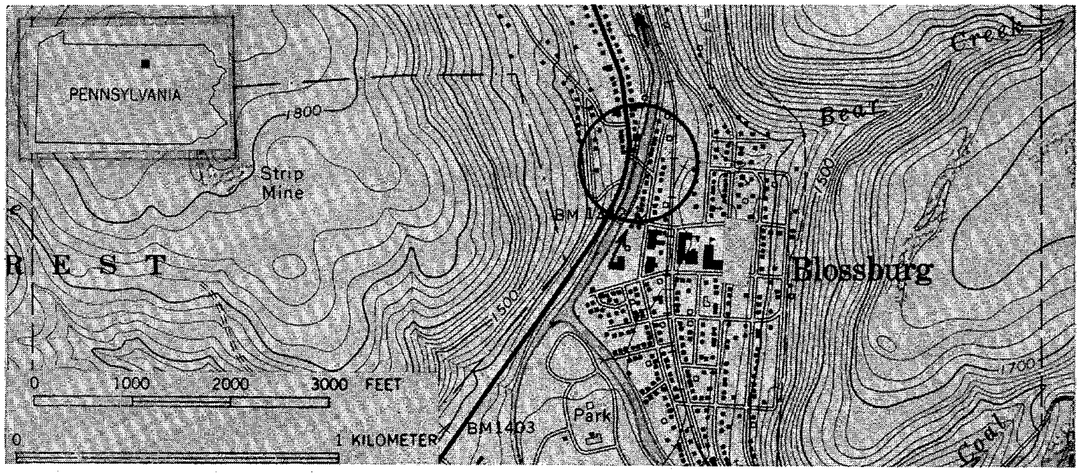


Figure 234. Map showing Tioga River at connecting road to US-15 (circled). (Base from U.S. Geol. Survey Blossburg, Pa., 7.5' quadrangle, contour interval 20 feet, 1970.)

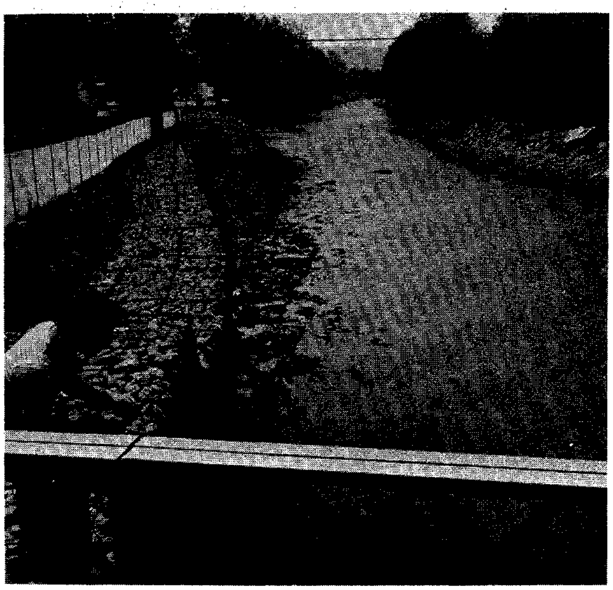


Figure 235. A, view of gabion retaining wall along right bank upstream from bridge, in July, 1976. B, View of toe gabion and adjoining riprap, at gabion slope revetment on right bank downstream from bridge.

SITE 253. TIOGA RIVER AT US-15 NEAR COVINGTON, PA.

Description of site: Lat $41^{\circ}44'$, long $77^{\circ}05'$, location as shown in fig. 236. Bridge is single-span, steel pony truss.

Drainage area, 90 mi^2 (233 km^2); valley slope, 0.007; channel width, about 150 ft (45 m). Stream is perennial, alluvial, gravel and cobble bed, in valley of moderate relief, narrow flood plain. Channel is sinuous, locally braided, wider at bends, cut banks general, gravel banks.

Hydraulic problem and countermeasure:

1930's Bridge built, no countermeasures known.

1972 Flood of 90-yr R.I. No damage to bridge, but severe bank erosion occurred along the Tioga River, and the Corps of Engineers subsequently deepened the channel and did remedial work along the banks. Along the left bank about 300 ft (90 m) downstream from the US-15 bridge, four spurs were built to control bank erosion at a bend (figs. 237 and 238). The spurs were constructed of dumped rock riprap, approximately equivalent to class III riprap, to a length of 40 ft (12 m), of which 20 ft (6 m) was keyed into the bank. The crest of each spur was sloped from the top of the bank to the streambed. Each spur was placed at an angle of about 45° with the bank, angled downstream, and the spacing factor (spacing distance/spur length) ranged from about 1 to about 4.

1975 Flood of 18-yr R.I., performance of spurs judged satisfactory. Deposition occurred at the upstream deflector, the middle two sustained no damage, and the fourth (farthest downstream) lost half its length to erosion.

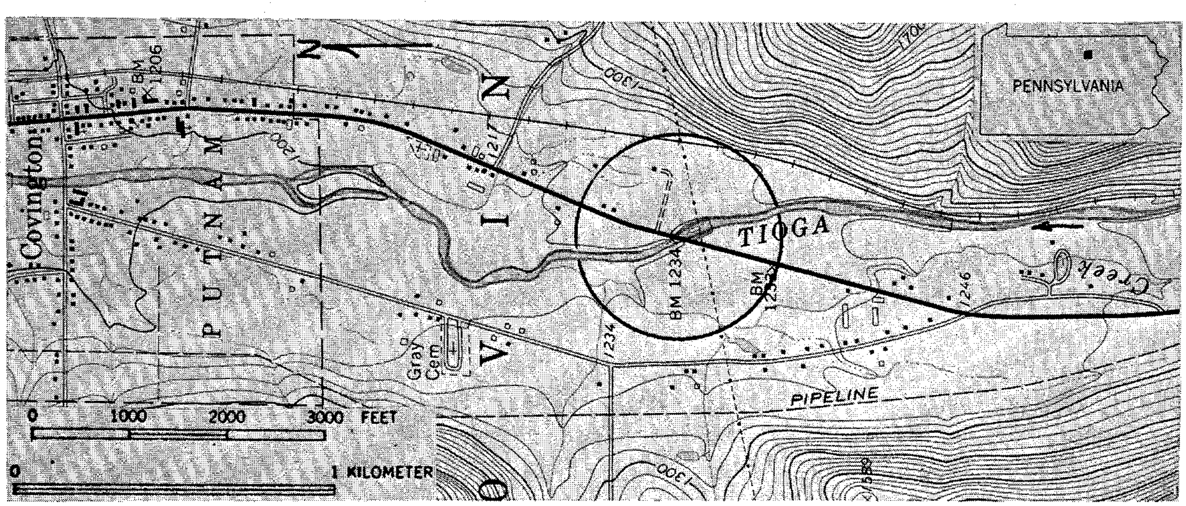


Figure 236. Map showing Tioga River at US-15 near Covington, PA. (Base from U.S. Geol. Survey Blossburg, PA., 7.5' quadrangle, contour interval 20 feet, 1970.)

Discussion: Although this problem is mostly unrelated to the bridge, it is reported here because spurs are an important countermeasure for control of erosion at bridges. In this case, well-constructed short spurs were effective in controlling bank erosion, and the amount of riprap in the spurs is evidently less than would be required to revet the length of bank protected.

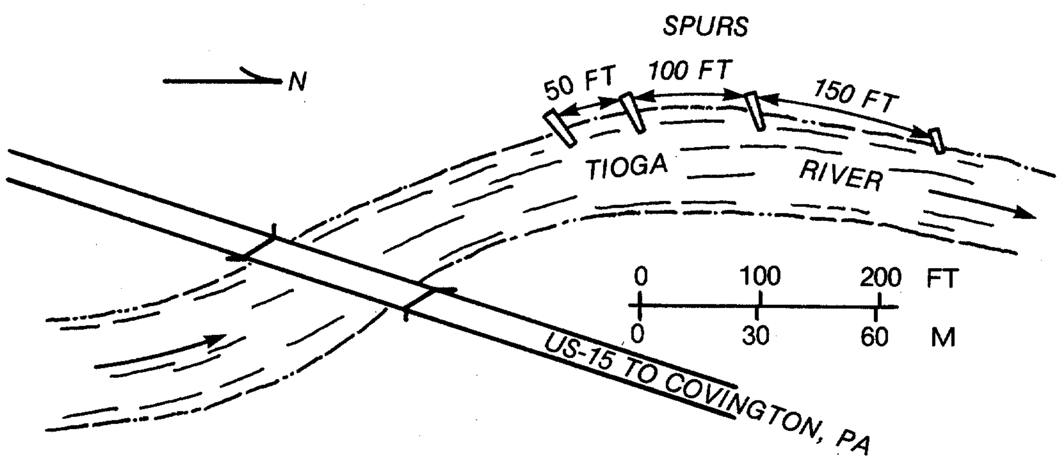


Figure 237. Plan sketch showing position of spurs, US-15 near Covington, PA.

Figure 238. Upstream view of bridge and spurs, US-15 near Covington, PA.

SITE 254. HUNTERS RUN AT SR-38 NEAR MOUNT HOLLY SPRING, PA.

Description of site: Lat 40°05', long 77°12', location as shown in fig. 239. Bridge is 36 ft (11 m) in length, of concrete box design, spread footings on bedrock.

Drainage area, 5 mi^2 (13 km^2); bankfull discharge, about 400 ft^3/s (11 m^3/s); valley slope, 0.014; channel width, about 40 ft (12 m). Stream is perennial but flashy, alluvial, gravel bed, in valley of moderate relief, little or not flood plain. Channel is sinuous, locally braided, gravel banks.

Hydraulic problem and countermeasure:

1926 Bridge built. The channel was relocated through the new bridge (fig. 239) but, because of difficulties in changing the alinement of the approach roadway, the new channel was designed with a sharp bend upstream from the bridge.

1970-72 Major floods, R.I. of one estimated at 70 yr and another at 30 yr. Bend in new channel moved gradually downstream, eroding a large area at the right abutment (fig. 239) and scouring the streambed at the abutment.

1975 As a countermeasure, the scoured area was filled and a retard of hand-placed riprap was built upstream from the abutment (fig. 240). Plans call for pressure grouting of the riprap retard and the placement of heavy riprap along its toe. The countermeasures have not yet been tested.

Discussion: The nearly parallel trends of stream and roadway, in a narrow valley, posed a difficult problem in crossing location. If the bend in the new channel had been less sharp and the banks had been revetted, the channel could probably have been held in position upstream from the bridge.

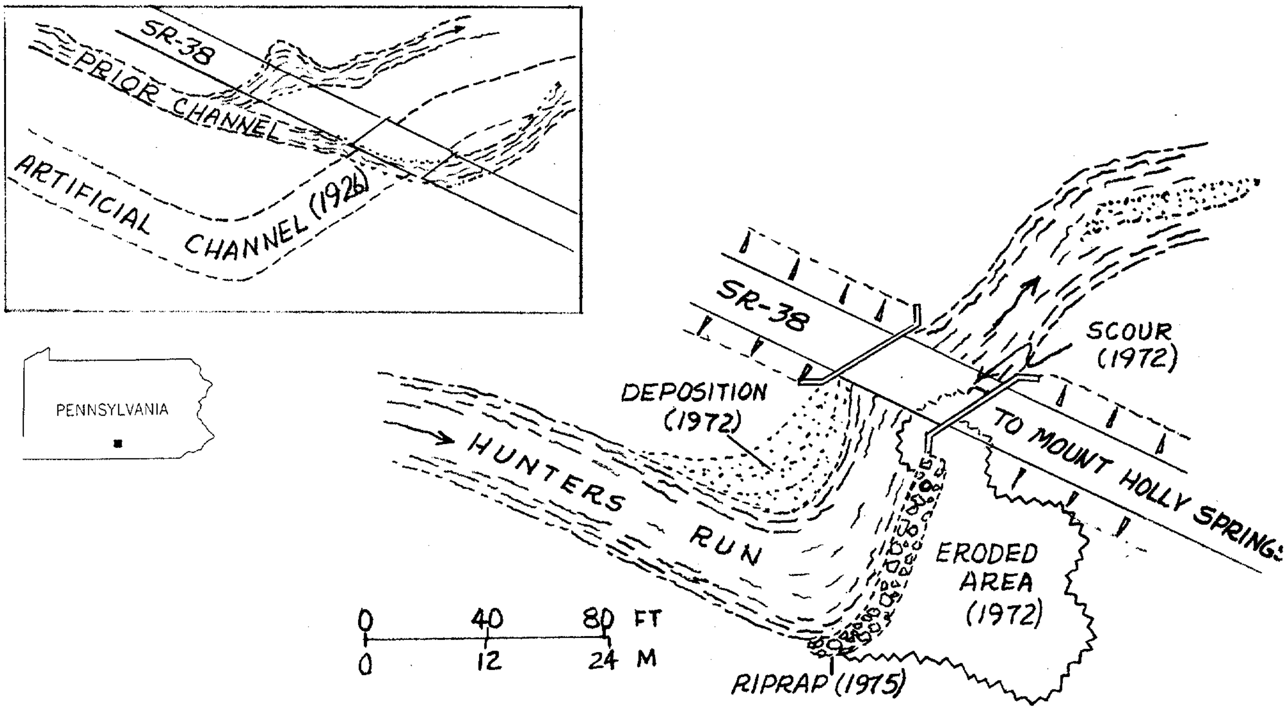


Figure 239. Right, plan sketch of SR-38 crossing at Hunters Run in 1976, and (upper left) channel alteration made in 1926 when SR-38 bridge was built.

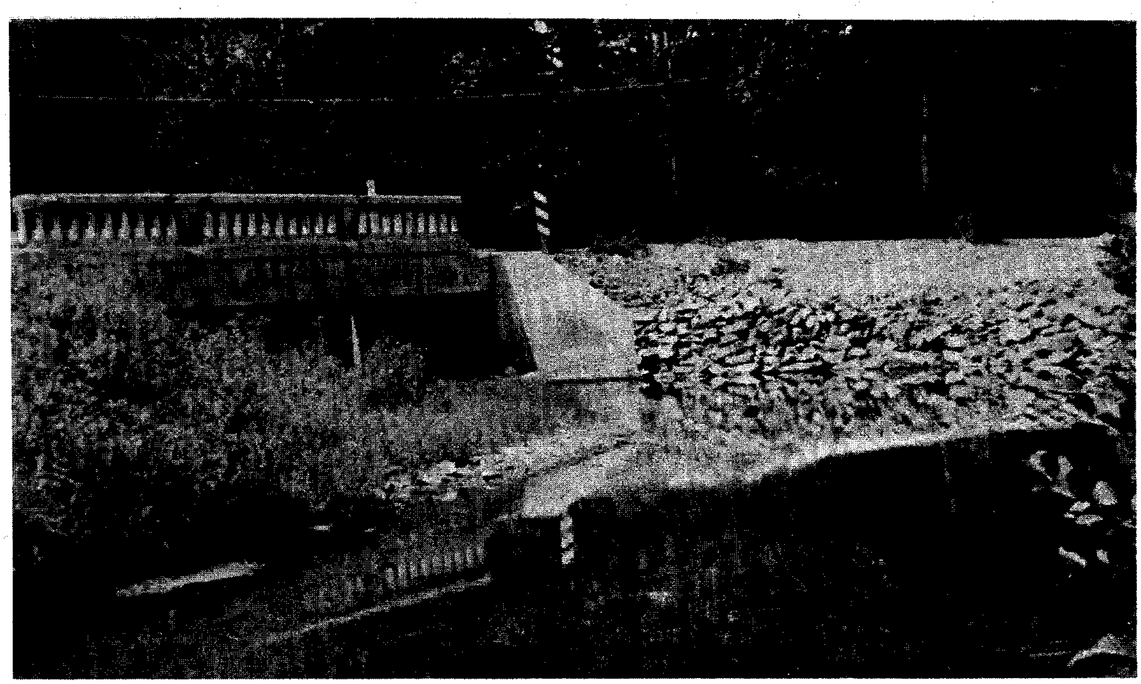


Figure 240. Downstream view of riprap retaining wall at SR-38 bridge in July, 1976.

SITE 257. BEAVER RIVER AT PH-26 NEAR FLAT VALLEY, SASKATCHEWAN, CANADA

<u>Description of site</u>: Location as shown in fig. 241. Bridge is 267 ft (81 m) in length, three 89-ft (27-m) precast, prestressed-concrete girder spans, two concrete wall-type piers in channel, round nose, not battered and with no metal shields on upstream side. Crossing is at a bend.

Channel width, about 250 ft (76 m). Stream is perennial, alluvial, sand bed, in valley of low relief, narrow flood plain. Channel is meandering (meanders are distorted because of confinement in narrow valley), equiwidth, not incised, cut banks general, silt-sand banks, tree cover at less than 50 percent of bankline (fig. 241).

<u>Countermeasures</u>:

1971 Bridge built. As part of crossing plan, a guide bank (spur dike) was built at the right abutment and three short spurs were built at the left bank upstream from the bridge. The guide bank is an earth embankment elliptical in plan, 200 ft (61 m) in length, revetted with riprap. The spurs (fig. 242) are of "fieldstone" (glacial boulders) having a maximum diameter of about 1.5 ft or 0.5 m), pointed downstream at about a 45^{0} angle, length about 40 ft (12 m), and separated by a distance of about 100 ft (30 m).

1974 Flood, R.I. estimated at 50 yr, with flow over roadway embankment south of bridge. Local scour occurred in bridge waterway, attributed to debris lodged against pier. Performance of spur dike considered good, and performance of spurs satisfactory. However, spurs were partly eroded and some bank erosion occurred between the spurs (fig. 243).

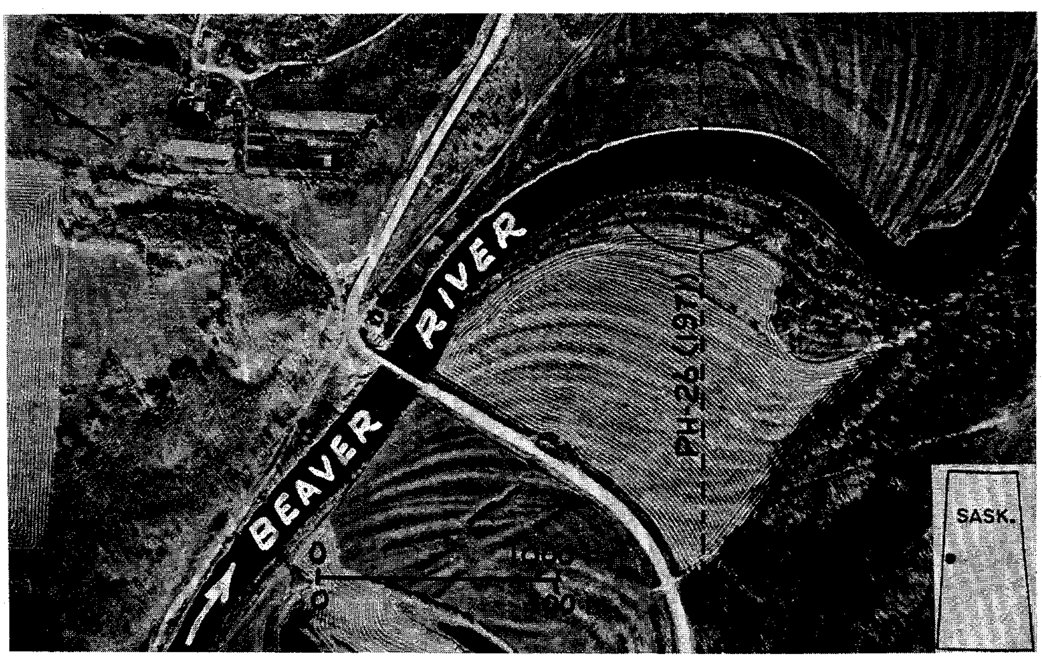


Figure 241. Aerial photograph of Beaver River at PH-26 crossing site, on May 10, 1968. Bridge shown was replaced by new bridge in 1971, at the location circled. (From Saskatchewan Dept. of Highways and Transportation.)

Discussion: Erosion of streambank between spurs is tentatively attributed to development of an eddy current downstream from each spur, which is considered a characteristic of impermeable spurs (jetties) by California Division of Highways (1970, p. 234).

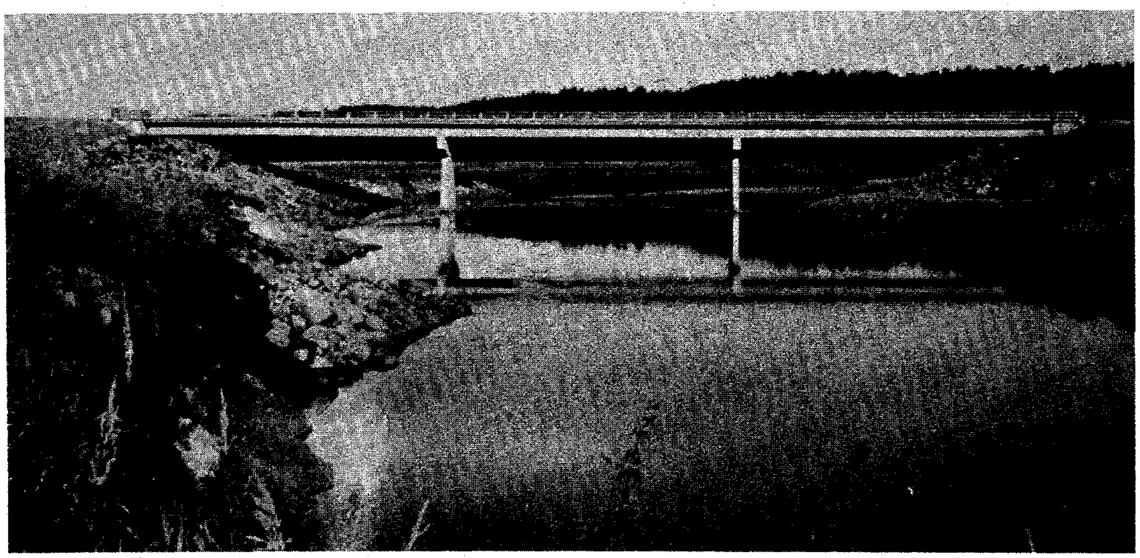


Figure 242. Downstream view of bridge built in 1971, showing spurs (at left) and part of guide bank (extreme right). (From Saskatchewan Dept. of Highways and Transportation.)

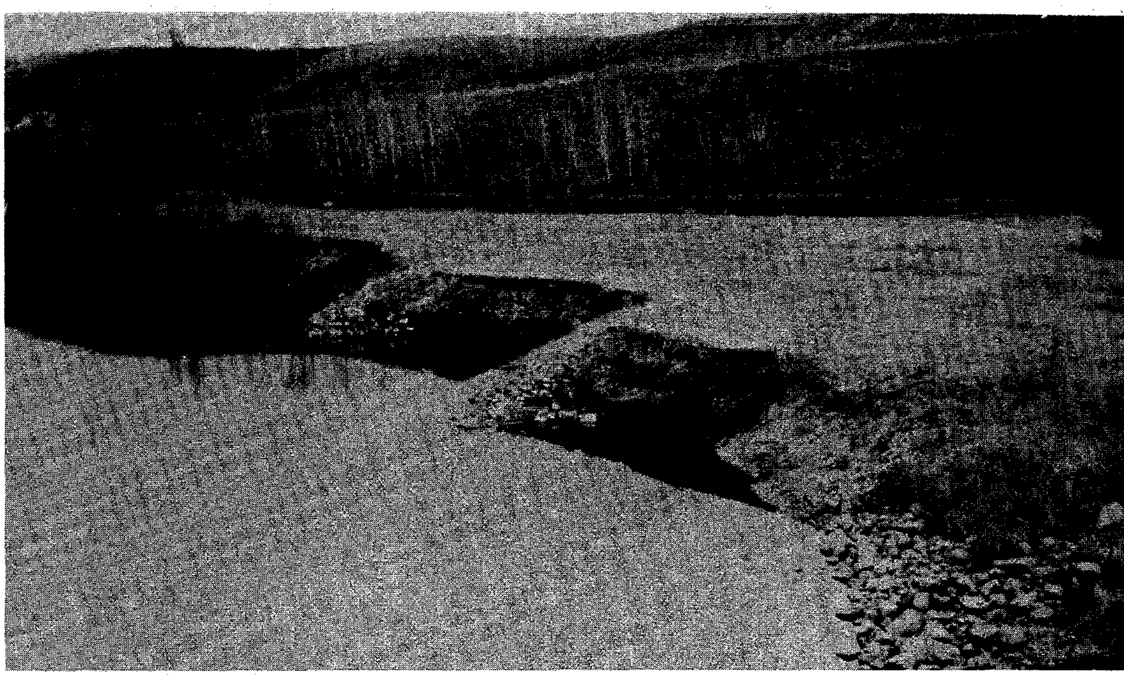


Figure 243. View of spurs, looking upstream from bridge, after 1974 flood. (From Saskatchewan Dept. of Highways and Transportation.)

SITE 258. NORTH SASKATCHEWAN RIVER AT PH-5 NEAR BORDEN, SASKATCHEWAN, CANADA

Description of site: Location as shown in fig. 244. Bridge is 852 ft (260 m) in length, concrete arch spans, three concrete wall-type piers in channel, pile foundations, spillthrough abutments. Piers have pointed concrete noses, not protected by metal shields.

Channel width, about 1,200 ft (360 m). Stream is perennial, alluvial, sand bed, in valley of moderate relief, little or no flood plain. Channel is sinuous, generally braided with narrow vegetated islands and bars (fig. 244), not incised, cut banks local, silt-sand banks.

Hydraulic problem and countermeasure:

1936 Bridge built. Braid on north side of crossing (dotted lines, fig. 244) was blocked off by approach embankment, thus confining flow to a single channel at bridge. "Fieldstone" riprap was placed at abutment fill slopes and at piers, but no river training works were constructed.

1937-59 Left bank receded about 150 ft (45 m), exposing piers (figs. 245 and 246).

1960 Six spurs, earth embankment revetted with "fieldstone", pointing upstream approximately at 45^{o} angle to bankline, were built at left bank upstream from bridge (figs. 247 and 248). Upstream spur about 50 ft (15 m) in length, longest spur about 150 ft (45 m), spacing about 120 ft (36 m).

1961-72 Sediment accumulated at spurs, which functioned satisfactorily as bank protection (fig 248). In May of 1972, about 80 yd^3 (61 m^3) of riprap were required to repair spurs.

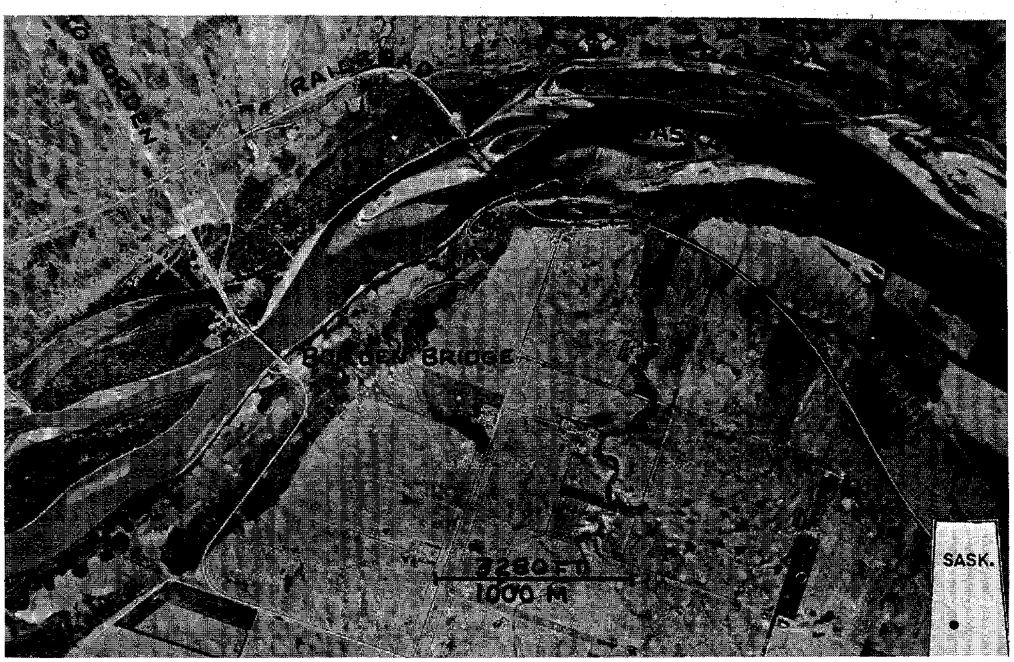


Figure 244. Aerial photograph of North Saskatchewan River at PH-5, in May, 1960. (From Saskatchewan Dept. of Highways and Transportation.)

Discussion: The braided channel is substantially constricted at the bridge, which has a length of 850 ft (259 m) as compared with 1,300-ft (396-m) length of the railroad bridge downstream (fig. 244). Erosion of the left bank at the highway bridge is attributed to the alinement of the larger of the two upstream channels. The upstream-pointing spurs have functioned satisfactorily as bank protection, but it is not possible to say how their performance would compare with similar spurs pointing downstream. It is noteworthy that the pointed concrete noses of the piers have not been damaged by ice, even though they are not protected by metal shield (fig. 249).

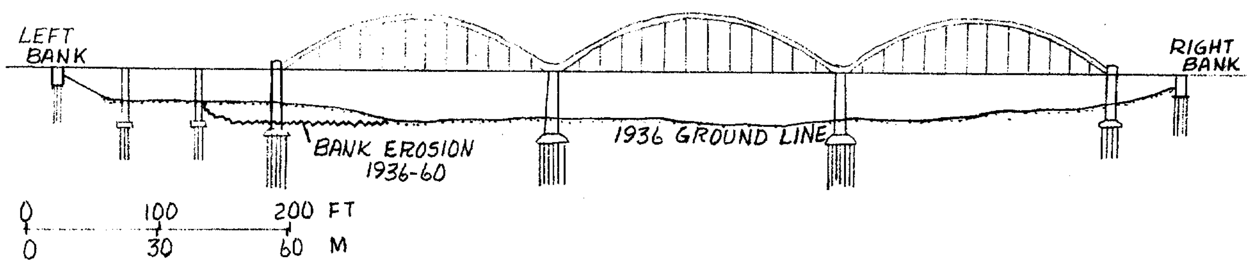


Figure 245. Elevation sketch of PH-5 bridge, North Saskatchewan River, showing bank erosion.

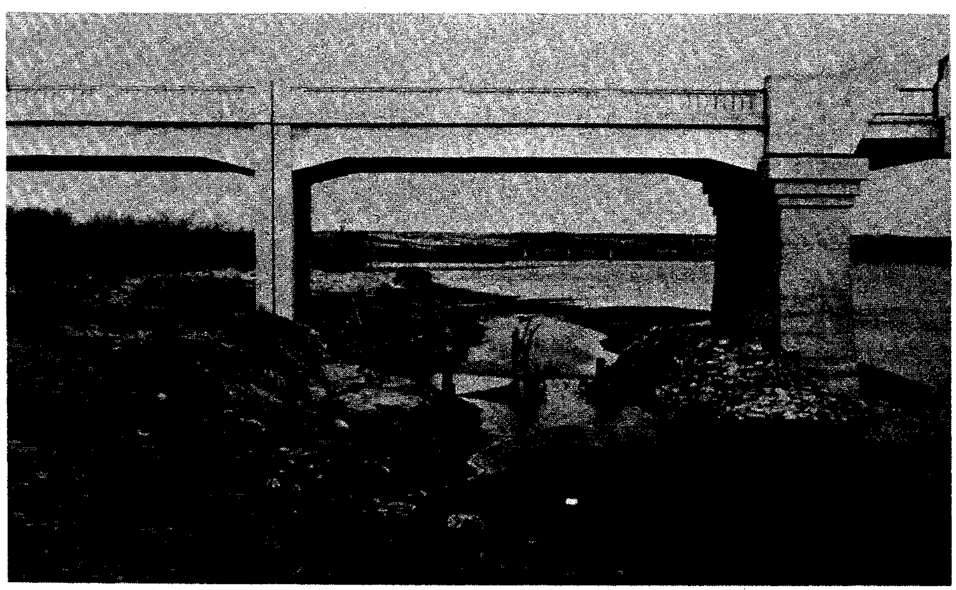


Figure 246. Erosion of left bank at PH-5 bridge, November 7, 1960.

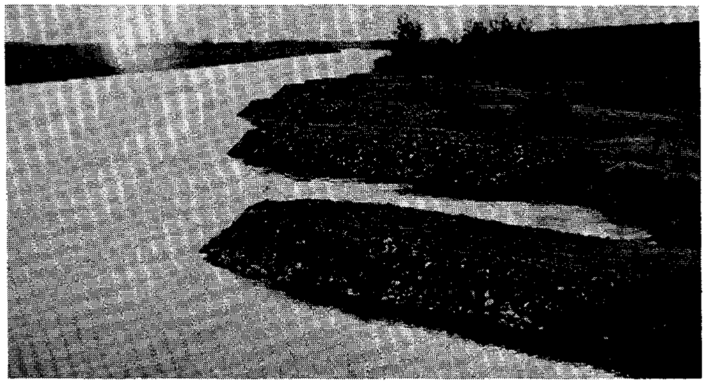


Figure 247. View, looking upstream, of spurs at PH-5 bridge, in 1961. (From Saskatchewan Dept. of Highways and Transportation.)

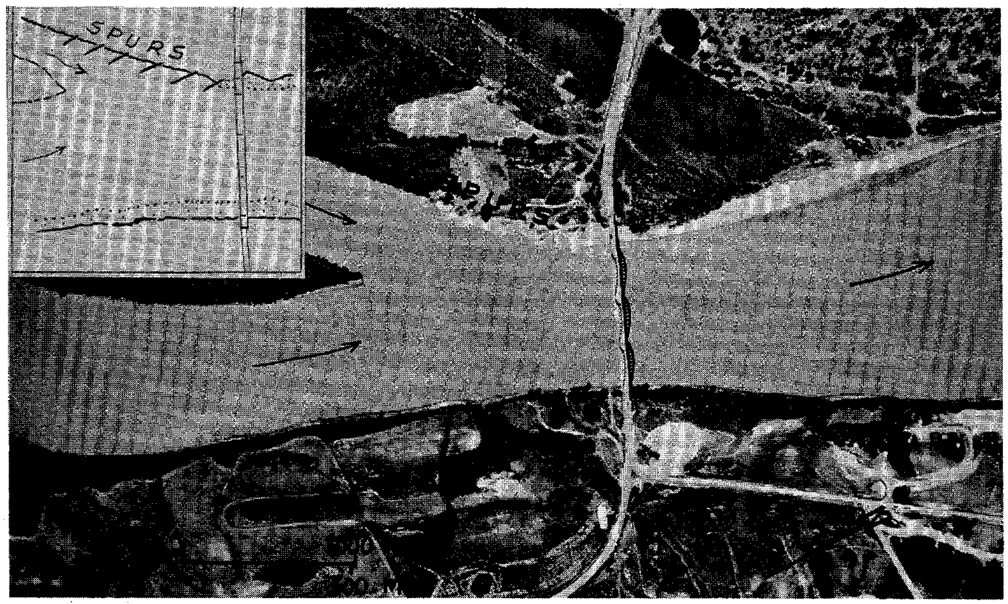


Figure 248. Aerial photograph showing accumulation of sediment at spurs. Inset, layout of spurs.

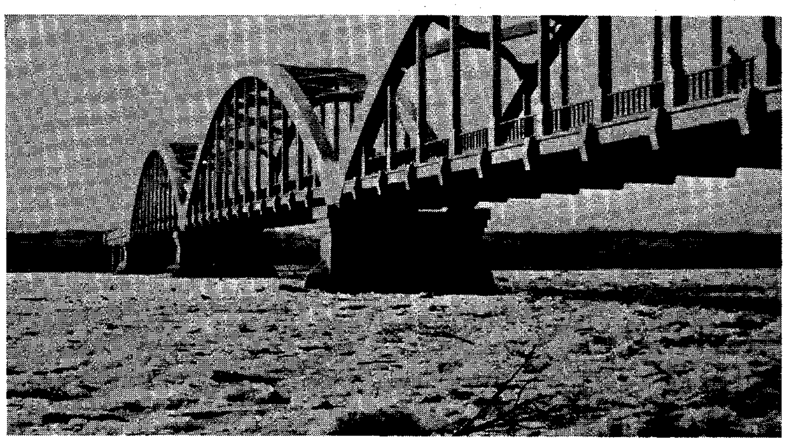


Figure 249. Floating ice, 10-18 in (25-46 cm) in thickness, at PH-5 bridge on April 14, 1969. Note pointed concrete nose on pier. (From Saskatchewan Dept. of Highways and Transportation.)

SITE 259. NORTH SASKATCHEWAN RIVER AT PH-376 NEAR MAYMONT, SASKATCHEWAN, CANADA

Description of site: Location as shown in fig. 250. Bridge is 1,035 ft (315 m) in length, three wall-type concrete piers on timber-pile foundations. Spillthrough abutments at 2.5:1 slope, protected with a fieldstone riprap blanket 1.5 ft (0.45 m) thick on a granular filter blanket 1 ft (0.3 m) thick.

Channel width, about 1,500 ft (457 m). Stream is perennial, alluvial, sand bed, in valley of moderate relief, little or no flood plain. Channel is sinuous, generally braided (fig. 250), not incised, tree cover at 50-90 percent of bankline. At the crossing site, there is no flood plain at the south bank, and the valley side is marked by many slump scarps, as indicated by the pattern of vegetation in fig. 250.

Countermeasure:

1974 Bridge built. To avoid construction of a very long bridge, the right approach embankment was extended into the channel for about 650 ft (198 m) and a guide bank, elliptical in plan, was built at the right bridge abutment (fig. 250). The guide bank is 500 ft (152 m) in upstream length, 140 ft (42 m) in downstream length, crest width 10 ft (3 m), side slopes 2.5:1, revetted with riprap and filter blanket as described above for abutment fill slopes. Top of guide bank is at elevation 1506 ft, as compared with elevation 1503 ft for the 100-yr R.I. design flood, which may reach 1510 ft if ice effects accompany the flood.

Discussion: Although the guide bank has not yet been tested by flood, the general feasibility of constricting wide braided channels at bridge crossings has been shown by experience in Saskatchewan. The guide-bank length was decided upon rather arbitrarily, as no firm hydraulic criteria for length have been established.

Figure 250. Aerial photograph of North Saskatchewan River at PH-376 crossing, on May 2, 1976. (From Saskatchewan Dept. of Highways and Transportation.)

SITE 260. QU'APPELLE RIVER AT PH-11 NEAR LUMSDEN, SASKATCHEWAN, CANADA

(Information on this site is from an unpublished case history prepared by L. J. Hamblin, Saskatchewan Department of Highways and Transportation, for presentation to the Project Committee on Bridge Hydraulics, Roads and Transportation Association of Canada).

<u>Description of site</u>: Lat 50°39', long 104°52', location as shown in fig. 251. Dual bridges, each 150 ft (46 m) in length, two concrete wall-type piers in channel, spillthrough abutments at channel banks. Crossing is at bend. Piers and abutments are skewed 15° from perpendicular to bridge center line, alined to low flow in channel but not alined with flood flow.

Bankfull discharge, about 2,500 ft^3/s (71 m^3/s); valley slope, about 0.0001; channel width, about 50 ft (15 m). Stream is perennial, partly regulated, silt-clay bed, in valley of moderate relief, wide flood plain with natural levees and flood basins. Channel is highly meandering, equiwidth, not incised, cut banks rare, silt-clay banks.

<u>Hydraulic problem and countermeasure</u>:

1961 Bridge built, designed for 50-yr R.I. flood of 4,200 ft^3/s (119 m^3/s), frequency determination based on 49-year period of record.

1969 Spring flood; maximum daily discharge averaged 6,600 ft^3/s (187 m^3/s) almost 60 percent higher than design discharge. No hydraulic problems at bridge, but need for a spur dike was apparent because of observed turbulence where flow along right roadway embankment entered channel 50 ft (15 m) upstream from bridge. However, since bridge had performed satisfactorily during this flood, which was substantially greater than the previously determed 50-yr R.I. flood, the spur dike was not constructed.

1974 Spring flood; maximum daily discharge averaged 15,400 ft^3/s (436 m^3/s). Flow entering channel along right roadway embankment produced eddy that eroded bank of channel, resulted in upstream flow adjacent to right abutment (of upstream bridge), eroded the abutment fill-slope, and undermined the abutment (figs. 251 and 252). Upstream bridge closed to traffic for 3 days, while a retaining wall of steel sheet-piling was driven back of the abutment. The left abutment of the downstream bridge was also undermined, to a lesser degree. Erosion of the left abutment of the downstream bridge, which is on the inside of the bend, is tentatively attributed to straightening of the flow alinement by the bridge.

An attempt was made to monitor scour during the flood, but no readings were obtained from the sonic equipment, presumably because of severe turbulence. An average velocity in the bridge waterway of 7.5 ft/s (2.3 m/s) and a maximum velocity of 10.4 ft/s was measured during the flood, but local scour was not detected (fig. 253).

1975 Countermeasures installed. The channel was restored with a slightly larger waterway area, both banks were riprapped, and a short spur dike (elliptical, revetted earth embankment, 75 ft or 23 m in length) was constructed at the right abutment of the upstream bridge (fig. 254).

Riprap consisted of rounded glacial stones, 20 percent less than 5 in (12.5 cm) in diameter, 80 percent less than 12 in (20 cm). Thickness of riprap blanket, 1 ft (0.3 m).

1976 Flood on April 7, peak discharge 5,950 ft^3/s (168 m^3/s), water level about 1 ft (0.3 m) below bottom of bridge girders. Countermeasures performed well, no reported damage at bridge.

Discussion: This case illustrates the uncertainties of the flood frequency determinations on which bridge designs are based. The alinement of the natural channel upstream from the bridge (note that the channel trends almost parallel to the roadway embankment for several hundred feet) would ordinarily be considered a hazard, but the natural channel transmits only a small part of extreme flood flows. Also, the channel is stable, as indicated by its morphologic features: meandering, equiwidth, natural levees, no bare point bars, not braided, silt-clay banks, banks vegetated. Canadian bridge engineers who have studied the site and the problem are of the opinion that no serious erosion would have occurred during the extreme 1974 flood if the spur dike had been present. Although the riprap is of rounded stones and rather smaller than class I, it performed well during both the 1969 and 1976 floods.

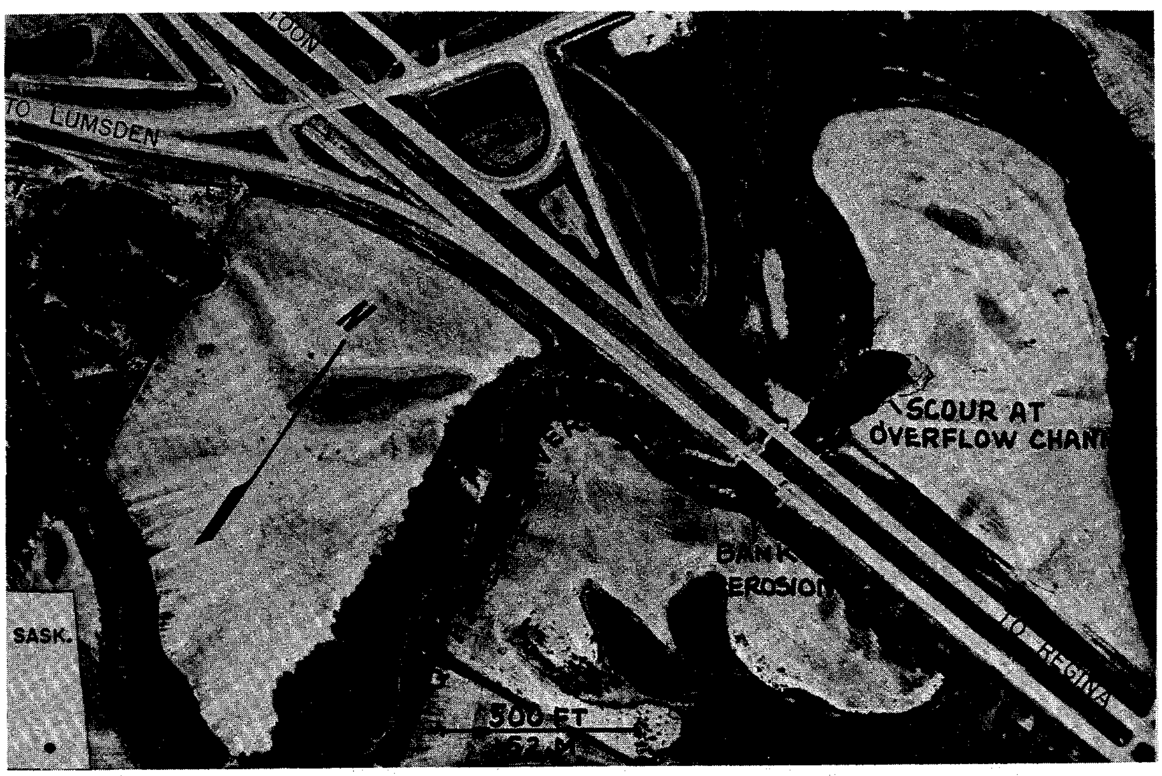


Figure 251. Aerial photograph of Qu'Appelle River after 1974 flood at PH-11 near Lumsden, Saskatchewan, Canada. (From Saskatchewan Dept. of Highways and Transportation.)

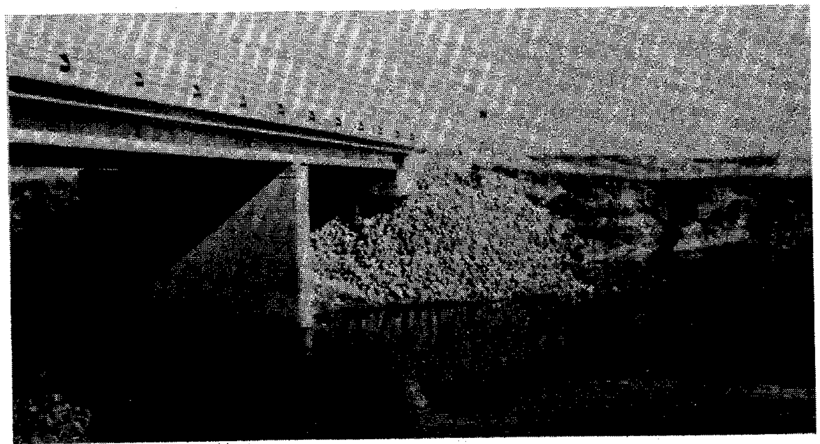


Figure 252. Bank erosion at upstream side of right abutment, south bridge of PH-11. (From Saskatchewan Dept. of Highways and Transportation.)

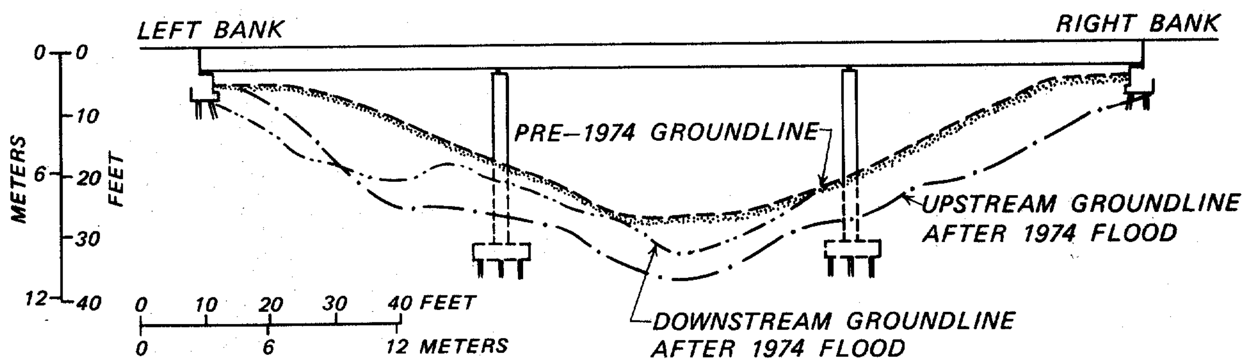


Figure 253. Cross section of Qu'Appelle River at PH-11, showing general scour as measured after 1974 flood.

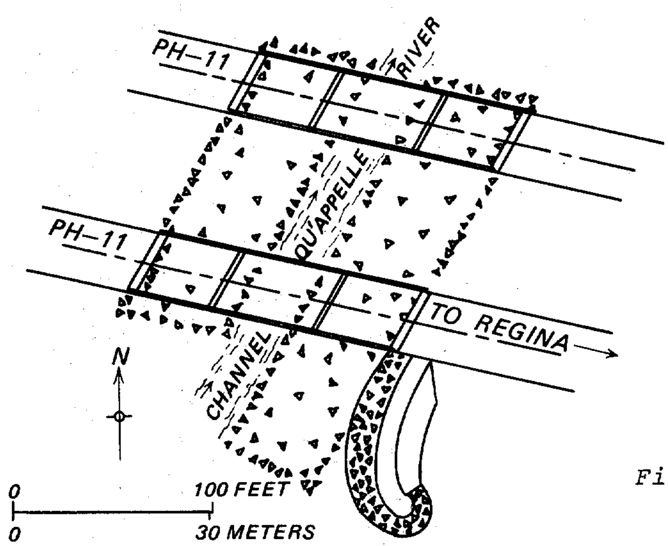


Figure 254. Guide bank and riprap revetment installed at PH-11 bridge after 1974 flood.

SITE 261. SOUTH FORK FORKED DEER RIVER AT US-51 NEAR HALLS, TENN.

Description of site: Lat 35°57', long 89°24', location as shown in fig. 255. Dual bridges, built in 1963, had a 53-ft (16-m) main span supported by wall-type piers in the main channel, and thirty 28-ft (8-m) approach spans supported by concrete pile bents. In 1975, both bridges over the main channel were rebuilt, with 75-ft (22.5-m) main span supported by hammerhead piers. Spillthrough abutments, set well back from the main channel, were protected with sacked concrete in 1963 and have remained stable.

Drainage area, 1,038 mi^2 (2,688 km^2); bankfull discharge, 1,000 ft^3/s (28 m^3/s); width where bordered by natural vegetation, 80 ft (24 m). Stream is perennial, alluvial, sand bed, in valley of moderate relief, wide flood plain. Natural channel had a sinuousity of about 2.5, but channel has been straightened. Natural channel is equiwidth, not incised, cut banks rare, silt-sand banks. Channel was first straightened and enlarged in the 1920's by local drainage districts; but, probably because the natural flood plain forest was not cleared, the banks remained stable. In 1969, the Corps of Engineers straightened and enlarged a reach about 3 mi (4.8 km) in length downstream from the bridge, reducing the length about 20 per cent. During the past few decades, and particularly in recent years, the flood plain has been cleared of trees for agricultural purposes.

Hydraulic problem and countermeasure:

1963 Bridge built, across previously straightened reach of channel, sacked concrete at abutments.

1969 Channel enlarged and straightened downstream from bridge, by U.S. Army Corps of Engineers.

1970-71 Left bank receded an average distance of 14 ft (4 m) during this period. Peak discharge during period, 7,590 ft^3/d (215 m^3/s), R.I. of flood, 1.5 yr. Timber-pile retard built at left bank near bent 7; single row of pile with wood face planks, extending from the downstream end of bent 7 for a distance of 125 ft (37.5 m) upstream.

1971-73 Peak discharge during period 26,540 ft^3/s (751 m^3/s), 17-yr R.I. Bankfull stage occurred several times, high flows sustained for periods of weeks. Left bank continued to erode behind retard, average distance of recession, about 7 ft (2 m). Bent 7 became exposed below ground line, concrete was poured at base to prevent further erosion. Slumping from the left bank deflected flow toward the right bank, causing rapid erosion and failure of bent 8. South lane of bridge closed.

1975 Both lanes of bridge rebuilt, with new piers having deeper footings and less area normal to flow (fig. 256). Single-row, timber pile retard built along both banks in vicinity of bridge (fig. 257). A large scour hole in the center of the channel downstream from the bridge, attributed to flow constriction during bridge construction, was filled with gravel.

1977 Effectiveness of timber-pile retard not yet tested, but area between retard and bank is not accumulating sediment. The lowermost face plank on the retard is about 5 ft (1.5 m) above the streambed. From experience in Oklahoma, Keeley (1971) recommends that face planks be extended to, or below, stream bed elevation. In addition, the upstream end of the retard seems to be keyed into the bank for an insufficient distance. Vegetation is becoming re-established on the banks, which appear more stable now than in the recent past.

Discussion: The 1963 bridge failed because of channel degradation and concurrent bank recession, which are directly attributable to straightening of the channel for drainage purposes and clearing of the banks and flood plain for agricultural purposes. Channel width increased by a factor of 2, approximately, between 1969 and 1976. The clearing of vegetation is apparently the most critical factor, because channel straightening in the 1920's, which was not accompanied by extensive clearing, did not result in significant bank instability. The timber pile retard installed in 1971 was inadequate, in view of the seriousness of the problem. Bank recession might have been controlled by an adequate retard or other countermeasure, but channel degradation is more difficult to control.

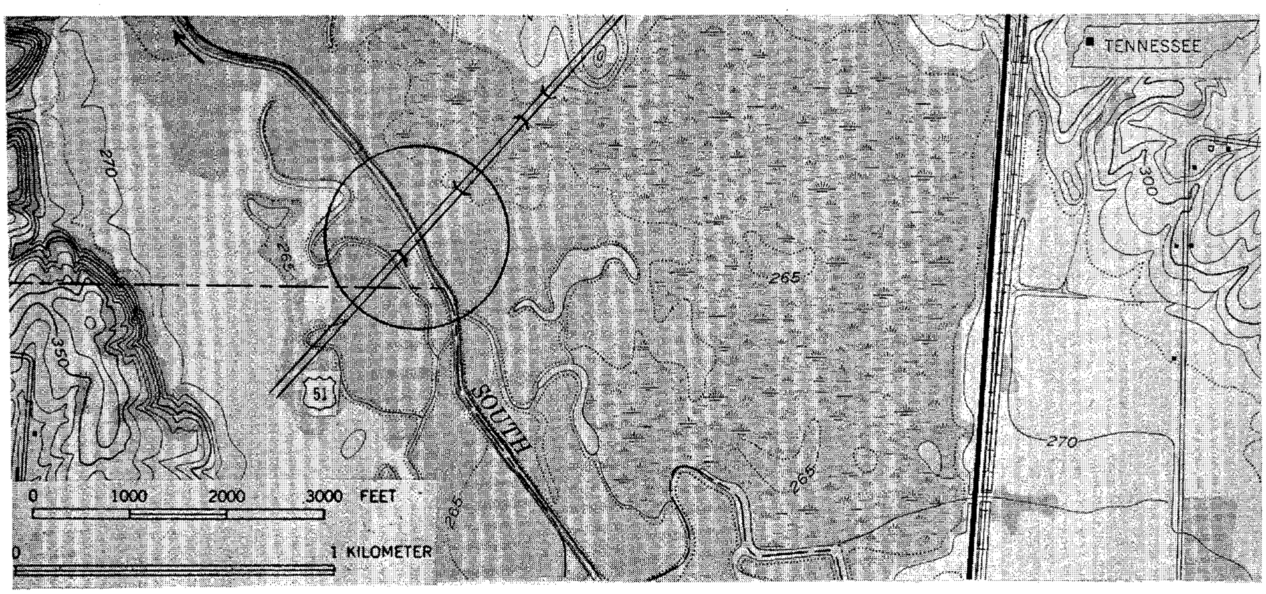


Figure 255. Map showing South Fork Forked Deer River at US-51 crossing (circled). (Base from U.S. Geol. Survey Fowlkes, Tenn., 7.5' quadrangle, contour interval 10 feet, 1952.)

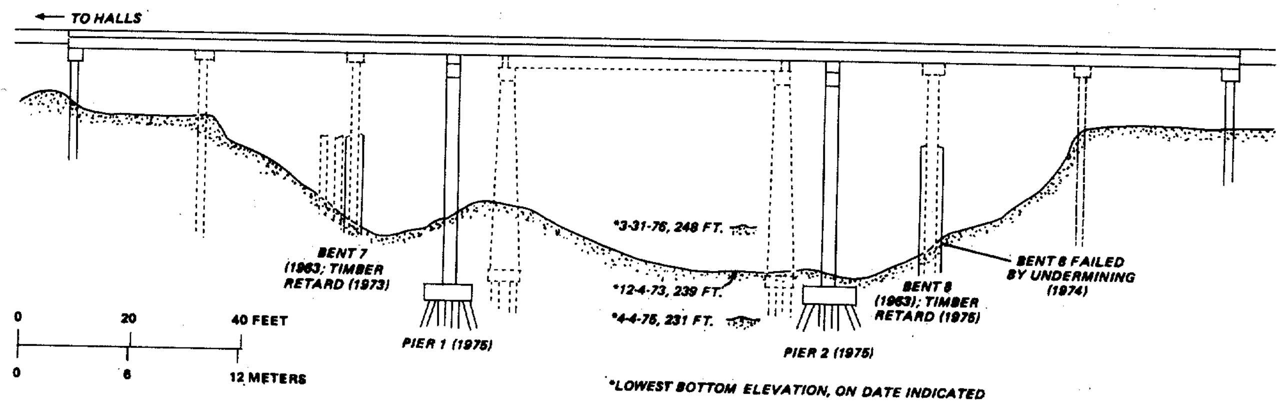


Figure 256. Elevation sketch of US-51 bridge. Piers and pile bents of 1963 bridge, removed during construction of 1975 bridge, are shown in dashed lines.

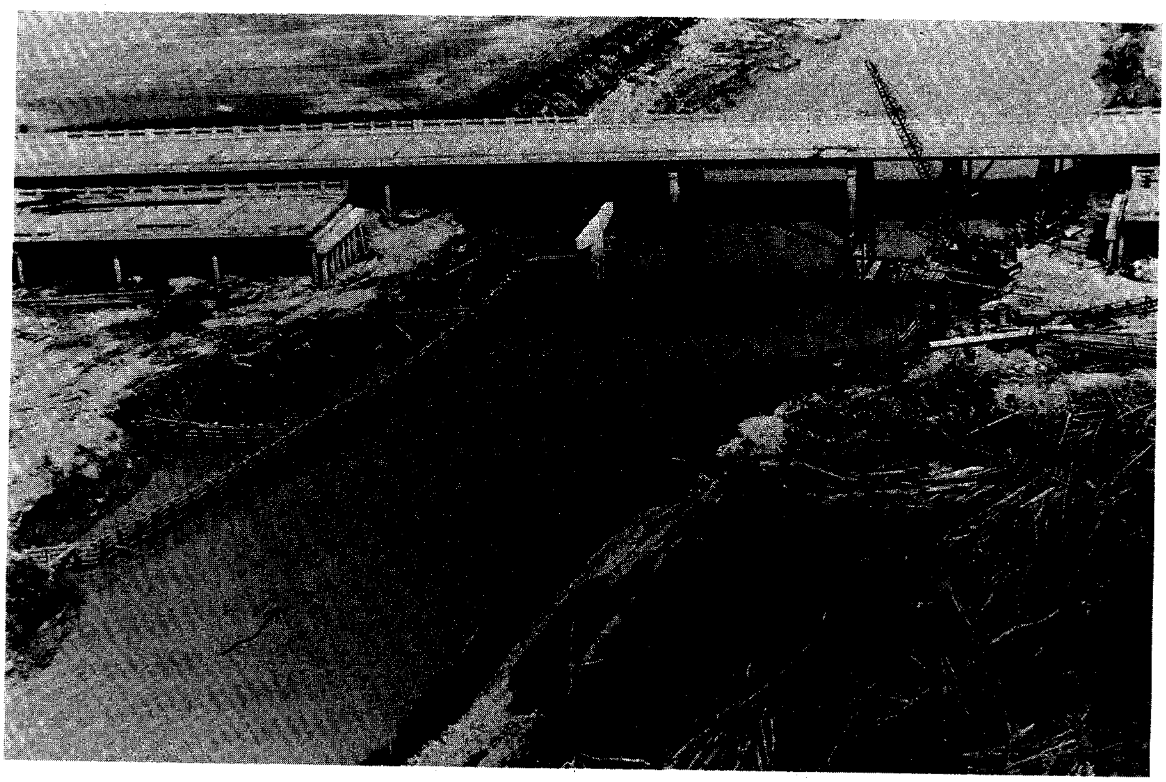


Figure 257. Aerial oblique view on June 3, 1975, showing reconstruction of upstream bridge, US-51 at South Fork Forked Deer River. (From Tennessee Dept. of Transportation.)

SITE 262. OBION RIVER AT COUNTY ROAD S-8005 AT LANE, TENN.

Description of site: Lat 36°11', long 89°22', location as shown in fig. 258. Bridge over main channel, built in 1963, 380 ft (116 m) in length, crossing included two 399-ft (122 m) relief bridges on left bank flood plain. Two concrete wall-type piers and 8 pile bents in channel, piers 1 and 2 and bents 4 and 5 had pile foundations. Spillthrough abutments, fill slopes protected with sacked concrete. Crossing was satisfactory at time of design and construction, but subsequent events, all man-induced, led to loss of bridge.

Drainage area, 2,000 mi^2 (610 km^2); bankfull discharge, about 13,000 ft^3/s (368 m^3/s); channel width of natural stream, about 125 ft (38 m) valley slope, 0.00024. Stream is alluvial, sand-silt bed, in valley of moderate relief, wide flood plain. Natural channel (before straightening) meandering, equiwidth, not incised, coherent silt-sand banks, dense hardwood forest on flood plain and along banks.

Hydraulic problem and countermeasure:

1959 Bridge built. Older bridge at same site replaced because of its poor structural condition, no hydraulic problems on record. New approach embankment south of bridge was offset upstream from older. Old embankment was deeded to private landowner, who subsequently converted it to a levee by filling relief openings and raising height, thus impeding flow through the relief bridges of the new crossing.

1963 Corps of Engineers completed enlarging and straightening of the Obion River along a reach extending 2 mi (3 km) downstream from the town of Lane, and 2.7 mi (4.5 km) upstream. Channel slope was increased from about 0.00017 to 0.00030. At the bridge, the bottom of the newly dredged channel was at the elevation of the pier footings.

1972-74 During this period of above normal rainfall, about 22 floods near or above bankfull stage occurred, and the largest two of these were of 5-yr R.I. General scour probably occurred during this period and particularly during the 5-yr R.I. flood of January 14, 1974, which had a peak discharge of 38,700 ft^3/s (1,095 m^3/s). A large amount of drift accumulated at each of the main piers. On January 27, 1974, during a flow of about 7,700 ft^3/s (218 m^3/s, bent 4, p-ers 1 and 2, and four spans of the bridge fell into the channel, with the loss of two lives. Field inspection indicated that the failure began with the subsidence of bent 4. Soundings made after bridge failure (fig. 259) indicate that about 8 ft (2.5 m) of general scour at the piers was not determined. Maximum depth of scour at the bridge is assumed to have reached pile tips, at about elevation 218 ft. A scour hole having a depth of 32 ft (10 m), downstream from the bridge, was tentatively attributed to flow disturbance from the fallen spans.

1974 New bridge built, having a longer center span, fewer bents in channel, and deeper footings. No further countermeasures applied.

1976 Lateral erosion of right bank at bridge is attributed to flow deflection at upstream bend, and erosion of left bank is attributed to overbank flow deflected into channel by left approach embankment. It seems that a spur dike is needed at the left embankment to avoid serious scour in the channel during a major flood.

Discussion: The major factors in the failure of the 1959 bridge are evidently man-induced. Diversion of overbank flow into the channel at the bridge was caused by conversion of the old approach embankment into a levee and also by building of a spoil bank along the left bank upstream from the bridge. The supply of drift was increased by not disposing of trees felled during dredging and agricultural land clearing. Exposure of pier footings is attributed to dredging of the channel, and to increase of channel slope by straightening.

Figure 258. Vertical aerial photograph of Obion River at S-8005 bridge, on November 3, 1971, just prior to bridge failure. (From U.S. Dept. of Agriculture.)

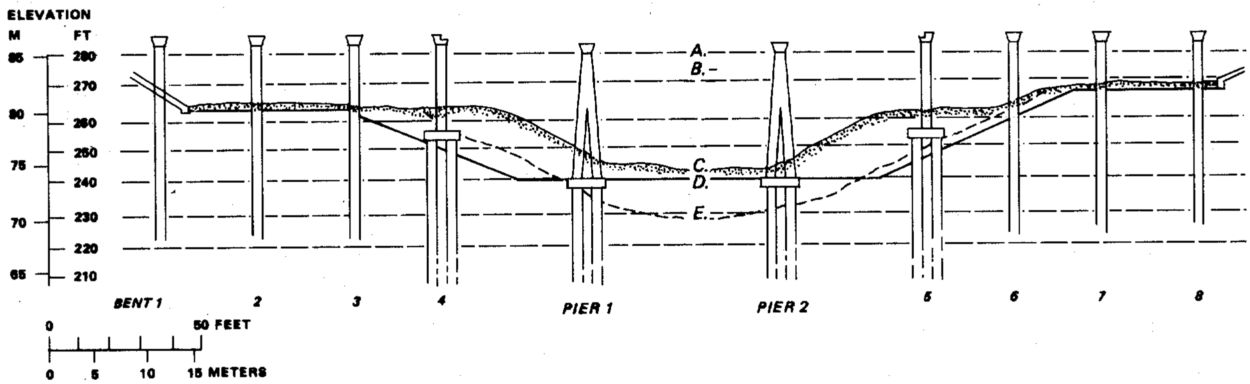


Figure 259. Channel cross section of Obion River at S-8005 bridge, showing bridge substructure, water levels, and channel bed at different dates. A, water surface elevation at bridge design flood, 50-yr R.I.; B, water surface at flood of 1-14-74; C, ground line from bridge plans, 1957; D, ground line from channel dredging plans, 1963; E, channel bed from soundings by Tennessee Dept. Transportation, after bridge failure.

Figure 260. Aerial photograph of turnpike bridges at Mountain Creek, prior to installation of bank revetment, on December 12, 1975. (From Howard, Needles, Tammen, and Bergendoff, engineering consultants.)

SITE 263. MOUNTAIN CREEK AT DALLAS-FT. WORTH TURNPIKE, TEX.

Description of site: Lat 32°46', long 96°56'. Dual bridges, each 700 ft (213 m) in length; each bridge has 10 pile bents with three round concrete pile, 3 ft (0.9 m) in diameter, cast in drilled holes, and founded a minimum distance of 10 ft (3 m) in shale bedrock. Spillthrough abutments, protected with concrete pavement. Bents are skewed 28° to flow direction of Mountain Creek, and a meander is located just upstream from the crossing site (fig.260).

Drainage area, 298 mi^2 (772 km^2); channel width, about 100 ft (30 m); stream is perennial, regulated by reservoir, alluvial, silt-clay bed, in valley of moderate relief; channel is sinuous, wider at bends with poorly developed point bars, somewhat incised, conspicuously slumped banks where tree cover is lacking, coherent silt-clay banks, tree cover 50-90 percent of bankline.

Hydraulic problem and countermeasure:

1956 Bridge built. Since that time, the bridge has been inspected annually by the engineering firm of Howard Needles Tammen and Bergendoff, which prepared in 1976, a comprehensive report on causes of damage and countermeasures recommended. Information given herein is mainly from that report.

1966-75 During annual inspections, cracks were noted in the columns of bents S6, S7, and S4 (fig. 261) and N7. In October of 1975, it was determined that the columns of bent N7 were about 3.5 in (9 cm) out of plumb as measured from ground line to cap beams. A detailed investigation was carried out, including test corings and laboratory strength tests of foundation soils, slope stability analyses, hydraulic study of the bridge waterway, and structural analysis of the columns.

As a result of this investigation, cracking of the columns was attributed to reduction in lateral support by bank erosion, which exposed columns to a maximum depth of about 15 ft (4.6 m) below construction ground line (fig. 261). The erosion was not attributed to contraction scour, because there is little or no flow contraction at the bridge. Instead, it is attributed to the skewness of the piers (figs. 261 and 262) and to flow deflection by the upstream meander. The banks, although they are cohesive, are undercut by the stream, become fissured, and fail mainly by slumping. Bank materials are described as "silty to fat waxy clays", and triaxial compression tests on undisturbed samples yielded cohesion values ranging from 360 to 1,680 lb/ft^2 (15 to 156 kg/m^2). The slumping does not occur because of internal failure (that is, without stream action) because even the steepest existing slopes have safety factors of 1.3 or higher.

Reduction in lateral support, at one side of a column, also accounts for the lateral column displacement. The position of the cracks on the columns is localized at the splice between the vertical steel reinforcement bars in the column and those in the drilled shaft.

Countermeasures were recommended and have been installed as described below. It was recognized that the hydraulic situation at the bridge would be much improved by realinement of the natural channel, but this was not recommended because of possible upstream or downstream effects.

1977 Cracked bent columns were repaired by the addition of a reinforced concrete collar (fig. 263A) 1 ft (30 cm) in thickness, extending above and below the intersection of the drilled shaft foundation and the column. Both banks were revetted with a rock-and-wire mattress about 6 in (15 cm) in thickness (serving as a filter blanket) overlain by dumped rock riprap having a size gradation similar to class I riprap (fig. 263B). The revetment was secured at the base by a footing trench and at the end, by a gabion wall (fig. 263C).

Discussion: Inasmuch as the pier columns are round, the bridge designers probably believed that skewness of the bents to flow direction would be of little consequence, but this evidently proved not to be the case. The banks of the natural channel upstream from the bridge are much scarred by slumping (fig. 260) and this might have served as a warning of bank instability at the bridge. Flow alinement through the bridge could have been (and could be) improved by a small amount of channel alteration. In view of the fact that the stream is not very sinuous and the banks are already unstable, straightening of a short reach would not likely have any significant consequences with respect to channel stability.

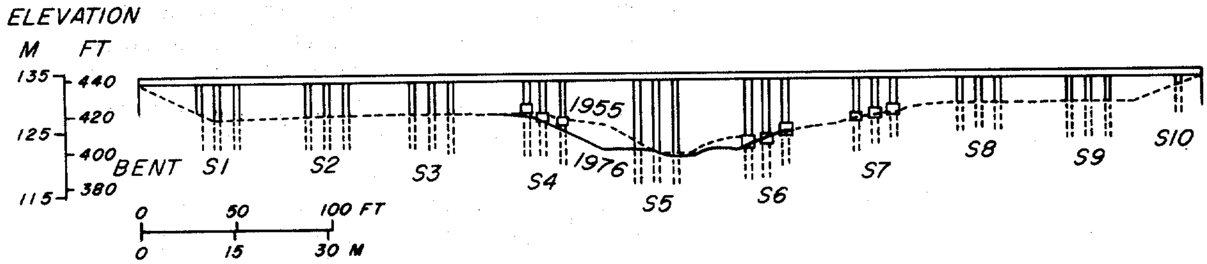


Figure 261. Elevation sketch of south bridge, as projected in a plane normal to flow direction of stream. Collars for strengthening of bent columns are shown.

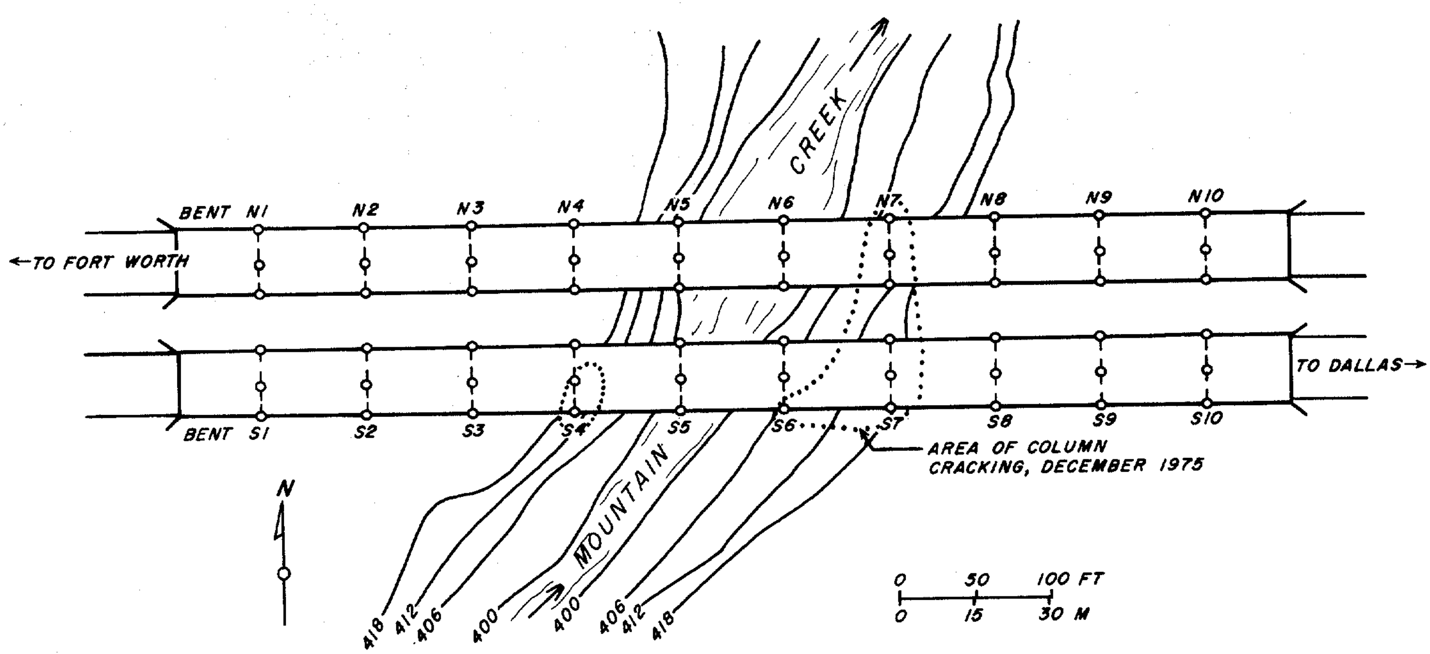


Figure 262. Plan sketch of turnpike bridges, showing contours of creek channel and location of cracked columns.

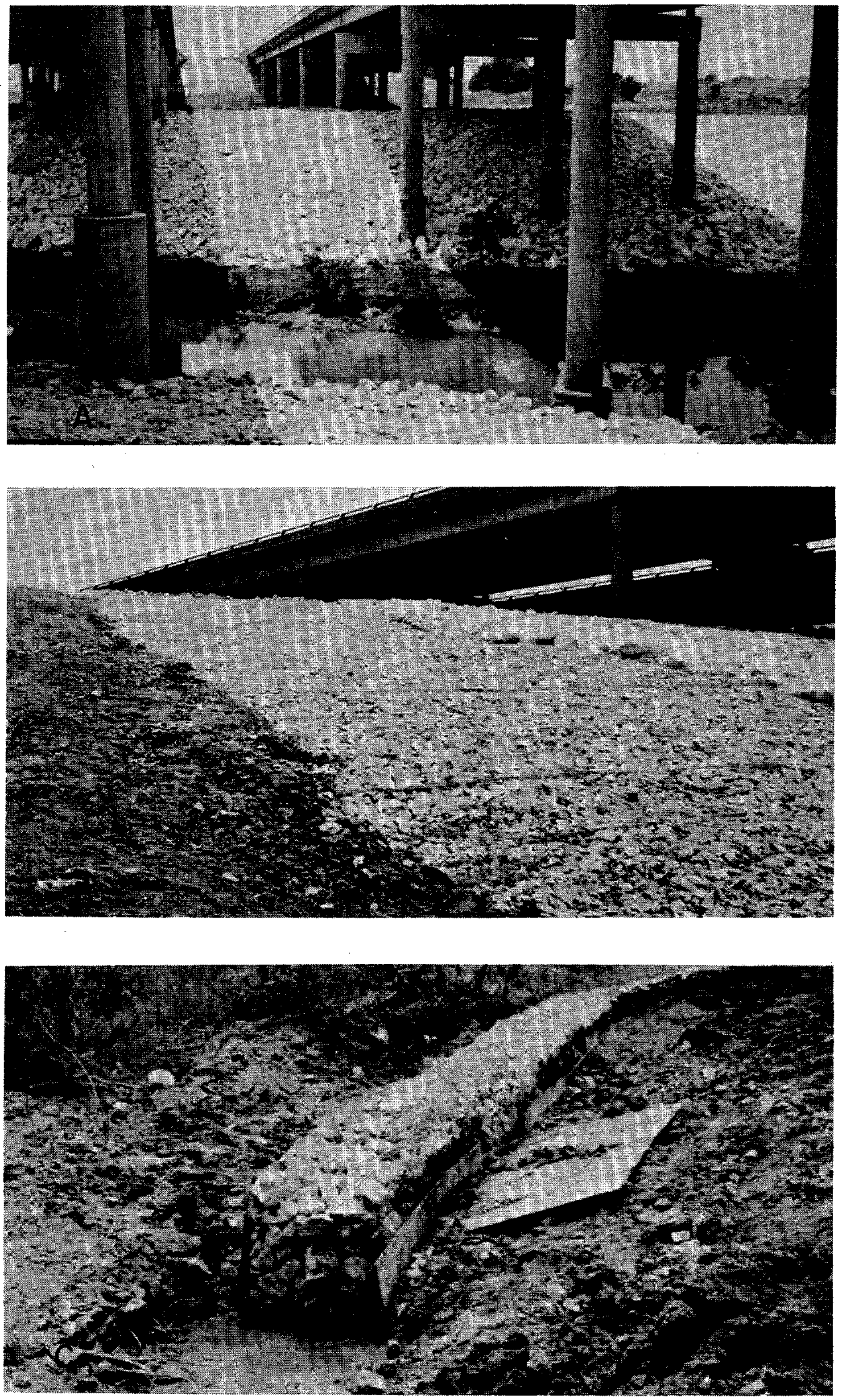


Figure 263. A, view of revetment installed on west bank and of collars on bent columns. B, view of installation of revetment on east bank, June 1, 1977. Rock-and-wire mattress, serving as a filter blanket, has been laid on graded slope at center and, at upper right, has been topped by rock riprap. C, view of gabion wall that will serve to anchor end of revetment on west bank, June 1, 1977.

SITE 264. SAUK RIVER AT MT. BAKER--SNOQUALMIE NATIONAL FOREST ROAD BRIDGE 3211-5.8 NEAR DARRINGTON, WASH.

Description of site: Lat 48°10', long 131°28', location as shown in fig. 264. Bridge is 135 ft (41 m) in length, native fir-log stringers, one wall-type pier in channel, set on bedrock, and one cross-braced timber-pile bent having spread footing on alluvium. Slopes of spillthrough abutments are natural bouldery bank materials, no wing walls. Crossing is at bend.

Drainage area, 152 mi^2 (394 km^2); bankfull discharge, about 300 ft^3/s ($8.5 m^3/s$); valley slope, 0.0075; width, about 120 ft (36.5 m). Stream is perennial, non-alluvial in reach crossed by bridge, cobble and boulder bed, in valley of high relief, little or no flood plain. Channel is sinuous, cobble and boulder banks.

Hydraulic problem and countermeasure: During a flood in 1975, (R.I. about 13 yrs) and a discharge of 19,000 ft^3/s (538 m^3/s), a log jam formed at the left side of the bridge and diverted the flow toward the timber-pile bent, the spread footing of which was undermined, causing the bridge to subside. As a countermeasure, the bent was rebuilt in 1976 on a foundation of eight 23-ft (7 m) steel H-pile; these were capped with a concrete footing and class II riprap was placed around the footing (fig 265). This countermeasure has not yet been tested by flood.

Discussion: The spread footing undermined by flood was set on loose sandy gravel containing small boulders and cobbles. Accumulation of the log jam is not attributed to low clearance but to location of the bridge on a bend in the stream.

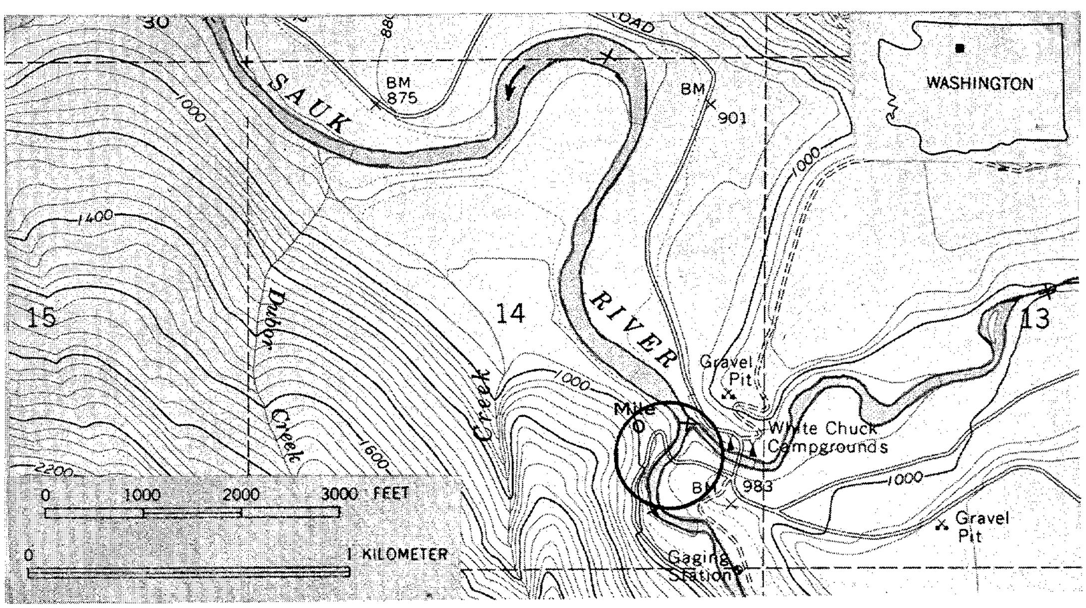


Figure 264. Map showing Sauk River at Forest Service bridge 3211-5.8, which is indicated by circle. (Base from U.S. Geol. Survey White Chuck Mtn., Wash., 7.5' quadrangle, contour interval 40 feet, topography from aerial photographs taken 1953).

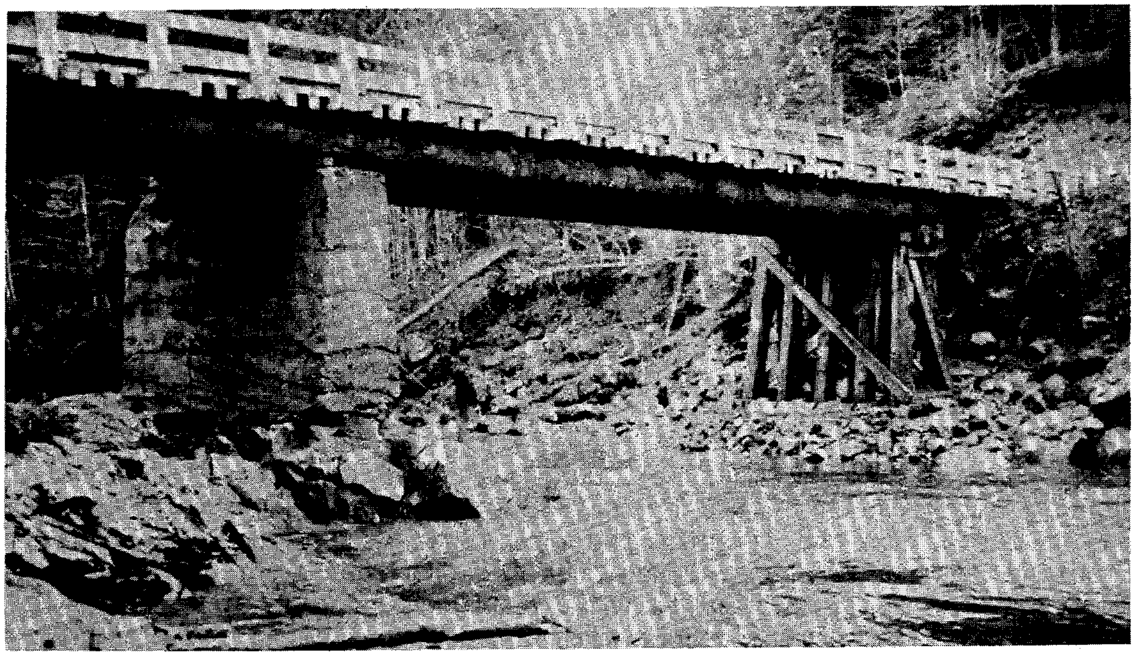


Figure 265. Downstream view of Sauk River bridge, showing (at right) timber-pile bent that was rebuilt in 1976.

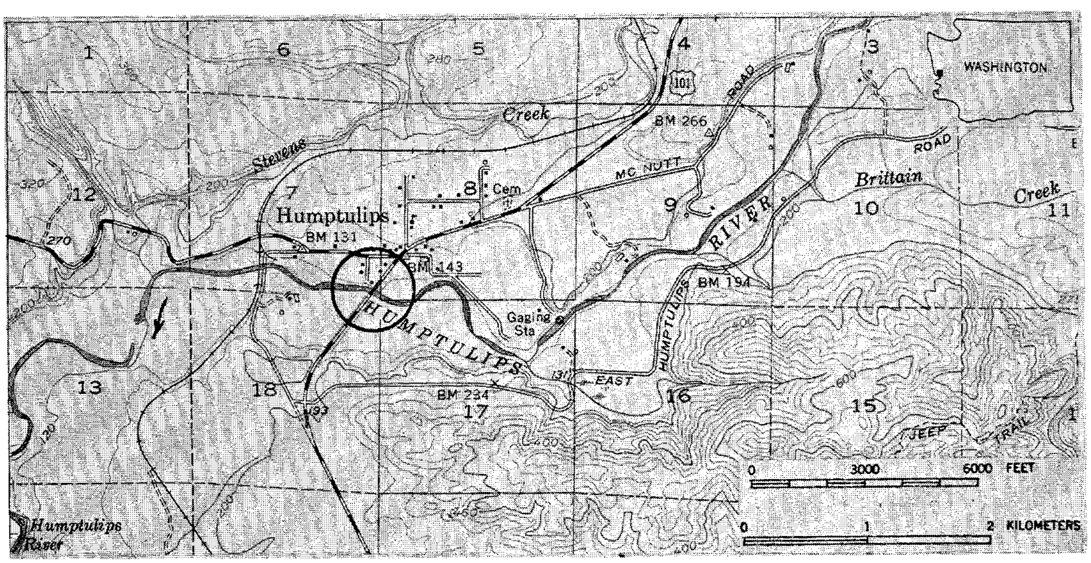


Figure 266. Map showing US-101 crossing (circled) at Humptulips River. (Base from U.S. Geol. Survey Humptulips, Wash., 15' quadrangle, contour interval 20 ft.)

SITE 265. HUMPTULIPS RIVER AT US-101 NEAR HUMPTULIPS, WASH.

Description of site: Lat 47°14', long 123°58', location as shown in fig. 266. Bridge is 444 ft (135 m) in length, has one steel pony truss span 260 ft (79 m) in length, supported by two concrete piers of wall-type with round nose and spread footings. Three 60-ft (18-m) spans are supported by open, square-column piers with spread footings. Abutments are spillthrough. Crossing is at bend and is skewed to trend of valley. Piers are perpendicular to bridge center line, but skewed to flood flow (figs. 267 and 268). Pier 4 is skewed about 45° to flow.

Drainage area, 133 mi^2 (344 km^2); bankfull discharge, 16,000 ft^3/s (453 m^3/s); valley slope, 0.002; width, about 200 ft (61 m). Stream is perennial, semi-alluvial, gravel bed, in valley of moderate relief, narrow flood plain. Channel is sinuous, locally anabranched, random width variation, cut banks locally (at outside of bends), silt-clay banks.

Hydraulic problem and countermeasure:

1950 Bridge built, for design flood of 100-yr R.I., no countermeasures known.

1951-56 Succession of small floods, R.I. about 2 yr. Local scour occurred at upstream, landward corner of pier 4 and channel bed was lowered to an elevation 1 ft (0.3 m) below bottom of footing, or about 14 ft (4.3 m) below ground line at time of construction. Part of this channel erosion is due to lateral migration of the channel. Local scour also occurred at the downstream, upstream, and streamward side of pier 3, also to an elevation about 1 ft (0.3 m) below bottom of footing. As a countermeasure, dumped rock riprap, approximately equivalent to class II, was placed over an area measuring about 10 by 10 ft (3 by 3 m) around pier 4 and extending downward to the top of the footing (fig. 269). Similar riprap was placed over a smaller area around pier 3, and grouted.

1957-76 Countermeasures tested by a succession of small floods, two near mean annual and one of about 5-yr R.I. No damage was noted. The left bank beneath the bridge, which is on the outside of the bend, receded about 50 ft (15 m) between 1966 and 1974, as indicated by comparison of aerial photographs. This has apparently caused no problems as yet, but it may eventually be troublesome.

Discussion: The local scour at piers is attributed to skewness of the crossing (which tends to direct flood flow toward the left bank), to location of the crossing at a bend (which also tends to direct flow toward the left bank), and to skewness of the piers. Contraction of flow is not involved in the scour problem, according to U.S. Geological Survey hydrologists in Washington state.

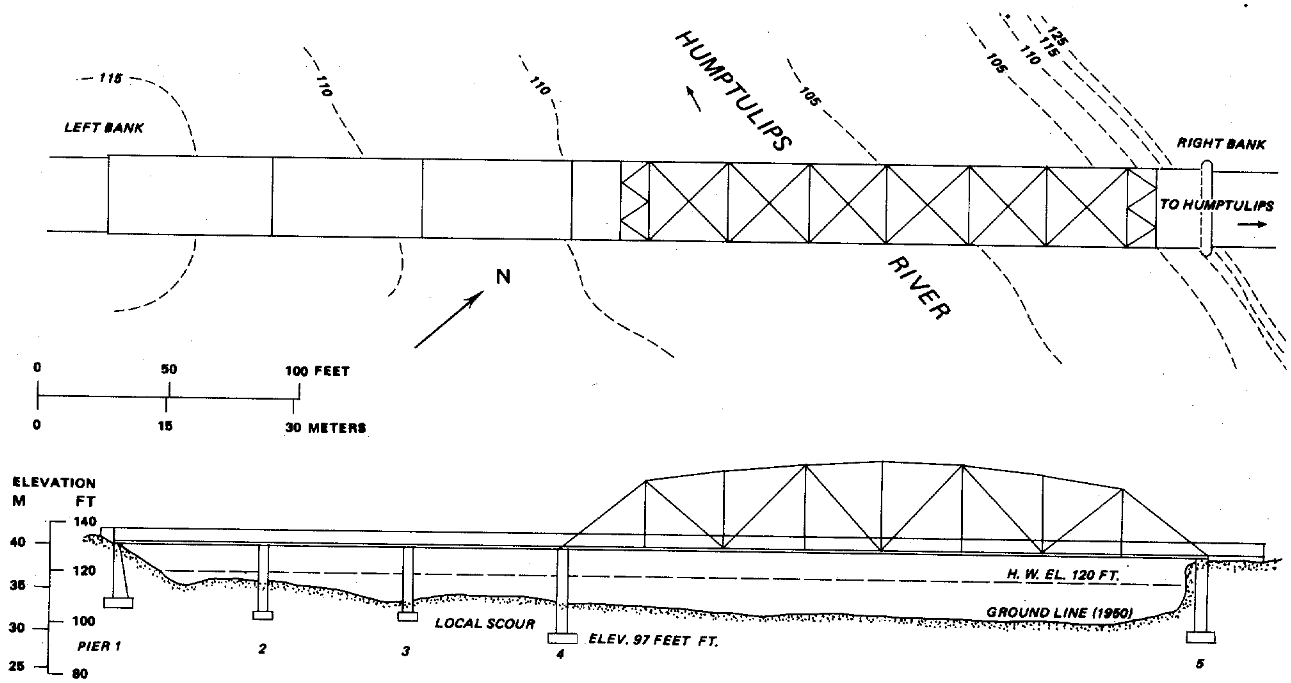


Figure 267. Plan, at top, and elevation, at bottom, of US-101 bridge at Humptulips River.

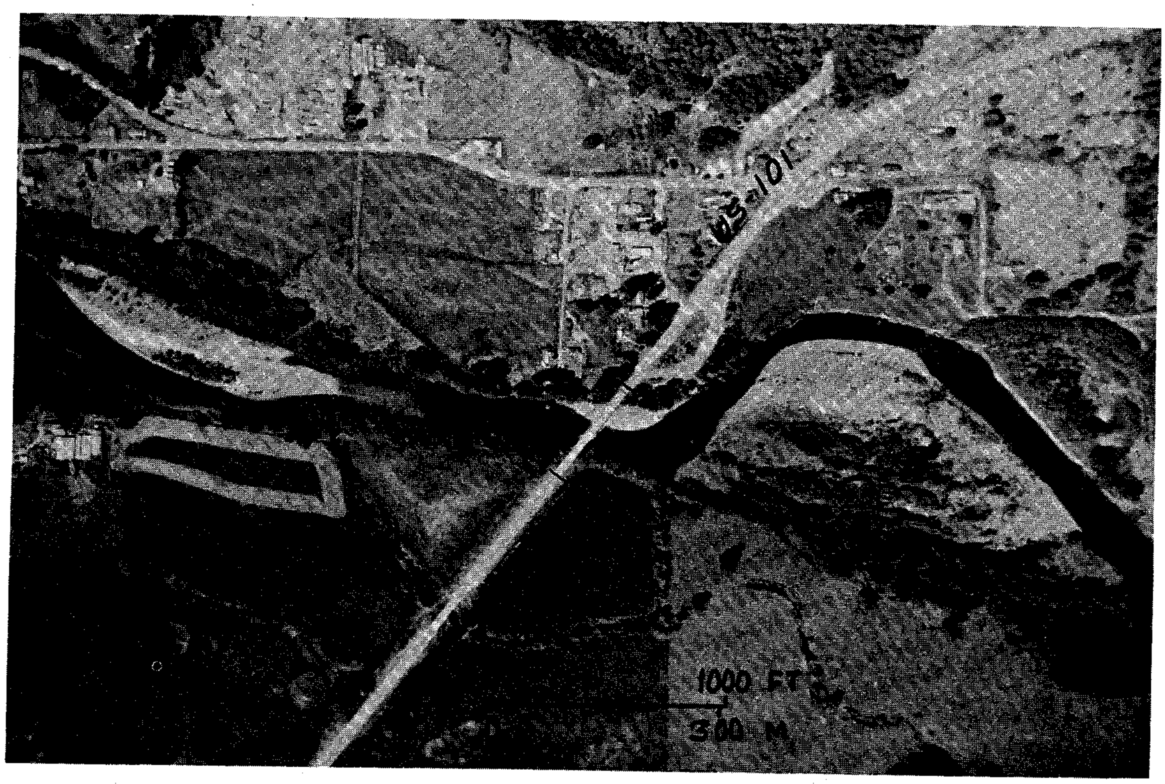


Figure 268. Aerial photograph of US-101 crossing at Humptulips River, May 31, 1974. (From Washington Dept. of Highways.)

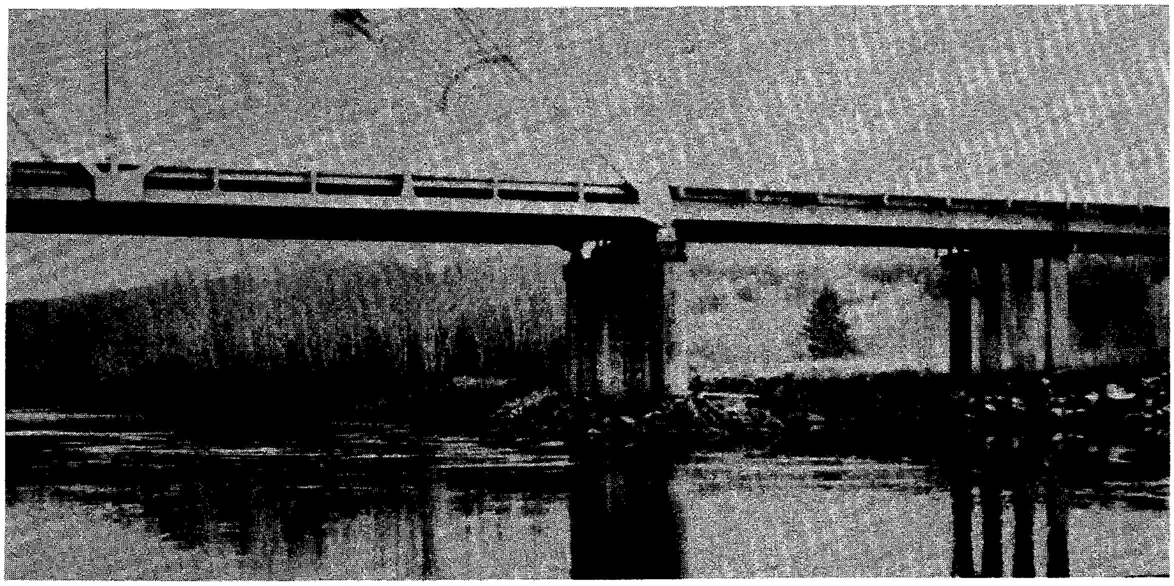


Figure 269. Upstream view of Humptulips River bridge, showing placement of riprap at piers 3 and 4, on January 6, 1977.

SITE 266. BOULDER CREEK AT BRIDGE 394-2.4, MT. BAKER-SNOQUALMIE NATIONAL FOREST, WASH.

Description of site: Lat 48°43', long 121°42', location as shown in fig. 270. Bridge is 220 ft (67 m) in length, single-span steel Pratt truss, left abutment set on rock ledge. Right abutment has spread footing on alluvial gravel, boulders, and sand.

Drainage area, about 10 mi^2 (26 km^2); bankfull discharge, 250 ft^3/s (7 m^3/s) width, 150 ft (46 m) valley slope 0.05. Stream is perennial, alluvial, cobble and boulder bed, in valley of high relief (but bridge is at head of an alluvial fan), no flood plain. Channel is straight, generally braided (fig. 271), bounded by steep valley sides.

Hydraulic problem and countermeasure:

1933 Approximate date of bridge construction; clearance was about 4 ft (1.2 m) above high water and about 8 ft (2.4 m) minimum above bed of channel. No countermeasures.

1934-73 No history of hydraulic problems, but channel gradually aggraded such that minimum clearance above channel bed was reduced from about 8 ft (2.4 m) to about 3 ft (1 m). Factors causing aggradation are not known, but landslides commonly occur in the drainage basin, in which timber has been harvested.

1974-75 As a result of a succession of flood events (R.I. of largest flood about 30 yr), logs and debris in the channel diverted the flow against the right abutment, and the resulting erosion exposed the abutment footing (figs. 272 and 273).

1975 As a countermeasure, debris was removed from the existing channel, stream boulders were placed against the abutment as riprap, and a secondary channel was formed beneath the bridge, to carry the flow in case the existing channel was again choked with debris. In December of 1975, after completion of the countermeasures, a single large flood (R.I. about 30 yr) occurred and no problems at the bridge resulted.

Discussion: Although aggradation has reduced the bridge clearance to a hazardous value, the actual problem that occurred--erosion at an abutment because of shift of a braid--might occur in any braided channel, whether aggrading or not. The measures applied are likely to be only temporarily effective, as the relocated channel is not likely to remain in position. Aggradation is not easy to control, but flow control measures such as spurs might be effective in maintaining the position of the thalweg.

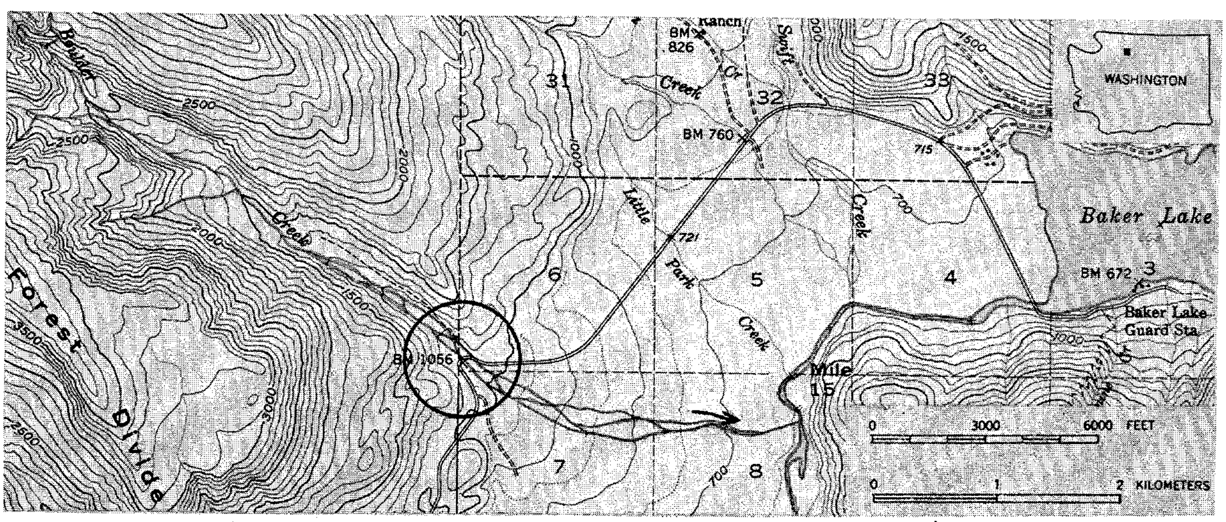


Figure 270. Map showing Boulder Creek at Forest Service bridge 394.2.4 (circled). (Base from U.S. Geol. Survey Lake Shannon Wash., 15' quadrangle, contour interval 100 feet, 1948.)

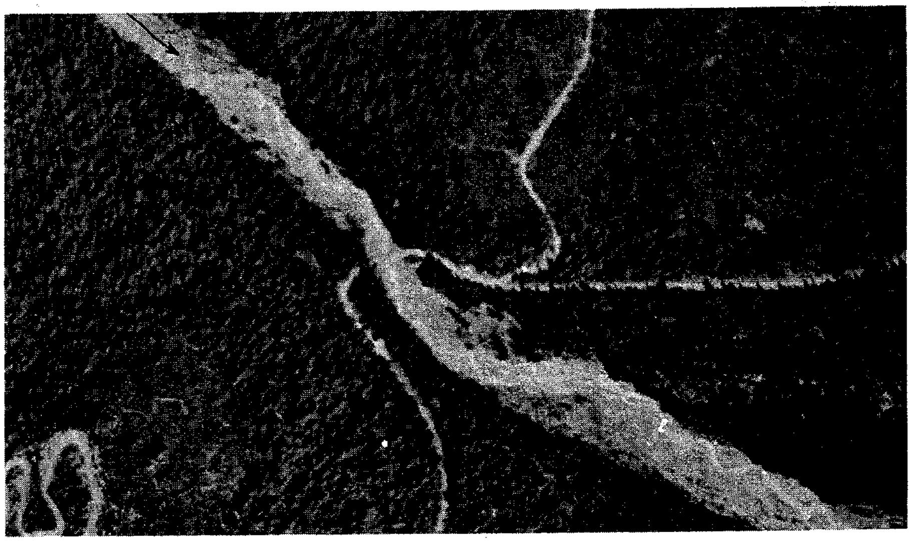


Figure 271. Aerial photograph showing Boulder Creek at Forest Service bridge 394-2.4, August 8, 1972. (From U.S. Forest Service.)

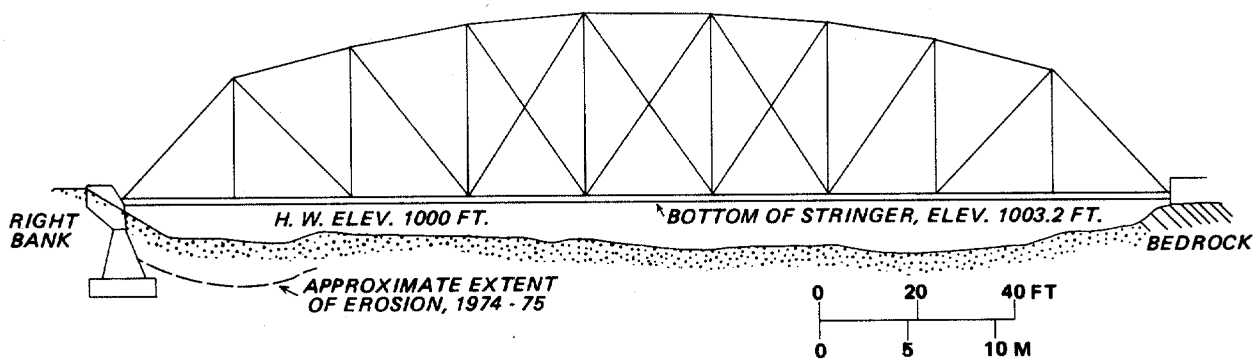


Figure 272. Elevation of Forest Service bridge 394.2.4, from original plans; on December 22, 1976, channel bed near center of crossing was at elevation 1,000 feet.

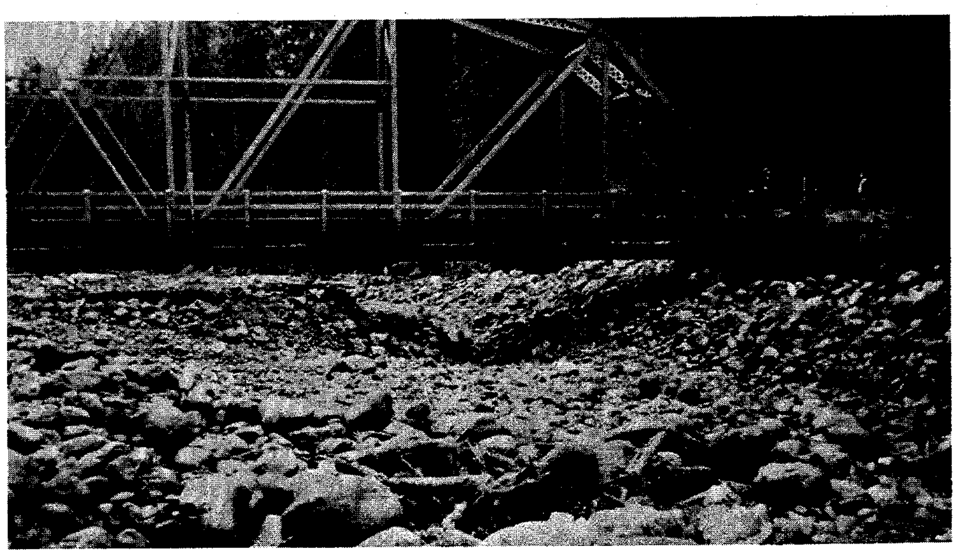


Figure 273. Downstream view showing right-bank abutment of Forest Service bridge 394-2.4, December 22, 1976.

SITE 267. NORTH FORK SKOKOMISH RIVER AT BRIDGE 2357-0.1 (LAKE CUSHMAN BRIDGE), OLYMPIC NATIONAL FOREST

Description of site: Lat 47°30', long 123°19', location as shown in fig. 274. Bridge is 200 ft (61 m) in length, four concrete wall-type piers in channel, spillthrough abutments, 1.5:1 fill slopes. Concrete footings of piers and abutments have steel H-pile foundations. Crossing is in backwater of Lake Cushman.

Drainage area, 58 mi^2 (150 km^2); bankfull discharge, 3,500 ft^3/s (98 m^3/s); valley slope, 0.021; width of braided channel between valley walls, 1,300 ft (396 m). Stream is perennial, semi-alluvial, gravel bed, in valley of high relief. Channel is generally braided, gravel banks.

Hydraulic problem and countermeasure:

1959 Bridge built. Lake Cushman was already in existence, and high water elevation of lake was known.

1960-67 General scour of channel bed, amounting to about 4 ft (1.2 m) of lowering on the east side and 2 ft (0.6 m) on the west, and exposure of H-pile foundations at piers 2 and 3 (figs 275 and 276). This is attributed to streamflow during lowered lake levels. During this period, there was a succession of small floods, ranging in R.I. from 2 to 5 yr. In addition, the foundation pile were exposed at piers 1 and 4, but this is attributed to wave action during high lake levels. Abutments and approaches also subjected to erosion by wave action, at different elevations, during about 9 months of the year.

1968 Dumped rock riprap was placed on abutment fill slopes at 2:1 slope ratio, to cover bases of all piers and extend to toe trenches in channel. Size gradation included some large boulders but a high percentage of fines (fig. 276).

1976 A gabion wall was placed around both abutments for protection against wave action.

1977 Foundation pile again exposed at all piers; riprap unsuccessful in preventing erosion by stream and by wave action. No erosion evident at abutments, but gabions not yet subjected to wave action.

Discussion: Regarding general scour by stream action in the bridge waterway, possible factors relating to this are the steep slope of the channel and the degree of constriction; the ratio of bridge length to natural channel width is about 1:6.5. In addition, the discharge of the stream may have been underestimated in the design. It also appears that the effects of wave action may have been underestimated. The high percentage of fines in the riprap placed as a countermeasure probably accounts for its failure to provide adequate protection against general scour and wave action. Besides the problems previously described, the stream thalweg has shifted and now flows along the left approach embankment before entering the bridge waterway; and the exposed steel H-pile are severely rusted.

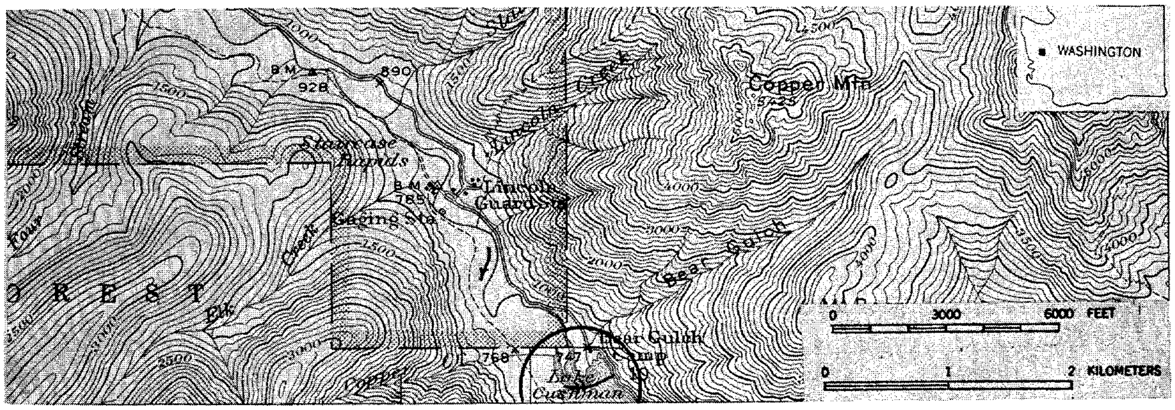


Figure 274. Map showing North Fork Skokomish River and Lake Cushman at Forest Service bridge 2357-0.1 (circled). (Base from U.S. Geol. Survey Mt. Steel, Wash., 15' quadrangle, contour interval 100 feet, 1948).

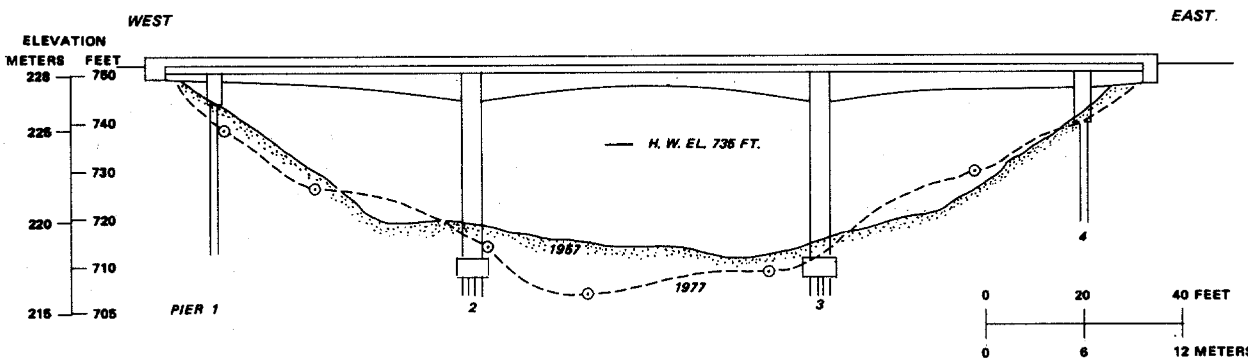


Figure 275. Elevation sketch of Lake Cushman bridge from original plans, with 1977 channel cross section added.

Figure 276. View of left (east) end of Lake Cushman bridge, on January 6, 1977. Footing and top seal of pier 3, center, are exposed and H-pile foundations of pier 2, background, also exposed. Note gabion wall at top of abutment fill-slope and approach alinement of stream, at left.

SITE 268. STEAMBOAT SLOUGH AT I-5 AT MARYSVILLE, WASH.

Description of site: Lat 48°02', long 122°11', location as shown in fig. 277. Bridge is dual, 1,026 ft (313 m) in length. Nine piers, each with four round concrete columns, in channel; square concrete footings on timber-pile foundation. Spillthrough abutments.

Drainage area, 1,714 mi^2 (4,439 km^2). Steamboat Slough is a tidal channel connected to the Snohomish River; channel is alluvial, sand-silt bed, sinuous.

Hydraulic problem and countermeasure:

1967 Bridge built, no countermeasures known.

1975 Local scour at piers 5, 6, 7, and 8 exposed pier footings and foundation pile beneath some footings (figs. 278 and 279). Scour occurred in connection with large accumulation of logs at bridge during winter of 1975. Log jam extended to bottom of channel and logs were lodged beneath the concrete seal and among the foundation piling of some piers. Timber-pile structures for protection of piers against boat traffic were also partly destroyed, some piling being broken off.

1976 As a countermeasure, steel sheet piling were driven on all sides of seals and to a depth of 10 ft (3 m) below seals; tremie concrete was pumped under the seals and the sheet pile was later cut off at top of seals.

1977 Countermeasures apparently effective but not adequately tested.

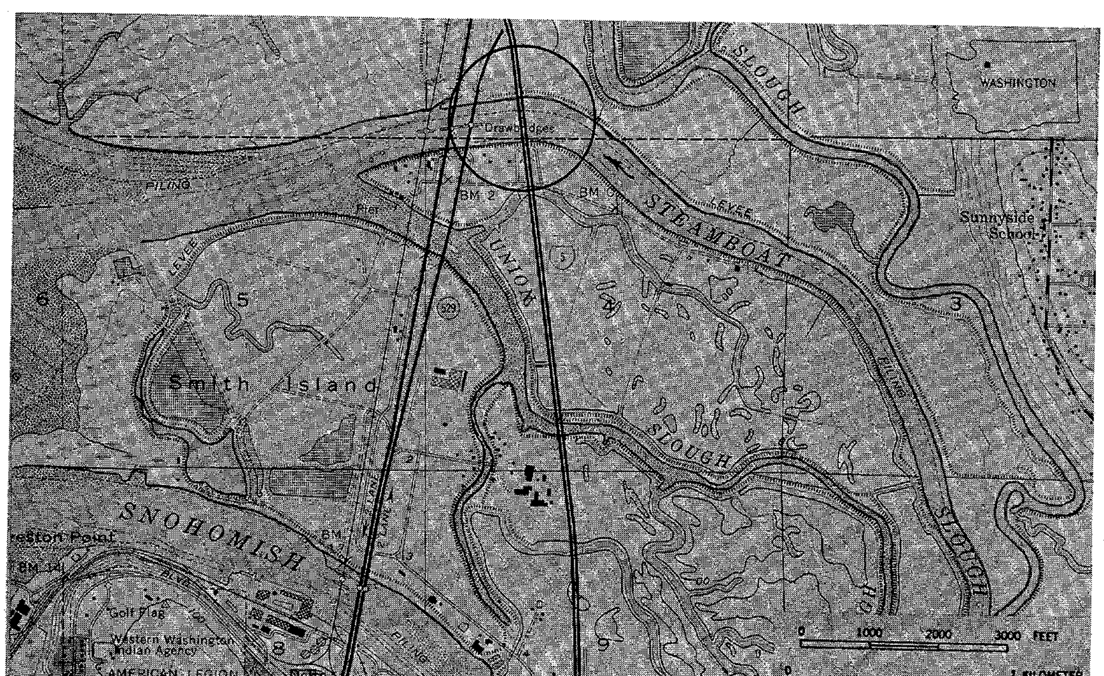


Figure 277. Map showing Steamboat Slough at I-5 crossing (circled). (Base from U.S. Geol. Survey Marysville, Wash., 7.5' quadrangle, contour interval 20 feet, photorevised 1973.)

Discussion: Accumulation of logs at bridge is attributed to streamflow, but scour is attributed, at least in part, to tidal flow. Because of tidal fluctuations, discharges and velocities are greater at Steamboat Slough than at the gage on the main stem Snohomish River, 10 miles (16 km) upstream from bridge.

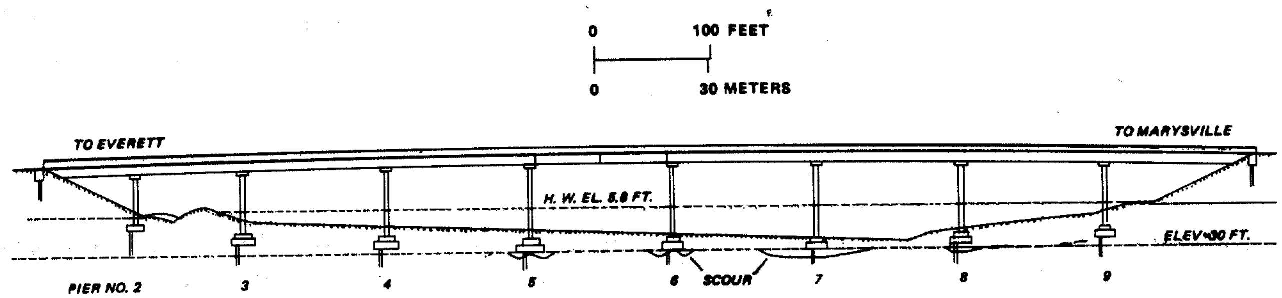


Figure 278. Elevation sketch of I-5 bridge at Steamboat Slough.

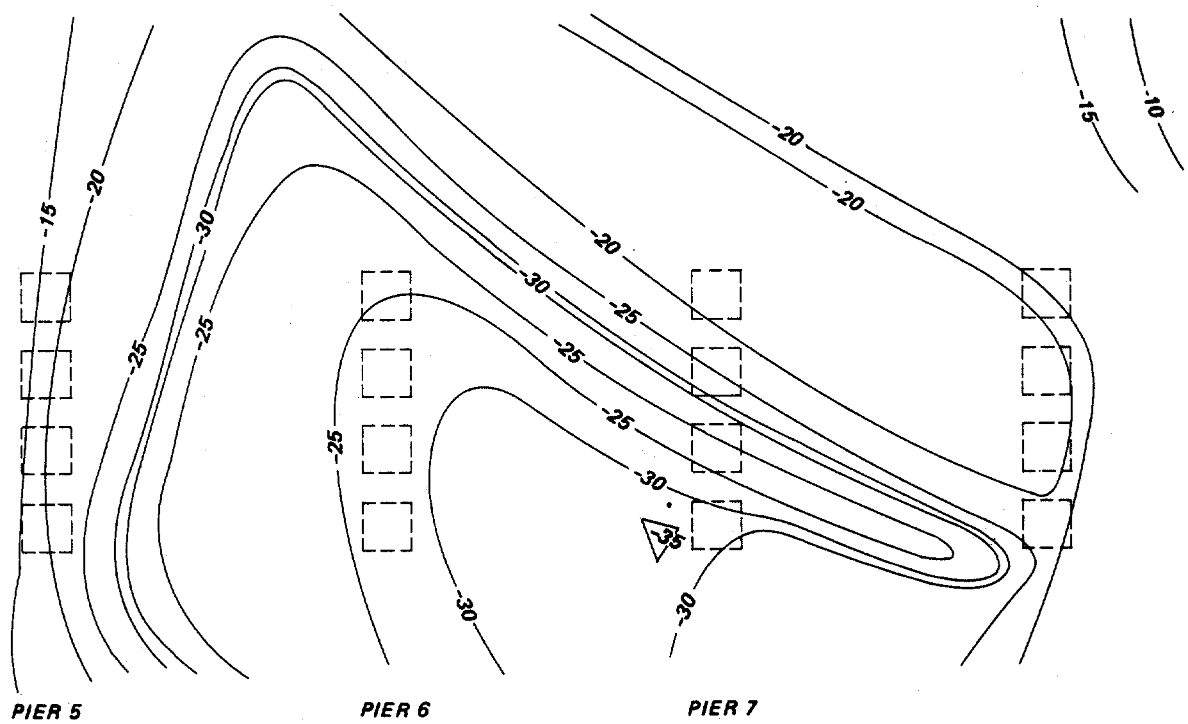


Figure 279. Contour map showing pattern of local scour at piers 5, 6, 7, and 8, I-5 bridge at Steamboat Slough. Soundings in feet, taken March 2, 1976.

SITE 269. SWINOMISH CHANNEL AT SR-20 NEAR BURLINGTON, WASH.

Description of site: Lat 48°27', long 122°30', location as shown in fig. 280. Bridge is 824 ft (251 m) in length, consists of 38 timber-pile trestle spans, two 30-ft (9-m) tower spans, and one 115-ft (35-m) vertical lift span; four concrete piers at towers have timber-pile foundations. Spillthrough abutments.

Swinomish Channel is an estuary that transmits large tidal flows between two salt-water bays, Padilla and Skagit.

Hydraulic problem and countermeasure:

1935 Bridge built, no countermeasures known.

1975 Local scour occurred to a maximum depth of 24 ft (7.3 m) below mean sea level at concrete pier on west side of main channel beneath lift span (fig. 281). Seal and foundation piling were exposed. The reason for scour at this pier, 40 years after bridge construction, is not known. At ebb tide on July 1, 1977, point velocities in the range of 1.6-2.8 ft/s (0.5-0.8 m/s) were measured adjacent to the piers.

As a countermeasure, steel sheet-piling were driven around existing seals to a depth of 10 ft (3 m) below the seals, the timber foundation-pile were exposed by excavation to a minimum depth of two feet (0.6 m) below the bottom of the seal, and the void was filled with tremie concrete. The countermeasure is apparently effective, during the short time that it has been in place.

Discussion: The top of the footing, at elevation -10 ft, was only two feet below the proposed elevation of the channel bottom on the original bridge plans. In view of the high tidal velocities known to occur in the channel, it would seem that the footings and seal were placed at rather shallow depth. Another case of scour in a tidal estuary, long after bridge construction, is reported herein (Site 164).

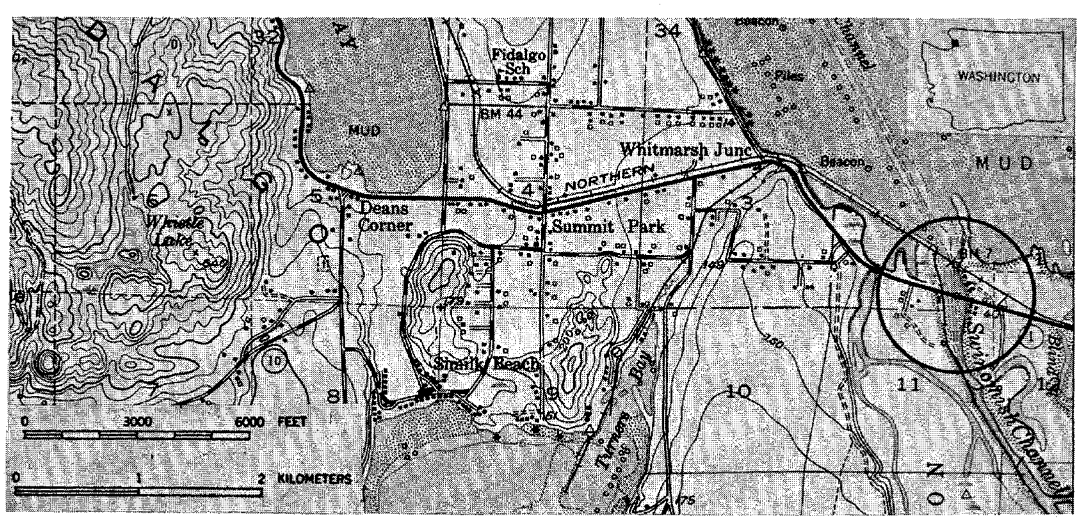


Figure 280. Map showing Swinomish Channel at SR-20 crossing (circled). (Base from U.S. Geol. Survey Deception Pass, Wash., 15; quadrangle, contour interval 50 feet, 1951.)

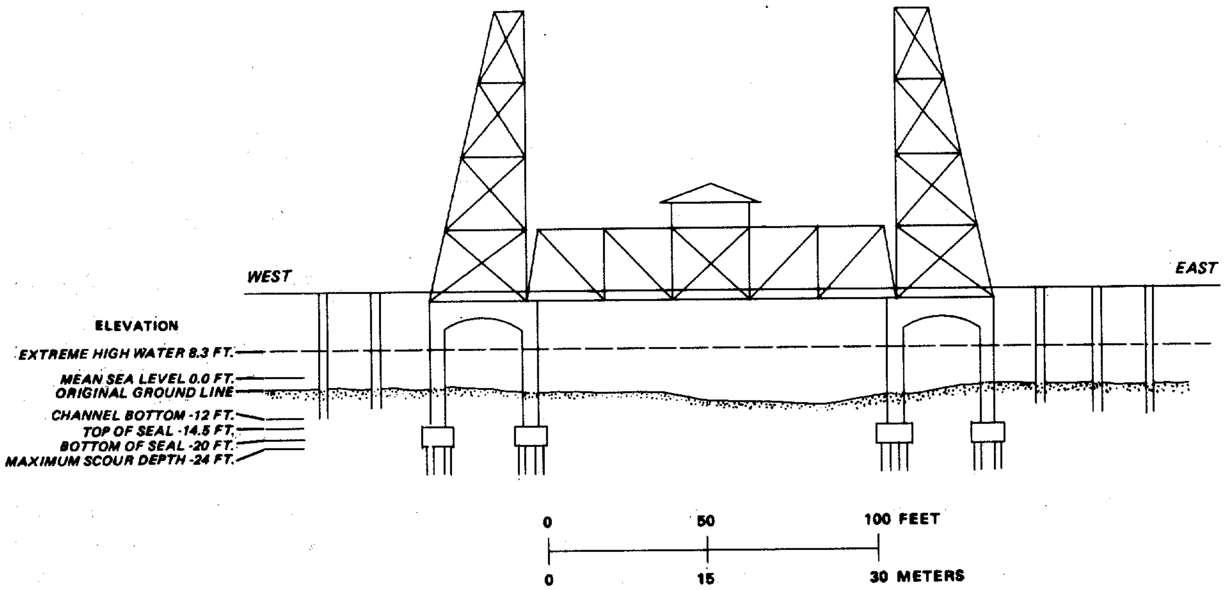


Figure 281. Elevation sketch of lift span of Swinomish Channel bridge.

SITE 270. SATUS CREEK AT US-97, FIRST CROSSING UPSTREAM FROM TOPPENISH, WASH.

Description of site: Lat 46°14', long 120°25', location as shown in fig. 282. Bridge, built in 1942, is 133 ft (40.5 m) in length, has steel I-beam center span, two-column open concrete piers with spread footings on clay alluvium, timber stringer end spans. Spillthrough abutments.

Drainage area, 270 mi^2 (700 km^2); valley slope, 0.01; width, 120 ft (36 m). Stream is perennial, alluvial, gravel bed, in valley of moderate relief, narrow flood plain. Channel is sinuous, locally anabranched, silt-clay banks.

Hydraulic problem: During a 1974 flood having a peak discharge of 21,000 ft^3/s (594 m^3/s), R.I. estimated at much greater than 100 yr, the right (south) abutment was undermined by scour and the end span collapsed (fig. 283). A substantial amount of drift had accumulated at the left (north) end of the bridge. Cost of repairs, $12,300. Bridge was rebuilt to original specifications.

Discussion: The bridge performed remarkably well in view of its age and the magnitude of the flood, and the installation of countermeasures or design features to prevent damage from a flood of such magnitude seems not to be economically practical.

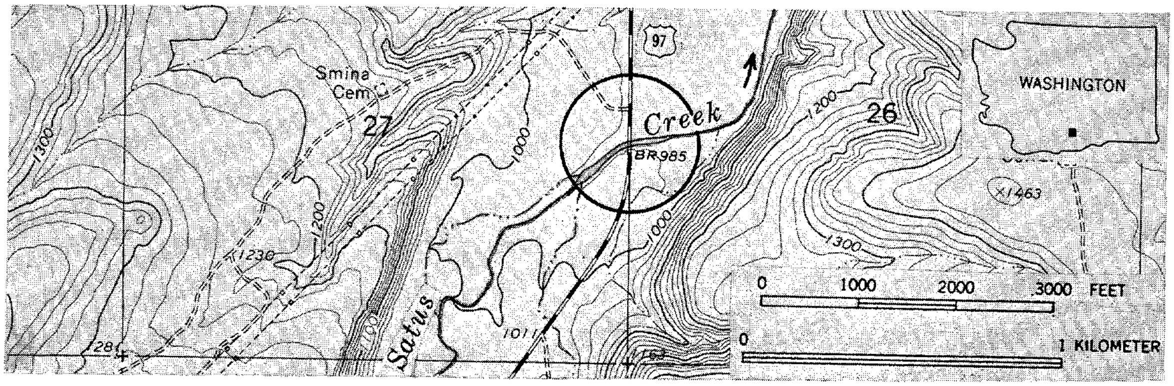


Figure 282. Map showing Satus Creek at first crossing of US-97 (circled). (Base from U.S. Geol. Survey Poisel Butte NW, Wash., 7.5' quadrangle, contour interval 20 feet, 1965.)

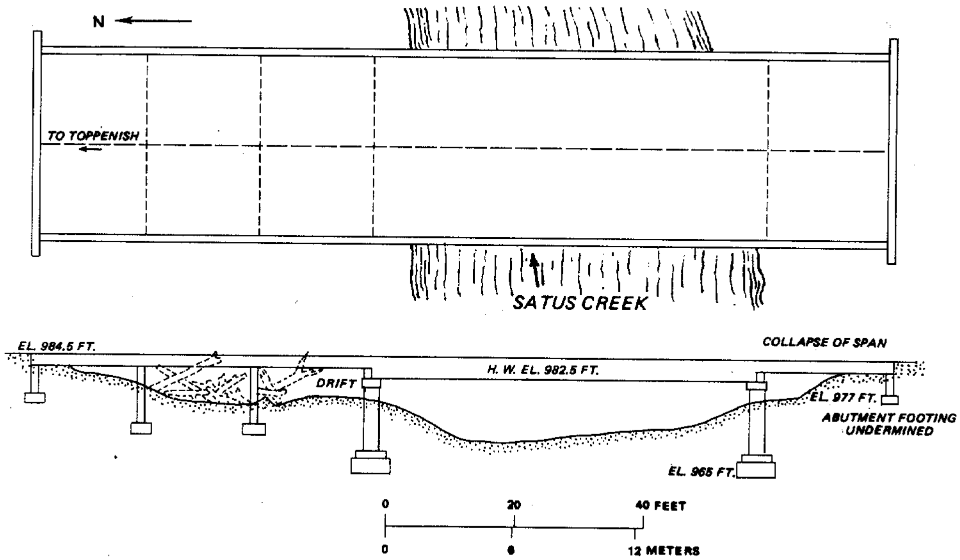


Figure 283. Sketch of plan (top) and elevation (bottom) for bridge at first crossing of Satus Creek.

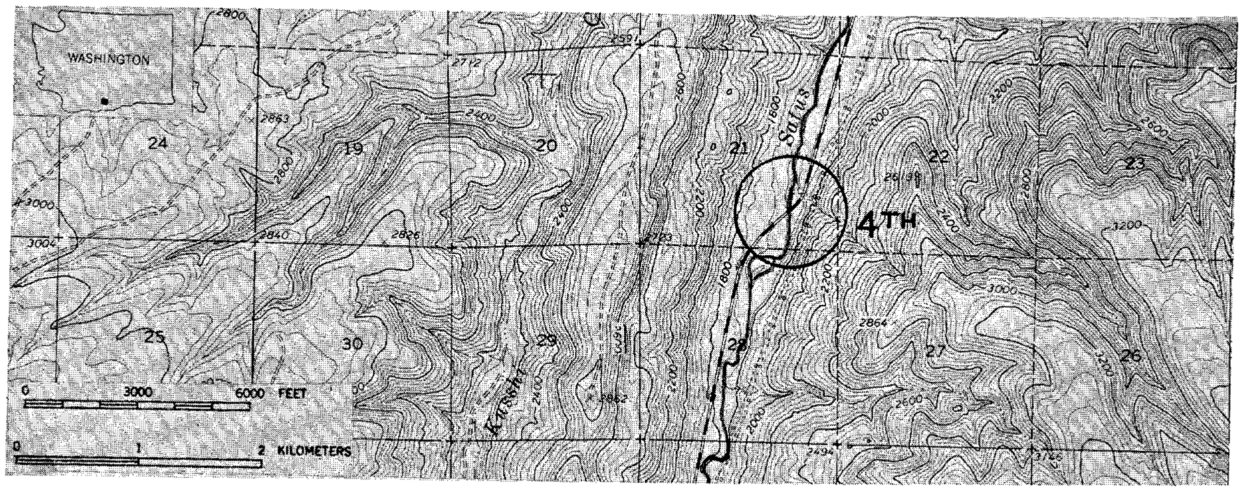


Figure 284. Map showing Satus Creek at US-97, fourth crossing. (Base from U.S. Geol. Survey Logy Creek, Wash., 15' quadrangle, contour interval 40 feet, 1965.)

SITE 272. SATUS CREEK AT US-97, FOURTH CROSSING UPSTREAM FROM TOPPENISH, WASH.

Description of site: Lat 46°04', long 120°33', location as shown in fig. 284. Bridge, built in 1942, is 180 ft (55 m), in length, has concrete wall-type piers in channel, spread footings on basaltic bedrock, piers skewed 40° to flowline of channel. Bridge designed for flood of 50-yr R.I.

Drainage area, 87 mi^2 (225 km^2); valley slope, 0.013. Stream is perennial, semi-alluvial, gravel bed, in narrow valley of high relief. Channel is sinuous, silt-clay banks.

Hydraulic problem: During a 1974 flood having a peak discharge of about 6,800 ft^3/s (192 m^3/s), R.I. estimated at much greater than 100 yr, bents 1 and 2, and pier 1, were undermined by scour (fig. 285). Bent 2 settled 8-10 in (20-25 cm) and moved forward 15 in (38 cm). Bridge closed, cost of repairs $34,300.

Discussion: Available information does not show whether or not the undermined footings were actually set in bedrock, but the skewness of the piers must have contributed to the scour. In reconstructing the bridge, the footing of bent 1 was lowered 7 ft (2 m) and set in bedrock, but the footing of bent 2 was set at about the same elevation. In view of the magnitude of the flood, damage to the bridge seems moderate, probably because of the semi-alluvial nature of the stream which places constraints on both scour and lateral erosion.

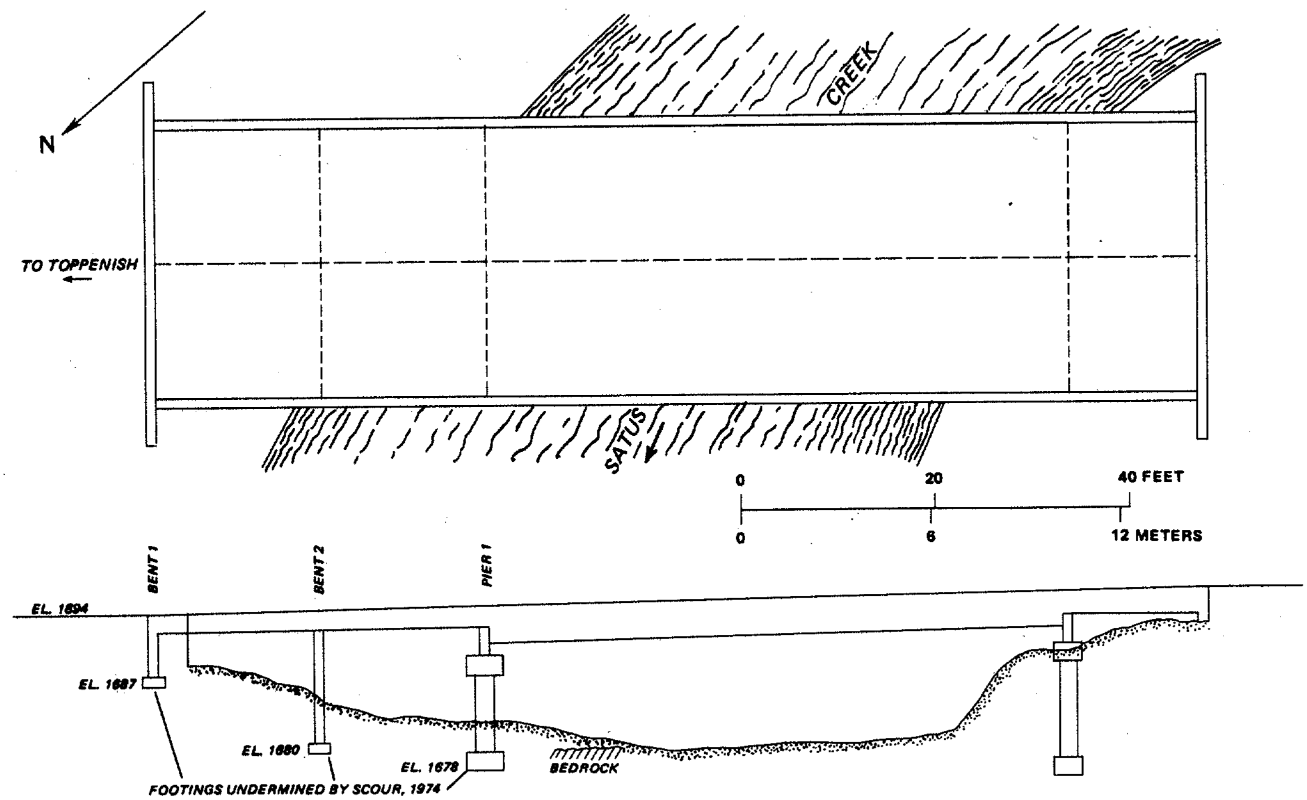


Figure 285. Sketch of plan (top) and elevation (bottom) for bridge at fourth crossing of Satus Creek.

SITE 273. SATUS CREEK AT US-97, FIFTH CROSSING UPSTREAM FROM TOPPENISH, WASH.

Description of site: Lat 46°02', long 120°38', location as shown in fig. 286. Bridge is 101 ft (31 m) in length, has two 3-column concrete piers in channel, spread footings on alluvium, spillthrough abutments. Crossing at curve in channel, piers and abutments approximately alined with flood flow.

Drainage area, 35 mi^2 (91 km^2); valley slope, 0.018. Stream is perennial, alluvial, gravel bed, in valley of high relief, little or no flood plain. Channel is sinuous, somewhat incised, silt-clay banks.

Hydraulic problem and countermeasure:

1959 Bridge built. Minor realinement of channel at time of bridge construction, "light" riprap placed at channel banks and abutment fill slopes. Recurrence interval of design flood, 100 yr; water surface elevation, 2,140 ft.

1974 Flood of discharge about 2,800 ft^3/s (79 m^3/s), R.I. estimated at much greater than 100 yr. Severe lateral erosion at left (east) abutment (fig. 287), which is at outside of bend. Abutment footing undermined, abutment settled about 1 ft (0.3 m), fill washed out from behind abutment. Bridge closed for repairs. Bridge was repaired and large rock riprap was placed along streambanks.

1977 Countermeasures not yet tested by flood. Abrasion of north (upstream) interior pier columns noted on bridge inspection report; exposure of steel reinforcement bar.

Discussion Although the flood that caused damage was far in excess of the design flood, there is a good chance that damage would have been prevented or lessened by heavy riprap bank protection.

Figure 286. Aerial photograph of fifth crossing of Satus Creek, prior to flood damage. (From Washington Dept. of Highways, 1966.)

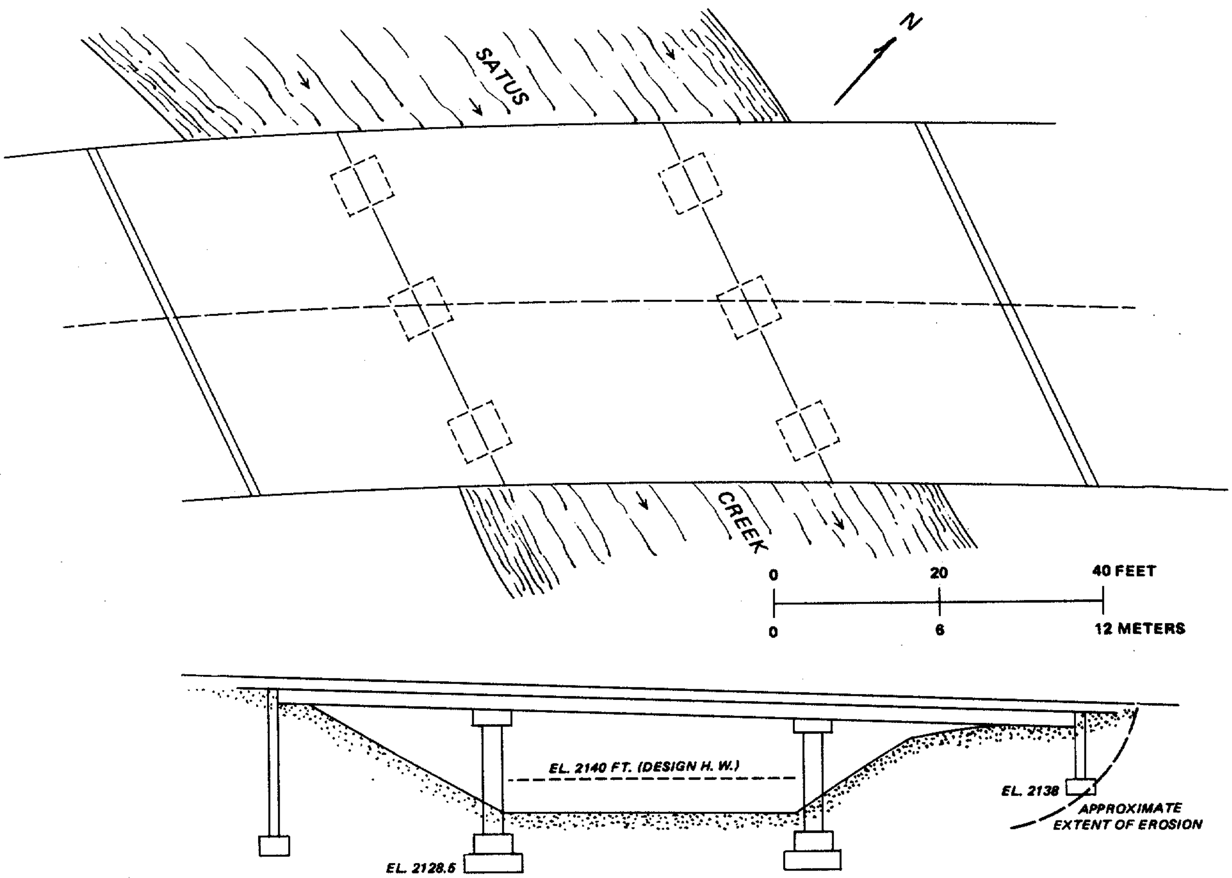


Figure 287. Sketch of plan (top) and elevation (bottom) for bridge at fifth crossing of Satus Creek.

SITE 274. SAGE CREEK AT SR-120 NEAR CODY, WYO.

Description of site: Lat 44°24', long 109°00'. Bridge, built in 1970, is 160 ft (48 m) in length, steel continuous girder; two pile bents, each with three round concrete pile, in channel; spillthrough abutments. Bridge crosses an artificial channel, about 800 ft (240 m) in length and 60 ft (18 m) wide, that cuts off a meandering reach of natural channel 1350 ft (412 m) in length (fig. 288).

Drainage area, about 60 mi^2 (155 km^2); width of natural channel, about 12 ft (3.6 m); stream is perennial, alluvial, silt bed, meandering.

Countermeasure: To offset the change in channel slope due to cutoff, a gabion drop structure flanked by wire-enclosed riprap on upstream and downstream sides, was constructed at the bridge. When inspected in March, 1977, the gabion structure was in good condition and functioning satisfactorily; and no unwanted effects of the channel cutoff were noted. However, no floods have occurred at the site during the period 1970-77.

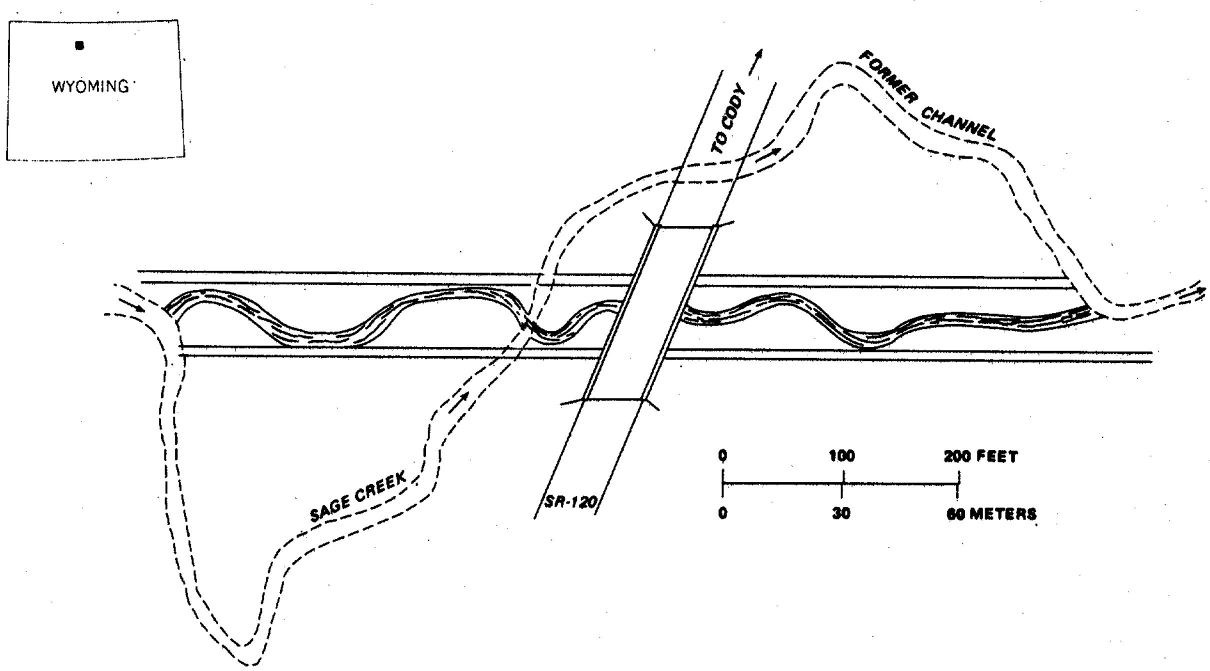


Figure 288. Plan sketch of channel alteration at Sage Creek.

SITE 275. SOUTH FORK POWDER RIVER AT I-25 NEAR KAYCEE, WYO.

Description of site: Lat 43°38', long 106°34'. Dual bridges, built in 1960, 265 ft (81 m) in length each with four 2-column concrete piers, spillthrough abutments. Abutment fill-slopes encroach on channel, bridge is considered to be underdesigned (fig. 289).

Drainage area, 1,150 mi^2 (2,978 km^2); valley slope, 0.0025, channel width about 350 ft (105 m). Stream is perennial, alluvial, sand bed, in valley of moderate relief, narrow flood plain but bordered by low, wide terrace not considered subject to overflow. Channel is sinuous, generally braided, silt-sand banks.

Countermeasures: Abutment fill-slopes are protected with rock-and-wire mattress. Left bank upstream from bridge is protected by a linear structure (called a training dike by the Wyoming Highway Department, but here classed as a retard) consisting of a double row of steel pile, connected by wire cable to each other and to the bank, faced with welded wire mesh and filled with cobbles. On the right flood plain is a similar structure, called a training dike by the Wyoming Dept. of Highways (fig. 290), intended to improve flow conveyance at the bridge.

Discussion: Although these countermeasures have been in place about 15 years, and were in good condition when inspected in April, 1977, no floods have occurred at the site since installation. The welded-wire mesh has a tendency to rupture and woven wire (chicken wire or hog wire) is now preferred.

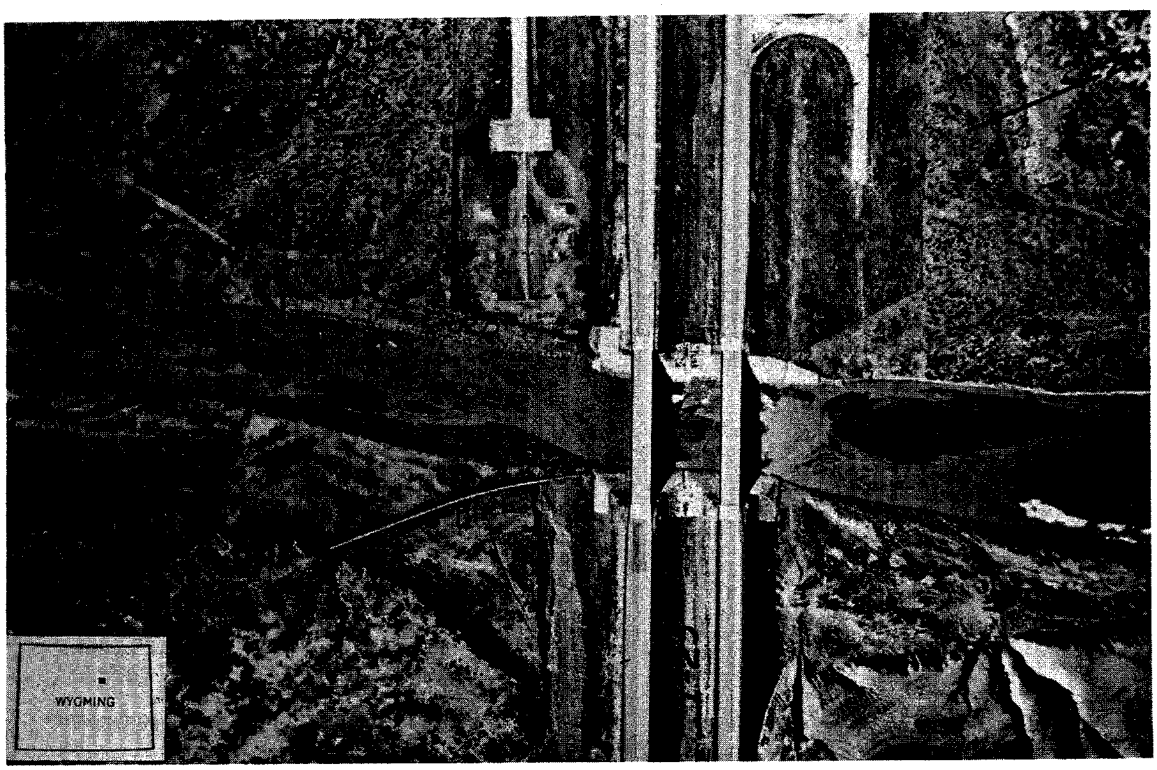


Figure 289. Aerial photograph of I-25 crossing, South Fork Powder River, March 1, 1968. (From Wyoming Dept. of Highways.)

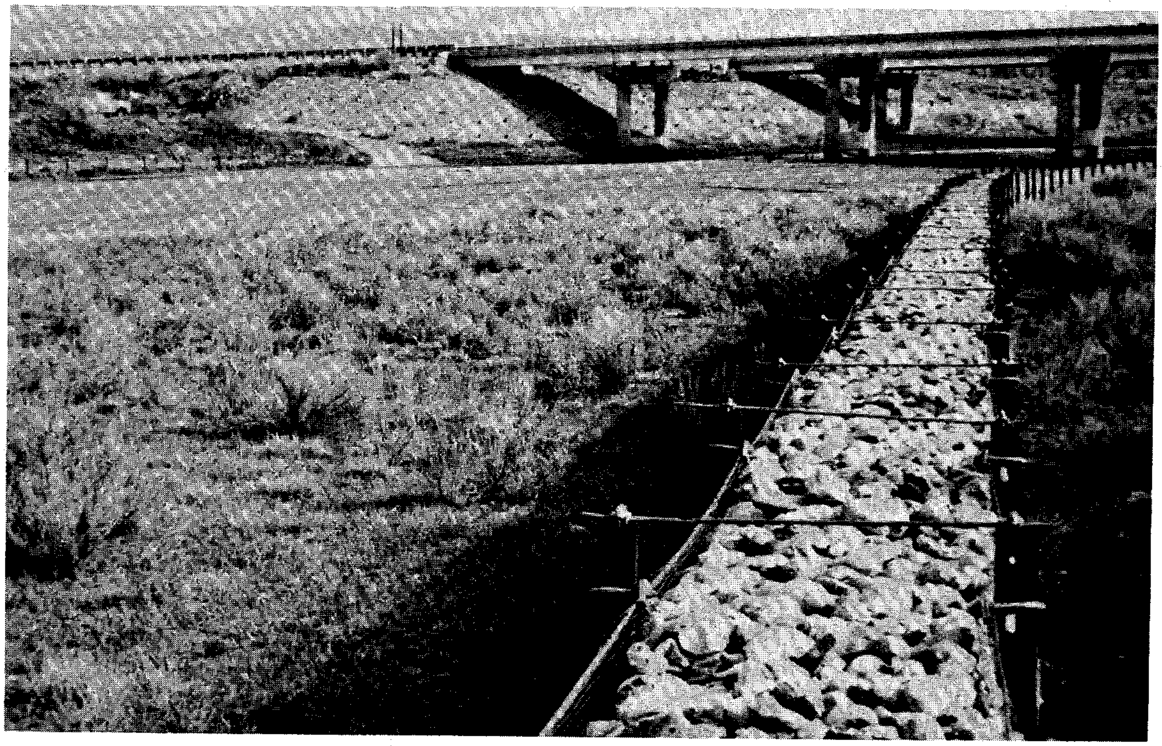


Figure 290. View looking downstream along spur dike (training dike) at I-25 bridge, in April, 1977. Rock-and-wire mattress on abutment fill-slopes is visible in background.

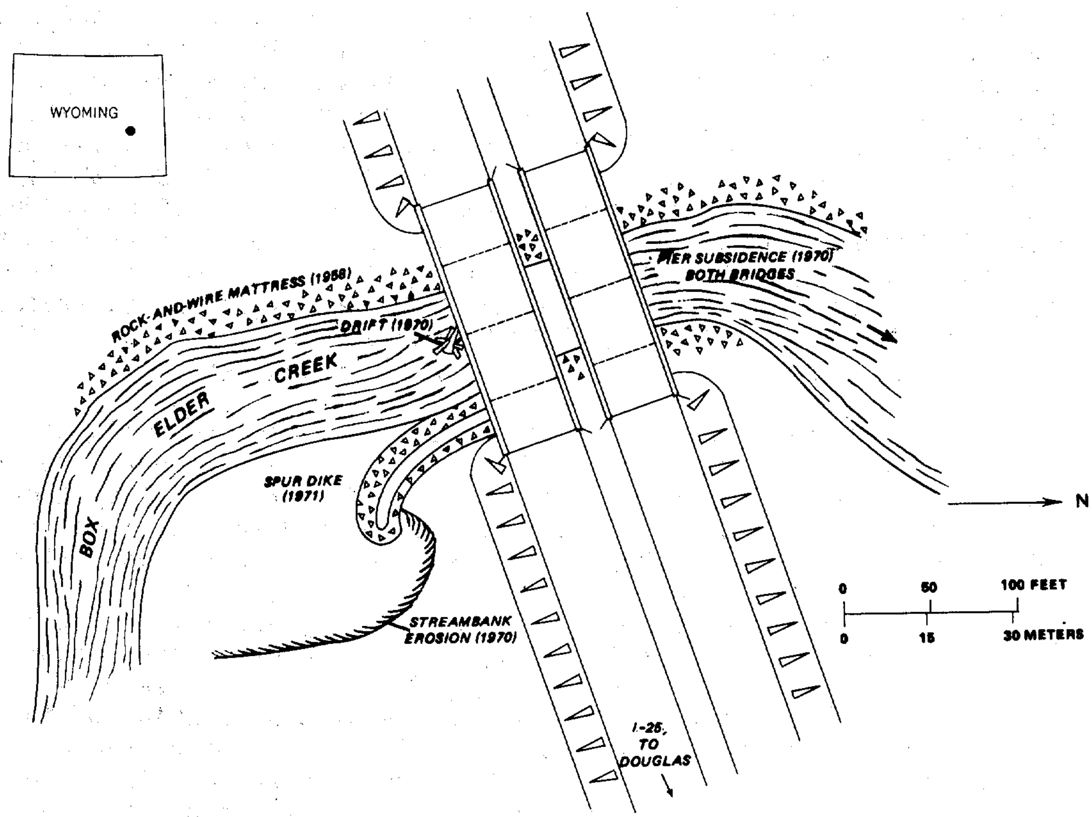


Figure 291. Plan sketch of I-25 bridge at Box Elder Creek.

SITE 276. BOX ELDER CREEK AT I-25 NEAR DOUGLAS, WYO.

Description of site: Lat 42°49', long 105°42'. Dual bridges, 155 ft (465 m) in length. Three 2-column (round) piers each bridge, spread footings on gravel, bottom of footing about 6 ft (1.8 m) below streambed. Spillthrough abutments. Abutment fill-slopes tend to encroach on channel.

Drainage area, about 200 mi^2 (518 km^2); valley slope, 0.005. Stream is perennial, alluvial, gravel bed, in valley of moderate relief, narrow flood plain. Channel is meandering, coherent silt-clay banks.

Hydraulic problem and countermeasure:

1958 Bridge built. Rock-and-wire mattress placed on abutment fill-slopes and on streambanks upstream and downstream from bridge (fig. 291).

1970 Flood on June 12, recurrence interval estimated at 7.5 times 50 yr. General scour occurred in bridge waterway, pier 2 of both bridges subsided, and the rock-and-wire mattress was eroded at the right bank beneath the bridge. In addition, lateral erosion of the right streambank occurred upstream from the bridge. Undermining of the spread footings of pier 2 (fig. 292) is attributed, at least in part, to accumulation of drift at the upstream pier column. High water elevation in the bridge waterway was 4,999 ft and mean velocity was about 10 ft^3/s (3 m^3/s).

A detailed study of the site was made by the engineering geologists of the Wyoming State Highway Department for the purpose of determining maximum depth of scour. It was found that gravels affected by scour contained little clay matrix and were penetrated by significantly fewer blows per foot on a drive point probe, than were clay-rich unscoured gravels beneath the channel bed. The scoured gravel had a maximum diameter of 3 in (7.6 cm) and the unscoured gravel, 7 in (18 cm). About 16 cores were taken, and a contour map on the surface of unscoured gravels beneath the channel bed was prepared. Scour occurred to a maximum depth of about 2 ft (0.6 m) beneath the base of the pier footing, or about 9 ft (2.7 m) beneath the channel bed as excavated for bridge construction (fig. 292). However, the elevation of the channel bed immediately before the flood is not known.

As countermeasures, the center piers were sheathed with concrete (fig. 293), converting them to wall-type piers, and a spur dike, revetted with rock-and-wire mattress, was constructed at the right upstream abutment. The rock-and-wire mattress is now (1977) plastered in some places with concrete, perhaps in places where the wire had failed.

Discussion: From the scour investigation by the Wyoming Highway Department, it was concluded that contraction scour occurred because of channel constriction by the abutment fill-slopes. As shown in fig. 292, the total waterway area was ample, and it seems that scour occurred because of constriction of the lower part of the waterway and the accumulation of drift. The rock-and-wire mattress, made by cobbles and small boulders enclosed in welded wire mesh, protected the abutment fill but was damaged, particularly along the right bank beneath the bridge. The spur dike has not yet been tested by flood.

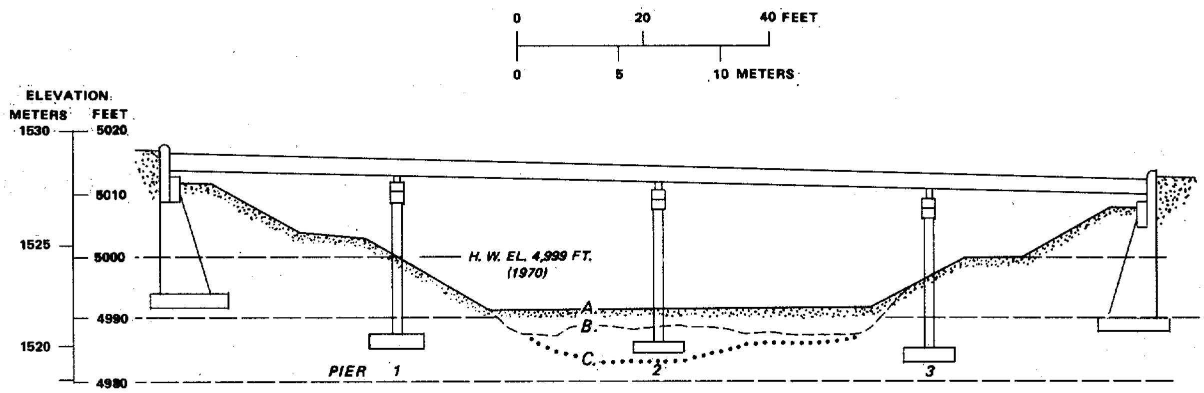


Figure 292. Elevation sketch of I-25 bridge at Box Elder Creek. A, channel bed as excavated at time of bridge construction; B, bed after flood, as surveyed in 1970; C, top of unscoured gravel, as determined by Wyoming State Highway Department.

Figure 293. Downstream view of I-25 bridge on February 9, 1977, showing configuration of center piers as rebuilt after the flood, and protection of abutment fill slopes by rock-and-wire mattress.

SITE 277. CARPENTERS DRAW AT SR-192 NEAR SUSSEX, WYO.

Description of site: Lat 43°41', long 106°17'. Bridge is about 80 ft (24 m) in length, steel pipe-pile bents, spillthrough abutments.

Drainage area, about 4 mi^2 (10.4 km^2); valley slope, 0.01; stream is ephemeral, alluvial, silt bed, in valley of low relief, little or no flood plain; channel is meandering, incised, cut banks general, silt-clay banks.

Hydraulic problem and countermeasure: In the 1960's, the bridge was severely damaged by channel degradation, manifested as migrating headcuts. At the time of bridge repair, several migrating headcuts were known to exist downstream from the bridge. Of the several alternatives available for protection of the bridge from damage by the headcuts as they migrated upstream to the crossing site, the one chosen consisted of a rock-and-wire mattress extending to the center of the channel from each abutment fill-slope, but not joined at the center. As a headcut passed through the bridge, leaving an inset channel behind, the mattress would subside and drape the sides of the new channel, thus protecting the pile bents and abutment fill-slope from lateral erosion.

On an inspection of the site in November of 1969 by representatives of the Wyoming State Highway Department, it was observed that one headcut had reached the bridge and that the mattresses were beginning to subside. This situation had not changed much by April 15, 1977, when the site was inspected by U.S. Geological Survey representatives. On May 15, 1977, a high flow, reaching a depth of about 7 ft (2 m) in the channel beneath the bridge, occurred. Subsequent inspection showed that the headcut, which is about 3 ft (1 m) in height, had advanced several feet, that the mattress had subsided further, and that, in places, the wire was not retaining its enclosed riprap (figs. 294 and 295).

Discussion: Channel degradation by headcutting, and widening that usually accompanies degradation, is common on the Great Plains and in much of the Rocky Mountain Region; the causes of the degradation are complex, but the effect on bridges is usually bad. Therefore, countermeasures for the protection of bridges against headcutting are of substantial interest. The countermeasure at Carpenters Draw seems capable of coping with a moderate amount of degradation (of the magnitude estimated from the heights of downstream headcuts at the time of bridge repairs), if the welded-wire mesh holds the riprap. Such mesh is now regarded as unsuitable for rock-and-wire mattress by the Wyoming State Highway Department, because the welds tend to break, and woven wire is used instead.

Figure 294. Upstream view of SR-192 bridge at Carpenters Draw in May, 1977, showing rock-and-wire mattress on abutment fill-slope, left, which extends to center of channel. Mattress on opposite slope also extends to center of channel. At lower left, wire is failing to hold riprap.

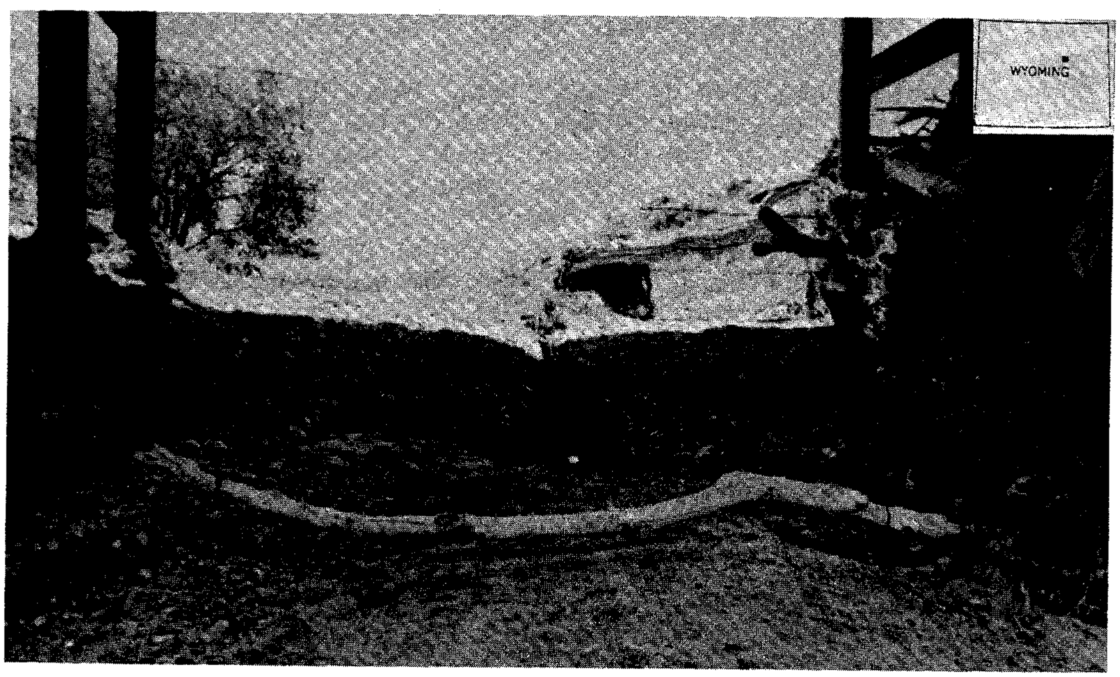


Figure 295. Upstream view beneath SR-192 bridge at Carpenters Draw in May, 1977. In foreground, mattresses have subsided and been covered with sediment; in background, narrow space between opposite mattresses is visible, at scarp of headcut. At right, wire is failing to hold riprap.

SITE 278. MURPHY CREEK AT I-25 NEAR KAYCEE, WYO.

Description of site: Lat 43°39', long 106°36'. Dual bridges, built about 1960, each with two 3-column concrete piers in channel, spillthrough abutments.

Drainage area, about 75 mi^2 (194 km^2); valley slope, 0.007. Stream is perennial, semi-alluvial, in valley of moderate relief, little or no flood plain. Channel is meandering.

Hydraulic problem and countermeasure: Minor erosion of abutment fill-slope and damage to riprap was noted by Wyoming State Highway Department in 1965, and some further damage was noted again in 1969. Fill-slope is protected by a combination of class I hand-placed riprap (which is mainly on the upper slope) and rock-and-wire mattress. The mattress is about 16 in (41 cm) thick and consists of cobbles enclosed in woven wire ("hog wire"). In 1969, the class I, hand-placed riprap was damaged, and failure was attributed to the small size; also, it was placed in a single layer, on silt, and without filter blanket.

In April and again in May of 1977, the site was inspected by a U.S. Geological Survey representative. In addition to the previously reported minor problems, an apparent failure of the rock-and-wire mattress at one place was noted (see fig. 296), although in most places it was performing satisfactorily.

Discussion: The hydraulic problems at this site are of little consequence, but some insight is provided into the effectiveness of different types of riprap. The rock-and-wire mattress has given satisfactory service for a period of about 17 years, although it has been broken at one place. The flood history at the site during this period is not known.

Figure 296. Downstream view beneath bridge of northbound lane in April 1977, I-25 at Murphy Creek. At left, hand-placed riprap has been eroded from area above rock-and-wire mattress. At center, wire has been displaced from mattress. At right hand-placed riprap has been eroded from fill-slope at piers.

SITE 279. SNAKE RIVER AT US-187 AND US-189 NEAR JACKSON, WYO.

Description of site: Lat 43°23', long 110°45'. Bridge, built about 1967, is about 390 ft (119 m) in length. Spillthrough abutments are protected by rock-and-wire mattress.

Drainage area, 2,620 mi^2 (6,787 km^2); valley slope, 0.006; channel width, about 600 ft (180 m). Stream is perennial, alluvial, gravel bed, in valley of high relief. Channel is sinuous, generally braided, locally anabranched.

Hydraulic problem and countermeasure: When the bridge at Snake River was designed, it was expected that overflow from Flat Creek would move along the roadway embankment and enter the Snake River. To prevent erosion of the embankment, eight spurs were built along the embankment transverse to the expected direction of flow (figs. 297 and 298). Only the spur adjacent to the Snake River was revetted. During the Snake River flood of June,1972, flow from the Snake River entered Flat Creek upstream from the Flat Creek bridge, and there seemed to be a danger that a substantial part of the Snake River flow would be diverted into Flat Creek. To prevent this, a system of dikes was built between Flat Creek and the Snake River. Flow diverted by the dikes and the roadway embankment has eroded the right streambank upstream from the abutment fill-slopes of the Snake River Bridge (fig. 298).

Discussion: The roadway embankment has not been eroded by overflow, either from the Snake River or from Flat Creek, but whether erosion would have occurred without the spurs is not clear. The system of dikes will probably prevent diversion of the Snake River into Flat Creek, but flow diverted by the dikes and the roadway embankment may erode the right abutment of the Snake River bridge.

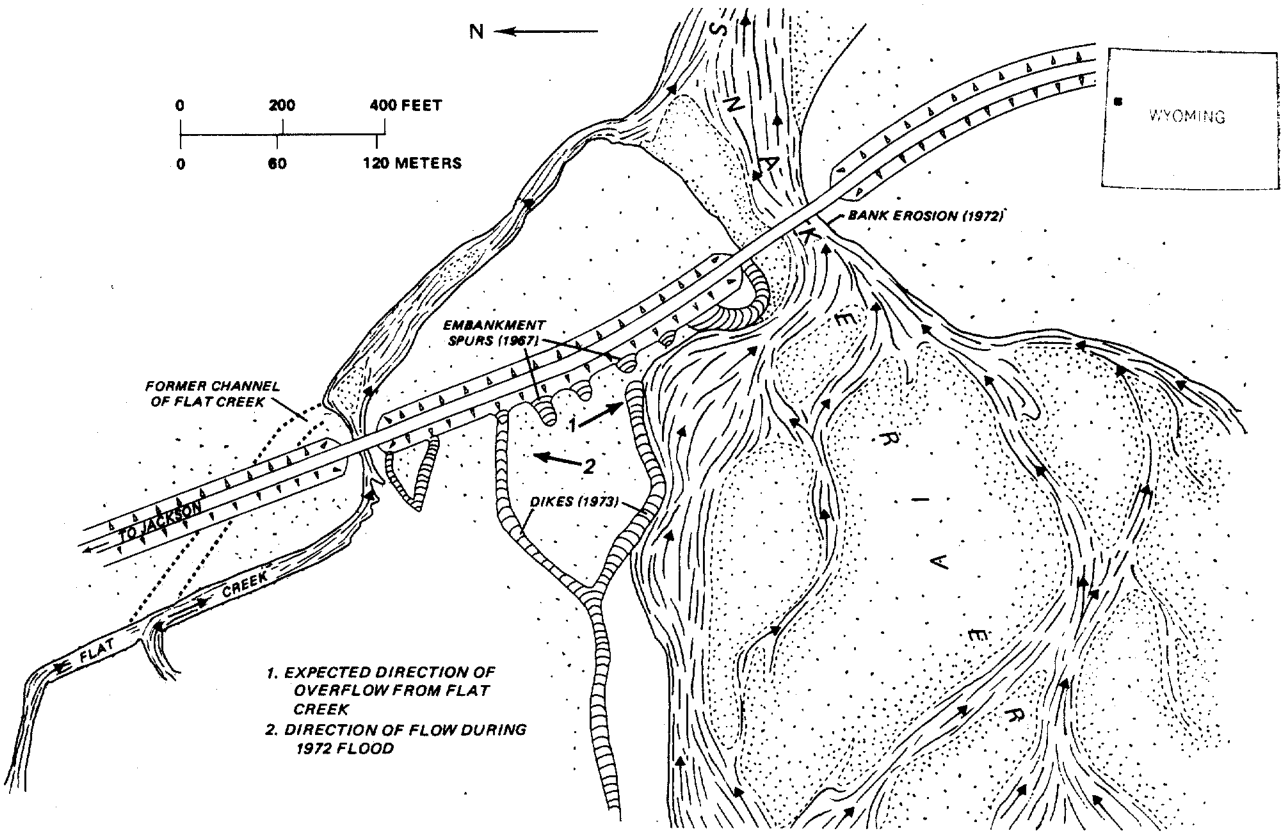


Figure 297. Plan sketch showing direction of overflow between Snake River and Flat Creek at the US-187, 189 bridge, and the use of countermeasures. (From an aerial photograph dated September 5, 1974.)

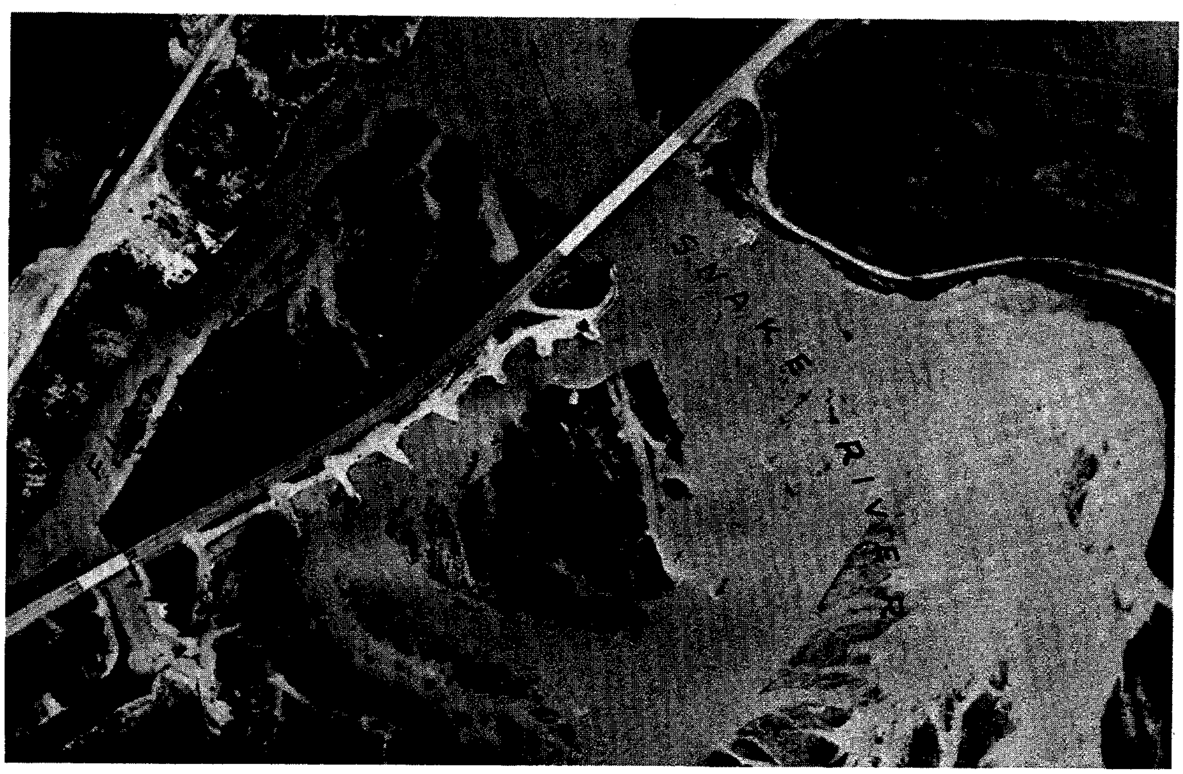


Figure 298. Aerial photograph showing diversion of flood flow from Snake River into Flat Creek on June 1, 1972. (From Wyoming Dept. of Highways.)

SITE 280. BOGUE CHITTO AT US-98 NEAR TYLERTOWN, MISS.

Description of site: Lat 31°11', long 90°17', location as shown in fig. 299. Bridge is 610 ft (183 m) in length, span over main channel is cantilevered steel girder 210 ft (63 m) in length, flanked by I-beam spans on steel H-pile bents. Webbed concrete piers in channel, spillthrough abutments set back from channel. Piers are perpendicular to bridge centerline, but skewed both to flow and low flow. General trend of river upstream from bridge is sharply skewed to bridge.

Drainage area, 502 mi^2 (1,300 km^2); bankfull discharge, about 6,000 ft^3/s (170 m^3/s); valley slope, 0.001, channel width, about 150 ft (45 m). Stream is perennial, alluvial, sand bed, in valley of low relief, wide flood plain. Channel is meandering, wider at bends, point bars narrow or absent, cut banks local, silt-sand banks.

Hydraulic problem and countermeasure:

1940 Bridge built. Shortly after construction, a moderately high flood of undetermined recurrence interval occurred. Fill-slope of right upstream abutment was severely eroded and emergency crew sandbagged approach embankment to prevent further damage.

1941 Abutment fill-slope was protected with grouted-concrete rock riprap. Tow spur dikes (or embankment spurs) of concrete rubble masonry were built at the right approach embankment (fig. 300). These are about 60 ft (18 m) long, 4 ft (1.2 m) wide, and 15 ft (4.5 m) high, with the crest about 8 ft (2.4 m) below road grade. One joins the roadway embankment about 50 ft (15 m) from the abutment, and the other about 200 ft (60 m) from the abutment.

1942-75 Many floods, largest of which had peak discharges in the range of 30-45,000 ft^3/s (850-1,275 m^3/s). No further erosion of abutment fill reported. During inspection of the site in 1975, minor scour of the flood plain under the bridge near the right abutment was observed.

Discussion: Although the spurs are unconventional in design, they have evidently been successful in protecting the abutment, under circumstances in which damage almost certainly would have occurred.

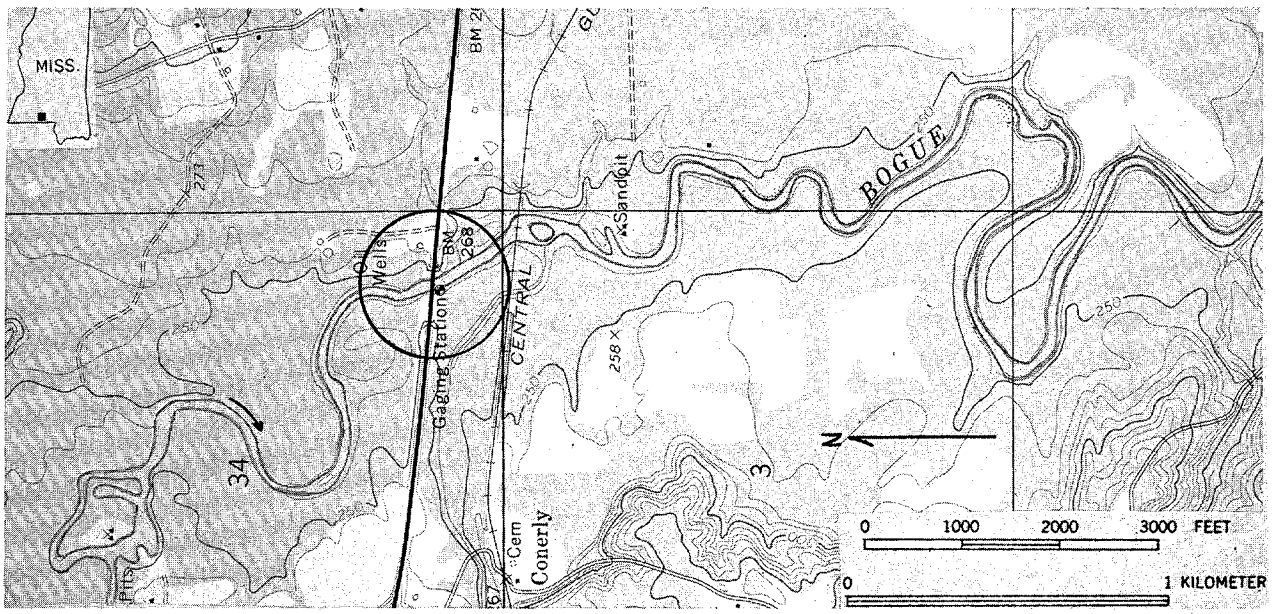


Figure 299. Map showing Bogue Chitto at US-98 crossing (circled) near Tylertown, Miss. (Base from U.S. Geol. Survey Holmesville, Miss., 7.5' quadrangle, contour interval 10 feet, 1971.)

Figure 300. Vertical aerial photograph of Bogue Chitto at US-98 crossing, on February 15, 1973. (From Mississippi Dept. of Highways.)

SITE 281. BUFFALO RIVER AT US-61 NEAR WOODVILLE, MISS.

Description of site: Lat 31°14', long 91°18', location as shown in fig. 301. Bridge is 1,750 ft (533 m) in length, 250-ft (75-m) cantilever girder span over main channel, concrete piers on pile foundations, spillthrough abutments. Crossing is at apex of large meander loop (figs. 301 and 302).

Drainage area, 182 mi^2 (471 km^2); bankfull discharge, about 40,000 ft^3/s (1,132 m^3/s; valley slope, 0.00085; channel width, about 200 ft (60 m). Stream is perennial, alluvial, sand-gravel bed, in valley of moderate relief, wide flood plain. Channel is meandering, wider at bends, point bars, cut banks general, silt-sand banks, tree cover along 50-90 percent of bankline.

Hydraulic problem and countermeasures:

1938 Bridge built, no countermeasures known.

1956 Bank erosion and scour (or degradation) at piers noted. Dumped rock riprap placed at north bank and at two main piers. At south bank, which is at the outside of the meander bend, a double row of jacks was placed, tied with 0.75 in (1.9 cm) steel cable (fig. 302).

1956-67 Series of minor floods, largest of 5-yr R.I. The steel jacks proved ineffective; steel cables tying them together and to the banks were broken and jacks became buried in channel. Left (south) bank just upstream from bridge eroded, and concrete slab riprap was placed at eroded bank.

1968 Eroded bank filled in and timber-pile retard placed at bank. Retard is double row of timber pile, 8 ft (2.4 m) wide, 4 x 6 in (10 x 15 cm) bracing, 4 x 10 in (10 x 25 cm) sheathing, top at elevation 116 ft.

1969-76 Series of minor floods and one (on March 25, 1973) flood of 50-yr R.I. Erosion of south bank and shift of channel to south still hazardous to piers (fig. 303). An additional countermeasure is planned but not yet built: extension of the timber-pile retard (built in 1968) to connect with pier 45 in order to reduce the eddy action at the pier.

Discussion: As shown in fig. 301, the current from the upstream bend is impinging directly on the south bank just upstream from the bridge. None of the countermeasures has been entirely successful in preventing bank erosion at this point. The jack field did not have much chance of success, because jacks tend to become buried when placed in a river channel (rather than in an overbank area). The timber-pile retard has been moderately effective, but the bank is eroding upstream from it, as well as on the downstream side of the bridge.

Figure 301. Aerial photograph of Buffalo River on February 10, 1973, at US-61 crossing. (Mississippi Highway Dept. photograph.)

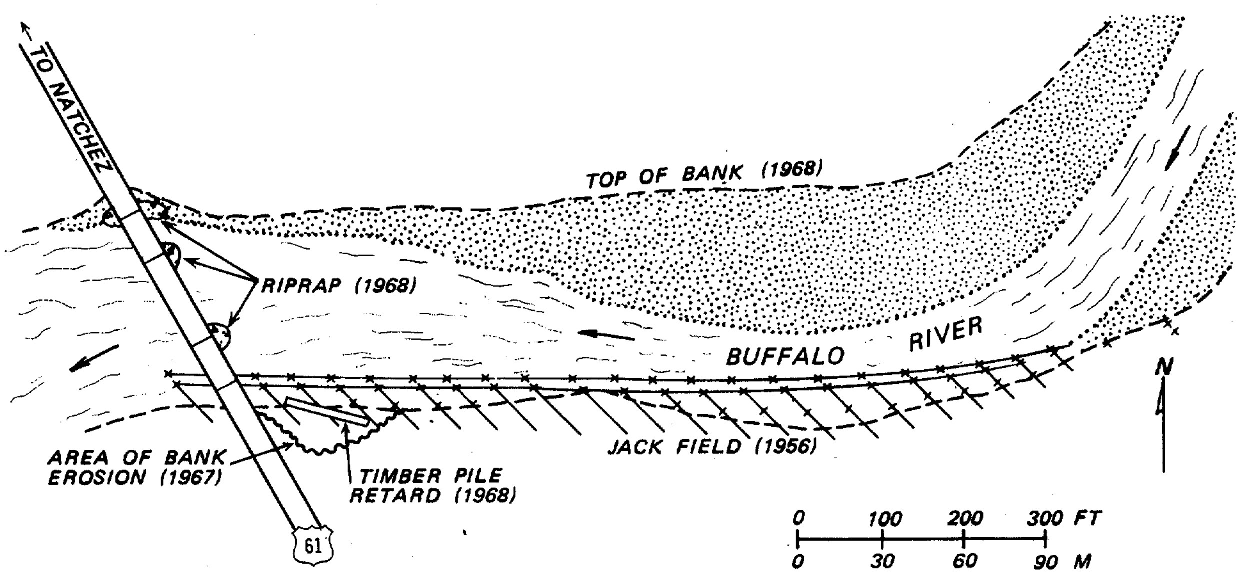


Figure 302. Plan sketch showing countermeasures at US-61 crossing.

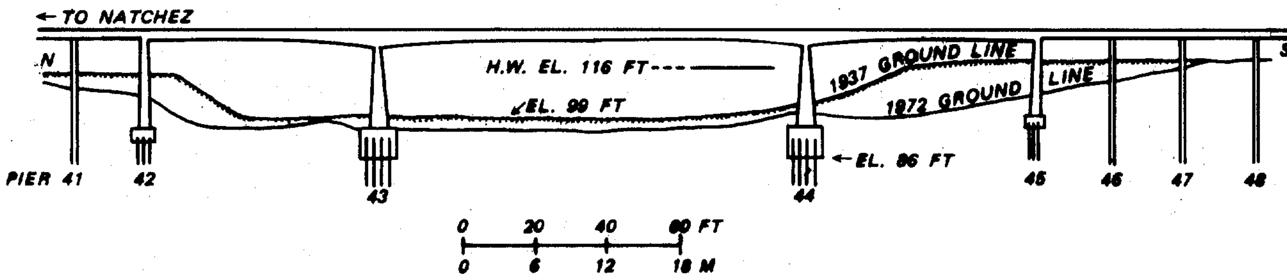


Figure 303. Elevation sketch of US-61 bridge, Buffalo River.

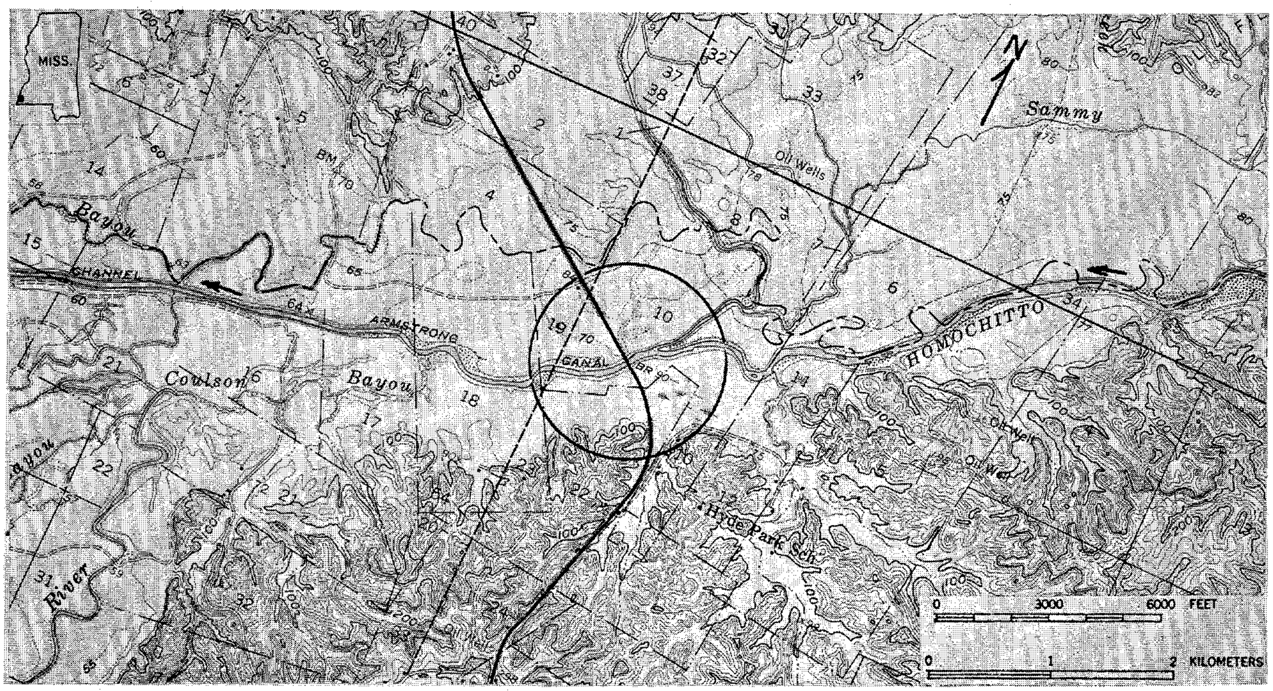


Figure 304. Map showing Homochitto River at US-61 crossing (circled). (Base from U.S. Geol. Survey Kingston, Miss., 15' quadrangle, contour interval 20 feet, 1958.)

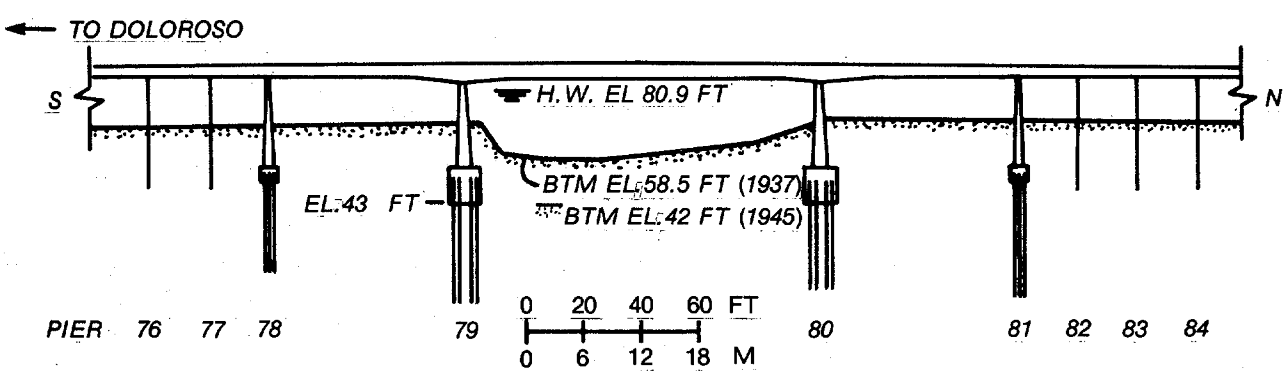


Figure 305. Elevation sketch of main-channel spans of US-61 bridge, which failed in 1955, from plans dated 1937.

SITE 282. HOMOCHITTO RIVER AT US-61 NEAR DOLOROSO, MISS.

Description of site: Lat 31°20', long 91°22', location as shown in fig. 304. Bridge was 3,090 ft (1,189 m) in length, consisted of 67 I-beam approach spans of 20-ft (6-m) length on right flood plain, and 55 approach spans of 21-ft (6-m) length on left flood plain, 250-ft cantilever girder span over main channel. Main piers were concrete, webbed, on timber-pile foundations. After failure of this bridge in 1955, the main-channel spans were replaced with a 550-ft (168-m) continuous truss span and the bottoms of footings were lowered from elevation 43 ft (13 m) to elevation 5 ft (2 m).

Drainage area, 1,120 mi^2 (2,900 km^2); bankfull discharge about 25,000 ft^3/s (707 m^3/s); valley slope, about 0.0004; channel width in 1973, about 400 ft (120 m). Stream is perennial, alluvial, sand bed, in valley of moderate relief, wide flood plain. Channel in 1973 was sinuous, equiwidth, cut banks local, silt-sand banks.

In 1938, the U.S. Army Corps of Engineers diverted the Homochitto River into a new channel from the US-61 bridge to the Mississippi River, shortening the river course from about 17 miles to about 9 miles. Channel degradation began at US-61 in 1944, had reached 16.5 ft (5 m) by 1945 (fig. 305) and increased by an additional 4.5 ft (1.3 m) during the period 1945-57. Since 1957, the channel has aggraded about 2 ft (0.6 m).

Hydraulic problem and countermeasure:

1937 Bridge built, no countermeasures used.

1944-49 North (right) bank of river riprapped (dumped rock approximately equivalent to class I riprap) to protect north main pier from lateral erosion. Steel sheet-piling driven around south main pier and pier footings rebuilt.

1955 Flood, peak discharge about 69,000 ft^3/s (1,952 ,^{3}/s). South main pier (pier 79; see fig. 305) undermined by degradation and scour. Pier failed and caused collapse of main span and south approach span. Both streambanks badly eroded for a distance of several hundred feet upstream from the bridge, except where dumped rock riprap had previously been placed at the north bank.

1956 Main spans rebuilt, center span increased in length from 250 ft (75 m) to 550 ft (168 m(, pier footings lowered by about 38 ft (11.6 m).

1957-74 Large floods during this period, but no maintenance problems reported at bridge.

Discussion: Although the bridge built in 1937 was lost because of severe channel degradation, the continuous maintenance problems encountered at the SR-33 bridge at Rosetta, about 16 valley miles upstream, have not occurred. The channel has been fairly stable laterally at Doloroso (fig. 306) but very unstable at Rosetta. Greater lateral stability at Doloroso is attributed to (1) backwater from the Mississippi River, which tends to reduce flow velocity (2) a somewhat higher clay content in the banks, which decreases erodibility and (3) the artificial channel was more nearly straight at Doloroso, such that re-establishment of the meandering pattern was longer delayed. Lateral instability of the channel, which begins shortly upstream from the US-61 bridge, will probably progress downstream and cause maintenance problems at the bridge unless bank protection measures are installed. The riprap placed at the north bank in 1955 has apparently been effective.

Figure 306. Aerial photograph of Homochitto River in 1971 at US-61 crossing. Note distinct increase in channel width, an indication of lateral instability, that begins upstream from crossing. (From U.S. Dept. of Agriculture.)

SITE 283. BRAZOS RIVER AT PIPELINE CROSSING NEAR US-90A, NEAR HOUSTON, TEX.

(Information and photographs for this case are from a published article by Henson, 1969, and from written communication with Henson, 1978. Although no bridge is involved, the problem of lateral bank erosion is similar to that affecting several bridges on the Brazos River; for example, see Sites 131 and 145).

Description of site: Crossing site of 20-in (51-cm) pipeline of the Houston Pipe Line Co. on the Brazos River about 0.5 mi (0.8 km) downstream from the US-90A crossing.

Drainage area, about 44,000 mi^2 (113,960 km^2) of which 9,000 mi^2 (23,310 km^2) is non-contributing; valley slope, 0.0002; channel width, 500 ft (152 m). Stream is perennial, regulated, alluvial, sand bed, in valley of low relief, wide flood plain. Channel is meandering, wider at bends, point bars, probably incised, cut banks general, silt-clay banks.

Hydraulic problem and countermeasure:

1926 First construction of pipeline crossing near this site.

1927-64 Earlier pipelines undercut several times by lateral bank erosion, were replaced by two 12-in (30.5 cm) pipelines at present site. New crossing was at apex of a broad bend, where the concave bank stood about 40 ft (12 m). Problems with lateral erosion of the bank continued through this period. Measures to protect pipeline included construction of timber-pile bents, also piers of sacked concrete, as supports.

1965 The two 12-in (30.5 cm) pipelines were replaced by one 20-in (51-cm) pipeline, and patented "Spur Jetties" were installed at the concave (right) bank of the crossing by Hold-That-River Engineering Co. of Houston, Texas. The "Spur Jetties" consist of a permeable panel made of treated timber planks fixed to horizontal steel pipe. This panel is bolted loosely to two vertical steel pile (salvaged 6-in (15-cm) steel pipe is sometimes used for pile) driven into the river bed. Each "Spur Jetty" is oriented normal to the direction of streamflow. The timber-and-pile panel has negative buoyancy, and slides on the vertical pile supports to maintain contact with the river bed. The function of the "Spur Jetties" is to reduce water velocity, induce sediment deposition, and shift the thalweg away from an eroding bank. As a panel is undercut by scour, it continues to shift downward until scour no longer occurs. Installation of the "Spur Jetties" was completed in July.

1966 After a flow of near bankfull stage, the river stage dropped and the performance of the "Spur Jetties" was assessed. Sediment had accumulated at the jetties, and some panels had subsided to maintain contact with the streambed (fig. 307A). The point bar deposit on the opposite bank was no longer visible, indicating that the thalweg had shifted toward the convex bank.

1969 The concave bank at the crossing continued to stabilize (fig. 307B), and vegetation was becoming established on the deposits at the "Spur Jetties".

1975 Further stabilization of the concave bank is evident from the growth of willows (fig. 307C).

<u>Discussion</u>: Photographic evidence indicates that the "Spur Jetties" have stabilized a high, eroding bank at a bend of the Brazos River. The difficulty of controlling lateral erosion on the Brazos is judged to be greater than average for meandering rivers because of the height and erodibility of the banks and the frequency of bankfull flows. However, conditions are favorable for the performance of permeable spurs, because the river transports a substantial sediment load and the climate promotes the establishment of vegetation. Although the "Spur Jetties" normally require routine maintenance, particularly if they are damaged by floating drift, this particular installation has required no maintenance since the first year.

The performance of an improved design of "Spur Jetty", installed in 1975 at the first bend downstream from the site described above, is documented in figs. 308A-C.

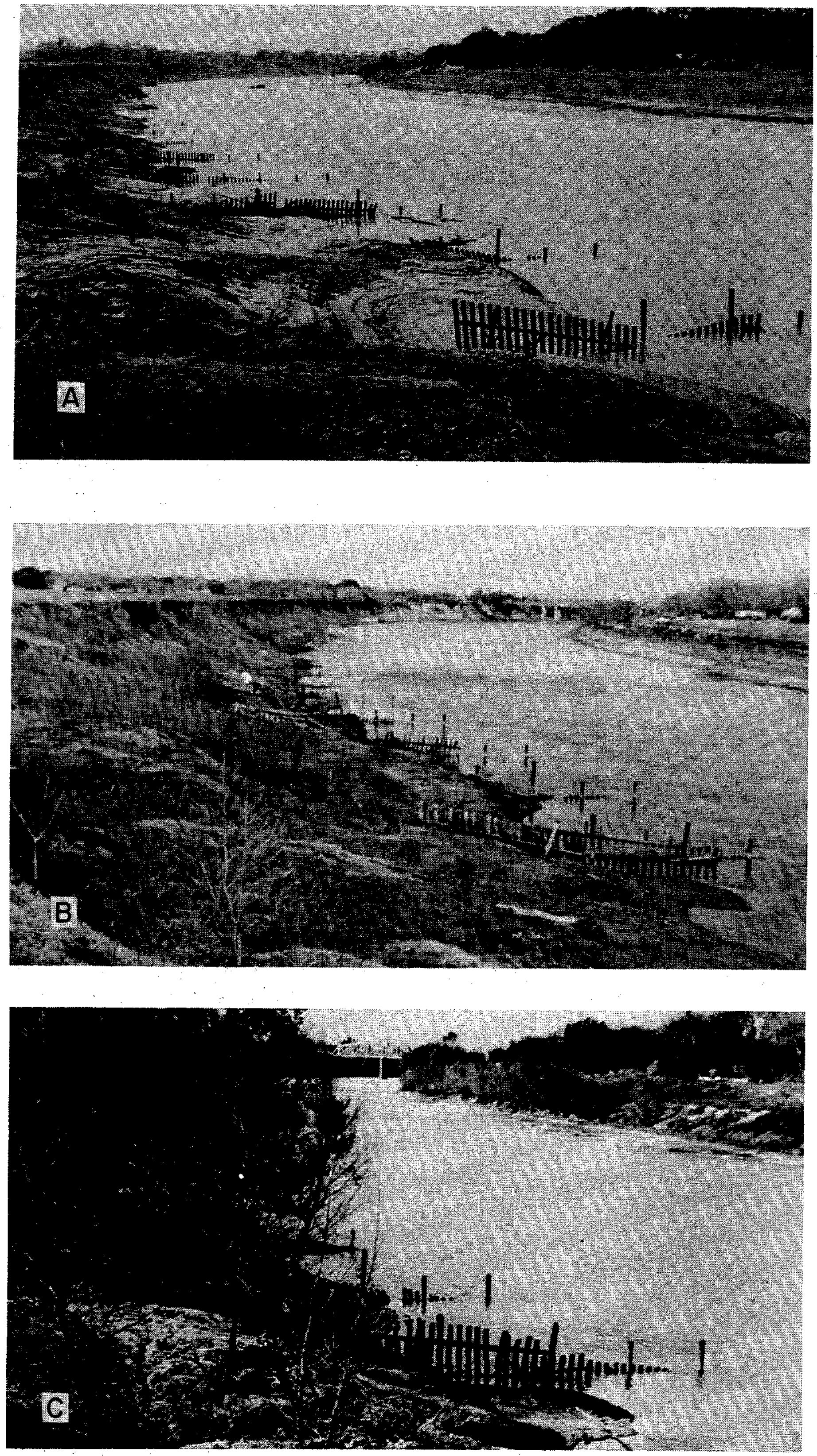


Figure 307. Sequential photographs of "spur jetties" at right bank of Brazos River, looking upstream, US-90A crossing in background. A, bank in 1966, after a flood; B, bank in 1969; C, bank in 1975.

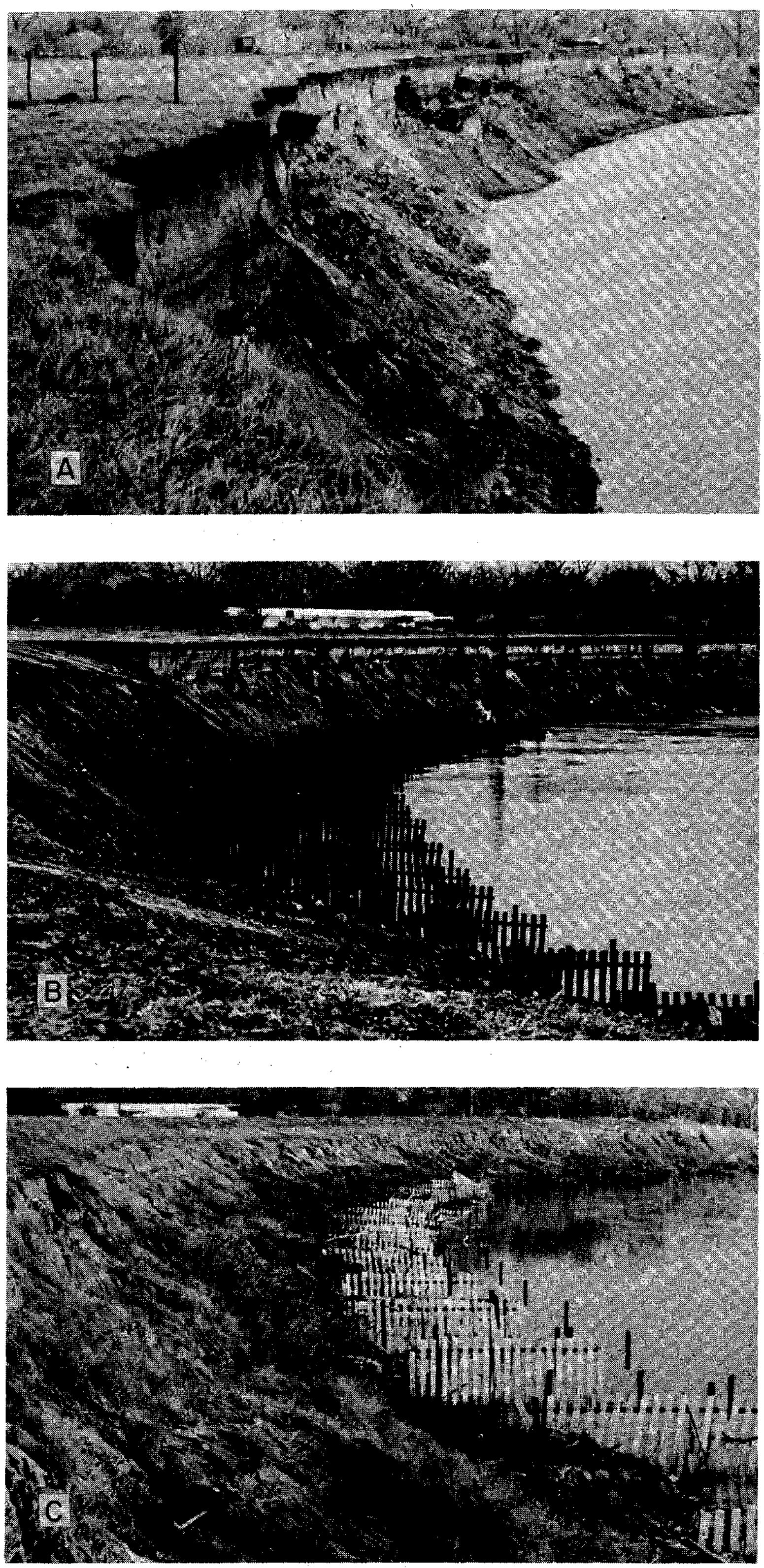


Figure 308. "Spur jetty" installation at eroding bank, Brazos River at R. E. Smith Ranch near Houston, Texas. A, downstream view of bank prior to installation, March 1975. B, installation in progress, 1975. C, view of jetties in 1978, after flows of near bankfull stage.

☆ U. S. GOVERNMENT PRINTING OFFICE : 1979 281-568/33

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.

* 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.

3. Environmental Considerations in Highway Design, Location, Construction, and Operation

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This category is concerned with developing and transferring research and technology into practice, or, as it has been commonly identified, "technology transfer."

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Maintenance R&D objectives include the development and application of new technology to improve management, to augment the utilization of resources, and to increase operational efficiency and safety in the maintenance of highway facilities.

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