HISTORICAL AND POTENTIAL SCOUR AROUND BRIDGE PIERS AND ABUTMENTS OF SELECTED STREAM CROSSINGS IN INDIANA

By David S. Mueller, Robert L. Miller, and John T. Wilson

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BRUCE BABBITT, Secretary

U.S. GEOLOGICAL SURVEY

Gordon P. Eaton, Director

For additional information write to:

District Chief
U.S. Geological Survey
Water Resources Division
5957 Lakeside Boulevard
Indianapolis, Indiana 46278-1996

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CONTENTS

CONTENTS--Continued

Page

ILLUSTRATIONS

ILLUSTRATIONS--Continued

TABLES

Page

TABLES--Continued

Page

CONVERSION FACTORS AND VERTICAL DATUM

Multiply	*By*	*To obtain*
inch (in.)	25.4	millimeter
foot (ft)	0.3048	meter
mile (mi)	1.609	kilometer
square mile (m^2)	2.590	square kilometer
cubic foot (ft^3)	0.02832	cubic meter
cubic foot per second (ft^3/s)	0.02832	cubic meter per second
foot per mile (ft/mi)	0.1894	meter per kilometer

Sea level: In this report, "sea level" refers to the National Geodetic Vertical Datum of 1929 (NGVD of 1929)--a geodetic datum derived from a general adjustment of the first-order level nets of both the United States and Canada, formerly called Sea Level Datum of 1929.

SYMBOLS

a--A coefficient based on the ratio of the shear velocity (u_*) to the fall velocity (ω) in the uncontracted channel.

a	u_*/ω	Mode of bed-material transport
0.25	<0.5	Mostly contact bed-material discharge
1.00	0.5-2.0	Some suspended bed-material discharge
2.25	>2.0	Mostly suspended bed-material discharge

A_e--Cross-sectional area of the flow obstructed by the embankment.

b--Width of the bridge pier.

b'--Width of the bridge pier projected normal to the approach flow.
$b' = b\cos(\alpha) + L\sin(\alpha)$.

B_c--Bottom width of the contracted section.

B_u--Bottom width of the uncontracted or approach section.

d_m--Mean grain size of the bed material.

d_{50}--Median grain size of the bed material.

F_a--Froude number of the flow defined as $F_a = \dfrac{\left(\dfrac{Q_e}{A_e}\right)}{\sqrt{gy_o}}$.

F_o--Froude number of the flow just upstream from the pier or abutment.

F_p--Pier Froude number, defined as, $\dfrac{V_o}{\sqrt{gb}}$.

f_b--Bed factor, defined as, $\dfrac{V^2}{y}$.

g--Acceleration of gravity.

K--A coefficient that is a function of boundary geometry, abutment shape, width of the piers, shape of the piers, and the angle of the approach flow. On the basis of numerous model studies, Ahmad (1962) suggested that, for calculation of scour at piers and abutments, the coefficient should be in the range of 1.7 to 2.0. For this investigation, it was assumed to be 1.8.

K_{sa}--A coefficient based on the geometry of the abutment (1.0 for a vertical abutment that has square or rounded corners and a vertical embankment, 0.82 for a vertical abutment that has wingwalls and a sloped embankment, and 0.55 for a spill-through abutment and a sloped embankment).

K_{S1}--A coefficient based on the shape of the pier nose (table 2).

K_{S2}--A coefficient based on the shape of the pier nose; 1.0 for cylindrical piers and 1.4 for rectangular piers.

K_1--A coefficient based on the shape of the pier nose; 1.1 for square-nosed piers, 1.0 for circular- or round-nosed piers, 0.9 for sharp-nosed piers, and 1.0 for a group of piers.

K_2--A coefficient based on the ratio of the pier length (L) to pier width (b) and the angle of the approach flow referenced to the bridge pier.

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

K_θ--A coefficient based on the inclination of an approach roadway embankment to the direction of the flow,

$$K_\theta = \left(\frac{\theta}{90}\right)^{0.13}.$$

$K_{\alpha L}$--A coefficient based on the angle of the approach flow referenced to the bridge pier (fig. 25).

L--Length of the bridge pier.

l--Length of an abutment, defined as, A_e/y_{oa}.

l_{ae}--Effective length of an abutment.

l_{at}--Abutment and embankment length measured at the top of the water surface and normal to the side of the channel from where the top of the design flood hits the bank to the other edge of the abutment (Richardson and others, 1991, p. B-7).

n_c--Manning's roughness coefficient for the part of the contracted channel represented by the specified bottom width.

n_u--Manning's roughness coefficient for the part of the uncontracted or approach channel represented by the specified bottom width.

q--Discharge per unit width just upstream from the pier.

q_{mc}--Discharge per unit width in the main channel.

Q--Discharge.

Q_c--Discharge in the part of the contracted channel represented by the specified bottom width.

Q_e--Discharge obstructed by the embankment.

Q_u--Discharge in the part of the uncontracted or approach channel represented by the specified bottom width.

r--A coefficient used to relate scour in a long contraction to scour at an abutment or pier.

$\boldsymbol{R}_p$--Pier Reynolds number, defined as, $\frac{V_o b}{\nu}$.

S--Dimensionless slope of the energy grade line near the bridge.

u_*--Shear velocity, defined as, $\sqrt{g y_u S}$.

V--Average velocity of the section.

V_o--Velocity of the approach flow just upstream from the bridge pier or abutment.

y--Average depth of the section.

y_b--Average depth of flow at the bridge.

y_c--Average depth of flow in the contracted channel.

y_{ca}--Depth of abutment scour, including contraction scour.

y_o--Depth of flow just upstream from the bridge pier or abutment, excluding local scour.

y_{oa}--Depth of flow at the abutment.

y_p--Depth of flow at the bridge pier, including local pier scour.

y_r--Regime depth of flow, defined as, $\left(\frac{q^2}{f_b}\right)^{1/3}$.

y_{sa}--Depth of abutment scour below the ambient bed.

y_{sc}--Depth of contraction scour below the reference bed level.

y_{sp}--Depth of pier scour below the ambient bed.

y_u--Average depth of flow in the uncontracted channel.

τ_c--Critical shear stress.

τ_o'--Boundary shear stress of the approach flow associated with the sediment particles.

ω--Fall velocity of the median grain size of the bed material.

ν--Kinematic viscosity of water.

α--Angle of the approach flow referenced to the bridge pier, in degrees.

θ--Angle of inclination of an embankment to the flow, in degrees; $\theta < 90°$ if the embankment points downstream.

ϕ--A coefficient based on the shape of the pier nose; 1.3 for square-nosed piers, 1.0 for round-nosed piers, 0.7 for sharp-nosed piers.

HISTORICAL AND POTENTIAL SCOUR AROUND BRIDGE PIERS AND ABUTMENTS OF SELECTED STREAM CROSSINGS IN INDIANA

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ABSTRACT

Historical scour data were collected by means of geophysical techniques and used to evaluate the scour-computation procedures recommended by the U.S. Federal Highway Administration and 12 other published pier-scour equations. Geophysical data were collected with a ground-penetrating radar system and a tuned transducer at 10 bridges in Indiana. Data obtained from soil-boring logs from the bridge-construction plans and by probing with a steel pipe were used to support the geophysical data. The approximate location and depth of subsurface interfaces indicating possible scour holes were identified at nine sites. These geophysical data were used to evaluate 13 pier-scour equations. For this comparison, it was assumed that the historical scour measured by use of geophysical techniques was associated with the peak historical discharge. The hydraulic conditions for the peak historical discharge were estimated by use of the Water-Surface Profile (WSPRO) computation model. Because the geophysical data were not sufficient to map the lateral extent of the refilled scour hole, local scour could not be separated from contraction scour. For the evaluation, the results of the contraction and pier-scour equations were combined to determine a computed bed elevation, which was compared to the minimum historical bed elevation at the upstream end of the piers estimated from the geophysical data. None of the pier-scour equations accurately represented the historical scour at all of the study sites. Only 3 of the 13 pier-scour equations commonly produced results that were grossly different from the historical data. On the basis of the limited data presented, the Federal Highway Administration procedures provided a combination of accuracy and safety, required by design equations, equal to or better than the other equations evaluated.

The potential scour resulting from the 100-year and 500-year peak discharges was computed according to the procedures recommended by the Federal Highway Administration. At two bridges, the procedures overpredicted historical scour by more than 10 feet, and at two other bridges, the procedure underpredicted historical scour by more than 5 feet; therefore, the potential-scour computations need to be verified by additional data and sediment-transport modeling. Computed abutment scour appeared to be excessive at about half of the sites; however, current Federal Highway Administration guidance suggests protection of abutments by riprap and, where appropriate protection is provided, abutment scour need not be computed.

INTRODUCTION

Scour of the streambed in the vicinity of bridge piers and abutments during floods has resulted in more bridge failures than all other causes in recent history (Murillo, 1987). The I-29 bridge over the Big Sioux River in Iowa failed because of scour in 1962, as did the I-64 bridge over the John Day River in Oregon in 1964. In 1985, 73 bridges were destroyed or damaged by scour resulting from floods in Pennsylvania, Virginia, and West Virginia. In 1987, 17 bridges in New York and New England were damaged or destroyed by scour, including the New York State Thruway bridge spanning Schoharie Creek that resulted in the loss of 10 lives (Harrison and Morris, 1991, p. 210). In 1989, eight people were killed when the U.S. Route 51 bridge over the Hatchie River in Tennessee failed because of a lateral shift of the stream. In 1990, the Troy Avenue bridge over Buck Creek near Indianapolis, Ind., failed because of scour of the streambed. Consequently, damage to bridges resulting from scour of the streambed is a serious problem of national concern.

The U.S. Federal Highway Administration (1988) recommended that, "Every bridge over an alluvial stream, whether existing or under design, should be evaluated as to its vulnerability to floods in order to determine the prudent measures to be taken for its protection." More than 35 equations for the prediction of scour around bridge piers, a significant number of abutment-scour equations, and several contraction-scour equations are published in the literature. Nearly all of these equations are empirical and are based on laboratory data collected by use of flumes with uniform cohesionless bed materials under steady-flow conditions. Minimal data have been collected to verify the applicability and accuracy of these equations for the range of soil conditions, streamflow conditions, and bridge designs that exist throughout the United States (Richardson and others, 1991). Anderson (1974) showed that, for identical conditions, the scour predicted by different pier-scour equations can vary by a factor of 6 or greater. The U.S. Federal Highway Administration (FHWA) has published two Hydraulic Engineering Circulars (Richardson and others, 1991; Lagasse and others, 1991) that provide guidance for evaluating scour and stream instabilities at highway stream crossings. Richardson and others (1991, p. 23) recommend that--

> Adequate consideration must be given to the limitations and gaps in existing knowledge when using currently available formulas for estimating scour. The designer needs to apply engineering judgment in comparing results obtained from scour computations with available hydrologic and hydraulic data to achieve a reasonable and prudent design. Such data should include:
> a. Performance of existing structures during past floods,
> b. Effects of regulation and control of flood discharges,
> c. Hydrologic characteristics and flood history of the stream and similar streams, and
> d. Whether the bridge is structurally continuous.

Therefore, to improve the accuracy of scour computations at a site, existing equations need to be evaluated and the results compared to field measurements at sites with similar hydraulic and geotechnical conditions. Before this study, no published data were available to assess the applicability of existing scour equations to hydraulic and geotechnical conditions at bridge sites in Indiana. Because scour holes often refill after the passage of a flood, simple bed surveys are not sufficient to determine the depth of scour holes that formed during previous floods. Geophysical techniques such as ground-penetrating radar and continuous high-resolution subbottom seismic profiling must be used to delineate the scour hole formed by a previous flood. To verify the FHWA procedures for use in Indiana, the U.S. Geological Survey, in cooperation with the Indiana Department of Transportation, evaluated the existing published equations to provide information on 10 bridge sites.

Purpose and Scope

This report provides an evaluation of techniques for measuring historical scour, assesses the ability of selected published scour-computation procedures to duplicate the measured historical scour, and presents estimates of potential scour resulting from the 100- and 500-year floods. This information will assist the Indiana Department of Transportation (INDOT) and the FHWA in making decisions about the safety of the selected bridges and determining if the procedures used in this study are efficient and reliable for future bridge-scour investigations in Indiana.

Acknowledgments

The technical assistance and review provided by John Pangallo of INDOT is greatly appreciated.

DESCRIPTION OF SITES

Ten sites were selected from a list of potential sites provided by INDOT. The sites were selected to represent different geographic regions and a wide range of drainage areas within Indiana (fig. 1). The description of bed material is based on the sediment grade scale shown in table 1.

Bridge 24-91-3731A, U.S. Route 24 over Tippecanoe River at Monticello, Ind.

This study site (fig. 2), which drains 1,768 mi^2, is in White County, approximately 75 mi northwest of Indianapolis (fig. 1). The site is

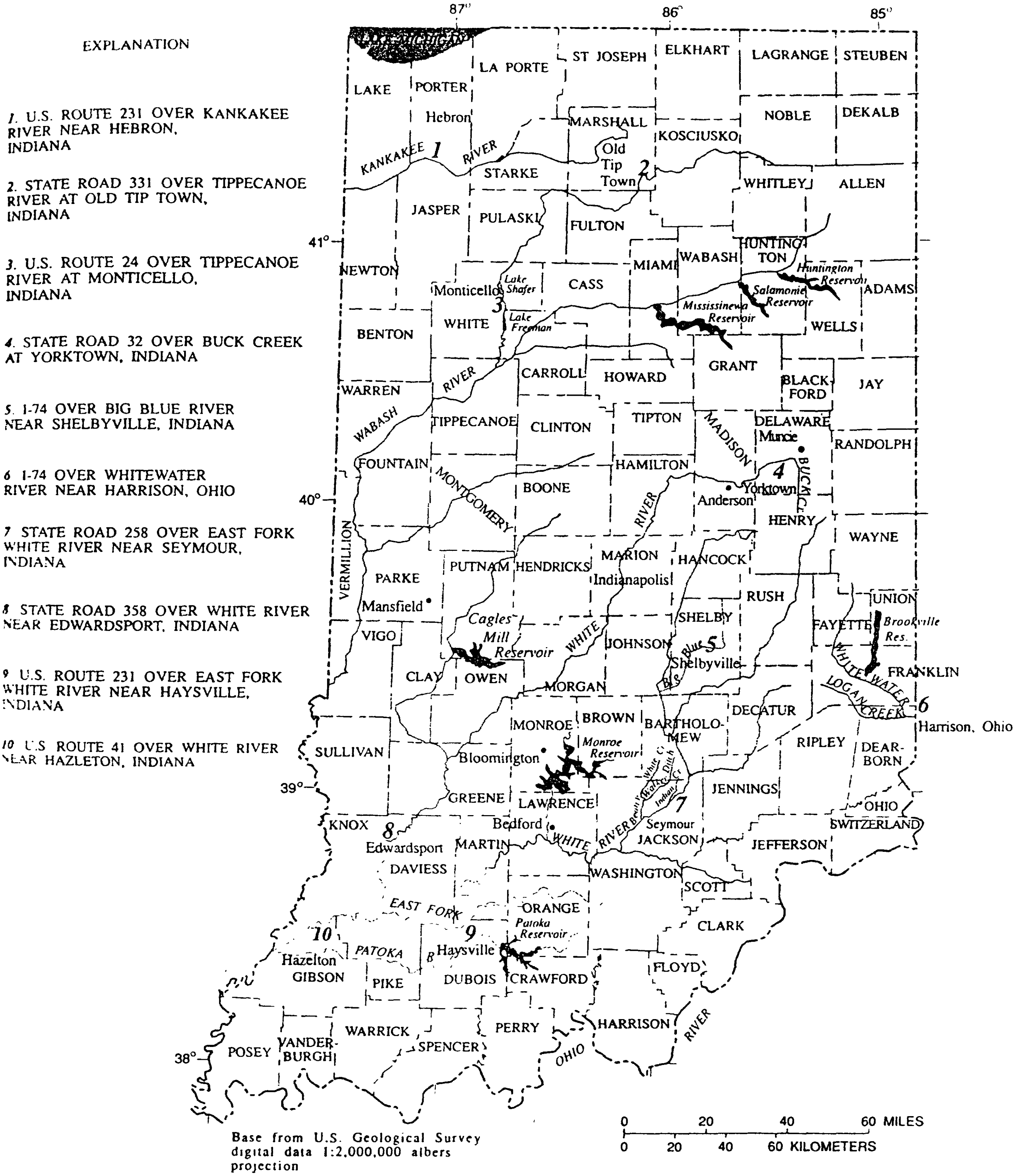


Figure 1. Location of study sites.

Table 1. *Sediment grade scale*

[Modified from Lagasse and others, 1991, p. 11]

Size			Sieve number		
Millimeters	Micrometers	Inches	Tyler	U.S. # standard	Class
4,100 -2,000	--	160 -80	--	--	Very large boulders
2,000 -1,000	--	80 -40	--	--	Large boulders
1,000 - 500	--	40 -20	--	--	Medium boulders
500 - 250	--	20 -10	--	--	Small boulders
250 - 130	--	10 - 5	--	--	Large cobbles
130 - 64	--	5 - 2.5	--	--	Small cobbles
64 - 32	--	2.5 - 1.3	--	--	Very coarse gravel
32 - 16	--	1.3 - .6	--	--	Coarse gravel
16 - 8	--	.6 - .3	2.5	--	Medium gravel
8 - 4	--	.3 - .16	5	5	Fine gravel
4 - 2	--	.16 - .08	9	10	Very fine gravel
2.00 - 1.00	2,000 -1,000	--	16	18	Very coarse sand
1.00 - .50	1,000 - 500	--	32	35	Coarse sand
.50 - .25	500 - 250	--	60	60	Medium sand
.25 - .125	250 - 125	--	115	120	Fine sand
.125 - .062	125 - 62	--	250	230	Very fine sand
.062 - .031	62 - 31	--			Coarse silt
.031 - .016	31 - 16	--			Medium silt
.016 - .008	16 - 8	--			Fine silt
.008 - .004	8 - 4	--			Very fine silt
.004 - .0020	4 - 2	--			Coarse clay
.0020 - .0010	2 - 1	--			Medium clay
.0010 - .0005	1 - .5	--			Fine clay
.0005 - .0002	.5 - .24	--			Very fine clay

in a commercially developed urban area; the topography of the basin is gently rolling, and land use is predominantly farmland. The channel at the study site is deeply entrenched in a narrow valley. The bridge is between Lake Shafer (upstream) and Lake Freeman (downstream). Both lakes are operated solely for water supply and recreation with no flood-control objectives; however, peak flows may be partially attenuated.

The channel approaching the bridge is fairly straight and directs flow through the bridge parallel to the piers. The approach is well developed; a retaining wall and boat docks are along the left bank. The overbank is a gravel parking lot and boat-storage area. The right bank is protected with large boulders, and the overbank consists of mowed grass and paved parking lots. The downstream right bank is natural, with trees and brush on the overbank. Boat docks have been built along the downstream left bank, which is unprotected; the overbank is mowed grass. All the banks appear to be stable. The bed material was not visible; however, soil-boring logs from the bridge plans indicate that the bed material is mostly sand and gravel (fig. 3).

Bridge 32-18-5441A, S.R. 32 over Buck Creek at Yorktown, Ind.

This study site (fig. 4), which drains 100 mi^2, is in Delaware County, approximately 45 mi northeast of Indianapolis (fig. 1). The site is in an urban area consisting of commercial structures and residences. The basin is gently rolling and drains cultivated and urban areas.

The channel in the vicinity of the bridge is dredged. The flood plain just upstream from the bridge is approximately the same width as the bridge opening; therefore, a constriction does not exist at the bridge. Below the bridge, the widening valley allows some expansion. The channel approach to the bridge is straight; however, the channel curves approximately 30° to the left 500 ft downstream from the bridge. The low-water control is a series of gravel riffles; the medium and high-water control is the channel. The channel slope measured from the USGS Muncie West quadrangle map is 0.0024 ft/ft.

The banks upstream are covered with small trees and brush. The banks downstream are covered with grass and weeds. All banks appear to be stable. The highway plans did not provide soil-boring logs at this site but, based on observation, the bed material is sand, gravel, and small boulders.

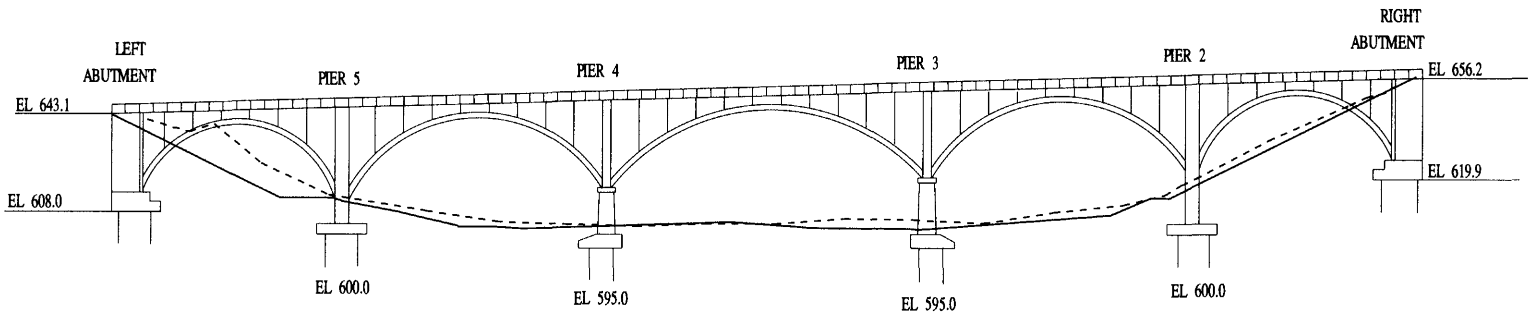
Bridge 41-26-3917C, U.S. Route 41 over White River near Hazleton, Ind.

This study site (fig. 5), which drains 11,305 mi^2, is on the Knox and Gibson County line, approximately 120 mi southwest of Indianapolis (fig. 1). The topography at the site is hilly with a wide flood plain, and land use is predominantly agricultural. The basin drains parts of central and east-central Indiana and much of southern Indiana, where the topography and land use range from rolling farmland to hilly forested areas. Several metropolitan areas are within the basin, including Indianapolis, Anderson, Bloomington, Columbus, and Bedford. Two flood-control reservoirs are within the basin: Cagles Mill Reservoir, draining 293 mi^2 and Monroe Reservoir, draining 432 mi^2.

The channel is fairly straight 0.5 mi upstream and downstream from the bridge, and flow through the bridge is parallel to the piers. The right bank is stable. The left bank, a vertical clay bank devoid of vegetation in many places, has slumped into the channel. Both banks are tree lined. The overbanks are cultivated fields. The channel slope measured from the USGS Iona, Union, Patoka, and Decker quadrangle maps is 0.00014 ft/ft. Although the bed material was not visible, soil-boring logs from the bridge plans indicate that the bed material is most likely sand (fig. 6).

Bridges I-74-114-4192B Eastbound and Westbound, I-74 over Big Blue River near Shelbyville, Ind.

This study site (fig. 7), which drains 314 mi^2, is in Shelby County, approximately 25 mi southeast of Indianapolis (fig. 1). Two bridges are at this site, a westbound bridge upstream from an eastbound bridge. The basin and the site are best characterized as gently rolling farmland.



EXPLANATION

- - - - - - - STUDY BED ELEVATION

——— CONSTRUCTION BED ELEVATION

EL 600.0 ELEVATION, IN FEET ABOVE SEA LEVEL-- Elevations below each pier refer to the bottom of that footing. Pier numbering is consistent with bridge plans

BRIDGE CHARACTERISTICS

SKEW OF DECK (DEGREES): 0
WIDTH (FEET): 40.75
LENGTH (FEET): 469.5
YEAR CONSTRUCTED: 1947

PIER CHARACTERISTICS

PIER	WIDTH (FEET)	LENGTH (FEET)	NOSE	FOUNDATION	SKEW (DEGREES)
2	5	27.5	ROUND	FOOTING ON PILES	0
3	5 TO 6.5	33.3	ROUND	FOOTING ON PILES	0
4	5 TO 6.5	33.3	ROUND	FOOTING ON PILES	0
5	5	27.5	ROUND	FOOTING ON PILES	0

ABUTMENT CHARACTERISTICS

TYPE OF ABUTMENT: SPILL THROUGH
TYPE OF FOUNDATION: FOOTING ON PILES

NOTE: PILES NOT SHOWN TO SCALE

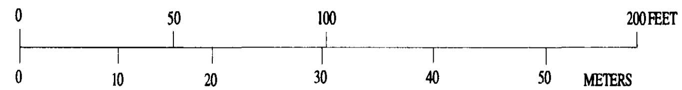


Figure 2. Characteristics of U.S. Route 24 over Tippecanoe River at Monticello, Indiana.

Boring No 2 -

Station 14+09			
Offset 35' Lt			
Surface Elevation 610.2			
ELEVATION	DEPTH	NUMBER OF BLOWS	DESCRIPTION
610.5	0		Water Surface
	2		
	4		
	6		Sand and Gravel
	8		
	10		
598.5	12		

Boring No 4 -

Station 17+05			
Offset 27' Rt			
Surface Elevation 610.2			
ELEVATION	DEPTH	NUMBER OF BLOWS	DESCRIPTION
610.5	0		Water Surface
	2		
	4		
	6		Sand and Gravel
	8		
	10		
598.2	12		

Figure 3. Soil-boring logs, U.S. Route 24 over Tippecanoe River at Monticello, Indiana (taken from Indiana State Highway Commission, 1947, Bridge plans, sheet 4).

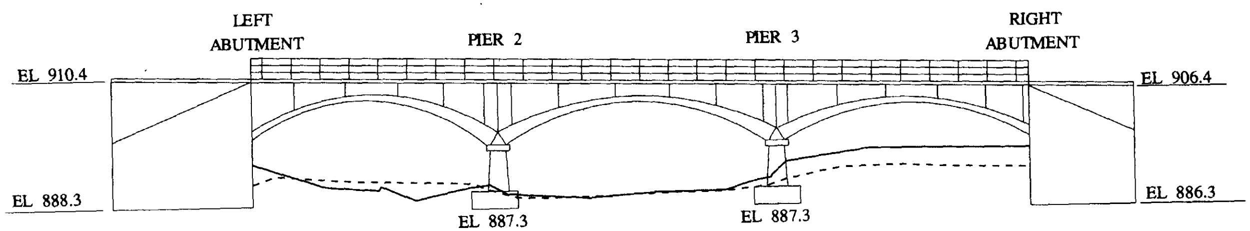
The low-water channel curves sharply to the left 400 ft upstream from the upstream bridge and flows through the bridge openings at a skew of 30°. A low-water island has formed between the bridges. The channel makes a gradual curve to the left just downstream from the downstream bridge. Along the downstream right bank is an earth levee that is parallel to the right abutment.

The upstream banks, which are steep and partly bare of vegetation, have slumped into the channel. Both upstream banks are unstable. The low-water control is a series of gravel riffles. The high-water control is the channel and levee on the right bank. The upstream overbanks are predominantly cultivated with a wooded area along the channel. Both downstream banks are wooded and appear to be stable. The area behind the levee consists of a cultivated field and a gravel pit. The channel slope measured from the USGS Shelbyville quadrangle map is 0.0012 ft/ft. On the basis of observation, the bed material is predominantly sand to coarse gravel, with occasional cobbles or boulders. Soil-boring logs from the highway plans are shown in figure 8.

Bridges I-74-170-4684A and I-74-170-4684JA, I-74 over Whitewater River near Harrison, Oh.

This study site (fig. 9), which drains 1,344 mi^2, is in Dearborn County, approximately 80 mi southeast of Indianapolis (fig. 1). Two bridges are at this site, a westbound bridge upstream from an eastbound bridge. The banks are steep, and surrounding land is hilly and predominantly forested. The Whitewater River valley is approximately 0.75 mi wide, and the flood plain is cultivated in most areas. The basin is characterized by hilly forests to rolling farmlands. A large multipurpose reservoir, Brookville Reservoir (operated since January 1974), drains 389 mi^2 of the basin.

The low-water channel is near the left bank; pier 6 of the downstream bridge and pier 15 of the upstream bridge are in this channel (fig. 9). Pier 5 of the downstream bridge and pier 14 of the upstream bridge are on a gravel bar above the low-water channel. Logan Creek, which drains 13.2 mi^2, flows into the Whitewater River from the right bank between the study bridges. Logan Creek approaches at a 30° angle,



EXPLANATION

- - - - - - - - - - -	STUDY BED ELEVATION
————————	CONSTRUCTION BED ELEVATION
EL 910.4	ELEVATION, IN FEET ABOVE SEA LEVEL-- Elevations below each pier refer to the bottom of that footing. Pier numbering is consistent with bridge plans.

BRIDGE CHARACTERISTICS

SKEW OF DECK (DEGREES): 0
WIDTH (FEET): 68
LENGTH (FEET): 134
YEAR CONSTRUCTED: 1928
WIDENED: 1968

PIER CHARACTERISTICS

PIER	WIDTH (FEET)	LENGTH (FEET)	NOSE	FOUNDATION	SKEW (DEGREES)
2	3.7	59.0	ROUND	SPREAD FOOTING	0
3	3.7	59.0	ROUND	SPREAD FOOTING	0

ABUTMENT CHARACTERISTICS

TYPE OF ABUTMENT: VERTICAL WITH WING WALLS
TYPE OF FOUNDATION: SPREAD FOOTING

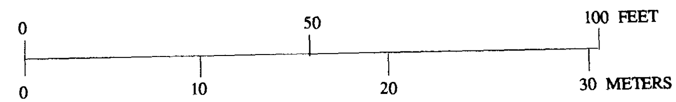
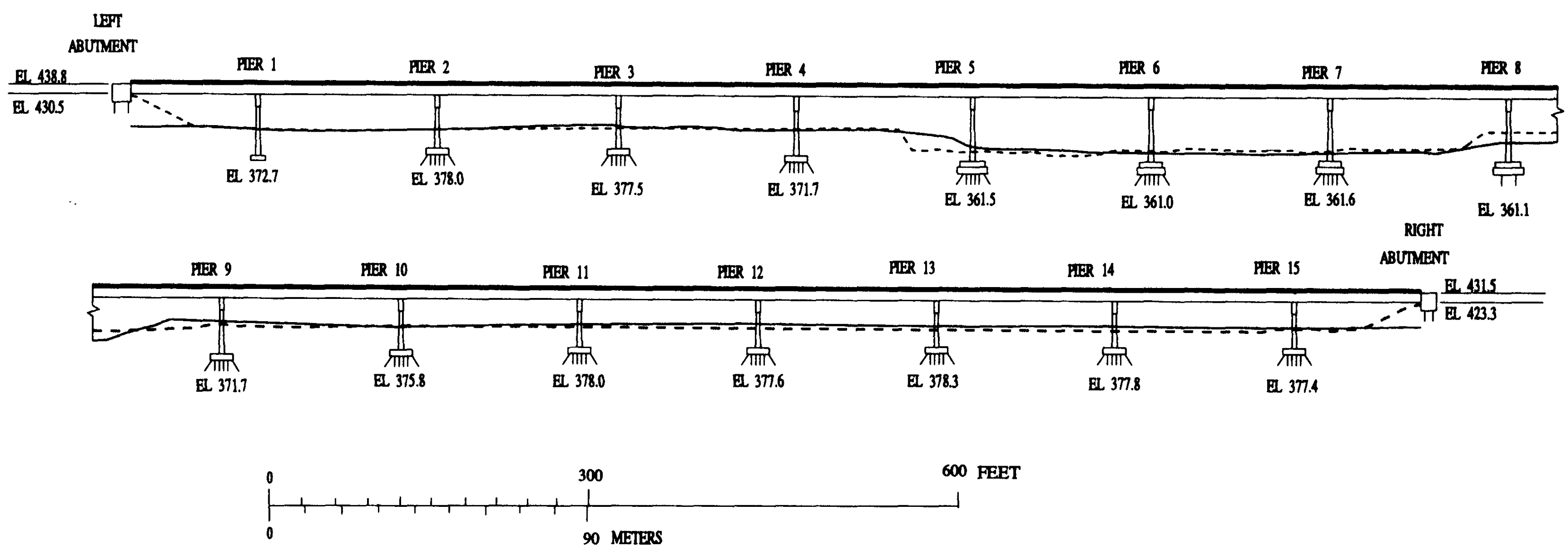


Figure 4. Characteristics of State Road 32 over Buck Creek at Yorktown, Indiana.



BRIDGE CHARACTERISTICS

SKEW OF DECK (DEGREES): 0
WIDTH (FEET): 63.75
LENGTH (FEET): 2403
YEAR CONSTRUCTED: 1958

ABUTMENT CHARACTERISTICS

TYPE OF ABUTMENT: SPILL THROUGH
TYPE OF FOUNDATION: FOOTING ON PILES

NOTE: PILES NOT SHOWN TO SCALE

PIER CHARACTERISTICS

PIER	WIDTH (FEET)	LENGTH (FEET)	NOSE	FOUNDATION	SKEW (DEGREES)
1	4.7	42.5	ROUND	SPREAD FOOTING	0
2	4.7	42.5	ROUND	FOOTING ON PILES	0
3	4.7	42.5	ROUND	FOOTING ON PILES	0
4	4.7	42.5	ROUND	FOOTING ON PILES	0
5	4.7	42.5	ROUND	FOOTING ON PILES	0
6	4.7	42.5	ROUND	FOOTING ON PILES	0
7	4.7	42.5	ROUND	FOOTING ON PILES	0
8	4.7	42.5	ROUND	FOOTING ON PILES	0
9	4.7	42.5	ROUND	FOOTING ON PILES	0
10	4.7	42.5	ROUND	FOOTING ON PILES	0
11	4.7	42.5	ROUND	FOOTING ON PILES	0
12	4.7	42.5	ROUND	FOOTING ON PILES	0
13	4.7	42.5	ROUND	FOOTING ON PILES	0
14	4.7	42.5	ROUND	FOOTING ON PILES	0
15	4.7	42.5	ROUND	FOOTING ON PILES	0

EXPLANATION

--------- STUDY BED ELEVATION

———— CONSTRUCTION BED ELEVATION

EL 438.8 ELEVATION, IN FEET ABOVE SEA LEVEL-- Elevations below each pier refer to the bottom of that footing. Pier numbering is consistent with bridge plans

Figure 5. Characteristics of U.S. Route 41 over White River near Hazleton, Indiana.

Boring No 3

Station 515+17.75			
Offset 25' Lt			
Surface Elevation 390.0			
ELEVATION	DEPTH	NUMBER OF BLOWS	DESCRIPTION
390.0	0		Water Surface
	2		
	4		
	6		
	8		
	10		
	12		14' Soft sandy Clayey silt
376.0	14		
	16		5' very dense well graded sand trace of silt and fine gravel
371.0	18		
	20		
	22		
	24		
	26		42' Very dense well graded sand, trace of fine gravel
	28		
	30		
	32		
	34		
	36		
	38		
	40		
	42		
	44		
	46		
	48		
	50		
	52		
	54		
	56		
	58		
329.0	60		

Boring No 4

Station 516+73.50			
Offset 25' Rt			
Surface Elevation 380.0			
ELEVATION	DEPTH	NUMBER OF BLOWS	DESCRIPTION
390.0	0		Water Surface
	2		
	4		
	6		10' Water
	8		
380.0	10		
	12		
	14		
	16		10' Fine to coarse sand, traces of fine gravel, dense
	18		
370	20		
	22		
	24		
	26		21' Medium to coarse sand, traces of fine gravel, dense
	28		
	30		
	32		
	34		
	36		
	38		
349.0	40		

Figure 6(a). Soil-boring logs, U.S. Route 41 over White River near Hazleton, Indiana (taken from Indiana State Highway Commission, 1958, Bridge plans, sheet 10).

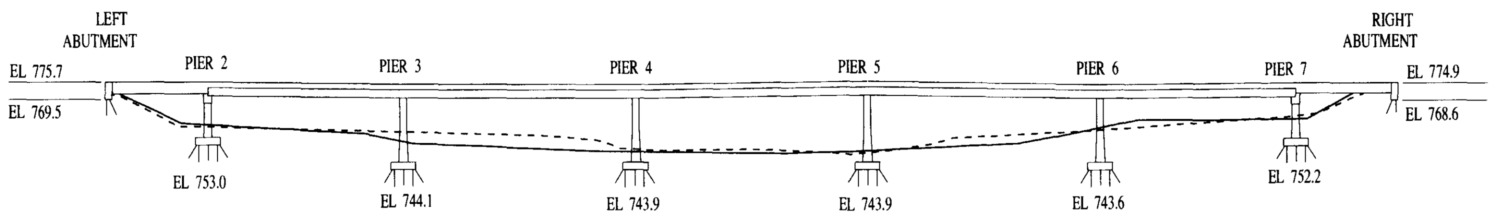
Boring No 5

Station 518+29.25			
Offset 25' Lt			
Surface Elevation 379.0			
ELEVATION	DEPTH	NUMBER OF BLOWS	DESCRIPTION
390.0	0		Water Surface
	2		
	4		
	6		
	8		11' Water
379.0	10		
	12		
	14		10' Fine to medium dense sand
	16		
	18		
369.0	20		
	22		
	24		
	26		
	28		
	30		
	32		
	34		
	36		31' Fine to medium sand, trace of fine gravel, dense
	38		
	40		
	42		
	44		
	46		
	48		
338.0	50		
	52		
	54		
	56		10' Blue shale very soft, decomposed
	58		
328	60		

Boring No 6

Station 519+85.00			
Offset 25' Rt			
Surface Elevation 389.0			
ELEVATION	DEPTH	NUMBER OF BLOWS	DESCRIPTION
390.0	0		Water Surface
			1' Water
387.0	2		2' Brn moist silt traces of clay
	4		
	6		
	8		8' Loose fine to med sand little silt
379.0	10		
	12		
	14		
	16		
	18		
	20		
	22		
	24		
	26		22' Well graded medium dense sand with traces of small gravel
	28		
	30		
357.0	32		
	34		
	36		
	38		
	40		
	42		
	44		
	46		
	48		
	50		
	52		
	54		29' Well graded dense sand
	56		
	58		
328.0	60		

Figure 6(b). Soil-boring logs, U.S. Route 41 over White River near Hazleton, Indiana (taken from Indiana State Highway Commission, 1958, Bridge plans, sheet 10).



EXPLANATION

- - - - - - - - STUDY BED ELEVATION

———— CONSTRUCTION BED ELEVATION

EL 743.9 ELEVATION, IN FEET ABOVE SEA LEVEL-- Elevations below each pier refer to the bottom of that footing. Pier numbering is consistent with bridge plans

BRIDGE CHARACTERISTICS

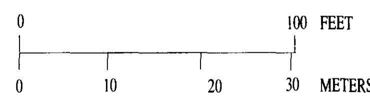
SKEW OF DECK (DEGREES): 0
WIDTH (FEET): 33
LENGTH (FEET): 462.8
YEAR CONSTRUCTED: 1959

PIER CHARACTERISTICS

PIER	WIDTH (FEET)	LENGTH (FEET)	NOSE	FOUNDATION	SKEW (DEGREES)
2	3.5	28.7	ROUND	FOOTING ON PILES	0
3	3.5	28.7	ROUND	FOOTING ON PILES	0
4	3.5	28.7	ROUND	FOOTING ON PILES	0
5	3.5	28.7	ROUND	FOOTING ON PILES	0
6	3.5	28.7	ROUND	FOOTING ON PILES	0
7	3.5	28.7	ROUND	FOOTING ON PILES	0

ABUTMENT CHARACTERISTICS

TYPE OF ABUTMENT: SPILL THROUGH
TYPE OF FOUNDATION: FOOTING ON PILES



NOTE: PILES NOT SHOWN TO SCALE

Figure 7. Characteristics of I-74 over Big Blue River near Shelbyville, Indiana.

Boring No 4

Station 373+65			
Offset 60' Lt			
Surface Elevation 752.4			
ELEVATION	DEPTH	NUMBER OF BLOWS	DESCRIPTION
765.0	0		Water Surface
	2		
	4		
	6		
	8		
	10		12.6' Water
752.4	12		
	14		4' Brown clay
748.4	16		
	18		
	20		
	22		10' Gravel and sand
	24		
738.4	26		
	28		
	30		7.9' Gray clay
	32		
730.5	34		

Rock

Boring No 5

Station 373+65			
Offset (on centerline)			
Surface Elevation 760.9			
ELEVATION	DEPTH	NUMBER OF BLOWS	DESCRIPTION
765.0	0		Water Surface
	2		4.1' Water
760.9	4		
	6		5' Brown clay
755.9	8		
	10		
	12		
	14		
	16		13' Gravel and sand
	18		
	20		
742.9	22		
	24		
	26		8.9' Gray clay
	28		
734.0	30		

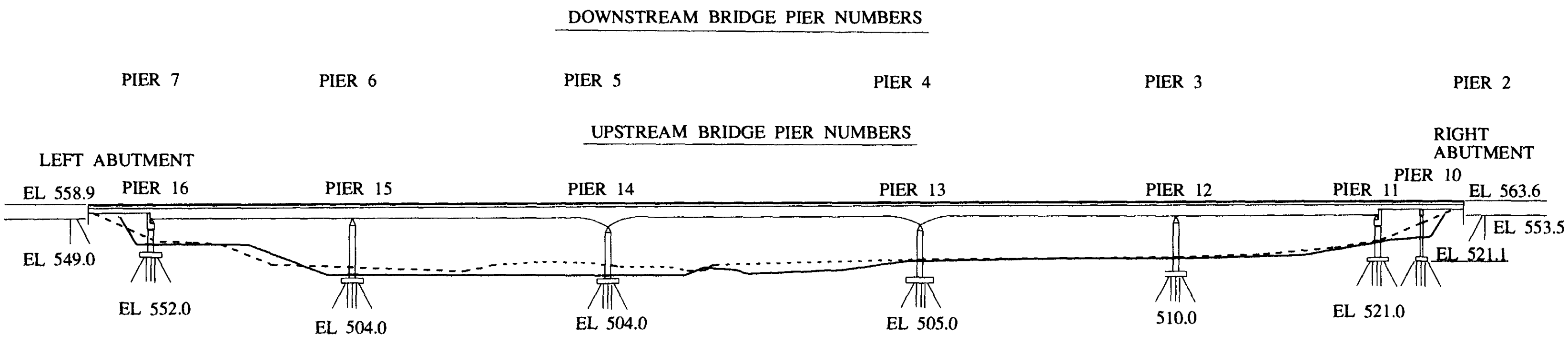
Rock

Boring No 6

Station 373+67			
Offset 60' Rt			
Surface Elevation 761.7			
ELEVATION	DEPTH	NUMBER OF BLOWS	DESCRIPTION
765.0	0		Water Surface
761.7	2		3.3' Water
	4		4.0' Brown clay
757.7	6		
	8		
	10		
	12		
	14		13' Gravel and sand
	16		
	18		
744.7	20		
	22		
	24		
	26		10.4' Gray clay
	28		
734.3	30		

Rock

Figure 8. Soil-boring logs, I-74 over Big Blue River near Shelbyville, Indiana (taken from Indiana State Highway Commission, 1958, Bridge plans, sheet 5).



EXPLANATION

STUDY BED ELEVATION

CONSTRUCTION BED ELEVATION

EL 505.0 ELEVATION, IN FEET ABOVE SEA LEVEL-- Elevations below each pier refer to the bottom of that footing. Pier numbering is consistent with bridge plans

BRIDGE CHARACTERISTICS

SKEW OF DECK (DEGREES): 0

WIDTH (FEET): 36.3

LENGTH (FEET): 968

YEAR CONSTRUCTED: 1962

PIER CHARACTERISTICS

PIER	WIDTH (FEET)	LENGTH (FEET)	NOSE	FOUNDATION	SKEW (DEGREES)
10	35.5	35.5	ROUND	FOOTING ON PILES	0
11	39.0	39.0	ROUND	FOOTING ON PILES	0
12	38.0	38.0	ROUND	FOOTING ON PILES	0
13	38.0	38.0	ROUND	FOOTING ON PILES	0
14	38.0	38.0	ROUND	FOOTING ON PILES	0
15	38.0	38.0	ROUND	FOOTING ON PILES	0
16	39.0	39.0	ROUND	FOOTING ON PILES	0

ABUTMENT CHARACTERISTICS

TYPE OF ABUTMENT: SPILL THROUGH

TYPE OF FOUNDATION: FOOTING ON PILES

0 100 200 300 400 FEET

0 30 60 90 METERS

NOTE: PILES NOT SHOWN TO SCALE

Figure 9. Characteristics of I-74 over Whitewater River near Harrison, Ohio.

and most high flow appears to enter the bridge opening between pier 14 on the upstream bridge and pier 4 on the downstream bridge.

The low-water control is a cobble riffle approximately 200 ft downstream from the downstream bridge. The high-water control is the channel. The channel slope measured from the USGS Harrison, Ohio-Indiana quadrangle map is 0.0020 ft/ft. The channel is fairly straight upstream from the bridge opening but bends gradually to the left downstream from the bridge. The banks are covered with trees and brush and appear to be stable. The left bank in the vicinity of the bridge is protected by large limestone blocks. On the basis of observation, the bed material is predominantly small cobbles with sand and gravel. Soil-boring logs from the highway plans are shown in figure 10.

Bridge 231-37-4980, U.S. Route 231 over Kankakee River near Hebron, Ind.

This study site (fig. 11), which drains 1,646 mi^2 (of which 201 mi^2 are noncontributing), is on the Porter and Jasper County line, approximately 110 mi northwest of Indianapolis (fig. 1). The study site is characterized by flat farmland. The basin is flat to gently rolling and is predominantly cultivated.

The channel is dredged and straight; spoil banks function as levees along both sides. Both banks are covered with trees and brush. The bridge spans from levee to levee; therefore, the bridge does not contract the flow. The channel slope measured from the USGS Kouts, Hebron, Demotte, and Shelby quadrangle maps is 0.00019 ft/ft. On the basis of observation, the bed material is predominantly sand; however, some construction debris (broken concrete with reinforcing steel) is visible along the right side of the upstream nose of pier 3 (fig. 11). Soil-boring logs from the highway plans are shown in figure 12.

Bridge 45-19-995D, U.S. Route 231 over East Fork White River near Haysville, Ind.

This study site (fig. 13), which drains 5,558 mi^2, is on the Dubois and Martin County line approximately 110 mi southwest of Indianapolis (fig. 1). The banks are steep, the flat flood plain is approximately 0.5 mi wide, and the surrounding topography is hilly. The banks and hills are mostly forested, and the flood plain is cultivated.

The basin drains the east-central part of Indiana, where the topography and land use range from rolling farmland to hilly forested areas. Monroe Reservoir, operated as a multipurpose facility for water supply, recreation, and flood control, is within the basin. Monroe Reservoir regulates the flow from 432 mi^2 of the drainage basin.

The channel curves to the right as it enters the bridge opening and flows through the opening at a 25° skew to the bridge and piers. Both banks are covered with trees and brush and appear to be stable. The channel slope measured from the USGS Rusk, Alfordsville, Jasper, Glendale, and Sandy Hook quadrangle maps is 0.000097 ft/ft. On the basis of observation, the bed material is sand to medium gravel. Soil-boring logs were not available for the main channel.

Bridge 258-36-4912, S.R. 258 over East Fork White River near Seymour, Ind.

This study site (fig. 14), which drains 2,347 mi^2, is in Jackson County, approximately 60 mi south of Indianapolis (fig. 1). The study site is characterized by rolling farmland and a flat flood plain approximately 5 mi wide. The basin drains the east-central part of Indiana, where topography and land use range from rolling farmland to hilly forested areas.

Within the valley, two relief bridges are in swales in the left overbank. In the right overbank, a relief bridge is in a swale, and the Indian Creek bridge also functions as a relief bridge. If the flow is high enough, water can escape through Beatty Walker Ditch and White

Boring No 10D

Station 40+92			
Offset 42' Lt			
Surface Elevation 511.5			
ELEVATION	DEPTH	NUMBER OF BLOWS	DESCRIPTION
535	0		Water Surface
	2		W
	4		a
	6		t
	8		e
	10		r
	12		23.5'
	14		
	16		W
	18		a
	20		t
	22		e
511.5			r
	24		
	26	18	
	28		
	30		10' Light brown fine to coarse sand to med. gravel
501.5	32	50	
	34		
	36		
	38	37	
	40		
	42	33	11' Light brown very fine sand
490.5	44		
	46	60	
	48		
	50	57	
	52		
	54		
	56	84	
	58		
	60		19' Light brown fine to coarse sand and fine to medium gravel with trace of silt
	62		
471.5		51	
	64		
	68		
	70	80	
	72		11' Light brown fine to coarse sand and fine to medium gravel, trace of silt
460.5	74	42	

Boring No 10E

Station 42+72			
Offset 42' Lt			
Surface Elevation 533.2			
ELEVATION	DEPTH	NUMBER OF BLOWS	DESCRIPTION
535	0		Water Surface
533.2			1.8' Water
	2		
531.2			2' Black organic top soil
	4		
	6	6	
	8		5.5' Brown moist soft sandy silt
525.7			
	10		
	12	26	
	14		10.5' Brown moist medium dense fine to coarse sand with some fine to medium gravel
	16		
	18	59	
515.2			
	20		
	22	19	
	24		
	26		9.0' Brown wet medium dense fine to medium gravel with some coarse sand
	28	26	
506.2			
	30		
	32	38	
	34		
	36		
	38	55	
	40		15' Brown wet fine to coarse sand with a trace of fine gravel
	42	61	
451.2			
	44		4.5' Brown wet dense fine to coarse sand with some fine gravel and trace of silt
	46	64	
486.7	48		

NOTE: Number of blows indicate blows required to drive a 2" O.D. split spoon sampler to a depth of 6" by a 140 lb. hammer falling 30 inches.

Figure 10(a). Soil-borings logs, I-74 over Whitewater River near Harrison, Ohio (taken from Indiana State Highway Commission, 1960, Bridge plans, sheet 11).

Boring No 10M

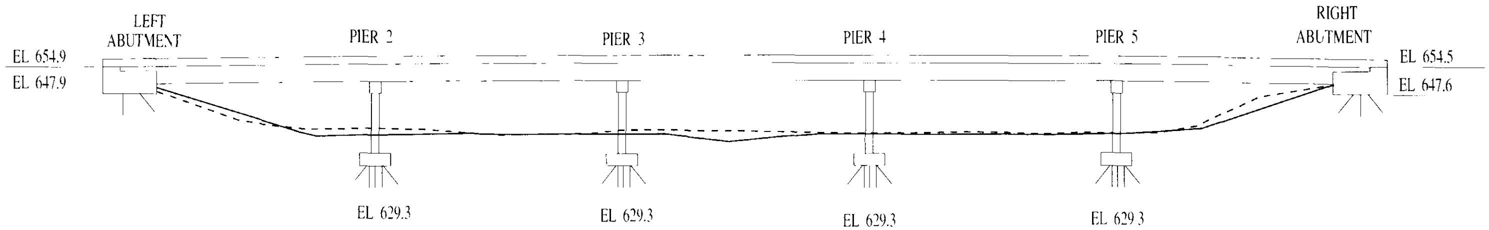
Station 42+72			
Offset 42' Rt			
Surface Elevation 533.6			
ELEVATION	DEPTH	NUMBER OF BLOWS	DESCRIPTION
535	0		Water Surface
533.6			1.4' Water
	2		
	4	3	
	6		7' Brown moist soft sandy silt
526.6	8		
	10	24	
	12		
	14		
	16		
	18	31	12' Brown moist medium dense fine to coarse sand and some fine to medium gravel
514.6	20		
	22	23	4' Brown wet medium dense fine to medium gravel and trace of coarse sand
510.6	24		
	26	25	
	28		
	30		
	32	33	
	34		
	36	60	
	38		
	40		
	42	40	21' Brown wet medium dense fine to coarse sand with a trace of fine gravel
489.6	44		
	46	50	
	48		
	50	51	
	52		
	54		
	56	74	17' Brown wet dense fine to coarse sand with some fine to medium gravel and a trace of silt
	58		
	60		
472.6	62	80	

NOTE: Number of blows indicate blows required to drive a 2" O.D. split spoon sampler to a depth of 6" by a 140 lb. hammer falling 30 inches.

Boring No 10L

Station 40+92			
Offset 42' Rt			
Surface Elevation 509.7			
ELEVATION	DEPTH	NUMBER OF BLOWS	DESCRIPTION
535.0	0		Water Surface
	2		
	4		Water
	6		
	8		
	10		
	12		25.3'
	14		
	16		Water
	18		
	20		
	22		
509.7	24		
	26	14	
	28		
	30		
502.7	32	27	7' Gray very fine sand
	34		
497.7	36	100/6"	5' Brown fine to coarse sand with little fine gravel
	38		
	40	57	
	42		
	44		
487.7	46	17	10' Brown fine to coarse sand with some fine to medium gravel
	48		
	50	70	
	52		
	54		
	56	126	
	58		
473.7	60	67	14' Light brown silty fine to coarse sand and fine to medium gravel
	62		
	64		
	66	69	
	68		
	70		
462.7	72	67	11' Light Gray silty fine to coarse sand

Figure 10(b). Soil-boring logs, I-74 over Whitewater River near Harrison, Ohio (taken from Indiana State Highway Commission, 1960, Bridge plans, sheet 11).



EXPLANATION

- - - - - - - - STUDY BED ELEVATION

———— CONSTRUCTION BED ELEVATION

EL 629.3 ELEVATION, IN FEET ABOVE SEA LEVEL-- Elevations below each pier refer to the bottom of that footing. Pier numbering is consistent with bridge plans

BRIDGE CHARACTERISTICS

SKEW OF DECK (DEGREES): 48
WIDTH (FEET): 46.5
LENGTH (FEET): 318.5
YEAR CONSTRUCTED: 1984

PIER CHARACTERISTICS

PIER	WIDTH (FEET)	LENGTH (FEET)	NOSE	FOUNDATION	SKEW (DEGREES)
2	2.0	49.0	ROUND	FOOTING ON PILES	0
3	2.0	49.0	ROUND	FOOTING ON PILES	0
4	2.0	49.0	ROUND	FOOTING ON PILES	0
5	2.0	49.0	ROUND	FOOTING ON PILES	0

ABUTMENT CHARACTERISTICS

TYPE OF ABUTMENT: SPILL THROUGH
TYPE OF FOUNDATION: FOOTING ON PILES

NOTE PILES NOT SHOWN TO SCALE

0 50 100 FEET

0 10 20 30 METERS

Figure 11. Characteristics of U.S. Route 231 over Kankakee River near Hebron, Indiana.

Boring No 2

Station 30+78.00 "A"

Offset 22' Rt

Surface Elevation 636.73

ELEVATION	DEPTH	NUMBER OF BLOWS	DESCRIPTION
641.5	0		Water Surface
	2		
636.73	4		Water
634.13	6	3-2-1	3.0' Brown wet very loose sand with little silty clay
	8		
631.73		2-2-3	3.0' Gray very loose fine sand with trace gravel
	10		
629.23	12	6-9-11	2.5' Brown & Gray wet loose to med. sand
626.73	14	8-10-12	
	16		4.8' gray wet med dense fine sand
	18		
621.73		10-13-13	
	20		14.5' Gray moist med dense silt
	22		
	24		
616.73		4-8-14	
	26		
	28		
611.73		11-9-12	
	30		14.5' Gray moist very stiff silty loam
	32		
	34		
606.73		7-8-11	
	36		
	38		1.7' Dk gray moist hd sandy clay loam w/little gravel
602.43		51-31-8.4	
600.93	40	51-0 06	
	42		
	44		
	46		IRC: from 43.3' to 48.3' RQD=50%
593.23	48		

NOTE: Number of blows indicate blows required to drive a 2" O.D. split spoon sampler to a depth of 6" by a 140 lb. hammer falling 30 inches. First figure represents blow counts thru disturbed soil and is not to be used.

The standard penetration test results can be obtained by adding the last two figures (i.e. 5/5=10 blows per foot)

Boring No 3

Station 30+94"A"

Offset 22' Lt.

Surface Elevation 636.93'

ELEVATION	DEPTH	NUMBER OF BLOWS	DESCRIPTION
641.5	0		Water Surface
	2		
636.93	4		Water
	6		4.5' Dark brown wet very loose sand.
633.43	8	3-3-1	
630.93	10	9-7-10	
628.43	12	8-10-11	
	14		
625.93		4-10-7	
	16		
	18		
620.93	20	15-15-18	14' Brown & gray wet med dense fine sand with trace gravel
	22		
	24		
615.93		5-9-10	4.5' Gray moist medium dense silt
	26		
	28		
610.93	30	7-7-10	5.0' Gray moist very stiff silty loam
	32		
605.93	34	4-13-14	3.7' Gray moist very stiff sandy clay loam with trace shale fragments
	36		
602.23	38	51-02	3.0' Dark brown slightly moist medium hard shale with pyrite nodules

Figure 12(a). Soil-borings logs, U.S. Route 231 over Kankakee River near Hebron, Indiana (taken from Indiana State Highway Commission, 1982, Bridge Plans, sheet 8).

Boring No 4

Station 32+06"A"			
Offset 28' Rt.			
Surface Elevation 637.12			
ELEVATION	**DEPTH**	**NUMBER OF BLOWS**	**DESCRIPTION**
641.5	0		Water Surface
	2		
637.12	4		Water
	6		
633.62	8	1-1-2	
631.12	10	4-6-5	6.3' Dark brown wet very loose to med. dense sand
628.62	12	7-6-10	
626.12	14	16-19-23	
	16		
	18		7.5' Gray wet to med. dense to dense fine sand
621.12	20	10-13-10	1.5' Gray wet med dense sand & gravel
	22		3.5' Gray moist med dense silt
616.12	24	26-38-40	
	26		
	28		
611.12	30	8-11-16	9.0' Gray wet very dense to med. dense fine sand with some silt
	32		
606.12	34		4.0' Gray very moist very stiff silty loam with trace gravel & trace shale fragments
	36	22-61-01	
	38		
602.02			3.3' Brown moist med. hard brittle shale

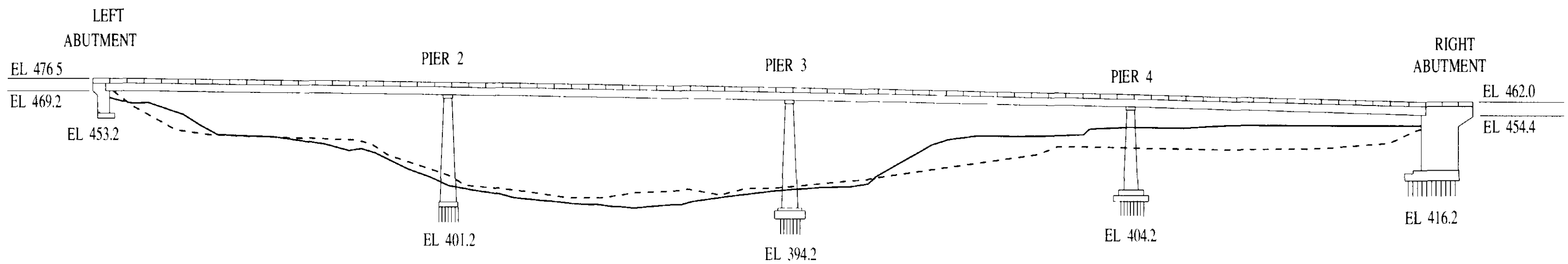
Boring No 5

Station 32+20"A"			
Offset 22' Lt.			
Surface Elevation 637.37			
ELEVATION	**DEPTH**	**NUMBER OF BLOWS**	**DESCRIPTION**
641.5	0		Water Surface
	2		
637.37	4		Water
	6		
632.87	8	4-5-5	5.1' Gray wet loose fine sand with little silt
630.37	10	8-9-7	
627.87	12	8-8-12	
	14		
625.37	16	4-11-13	
	18		9.5' Gray moist dense fine sand with trace gravel
620.37	20	12-22-19	
	22		4.5' Gray moist med dense silt
	24		
615.37	26	34-41-44	
	28		
610.37	30	17-31-40	7.7' Brownish gray very dense fine sand with some silt
	32		2.3' Gray moist dense silt with trace shale fragments
	34		2.8' Dark gray moist hard silty loam with trace shale fragments
605.37	36	15-23	
	38		4.1' Dark brown moist med. hard brittle weathered shale
601.37	40		

NOTE: Number of blows indicate blows required to drive a 2" O.D. split spoon sampler to a depth of 6" by a 140 lb. hammer falling 30 inches. First figure represents blow counts thru disturbed soil and is not to be used.

The standard penetration test results can be obtained by adding the last two figures (i.e. 5/5=10 blows per foot)

Figure 12(b). Soil-borings logs, U.S. Route 231 over Kankakee River near Hebron, Indiana (taken from Indiana State Highway Commission, 1982, Bridge Plans, sheet 8).



EXPLANATION

- - - - - - STUDY BED ELEVATION

——— CONSTRUCTION BED ELEVATION

EL 394.2 ELEVATION, IN FEET ABOVE SEA LEVEL-- Elevations below each pier refer to the bottom of that footing. Pier numbering is consistent with bridge plans

BRIDGE CHARACTERISTICS

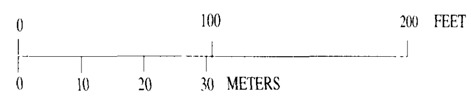
SKEW OF (DEGREES): 25
WIDTH (FEET): 40
LENGTH (FEET): 800
YEAR CONSTRUCTED: 1932

PIER CHARACTERISTICS

PIER	WIDTH (FEET)	LENGTH (FEET)	NOSE	FOUNDATION	SKEW (DEGREES)
2	4.5 TO 9.5	41.0	ROUND	FOOTING ON PILES	25
3	4.5 TO 9.5	41.7	ROUND	FOOTING ON PILES	25
4	4.5 TO 8.3	40.5	ROUND	FOOTING ON PILES	25

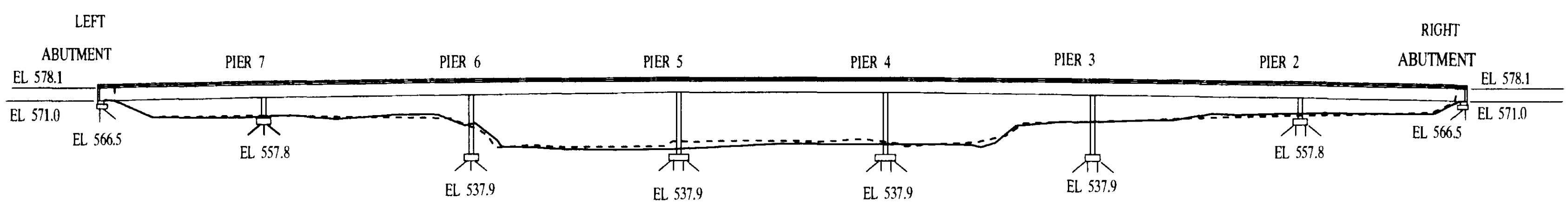
ABUTMENT CHARACTERISTICS

TYPE OF ABUTMENT: SPILL THROUGH
TYPE OF FOUNDATION: FOOTING ON PILES
Except the left abutment which is a spread footing



NOTE: PILES NOT SHOWN TO SCALE

Figure 13. Characteristics of U.S. Route 231 over East Fork White River near Haysville, Indiana.



EXPLANATION

- - - - - - - - - - - STUDY BED ELEVATION

———————— CONSTRUCTION BED ELEVATION

EL 537.9 ELEVATION, IN FEET ABOVE SEA LEVEL-- Elevations below each pier refer to the bottom of that footing. Pier numbering is consistent with bridge plans

BRIDGE CHARACTERISTICS

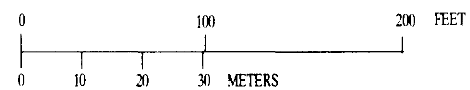
SKEW OF DECK (DEGREES): 15
WIDTH (FEET): 42.5
LENGTH (FEET): 755
YEAR CONSTRUCTED: 1970

PIER CHARACTERISTICS

PIER	WIDTH (FEET)	LENGTH (FEET)	NOSE	FOUNDATION	SKEW (DEGREES)
2	2.5	24.5	ROUND	FOOTING ON PILES	0
3	2.5	24.5	ROUND	FOOTING ON PILES	0
4	2.5	24.5	ROUND	FOOTING ON PILES	0
5	2.5	24.5	ROUND	FOOTING ON PILES	0
6	2.5	24.5	ROUND	FOOTING ON PILES	0
7	2.5	24.5	ROUND	FOOTING ON PILES	0

ABUTMENT CHARACTERISTICS

TYPE OF ABUTMENT: SPILL THROUGH
TYPE OF FOUNDATION: FOOTING ON PILES



NOTE: PILES NOT SHOWN TO SCALE

Figure 14. Characteristics of State Road 258 over East Fork White River near Seymour, Indiana.

Creek as well. A historical covered bridge 200 ft upstream is maintained as a point of interest. Upstream from this covered bridge, an abandoned railroad fill crosses the left overbank 500 to 2,000 ft upstream from S.R. 258. This fill has four openings that allow floodwaters to pass through this obstruction. Most of the railroad fill on the right bank has been removed.

The banks downstream from the study bridge and upstream between the study bridge and the covered bridge are steep and partly bare of vegetation; however, these banks appear to be fairly stable. The banks upstream from the covered bridge are eroding and unstable. The erosion of the left bank may endanger the left abutment of the covered bridge. A gravel bar has formed around and to the left of pier 5 of the study bridge. This bar formation appears to be the effect of a large pier under the covered bridge. The channel slope measured from the USGS Jonesville and Seymour quadrangle maps is 0.00038 ft/ft. On the basis of observation, the bed material is sand and gravel. Soil-boring logs taken from the highway plans are shown in figure 15.

Bridge 331-50-6627, S.R. 331 over Tippecanoe River at Old Tip Town, Ind.

This study site (fig. 16), which drains 389 mi^2, is in Marshall County, approximately 100 mi north of Indianapolis (fig. 1). The topography is rolling to hilly. Wooded areas parallel both banks, and the flood plain is narrow and wooded. The surrounding area is predominantly cultivated. The basin is rolling to hilly and contains numerous small lakes; land use is predominantly farmland.

The channel is fairly straight immediately upstream and downstream from the bridge, but approximately 1,000 ft downstream the channel turns sharply to the left. Both banks are vegetated and stable. The channel slope measured from the USGS Mentone and Argos quadrangle maps is 0.00021 ft/ft. On the basis of observation, the bed material is sand to coarse gravel. Soil-boring logs from the highway plans are shown in figure 17.

Bridge 358-42-6779, S.R. 358 over White River near Edwardsport, Ind.

This study site (fig. 18), which drains 5,013 mi^2, is on the Knox and Daviess County line, approximately 90 mi southwest of Indianapolis (fig. 1). Rolling topography flanks a flat valley approximately 5 mi wide.

The basin drains central to west-central Indiana, where the topography and land use range from rolling farmland to hilly forested areas. Several metropolitan areas are within the basin, including Indianapolis, Anderson, and Muncie. Cagles Mill Reservoir, operated for flood control, drains 293 mi^2 of the total basin area.

The channel banks are forested and appear to be stable. The channel is straight immediately upstream and downstream from the bridge, and flow through the bridge is parallel to the piers. The flow at this site is confined by levees on both banks. The levee on the right begins at S.R. 358 and parallels the channel until it ends approximately 0.6 mi downstream. The levee on the left bank crosses S.R. 358 approximately 3,000 ft left of the channel and converges to a point 200 ft left of the channel approximately 0.6 mi downstream from the bridge. S.R. 358 blocks the flow on the right bank; however, the left overbank is at an elevation close to that of the valley floor between the bridge and the levee, thus allowing floodwater to flow across the road. The channel slope measured from the USGS Plainville, Bicknell, Washington, and Wheatland quadrangle maps is 0.00023 ft/ft. On the basis of observations, the bed material is sand to medium gravel. Soil-boring logs from the highway plans are shown in figure 19.

HISTORICAL SCOUR AROUND BRIDGE PIERS AND ABUTMENTS

Each bridge opening was surveyed with the ground-penetrating-radar system (GPR) and/or a tuned transducer to locate evidence of scour holes that may have refilled. The GPR was used with dual 80-megahertz antennae that transmit electromagnetic pulses into the subsurface. Ideally, this energy would be reflected from subsurface interfaces where electrical properties differ. The GPR technique was successful on the gravel bars and in water less than 4 ft deep. In

Boring No T.B.#4

Station 544+90
Offset 15' Lt
Surface Elevation 550.8

ELEVATION	DEPTH	NUMBER OF BLOWS	DESCRIPTION
555	0		Water Surface
	2		
550.8	4		4.2' Water
549.8			1' Brown moist loose fine to coarse sand with a trace of fine gravel
	6		
	8		
	10	4/6/5	8' Brown moist to wet medium dense fine to coarse sand with some fine to medium gravel
541.8	12		
	14	1/3/4	
	16		
	18	3/3/4	9' Brown and gray wet loose fine to coarse sand with some fine gravel
	20		
532.8	22		
	24	4/3/1	5' Gray wet very loose fine to coarse sand with a trace of fine gravel
527.8	26		
	28	4/4/4	
	30		5' Gray wet loose fine to medium sand
522.8	32		
	34	6/6/17	5' Gray wet medium dense fine to coarse sand with some fine gravel
517.8	36		
	38	6/9/13	5' Gray wet medium dense fine to coarse sand with a trace of fine gravel
	40		
512.8	42		
510.8	44	5/12/18	2' Gray wet medium dense fine to medium sand
507.8	46		3' Gray wet medium dense fine to coarse sand with a trace of fine gravel
	48	10/11/10	
	50		
	52	12/14/8	7.5' Gray wet medium dense fine to coarse sand with a little fine to medium gravel
500.3	54		

Boring No T.B.#5

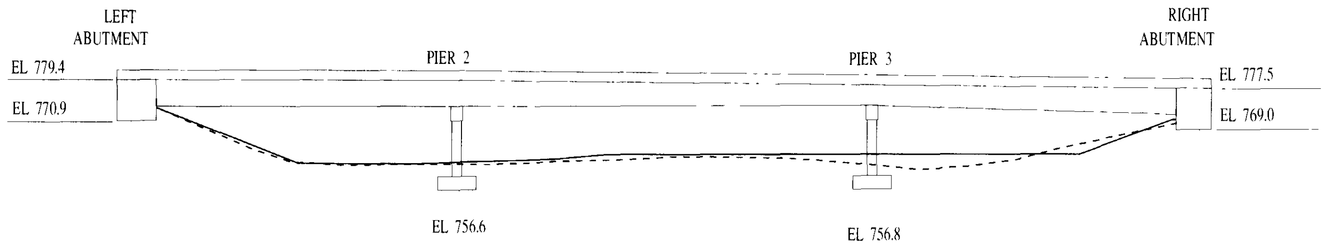
Station 546+09
Offset 3' Rt
Surface Elevation 547.0

ELEVATION	DEPTH	NUMBER OF BLOWS	DESCRIPTION
555.0	0		Water Surface
	2		
	4		
	6		
547.0	8		8' Water
	10	1/2/2	
	12		
	14	4/3/3	
	16		
	18	1/3/5	
	20		17' Brown wet very loose fine to coarse sand with some fine to medium gravel
	22	3/4/4	
530.0	24		
	26		
	28	9/17/20	5' Gray wet dense fine sand
525.0	30		
	32	6/10/13	
	34		9' Gray wet medium dense fine to medium sand with a trace of fine to medium gravel
	36	3/6/8	
516.0	38		
	40		4.5' Gray wet medium dense fine to coarse sand with some fine to medium gravel
511.5	42	11/13/13	

NOTE: Number of blows indicate blows required to drive a 2" O.D. split spoon sampler to a depth of 6" by a 140 lb. hammer falling 30 inches. First figure represents blow counts thru disturbed soil and is not to be used.

The standard penetration test results can be obtained by adding the last two figures (i.e. 5/5=10 blows per foot)

Figure 15. Soil-boring logs, State Road 258 over East Fork White River near Seymour, Indiana (taken from Indiana State Highway Commission, 1969, Bridge plans, sheets 8-9).



EXPLANATION

- - - - - - - - - - STUDY BED ELEVATION

———————— CONSTRUCTION BED ELEVATION

EL 756.8 ELEVATION, IN FEET ABOVE SEA LEVEL-- Elevations below each pier refer to the bottom of that footing. Pier numbering is consistent with bridge plans

BRIDGE CHARACTERISTICS

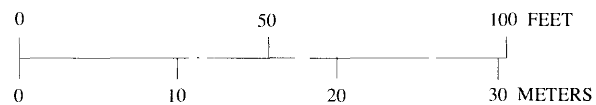
SKEW OF DECK (DEGREES): 41.75
WIDTH (FEET): 46.5
LENGTH (FEET): 223.5
YEAR CONSTRUCTED: 1986

PIER CHARACTERISTICS

PIER	WIDTH (FEET)	LENGTH (FEET)	NOSE	FOUNDATION	SKEW (DEGREES)
2	2	42	ROUND	FOOTING ON PILES	0
3	2	42	ROUND	FOOTING ON PILES	0

ABUTMENT CHARACTERISTICS

TYPE OF ABUTMENT: SPILL THROUGH
TYPE OF FOUNDATION: FOOTING ON PILES



NOTE: PILES NOT SHOWN TO SCALE

Figure 16. Characteristics of State Road 331 over Tippecanoe River at Old Tip Town, Indiana.

Boring No 2

Station 169+10"A"			
Offset 5.5' Rt			
Surface Elevation 762.1			
ELEVATION	DEPTH	NUMBER OF BLOWS	DESCRIPTION
765.0	0		Water Surface
762.1	2		Water
760.6	4	5/3/3	2.7' Brown wet loose sand and gravel. Test No.3 A-1-A
	6		
756.9	8	8/8/7	
755.4	10	8/11/16	4.9' Gray wet medium dense sand test No.2A-3
753.9		9/12/15	
752.4	12	10/10/15	Gray wet medium dense sand and gravel.Test No.3 A-1-A 1.6'
750.9	14	11/11/13	
749.4		8/13/14	
747.9	16	9/11/17	
	18		
	20		
741.9	22	8/10/11	
	24		
	26		
736.9	28	8/18/10	
	30		
731.9	32	5/7/11	20.5' Gray wet medium dense sand with gravel seams Test No.2 A-3
	34		
	36		
726.9	38	12/14/19	
	40		
721.9	42	28/17/14	14' Gray brown dense to medium dense gravelly sand (visual)
	44		
	46		
716.9	48	7/9/13	

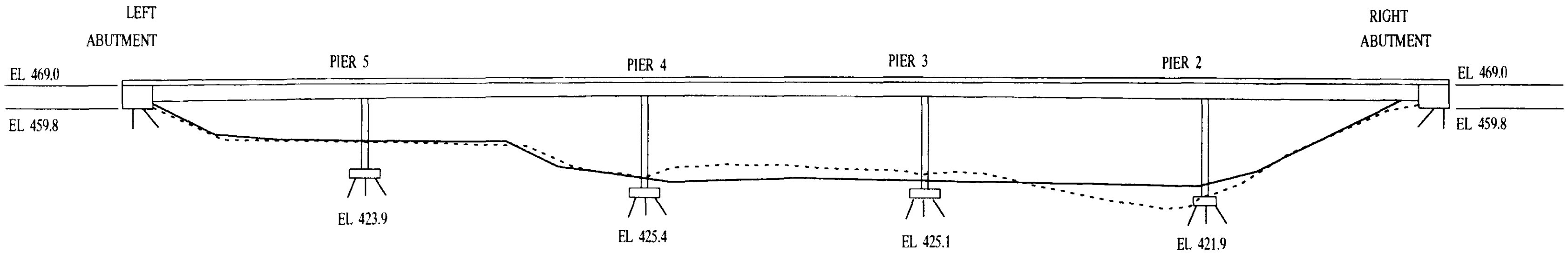
Boring No 3

Station 169+93"A"			
Offset 6' Lt			
Surface Elevation 763.6			
ELEVATION	DEPTH	NUMBER OF BLOWS	DESCRIPTION
765.0	0		Water Surface
763.6		4/4/3	Water
	2		0.5'Brn wet loose Grv.sand 0.6'Brn/blk wet loose sand
	4		
	6		
756.8	8	4/5/6	9.5' Gray wet medium dense sand. Test No.5 A-3
755.3		5/7/9	
753.8	10	4/6/7	
752.3	12	7/8/9	.9'Gray brown wet med.dense sand trace organic material (visual)
750.8	14	7/10/11	
749.3		6/8/12	
	16		
746.8	18	13/11/15	14.7' Gray wet medium dense sand Test No.5 A-3
	20		
741.8	22	6/10/13	
	24		
	26		
736.8	28	7/12/12	
	30		
731.8	32	8/8/9	9.9' Gray wet medium dense sandy gravel Test No. 4 A-1-A
	34		
	36		
726.8	38	8/11/16	
	40		
721.8	42	26/37/41	6.7' Brown wet very dense sand Test No.5 A-3
	44		
	46		
716.8	48	15/10/11	4' Brown wet very dense sand test No.5 A-3

NOTE: Number of blows indicate blows required to drive a 2" O.D. split spoon sampler to a depth of 6" by a 140 lb. hammer falling 30 inches. First figure represents blow counts thru disturbed soil and is not to be used.

The standard penetration test results can be obtained by adding the last two figures (i.e. 5/5=10 blows per foot)

Figure 17. Soil-boring logs, State Road 331 over Tippecanoe River at old Tip Town, Indiana (taken from Indiana State Highway Commission, 1983, Bridge plans, sheet 9).



EXPLANATION

- - - - - - - - STUDY BED ELEVATION

——— CONSTRUCTION BED ELEVATION

EL 425.1 ELEVATION, IN FEET ABOVE SEA LEVEL-- Elevations below each pier refer to the bottom of that footing. Pier numbering is consistent with bridge plans

BRIDGE CHARACTERISTICS

SKEW OF DECK (DEGREES): 0
WIDTH (FEET): 35
LENGTH (FEET): 499.5
YEAR CONSTRUCTED: 1985

PIER CHARACTERISTICS

PIER	WIDTH (FEET)	LENGTH (FEET)	NOSE	FOUNDATION	SKEW (DEGREES)
2	2.5	18.5	ROUND	FOOTING ON PILES	0
3	2.5	18.5	ROUND	FOOTING ON PILES	0
4	2.5	18.5	ROUND	FOOTING ON PILES	0
5	2.5	18.5	ROUND	FOOTING ON PILES	0

ABUTMENT CHARACTERISTICS

TYPE OF ABUTMENT: SPILL THROUGH
TYPE OF FOUNDATION: FOOTING ON PILES

NOTE: PILES NOT SHOWN TO SCALE

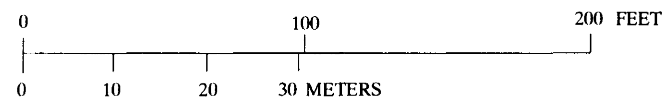


Figure 18. Characteristics of State Road 358 over White River near Edwardsport, Indiana.

Boring No 2

Station 19+47.00
Offset 17' Rt Line "D"
Surface Elevation 433.8

ELEVATION	DEPTH	NUMBER OF BLOWS	DESCRIPTION
438.3	0		Water Surface
	2		
433.8	4		4.5' Water
	6		
	8	1 1/2-1/2-1	5.0' Brown wet very loose sand
428.8	10		
426.3	12	2-3-3	2.5' Brown wet loose sand and gravel
	14	5-6-8	5.0' Grayish brown wet med. dense to loose sand with trace organics
421.3	16	4-4-5	
	18	6-5-4	13.5' Brown wet loose to med. dense sand w/ trace gravel
	20		
	22	5-4-6	
	24	8-7-10	
	26		
	28	8-8-11	
407.8	30	9-10	
406.8		41	1' Gray soft siltstone
	32		
	34		
401.8	36		5.0' Gray to dark gray sandy siltstone

Downstream Nose Pier 2

NOTE: Number of blows indicate blows required to drive a 2" O.D. split spoon sampler to a depth of 6" by a 140 lb. hammer falling 30 inches. First figure represents blow counts thru disturbed soil and is not to be used.
The standard penetration test results can be obtained by adding the last two figures (i.e. 5/5=10 blows per foot)

Boring No 3

Station 20+55.50
Offset 17' Lt Line "D"
Surface Elevation 432.3'

ELEVATION	DEPTH	NUMBER OF BLOWS	DESCRIPTION
438.3	0		Water Surface
	2		
	4		
432.3	6		6.0' Water
	8	WT.Tools 1/2-1/2-1 2-3-2	7.5' Dark brown wet very loose sand
	10		
	12		
424.8	14		
	16	3-4-5 6-9-14	5.0' Brown and gray wet loose to med. dense sand
419.8	18		
	20	8-14-8	
	22	10-13-17	8.0' Brown and gray wet dense to med. dense sand
	24	9-11-14	
411.8	26	8-13-16	
	28		19.5' Gray wet medium dense, fine sand
	30	9-11-13	
	32		
	34		
	36		
	38	14-11-12	
	40		
	42		
	44	10-13-16	
392.3	46		
390.8	48	11-17-15	1.5' Gray wet dense sand

Upstream Nose Pier 3

Boring No 4

Station 21+64.00
Offset 17' Rt Line "D"
Surface Elevation 432.3'

ELEVATION	DEPTH	NUMBER OF BLOWS	DESCRIPTION
438.3	0		Water Surface
	2		6.0' Water
	4		
432.3	6		
429.8	8	1	2.5' Gray wet very loose fine sand
	10	2-1-2	5.0' Gray and brown wet very loose to loose sand with trace gravel
424.8	12	2-3-3	
	14		
	16	7-9-9	22.5' Gray wet medium dense fine sand
	18	6-8-7	
	20	8-10-13	
	22	9-8-12	
	24		
	26	10-12-14	
	28	8-13-15	
	30		
	32		
	34	9-11-14	
402.3	36		
400.8	38	11-13-16	1.5' Gray wet medium dense fine sand with trace organics

Downstream Nose Pier 4

Note: Number of blows required to drive a 2" O.D. split spoon sampler 6" with a 140 lb. weight falling 30"

Figure 19. Soil-boring logs, State Road 358 over White River near Edwardsport, Indiana (taken from Indiana State Highway Commission, 1982, bridge plans, sheet 9).

water depths greater than 4 ft, however, the signal was rapidly attenuated in the water column because of high specific conductance of the water, and no useful data were recorded. The data sometimes contained interference from debris, side echo, and point reflections from cobbles and boulders. Furthermore, the dual-80 megahertz antennae are not shielded on top, and a reflection from the bridge deck may have caused interference in some of the record.

The tuned transducer was used with a 3.5- to 7-kilohertz (kHz) and a 14-kHz transducer to send and receive an acoustic signal. The acoustic signal is reflected from subsurface interfaces where acoustic impedances change. The transducer was suspended 6 to 12 in. below the water surface. This equipment was usable in water deeper than 5 ft. The data were sometimes obscured by the effects of side echo, debris, point reflections from cobbles and boulders, and multiple reflections.

The surveys were done in the main-channel part of the bridge opening and around each pier. In shallow channels, investigators maneuvered the equipment around the piers and across the channel by wading. At locations too deep to wade, the antennae or the transducer was attached to a 16-ft flat-bottom boat and maneuvered around the piers and across the channel. Sections were recorded across the upstream and downstream side of the bridge, along each side of each main-channel pier, and along the upstream and downstream end of each main-channel pier. The piers on the overbanks were not surveyed. To support the geophysical data, investigators probed the area around each surveyed pier with a steel pipe (0.5 in. inside diameter) to locate subsurface interfaces. From the 10 sites surveyed, 9 produced results adequate for interpretation.

The data were assessed to identify any subsurface interface that would indicate that the bed had scoured at some time in the past and subsequently refilled. Because GPR and tuned transducer record indicate interfaces where the electrical and acoustic properties change, correct interpretation of the record is critical to ensure that construction fill or other changes in subbottom material is not interpreted as scour. The data were adequate to determine the approximate location and depth of the interface; however, the data were not of sufficient resolution for mapping the lateral extent of refilled scour holes.

These data were interpreted by Robert L. Miller (USGS, Indianapolis, Ind.) in consultation with F. Peter Haeni (USGS, Hartford, Conn.), and Kenneth J. Hollett (USGS, Reston, Va.). Other interpretations of these data may be possible. Historical scour at the sites is discussed in the following sections.

U.S. Route 24 over Tippecanoe River at Monticello, Ind.

Because the depth of water at this site prevented use of the GPR, the main channel in the bridge opening was surveyed by use of the tuned transducer with a signal frequency of 7 kHz. Piers 3 and 4 (fig. 2) were surveyed, and the record indicates a cobble-and-boulder layer beneath a veneer of sand and gravel. Interpretation of the record did not give evidence that the veneer penetrated into the coarse layer; therefore, it is estimated that this site has not scoured below an elevation of about 601 ft. The probe penetrated only a few inches, thereby supporting the results of the geophysical survey.

S.R. 32 over Buck Creek at Yorktown, Ind.

The bridge opening was surveyed by use of the GPR; however, the data were not adequate for interpretation. The signal was affected by scattered boulders and debris and by the antennae being jostled as they were moved over and around the boulders. The water was too shallow to use the tuned transducer.

The area around piers 2 and 3 (fig. 4) was probed. Penetration around pier 2 ranged from 0 to 2.7 ft. The lowest elevation reached was 885.8 ft at a point 2.5 ft upstream and 3.5 ft to the right of the downstream end of the pier. Penetration around pier 3 ranged from 0 to 3.0 ft. The lowest elevation found by probing was 886.1 ft at a point 3 ft upstream from the upstream end of the pier. Results of the probing indicate that scour may have occurred at an elevation below the elevation of the bottom of the footing.

U.S. Route 41 over White River near Hazleton, Ind.

The main channel in the bridge opening was surveyed by use of the tuned transducer with a signal frequency of 7 kHz. Piers 5, 6, and 7 (fig. 5) are within this area. Because of limited penetration, the observations were restricted to elevations above 370 ft, which is above the bottom of the footing. Nevertheless, an interface was detected at several locations between elevations of 373 and 378 ft.

The deepest probe penetration in the vicinity of pier 5 was to an elevation of 369.4 ft at a point 12 ft right of the center of the pier. In the vicinity of pier 6, the probe penetrated to an elevation of 373.4 ft at a point 4 ft left of the center of the pier. In the vicinity of pier 7 the probe penetrated to an elevation of 371.8 ft at a point 1 ft and 2 ft left of the upstream end and 5 ft left of the center of the pier. Therefore, scour below the footings was not detected at this site.

I-74 over Big Blue River near Shelbyville, Ind.

The main channel in the bridge opening was surveyed by use of the GPR because shallow water prevented the use of the tuned transducer. Piers 4 and 5 (fig. 7) on both the downstream bridge (eastbound lane) and the upstream bridge (westbound lane) were surveyed. An interface was detected around all four piers between elevations of 738 to 744 ft. The interface appears to penetrate a layer of gravel and cobble and terminate in a clay layer as shown by the soil-boring logs (fig. 8). This interface is interpreted to be the result of scour and fill, on the basis of its shape and location (fig. 20). This indicates that scouring has occurred below the footings in the vicinity of piers 4 and 5 on both bridges; however, the data also indicate that the clay layer has not been significantly penetrated by scour. The bed material was too coarse to allow probe penetration.

I-74 over Whitewater River near Harrison, Oh.

The main channel in the bridge opening was surveyed by use of the GPR and the tuned transducer. The GPR survey around piers 6 and 15 (fig. 9) did not produce any conclusive results because the water depth and signal attenuation did not allow sufficient penetration. The surveys around piers 5 and 14 were done on a gravel bar. At a location 25 ft downstream from the end of pier 5, an interface was detected at an elevation of 503 ft. A deeper interface may exist, but the signal was too weak to allow for confident interpretation.

Subsurface interfaces were detected around piers 6 and 15 by use of the tuned transducer set to a signal frequency of 7 kHz, indicating the possibility of deep scour (fig. 21). Because of shallow water and cobble-sized bed material, the record has multiple reflections that mask the subbottom. Side echoes from the piers and stone protection along the left bank and distortion from debris also made the interpretation of this record difficult.

The deepest interface detected is at elevation 490 ft at a point 20 ft upstream from pier 15. Interfaces near pier 15 also were detected at elevation 492 ft, 15 ft upstream; at elevation 492 ft, 3 ft right of the upstream end; and at elevation 500 ft, downstream from the pier. At 12 ft upstream from pier 6, an interface was detected at an elevation of 497 ft. At 15 ft downstream and 6 ft left of pier 6, the record indicates an interface below an elevation of 495 ft; however, the record did not delineate this interface to its lowest elevation. The shapes and locations of these interfaces are consistent with patterns expected from local scour around piers.

Maximum probe penetration, 8.8 ft, resulted in a minimum elevation of 499.6 ft at a point 17 ft upstream from pier 15. Coarse bed material limited most probe penetration to less than 1 ft. The bed material restricted the probing around piers 5 and 14 to the upstream end of pier 14 and the downstream end of pier 5. The probe penetrated to an elevation of 509.2 ft at the downstream end of pier 5. Comparison of the streambed elevation from the construction plans to elevations surveyed for this study indicates that 6 ft of aggradation has occurred.

The data at this site indicate scour has removed bed material upstream from pier 15 to a level several feet below the bottom of the

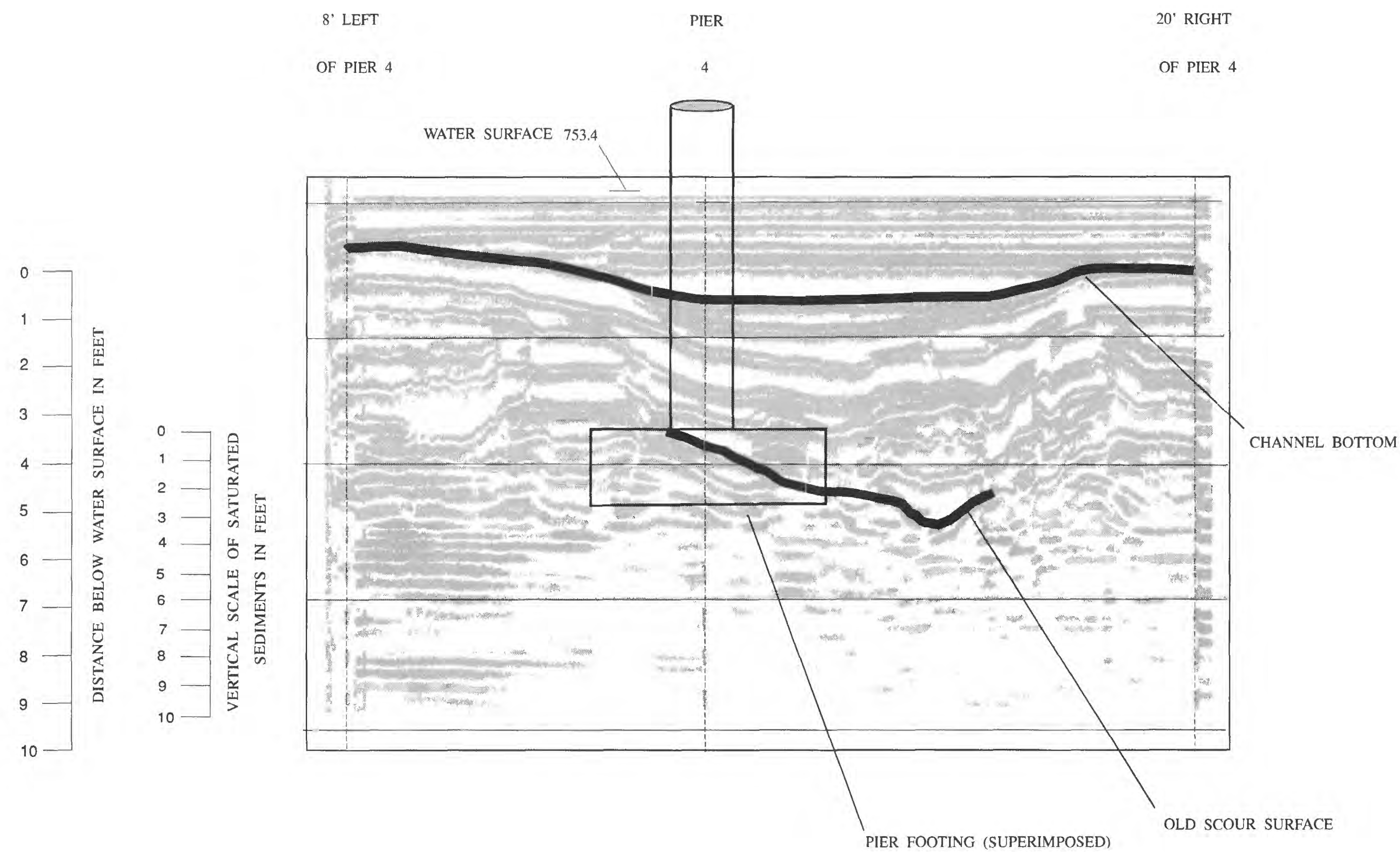


Figure 20. Ground-penetrating-radar cross section at I-74 over Big Blue River near Shelbyville, Indiana: left to right 2.5 ft downstream of pier 4.

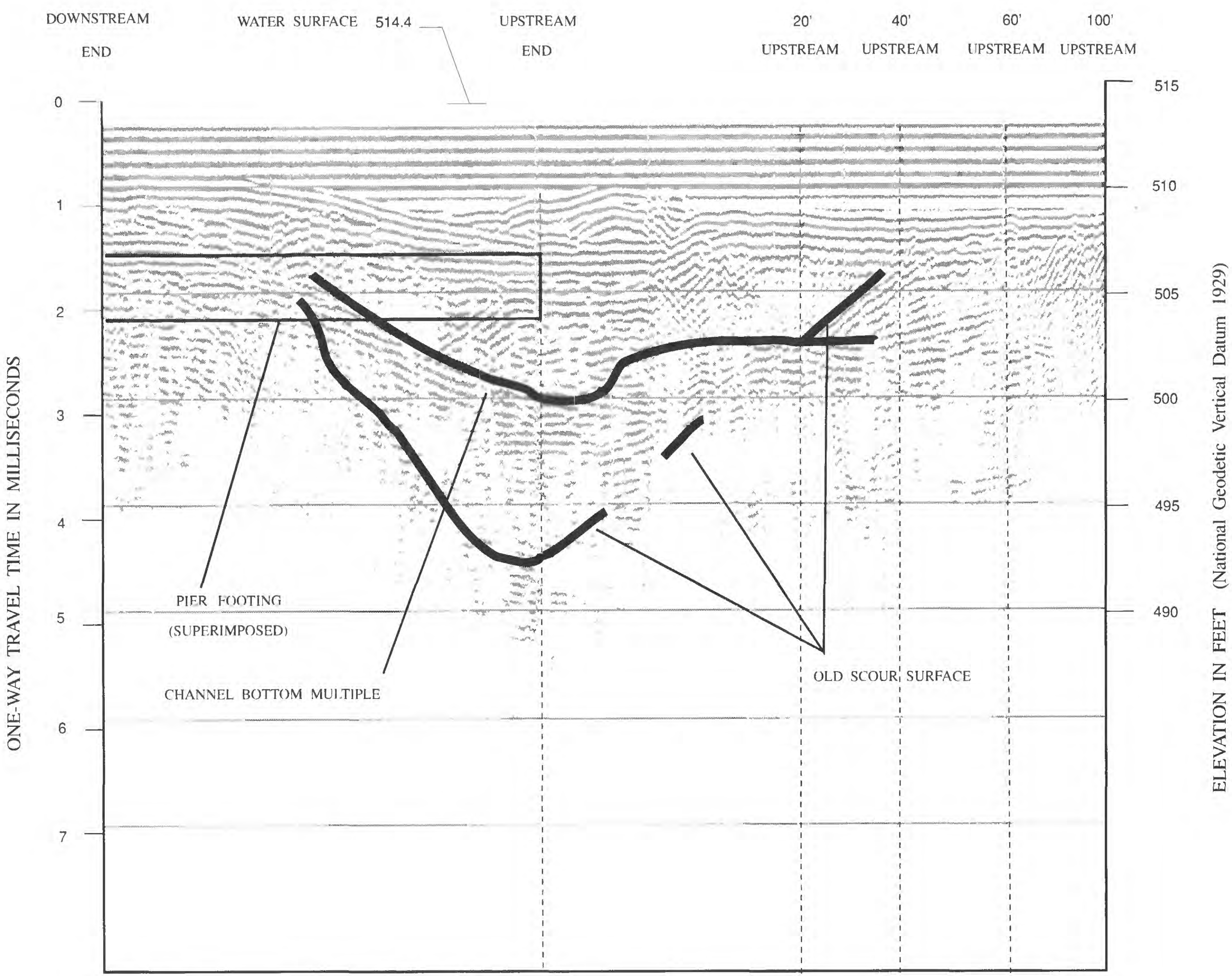


Figure 21. Interpretation of 7-kilohertz tuned-transducer cross section at I-74 over Whitewater River near Harrison, Ohio: downstream to upstream, 3 ft right of pier 15.

footing; however, observations in the field indicate that the scour 20 ft upstream from the pier was caused by a debris pile at pier 15.

U.S. Route 231 over Kankakee River near Hebron, Ind.

The main channel in the bridge opening was surveyed by use of the GPR and the tuned transducer at a signal frequency of 14 kHz. The water was too shallow for the tuned transducer, so no usable record was obtained. Water depth and conductivity limited penetration of GPR to approximately 10 ft. Therefore, interfaces more than 10 ft below the channel bottom were not detected. Adjacent to pier 3 (fig. 11), an interface was detected on the right side of the upstream end that indicates possible scour to an elevation of 629 ft (fig. 22). This was the lowest interface observed at this site. Around piers 4 and 5, interfaces were detected between elevations of 631 to 632 ft.

The areas around all of the piers were probed. Penetration around pier 2 ranged from 0 to 6.7 ft. The lowest elevation reached was 632.6 ft at a point one-quarter of the pier length downstream from the upstream end and 2 ft left of the pier. Penetration around pier 3 ranged from 0.7 to 5.9 ft. The lowest elevation found was 632.2 ft, which is believed to be the top of the footing. This elevation was reached on both sides of pier 3. The penetration around pier 4 ranged from 2.0 to 5.4 ft. The lowest elevation was 632.4 ft, which also is believed to be the top of the footing. This elevation was reached on both sides of pier 4. The penetration around pier 5 ranged from 0.5 to 5.5 ft. The lowest elevation, 632.5 ft, was along the left side of the pier.

The data indicate a probable scour hole at the upstream end of pier 3. This scour hole appears to have a bottom elevation that is equal to the elevation of the bottom of the footing. This probable scour hole was the only location at this site for which an interface was observed to be at or below the footing.

U.S. Route 231 over East Fork White River near Haysville, Ind.

The main channel in the bridge opening was surveyed by use of the GPR and the tuned transducer. Because of the water depth and signal attenuation, the GPR did not produce any conclusive results. By use of the tuned transducer, with a signal frequency of 14 kHz, a subsurface interface was detected between the elevations of 402 and 404 ft, located between piers 2 and 3 (fig. 13) and 25 to 40 ft downstream of pier 3. On the basis of the shape and location of this interface, it is likely a former position of the thalweg which has subsequently refilled.

Probing by use of a steel pipe around pier 3 resulted in penetrations ranging from 0.9 to 5.5 ft. The lowest elevation reached was 404.2 ft, 3 ft to the left of the center of the pier. Probing was not attempted at the location where the interface was detected with the tuned transducer. All scour detected was above the elevation of the pier footings.

S.R. 258 over East Fork White River near Seymour, Ind.

The main channel in the bridge opening, which includes piers 4 and 5 (fig. 14), was surveyed by use of the GPR. The water was too shallow to use the tuned transducer. From 5 to 6 ft of deposition was observed adjacent to pier 5. This deposition was verified by comparing the surveyed cross section to the cross section from the bridge plans.

The area around piers 4 and 5 was probed to verify the results of the geophysical survey. Penetration around pier 5 ranged from 1 to 7 ft. The lowest elevation found was 541.7 ft at a point 2 ft downstream and 3 ft left of the upstream end of the pier. This low point is consistent with the infilling indicated by the geophysical survey. Penetration around pier 4 ranged from 1 to 3 ft. The lowest elevation found was 542.0 ft at a point 2 ft right of and 8 ft downstream from the upstream end of the pier. Results from the probing were consistent with the geophysical survey, and neither technique detected scouring at this site.

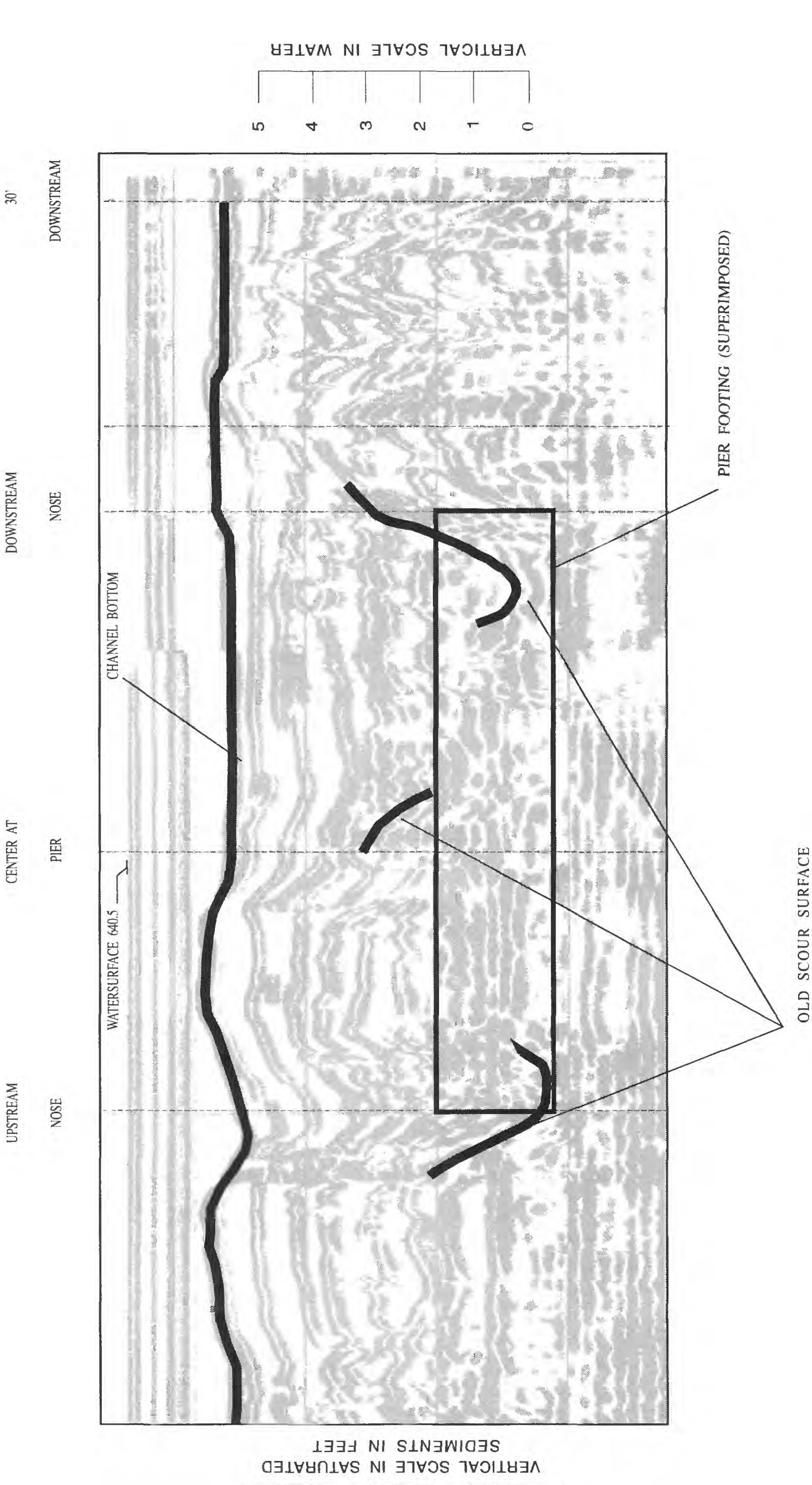


Figure 22. Ground-penetrating-radar cross section at U.S. Route 231 over Kankakee River near Hebron, Indiana: upstream to downstream, 3 ft right of pier 3.

S.R. 331 over Tippecanoe River at Old Tip Town, Ind.

The main channel in the bridge opening was surveyed by use of the GPR; shallow water prevented use of the tuned transducer. Piers 2 and 3 (fig. 16) are in this area. At pier 2, an interface was detected between the elevations of 753 and 755 ft. At a point 5 ft upstream from the upstream end and along the right side of the pier, the interface was observed at an elevation of 753 ft. At pier 3, the lowest interface observed was at 751 ft, just downstream from the end of the pier (fig. 23). Along the left side of the pier, the interface was observed at an elevation of 753 ft. At a point 15 ft upstream from the upstream end, the interface was at 752 ft.

The lowest elevation found by probing near pier 2 was 757.2 ft at a point 3 ft to the right of the upstream end of the pier. The lowest elevation at pier 3 was 757.1 ft at a point 12 ft upstream from the upstream end of the pier. The geophysical data indicate an interface 4 to 6 ft below the bottom of the footing, which, on the basis of its shape and location, is likely the result of scour.

S.R. 358 over White River near Edwardsport, Ind.

The main channel in the bridge opening, which includes piers 2 and 3 (fig. 18), was surveyed by use of the tuned transducer with a signal frequency of 14 kHz. The water was too deep for the GPR. Aggradation was observed in the left part of the main channel, with 1 to 3 ft of deposition between the left bank and pier 3.

The area of most concern is along the left side of pier 2. The deepest interface, at an elevation of 420 ft, is at points 10 ft downstream from and 12 ft to the left of the pier (fig. 24) and 2 ft downstream from the downstream end of the pier. Interfaces also were detected at an elevation of 421 ft near pier 2 at points 250 ft upstream and 8 ft to the left, 6 ft upstream and 22 ft to the left, 5 ft upstream and 4 ft to the left, and 2 ft downstream and 14 ft to the left. On the basis of the location and shape of the interfaces, these interfaces are likely the results of a thalweg that has partially refilled. The thalweg at this site is below the bottom of the footing and may be close to undermining the footing during extremely high flows.

POTENTIAL SCOUR AROUND BRIDGE PIERS AND ABUTMENTS

Bridge failures resulting from the scour of bed material have resulted in numerous investigations into the mechanisms responsible for controlling the depth of scour around bridge piers and abutments. Total scour at bridges is a result of the combination of three components: (1) aggradation and degradation, (2) contraction scour, and (3) local scour (Richardson and others, 1991, p. 7-8). A literature review of bridge-scour equations by McIntosh (1989, p. 85) found that more than 35 equations have been proposed for estimating the local scour at interior bridge piers. Numerous equations also have been developed for prediction of scour at abutments and scour that is a result of channel-width contractions. Most local-scour equations are based on research with scale models in laboratory flumes with cohesionless, uniform bed material and limited field verification (McIntosh, 1989, p. 85). The contraction- and local-scour equations produce a wide range of scour-depth estimates for the same set of conditions (Anderson, 1974; Hopkins and others, 1980; Richards, 1991). Therefore, before computing potential scour by use of published equations, one must evaluate the applicability of these equations to the area of interest.

Description of Equations

Review and evaluation of all published equations were beyond the scope of this study; therefore, a limited number of equations were selected. A consistent notation for variables is used for presentation and discussion of the equations in this report. Consequently, the notation used herein may not be identical to the notation in the references cited. The variables are defined in the text the first time they are introduced. A complete listing of the variables is provided in the "Symbols" section at the front of this report. Many of the equations are dimensionless; therefore, any units can be used as long as they are consistent. If an equation

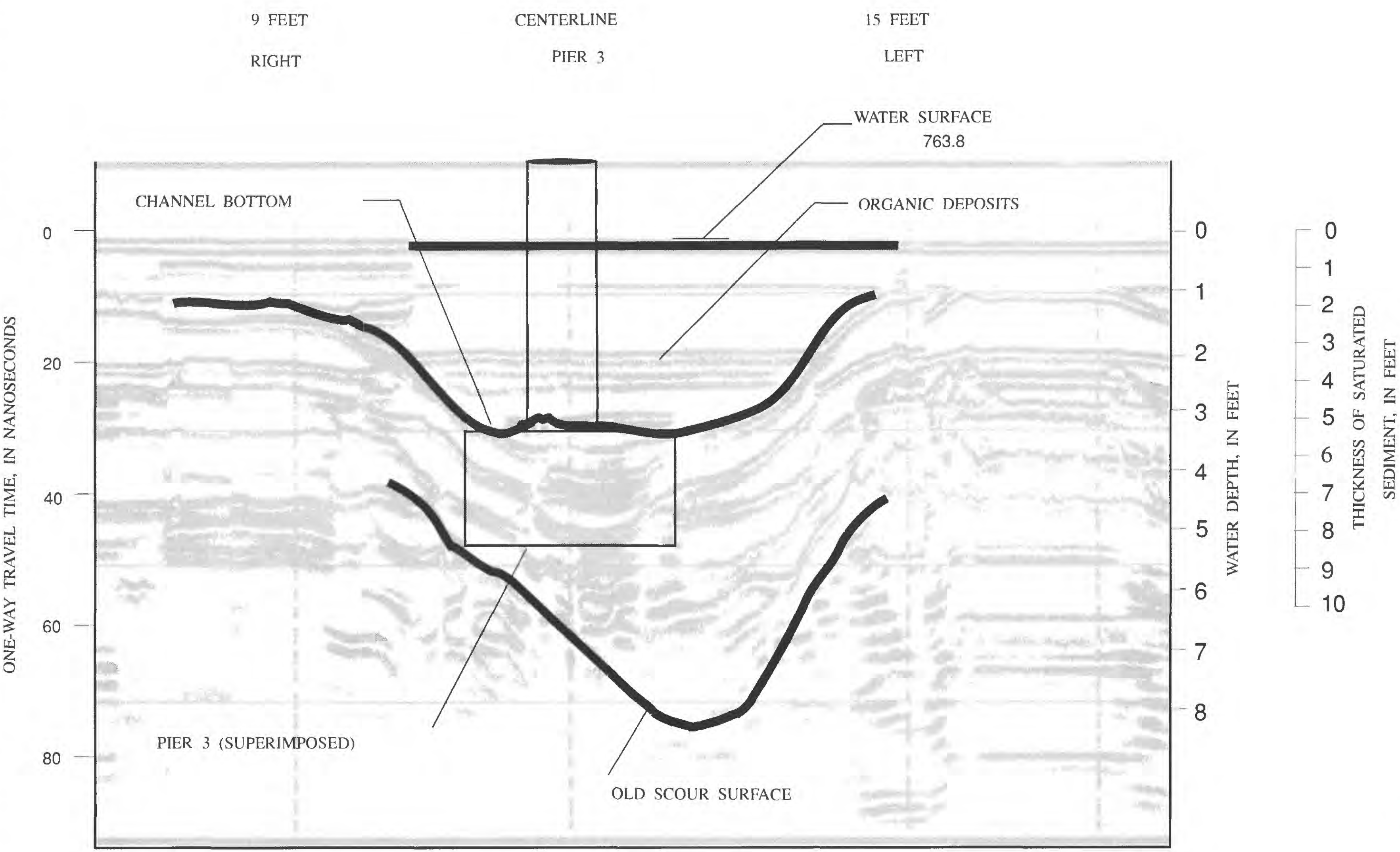


Figure 23. Ground-penetrating-radar cross section at State Road 331 over Tippecanoe River at Old Tip Town, Indiana: Right to left, 4 ft downstream at pier 3.

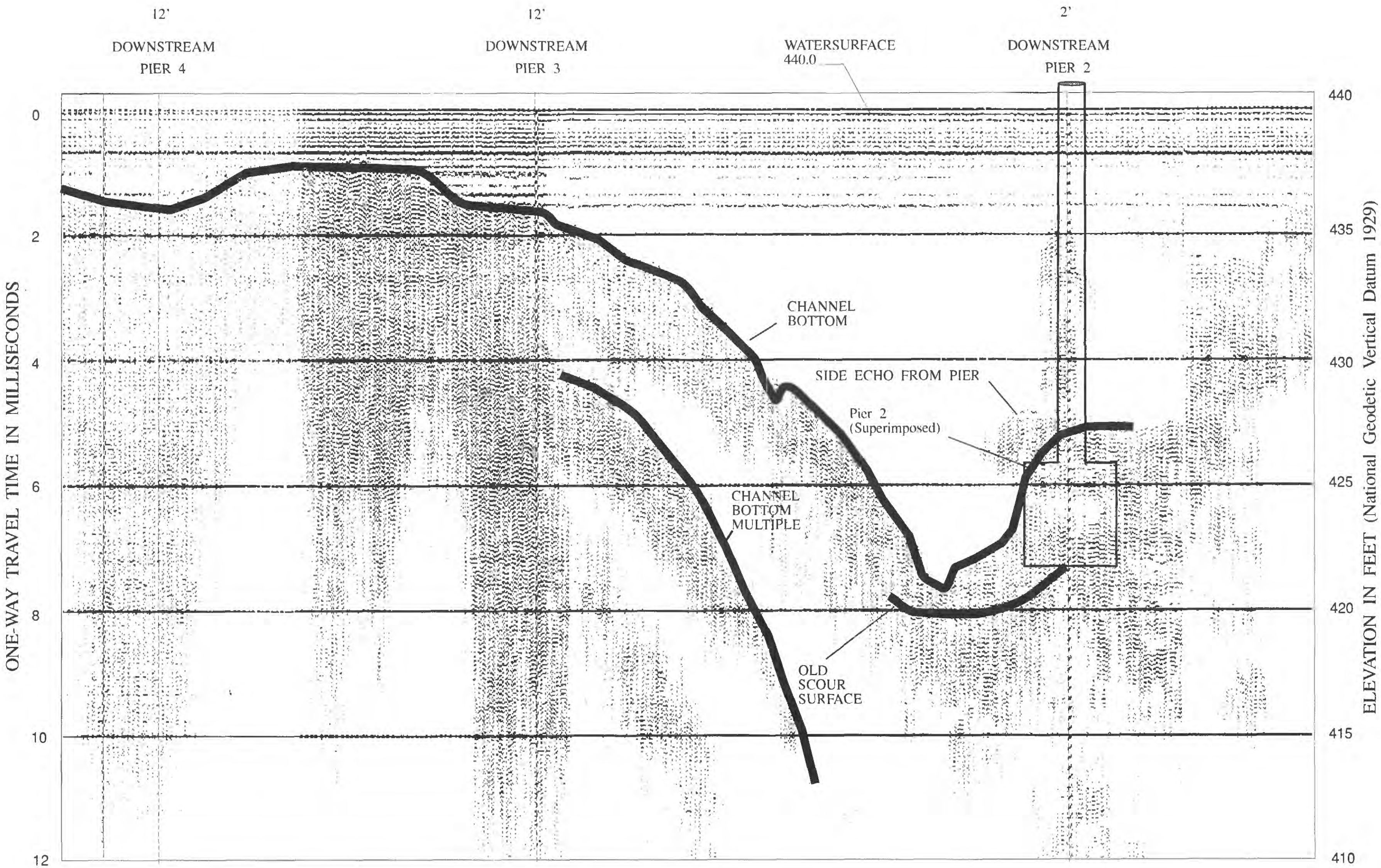


Figure 24. Interpretation of 14-kilohertz tuned-transducer cross section at State Road 358 over White River near Edwardsport, Indiana: left to right, downstream side of bridge.

requires a particular set of units, the units are defined with the equation in which they are required.

Contraction-Scour Equations

Contraction scour traditionally has been classified as either live-bed or clear-water scour. The live-bed condition occurs when the flow upstream from the contracted section is transporting bedload into the scour hole and when material transported from the scour hole consists of material from the scour hole and material transported from upstream. The clear-water condition occurs when the flow upstream does not transport bedload and when the only bedload material being transported from the scoured area is the material being scoured. Separate equations have been developed to estimate scour for these two conditions.

Live-bed scour

Laursen (1962) used a discharge equation (Manning's equation), a sediment-transport equation (Laursen, 1958), and discharge and sediment continuity to solve for the ratio of depth in a long contraction to that of a uniform reach. On the basis of these equations and the assumptions associated with them (steady-uniform flow, noncohesive material, and sufficient length of time to achieve equilibrium sediment transport), Laursen developed the following equation:

$$\frac{y_c}{y_u} = \left(\frac{Q_c}{Q_u}\right)^{6/7} \left(\frac{B_u}{B_c}\right)^{6/7\left(\frac{2+a}{3+a}\right)} \left(\frac{n_c}{n_u}\right)^{6/7\left(\frac{a}{3+a}\right)} \quad (1)$$

where y_c is depth of flow in the contracted channel,

y_u is average depth of flow in the uncontracted channel,

Q_c is discharge in the part of the contracted channel represented by the specified bottom width,

Q_u is discharge in the part of the uncontracted or approach channel represented by the specified bottom width,

B_u is bottom width of the uncontracted or approach section,

B_c is bottom width of the contracted section,

n_c is Manning's roughness coefficient for the part of the contracted channel represented by the specified bottom width,

n_u is Manning's roughness coefficient for the part of the uncontracted or approach channel represented by the specified bottom width, and

a is a coefficient based on the ratio of the shear velocity to the fall velocity in the uncontracted channel.

a	u_*/ω	Mode of bed-material transport
0.25	<0.5	Mostly contact bed-material discharge
1.00	0.5-2.0	Some suspended bed-material discharge
2.25	>2.0	Mostly suspended bed-material discharge

where u_* is shear velocity, defined as, $\sqrt{gy_uS}$; (g is acceleration of gravity; and S is dimensionless slope of the energy grade line near the bridge), and

ω is fall velocity of the median grain size of the bed material (Richardson and others, 1991, p. 44, figure 4.2).

According to Laursen (1960, p. 44), "A bridge crossing is in effect a long contraction foreshortened to such an extreme that it has only a beginning and an end." Therefore, the depth of contraction scour at a bridge for live-bed conditions can be derived from equation 1 as

$$y_{sc} = y_u \left(\frac{Q_c}{Q_u}\right)^{6/7} \left(\frac{B_u}{B_c}\right)^{6/7\left(\frac{2+a}{3+a}\right)} \left(\frac{n_c}{n_u}\right)^{6/7\left(\frac{a}{3+a}\right)} - y_b \,, \quad (2)$$

where y_{sc} is depth of contraction scour below the existing bed, and

y_b is average depth of flow at the bridge before contraction scour.

Richardson and others (1991, p. 38) provided two warnings on the use of this equation.

> The Manning's n ratio can be significant for a condition of dune bed in the main channel and a corresponding plain bed, washed out dunes, or antidunes in the contracted channel (Richardson and others, 1990). However, Laursen's equation does not correctly account for the increase in transport that will occur as the result of the bed planing out, which decreases resistance to flow and increases velocity and the transport of bed material at the bridge. That is, Laursen's equation indicates a decrease in scour for this case whereas in reality there is an increase in scour depth. Therefore, set the two n values equal.
>
> Laursen's equation will overestimate the depth of scour at the bridge if the bridge is located at the upstream end of the contraction or if the contraction is the result of the bridge abutments and piers. At this time, however, it is the best equation available.

Clear-water scour

On the basis of the proposition that the limiting condition of clear-water scour is a boundary shear equal to the critical tractive force, Laursen (1963) developed a relation for the scour in a long contraction as a function of channel geometry, flow, and sediment. Laursen assumed that the critical shear stress for noncohesive bed materials could be approximated as

$$\tau_c = 4d_m \,, \quad (3)$$

where τ_c is critical shear stress, in pounds per square foot, and

d_m is mean grain size of the bed material, in feet.

This relation is consistent with work done by White (1940) and Shields (1936). Laursen then set the ratio of the shear stress in the uncontracted section to the critical shear stress equal to one and solved for the dimensionless depth of scour,

$$\frac{y_c}{y_u} = 0.13\left(\frac{Q_c}{d_m^{1/3} y_u^{7/6} B_c}\right)^{6/7} - 1 \quad . \quad (4)$$

If the mean diameter of the sediment is represented by the more common median diameter, then the depth of scour, y_{sc}, yields

$$y_{sc} = \left(\frac{Q_c^2}{120 d_{50}^{2/3} B_c^2}\right)^{3/7} - y_b \,, \quad (5)$$

where d_{50} is median grain size of the bed material.

Note: This equation is not dimensionless; y_{sc}, d_{50}, B_c, y_b are in feet, and Q_c is in cubic feet per second.

Equation 5 is applicable for computing contraction scour at relief bridges for the overbank areas beneath bridges. For relief bridges, Richardson and others (1991, p. 42) recommend that $1.25d_{50}$ be used for the grain diameter in equation 5.

Pier-Scour Equations

Before the selected equations are discussed, it is necessary to explain how the data and method of analysis affect the computed depth of scour. Equations typically compute equilibrium, maximum, or design depths of scour. The measured scour, particularly in the flume, is often taken to be the equilibrium depth of scour, which is measured after equilibrium sediment transport has occurred and which averages the periodic change in bed elevation caused by the movement of bedforms. Therefore, equations that are based on laboratory data often compute equilibrium scour. Because it is impossible to determine exactly what depth of scour has been measured in the field without continuous monitoring, errors in the measured depth equal to the height of the bedforms could result. Some researchers have assumed that the scour measured in the field represents equilibrium conditions, whereas others have assumed that it represents maximum conditions. Many papers in the literature lack a good explanation of the depth of scour that is computed or measured--equilibrium or maximum. The method used to develop the pier-scour equations further complicates the description of which depth of scour is computed by the equations. If a regression analysis is used and no additional corrections are added, the depth of scour computed would not be a maximum scour for all sites. If an envelope curve were drawn above the data and used to develop the equation, then the depth of scour from this equation would, by design, exceed all measured depths of scour. For design purposes, it is desirable to use an equation that produces the maximum depth of scour that could be expected, thereby ensuring that the design achieves an acceptable factor of safety.

Ahmad

On the basis of previous work on scour around spur dikes, Ahmad (1953) concluded that local scour does not differ with grain size in the range usually found in the alluvial plains of West Pakistan (0.1 to 0.7 mm). He admitted, however, that this conclusion may not be valid for the entire range of bed-material grain sizes. Ahmad (1962) re-analyzed the work of Laursen (1962) with special emphasis on his experience with scour in sand-bed streams in West Pakistan and developed the following equation:

$$y_p = Kq^{2/3} , \tag{6}$$

where

$$y_p = y_o + y_{sp} , \tag{7}$$

and y_p is depth of flow at the bridge pier, including local pier scour;

y_o is depth of flow just upstream from the bridge pier or abutment, excluding local scour;

y_{sp} is depth of pier scour below the ambient bed; and

K is a coefficient that is a function of boundary geometry, abutment shape, width of the piers, shape of the piers, and the angle of the approach flow. On the basis of numerous model studies, Ahmad (1962) suggested that the coefficient should be in the range of 1.7 to 2.0 to calculate scour at piers and abutments. For this investigation, it was assumed to be 1.8.

Note: Equation 6 is not dimensionless; y_p is in feet and q is in cubic feet per second per foot.

Solving equations 6 and 7 for y_{sp} yields

$$y_{sp} = Kq^{2/3} - y_o \quad . \tag{8}$$

Note: Equation 8 is not dimensionless; y_{sp}, y_o are in feet and q is in cubic feet per second per foot.

Equation 8 is referred to as the "Ahmad equation."

Blench-Inglis

Inglis (1949) performed numerous experiments on model bridge piers and developed an empirical formula by fitting an equation to the plotted data. Blench (1962) reduced Inglis' (1949) original formula to the form

$$\frac{y_p}{y_r} = 1.8\left(\frac{b}{y_r}\right)^{0.25} , \tag{9}$$

where

$$y_r = \left(\frac{q^2}{f_b}\right)^{1/3} \tag{10}$$

and b is width of the bridge pier,

y_r is regime depth of flow,

q is discharge per unit width just upstream from the pier, and

f_b is the bed factor.

Blench (1951) stated that the bed factor was related to the nature of the sediment load and defined it as

$$f_b = \frac{V^2}{y} , \tag{11}$$

where V is average velocity of the section, and

y is average depth of the section.

Equation 11 is not acceptable for estimating the bed factor in the design of regime channels because the velocity will have a direct effect on the width and depth of the channel. Lacey (1936) proposed a rough estimate for the bed factor based on grain size; this relation was modified by other researchers including Blench (1951, 1969). Although the value of the coefficient varies in the literature, a value of 1.9 is common, and will be used herein:

$$f_b = 1.9 d_{50}^{0.5} \tag{12}$$

Note: This equation is not dimensionless; d_{50} is in millimeters.

If, in applying regime theory to bridge scour, the average velocity and depth in equation 11 can be approximated by the conditions just upstream of the pier, then equations 7, 9, 10, and 11 can be solved for y_{sp}, and the result is equation 13 which will be referred to as the "Blench-Inglis I equation":

$$y_{sp} = 1.8 b^{0.25} q^{0.5}\left(\frac{y_o}{V_o^2}\right)^{0.25} - y_o , \tag{13}$$

where V_0 is velocity of the approach flow just upstream from the bridge pier or abutment.

However, applying the empirical formula to estimate the bed factor and solving equations 7, 9, 10, and 12 for y_{sp} results in equation 14, which will be referred to as the "Blench-Inglis II equation":

$$y_{sp} = 1.8 b^{0.25}\left(\frac{q^2}{1.9 d_{50}^{0.5}}\right)^{0.25} - y_o . \tag{14}$$

Note: Because equation 12 was used in the derivation, equation 14 is not dimensionless; y_{sp}, b, and y_o are in feet, q is in cubic feet per second per foot, and d_{50} is in millimeters.

Chitale

A series of experiments on a 1:65 scale model of the Hardings Bridge was done to determine the influence of the upstream depth and sand diameter on scour around piers. The bed of the flume contained 0.32 mm sand, but four different sands having mean diameters of 0.16 mm, 0.24 mm, 0.68 mm, and 1.51 mm were used in the immediate vicinity of the piers. Each experiment was run until the scour depth reached equilibrium. Chitale (1962, p. 196) observed that

> 1. With axial flow, maximum depth of scour was always at the nose of the pier, scour at the sides being less 5 to 15%.
>
> 2. The ratio of scour at the nose and depth of flow in the channel bears a simple relation with the approach velocity in the channel.
>
> 3. The depth of flow on the upstream also has an influence on the scour at the pier nose.

Although some scatter of the data was evident, Chitale (1962) found that the Froude number provided the best criterion with which to characterize the relative depth of the scour hole and developed the following equation:

$$\frac{y_{sp}}{y_o} = -5.49F_o^2 + 6.65F_o - 0.51 \;, \qquad (15)$$

where

$$F_o = \frac{V_o}{\sqrt{g y_o}} \;, \qquad (16)$$

F_o is the Froude number of the flow just upstream of the pier, and

g is the acceleration of gravity.

Solving equation 15 for y_{sp} results in,

$$y_{sp} = y_o(-5.49F_o^2 + 6.65F_o - 0.51) \;, \qquad (17)$$

which will be referred to as the "Chitale equation."

Although one of the objectives of the model experiments was to determine the influence of sediment size on the depth of scour, the final equation does not account for sediment size. However, a visual analysis of the scatter of data around equation 15 showed that bed-material size can affect the relative depth of scour by as much as a factor of 2 for Froude numbers less than 0.2 but to a lesser extent for Froude numbers greater than 0.2.

Colorado State University

By use of all of the available laboratory data for scour at circular piers, Richardson and others (1975) developed the following equation:

$$\frac{y_{sp}}{y_o} = 2.0K_1K_2\left(\frac{b}{y_o}\right)^{0.65} F_o^{0.43} \;, \qquad (18)$$

where K_1 is a coefficient based on the shape of the pier nose (1.1 for square-nosed piers, 1.0 for circular- or round-nosed piers, 0.9 for sharp-nosed piers, and 1.0 for a group of piers), and

K_2 is a coefficient based on the ratio of the pier length to pier width and the angle of the approach flow referenced to the bridge pier.

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

Solving equation 18 for y_{sp} yields

$$y_{sp} = 2.0y_oK_1K_2\left(\frac{b}{y_o}\right)^{0.65} F^{0.43} \;, \qquad (19)$$

which will be referred to as the "CSU equation." Although Richardson and others (1975) made no restrictions on the use of K_1 and K_2, Richardson and others (1991) stated that no correction for pier shape should be made if the angle of attack of the approach flow is greater than 5° because, at these greater angles, the pier shape loses its effect.

According to Richardson and others (1990, p. V-105),

> For the determination of pier scour, the Colorado State University's equation is recommended for both live-bed and clear water scour. With a dune bed configuration, the equation predicts equilibrium scour depths and maximum scour will be 30 percent greater. For flow with plane bed configuration given by Colorado State University's equation gives the maximum scour. And for antidunes the computed scour depths should be increased by 20 percent.

More recently, however, Richardson and others (1991) recommended a 10-percent increase to compute maximum scour for both antidunes and plane-bed configurations. This later recommendation was used for the computations presented in this report.

Froehlich

Froehlich (1988) compiled a number of onsite measurements of local scour at bridge piers. All of the data were collected during sustained high flows and are assumed to represent equilibrium sediment transport through the scour hole. The critical mean-velocity relation presented by Neill (1968) was used to extract only live-bed data from the data set. Linear regression analysis of these live-bed data was used to develop an equation for the maximum relative depth of scour at a bridge pier:

$$\frac{y_{sp}}{b} = 0.32\phi\left(\frac{b'}{b}\right)^{0.62}\left(\frac{y_o}{b}\right)^{0.46} F_o^{0.2}\left(\frac{b}{d_{50}}\right)^{0.08}, \quad (20)$$

where b' is width of the bridge pier projected normal to the approach flow
$b' = b\cos(\alpha) + L\sin(\alpha)$;

ϕ is a coefficient based on the shape of the pier nose (1.3 for square-nosed piers, 1.0 for round-nosed piers, 0.7 for sharp-nosed piers);

α is angle of the approach flow referenced to the bridge pier, in degrees; and

L is length of the bridge pier.

Solving equation 20 for y_{sp} results in

$$y_{sp} = 0.32b\phi\left(\frac{b'}{b}\right)^{0.62}\left(\frac{y_o}{b}\right)^{0.46} F_o^{0.2}\left(\frac{b}{d_{50}}\right)^{0.08}, \quad (21)$$

which will be referred to as the "Froehlich equation." Although Raudkivi (1986) showed the standard deviation of the bed material to have a significant influence on the depth of scour, this information was not available for most of the data used to develop equation 20 and was not included in the regression analysis. All of the measured depths of scour were less than the depth of scour computed by equation 21 when the width of the pier was added to the result. Therefore, Froehlich (1988, p. 538) recommended for design purposes that the depth of scour computed by equation 21 be increased by the width of the pier. For the purposes of comparing the Froehlich equation to scour depths measured in Indiana, the factor of safety will not be included.

Inglis-Lacey

The application of the pier-scour equation developed by Inglis (1949) was determined to be difficult because of the effect of local stream geometry on the unit discharge (Joglekar, 1962, p. 184). In addition,

> it has to be remembered that the angle of repose of the bed material in the model and the prototype is the same, hence, the extent of scour in plan in the vertically distorted model is found always relatively greater than in the prototype. This in effect reduces the discharge intensity at the pier due to greater dispersion of flow and hence the depths of scour obtained in the model would be relatively less. (Joglekar, 1962, p. 184)

Data were collected for scour around bridge piers at 17 bridges in India. The discharges at these 17 sites ranged from 29,063 to 2,250,00 ft^3/s, the mean diameter of the bed material ranged from 0.17 to 0.39 mm, and measured scour depths ranged from 25 to 115 ft (Richards, 1991, p. 35). On the basis of this data, the following formula was developed (Joglekar, 1962, p. 184; Lacey, 1930):

$$y_p = 0.946\left(\frac{Q}{f_b}\right)^{1/3}, \tag{22}$$

where

$$f_b = 1.76 d_m^{0.5} \tag{23}$$

and Q is discharge.

Note: Equations 22 and 23 are not dimensionless; y_p is in feet, Q is in cubic feet per second, and d_m is in millimeters. Equation 23 is another published variation of equation 12.

Solving equations 7, 22, and 23 for y_{sp} and substituting the median grain size for the mean grain size results in

$$y_{sp} = 0.946\left(\frac{Q}{1.76 d_{50}^{0.5}}\right)^{1/3} - y_o . \tag{24}$$

Note: Equation 24 is not dimensionless; y_{sp}, y_o are in feet, Q is in cubic feet per second, and d_{50} is in millimeters.

Equation 24 will be referred to as the "Inglis-Lacey equation."

Joglekar (1962, p. 184) stated, "a representative f_b value has to be used. From bore data, values of f_b for each strata is to be worked out to ascertain that the anticipated depth is not based on the f_b value which is higher than that appropriate at that depth." Because the total discharge and depth of flow is included but the width of the channel is not, the approach velocity is not defined. This would seem to limit the application of this formula to streams whose geometric and hydraulic features are similar.

Inglis-Poona

Experiments were done at the Central Water and Power Research Station in Poona, India, in 1938 and 1939 to study scour around a single pier. These studies were done in a flume with sand having a mean diameter of 0.29 mm. On the basis of these studies, Inglis (1949) presented this formula (Joglekar, 1962, p. 184):

$$\frac{y_p}{b} = 1.7\left(\frac{q^{2/3}}{b}\right)^{0.78} . \tag{25}$$

Making the appropriate substitutions and solving equation 25 for y_{sp} results in

$$y_{sp} = 1.7b\left(\frac{q^{2/3}}{b}\right)^{0.78} - y_o , \tag{26}$$

Note: Equations 25 and 26 are not dimensionless; y_p, y_s, y_o, b are in feet, and q is in cubic feet per second per foot.

which will be referred to as the "Inglis-Poona I equation." This relation is not dimensionally homogeneous; therefore, it is unlikely that it is universally applicable to other bridge-scour data.

From this same set of experiments, Inglis (1949) developed a dimensionally homogeneous equation,

$$\frac{y_p}{b} = 1.73\left(\frac{y_o}{b}\right)^{0.78} , \tag{27}$$

which, when solved for y_{sp}, yields

$$y_{sp} = 1.73b\left(\frac{y_o}{b}\right)^{0.78} - y_o , \tag{28}$$

which will be referred to as the "Inglis-Poona II equation."

Larras

Larras (1963) defined a stable river as one that transports enough material to maintain the bed at a constant level and an unstable river as one that has inadequate sediment transport to maintain the bed at a constant level. According to Hopkins and others (1980),

> Larras concluded that maximum scouring is independent of the water depth and bed material size if the bed is stable, water depth is greater than 30 to 40 times the size of the bed material, and the channel constriction is less than 10% at the bridge site. The scour depth is a function of the maximum width of the pier, its shape, and flow direction.

Larras (1963) analyzed available scour data from various French rivers and model studies and developed the equation that will be referred to as the "Larras equation":

$$y_{sp} = 1.42 K_{S2} b^{0.75} , \tag{29}$$

where K_{S2} is a coefficient based on the shape of the pier nose (1.0 for cylindrical piers and 1.4 for rectangular piers).

Larras stated that the depth of scour would be greater in unstable riverbeds than for stable riverbeds because of the inadequate supply of bed material to the scoured area in unstable beds. Because Larras' field measurements were only point measurements of scour depth made after a flood had passed, those data may not properly represent the depth of equilibrium scour (Shen and others, 1969). Equation 29 depends only on pier width and is independent of the hydraulics.

Laursen

"The flow at the crossing cannot be considered uniform, but the solutions for the long contraction can be modified to describe the scour at bridge piers and abutments with the use of experimentally determined coefficients" (Laursen, 1962, p. 170). Laursen manipulated equation 1 to develop a formula which could be used to predict scour at abutments. If a live-bed condition is assumed, the formula is

$$\frac{l_{ae}}{y_o} = 2.75\left(\frac{y_{ca}}{y_o}\right)\left(\left[\left(\frac{1}{r}\right)\left(\frac{y_{ca}}{y_o}\right)+1\right]^{1.70} - 1\right) \tag{30}$$

where l_{ae} is effective length of an abutment;

y_{ca} is depth of abutment scour including contraction scour; and

r is a coefficient used to relate scour in a long contraction to scour at an abutment or pier.

Numerous flume experiments were done to evaluate the importance of the length-width ratio of the piers, the angle of attack of the stream against the piers, the approach velocity, the depth of flow, and the sediment size. All data on piers were adjusted to represent scour around a rectangular pier aligned with the flow. Laursen (1962) concluded that the abutment-scour equation with r = 11.5 and l_{ae} = $b/2$ fit the data reasonably well. Therefore, the Laursen equation for pier scour is

$$\frac{b}{y_o} = 5.5\left(\frac{y_{sp}}{y_o}\right)\left(\left[\left(\frac{1}{11.5}\right)\left(\frac{y_{sp}}{y_o}\right)+1\right]^{1.70} - 1\right) . \tag{31}$$

Laursen found that the most important aspect of the geometry of the pier was the angle of attack between the pier and the flow, coupled with the length-width ratio of the pier. The shape of the pier also is important if the pier is aligned with the flow. Therefore, the depth of scour from equation 31 must be corrected for pier shape if the pier is aligned with the flow

$$y_{sp} = K_{S1} y_{sp} , \tag{32}$$

and, for angle of attack if the pier is not aligned with the flow,

$$y_{sp} = K_{\alpha L} y_{sp} \ , \quad (33)$$

where $K_{\alpha L}$ is a coefficient based on the angle of the approach flow referenced to the bridge pier (fig. 25) and

K_{S1} is a coefficient based on the shape of the pier nose (table 2).

For live-bed conditions, Laursen found no significant influence of the velocity or sediment size on the depth of scour. Laursen (1962) concluded that the maximum depth of scour was uniquely determined by the geometry and that the width of the scour holes was approximately 2.75 y_{sp}.

Shen

Through a series of experiments, Shen and others (1969) determined that the basic mechanism of local scour was the vortex systems caused by the pressure field induced by the pier. Further analysis of the vortex systems showed that the strength of the horseshoe vortex system was a function of the pier Reynolds number,

$$\boldsymbol{R}_p = \frac{V_o b}{\nu} \ , \quad (34)$$

where $\boldsymbol{R}_p$ is the pier Reynolds number and

ν is the kinematic viscosity of water.

According to Shen and others (1969, p. 1925), "Since the horseshoe vortex system is the mechanism of local scour and the strength of the horseshoe vortex system is a function of the pier Reynolds number, the equilibrium depth of scour should be functionally related to the pier Reynolds number."

All known data at the time were used to investigate the influence of the pier Reynolds number on the depth of scour around bridge piers. The analysis showed that the depth of scour rises sharply as the pier Reynolds number increases to a point, then begins to decline as the

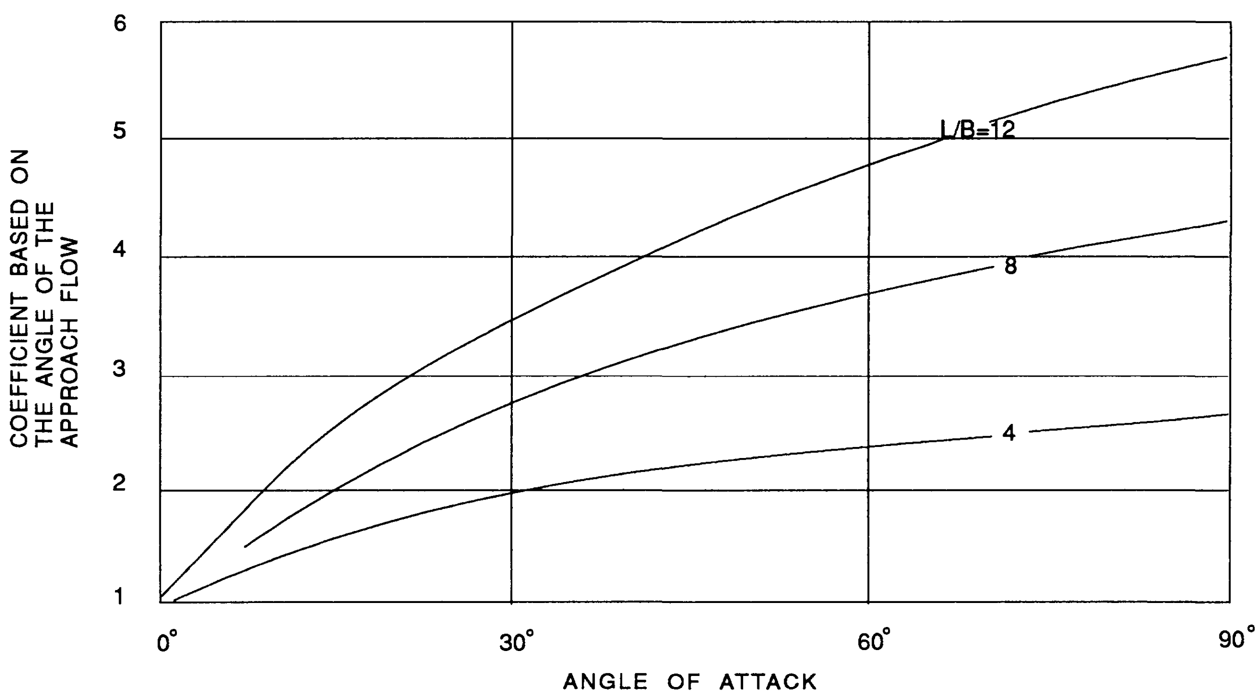


Figure 25. Effect of angle of attack (From Laursen, 1962, p. 177).

Table 2. *Pier-shape coefficients*

[K, shape coefficients for nose forms (to be used only for piers aligned with flow). From Laursen, 1962, p. 177]

Nose form	Length-width ratio	K_s
Rectangular	——	1.0
Semicircular	——	0.90 0.80
elliptic	2:1	0.75
	3:1	
Lenticular	2:1	0.80
	3:1	0.70

pier Reynolds number continues to increase. A least-squares regression of the data resulted in the following equation:

$$y_{sp} = 0.00073\boldsymbol{R}_p^{0.619} , \qquad (35)$$

which will be referred to as the "Shen equation." Evaluation of this equation showed that the effect of pier size prevented the equation from collapsing all of the data into one line, even for a given grain size. A definite separation of the data by sand size also was observed. Therefore, the Shen equation does not adequately account for the pier shape and the size of the bed material. Shen and others (1969) concluded that this equation could be used to provide a conservative estimate of clear-water scour, but that it was too conservative to be used for live-bed conditions. They suggested use of the equations by Larras (1963) and Breusers (1964-1965) for live-bed conditions.

Maza and Sanchez (1964) presented a relation between the ratio of depth of scour to pier width and the pier Froude number. Shen and others (1969) used all the available data in which median grain diameter of bed material was smaller than 0.52 mm in further investigations of the effects of the pier Froude number. They found that, for pier Froude numbers less than 0.2 and fine sands, the depth of scour increased rapidly as the pier Froude number increases; however, for pier Froude numbers greater than 0.2 and coarser sands, the depth of scour increased only moderately for increases in the pier Froude number. Therefore, two equations, which will be referred to as "the Shen-Maza equations," were used to fit the data:

$$y_{sp} = 11.0b\boldsymbol{F}_p^2 \quad for \quad \boldsymbol{F}_p < 0.2 \qquad (36)$$

$$y_{sp} = 3.4b\boldsymbol{F}_p^{0.67} \quad for \quad \boldsymbol{F}_p > 0.2 , \qquad (37)$$

where $\boldsymbol{F}_p$ is pier Froude number, defined as, $\frac{V_o}{\sqrt{gb}}$.

Equation 36 is fundamentally the same equation developed by Maza and Sanchez (1964) and is applicable when the pier Froude numbers are less than 0.2 (Shen and others, 1969). Equation 37 was developed by Shen and others (1969) for pier Froude numbers greater than 0.2.

Although the pier width is included in the pier Froude number, by squaring the pier Froude number and multiplying by the pier width, the pier width cancels. Therefore, these equations are independent of pier width, are based only on velocity, and are unlikely to be generally applicable to other situations.

Abutment-Scour Equations

Only abutment-scour equations presented in Richardson and others (1991) are discussed in this report. The alternative method presented first consists of a number of cases for which different equations are applicable. The Froehlich live-bed abutment-scour equation, however, is the primary equation recommended by Richardson and others (1991) for computing abutment scour.

Abutment projects into main channel, no overbank flow

Liu and others (1961) used dimensional analysis to design a laboratory experiment to study the mechanics of scour at abutments. Two tilting flumes were used in the investigation: one was 160 ft long and 8 ft wide, and the other was 80 ft long and 4 ft wide. River sand that had a median diameter of 0.56 mm was used in the 8-ft wide flume. Two different sands were used in the 4-ft wide flume, a filter sand that had a median diameter of 0.65 mm and Black Hills sand that had a median diameter of 0.56 mm. Four different abutment configurations were tested: (1) vertical-board, (2) vertical-wall, (3) wingwall, and (4) spill-through. The depth of scour was measured with respect to the average normal bed surface. Analysis of the major dimensionless parameters by use of data collected for the vertical-wall configuration resulted in the following equation:

$$\frac{y_{sa}}{y_o} = 2.15\left(\frac{l_{at}}{y_o}\right)^{0.4} F_o^{1/3} \quad , \qquad (38)$$

where y_{sa} is depth of abutment scour below the ambient bed, and

l_{at} is abutment and embankment length measured at the top of the water surface and normal to the side of the channel from where the top of the design flood hits the bank to the other edge of the abutment (Richardson and others, 1991, p. B-7).

Solving equation 38 for y_{sa} results in

$$y_{sa} = 2.15\left(\frac{l_{at}}{y_o}\right)^{0.4} F_o^{1/3} y_o \quad , \qquad (39)$$

which can be used to compute live-bed scour at vertical abutments.

Although wingwall and spill-through abutment configurations were studied and the data were presented, Liu and others (1961, p. 43) did not present an equation with the suitable exponents because "such an effort is not fully justified due to the limited amount of data." They did find, however, that the depth of scour for the wingwall and spill-through abutment configurations generally are less than those for the vertical-wall and vertical-board abutment configurations. Richardson and others (1991, p. B-7) presented the following equation based on Liu and others (1961) for spill-through abutments:

$$\frac{y_{sa}}{y_o} = 1.1\left(\frac{l_{at}}{y_o}\right)^{0.4} F_o^{1/3} \quad , \qquad (40)$$

which, when solved for y_{sa}, results in

$$y_{sa} = 1.1\left(\frac{l_{at}}{y_o}\right)^{0.4} F_o^{1/3} y_o \quad , \qquad (41)$$

which can be used to compute live-bed scour at spill-through abutments.

Liu and others (1961) developed their equations on the basis of equilibrium scour for a dune-bed configuration. The maximum depth of scour depends on the bed configuration of the natural stream. Richardson and others (1991, p. B-10) recommend that the equilibrium scour be increased 30 percent for dune-bed configurations and 10 percent for antidune-bed configurations.

Laursen (1962) manipulated equation 1 to develop the following formula, which can be used to predict live-bed scour at vertical-wall abutments:

$$\frac{l_{ae}}{y_o} = 2.75\left(\frac{y_{ca}}{y_o}\right)\left(\left[\left(\frac{y_{ca}}{11.5y_o}\right)+1\right]^{1.70} - 1\right) \quad . \qquad (42)$$

Equation 42 must be solved by an iterative procedure; however, Richardson and others (1991, p. B-8) presented a simplified form,

$$\frac{y_{ca}}{y_o} = 1.5\left(\frac{l_{ae}}{y_o}\right)^{0.48} , \qquad (43)$$

which can be solved directly for y_{ca} as

$$y_{ca} = 1.5\left(\frac{l_{ae}}{y_o}\right)^{0.48} y_o . \qquad (44)$$

Laursen's abutment-scour equations are presented for vertical abutments; however, the following factors are suggested for other abutment types of small encroachment lengths (Richardson and others, 1991, p. B-8):

Abutment type	Multiplying factor
45-degree wingwall	0.90
Spill-through	.80

Abutment at relief bridges

Laursen (1963, p. 100) extended his clear-water contraction-scour equation to abutments, stating:

> The solution for the long contraction serves only as a minimum estimate of the scour to be expected at a relief bridge. However, if the same assumptions can be made concerning the nature of the flow in the clear-water case as in the case with sediment supply by the stream, the solution for the long contraction can be adapted to the case of the abutment (and the case of the pier). The key observations in the case of sediment-transporting flow were that the flow approaching the obstruction dived beneath the surface and passed through the constriction in a somewhat distorted conical scour hole centered at the upstream corner of the abutment, and that the flow approaching the clear opening was little disturbed.

Laursen (1963, p. 102) presented the following equation for computing clear-water scour at vertical abutments,

$$\frac{l_{ae}}{y_o} = 2.75\left(\frac{y_{ca}}{y_o}\right)\left(\frac{\left(\frac{y_{ca}}{12y_o}+1\right)^{7/6}}{\left(\frac{\tau'_o}{\tau_c}\right)^{0.5}} - 1\right) , \qquad (45)$$

where τ'_o is shear stress for the approach flow associated with the sediment particles, and

τ_c is critical shear stress, which can be obtained from figure 26.

An iterative solution is required to solve equation 45 for y_{ca}. Laursen (1963, p. 102) assumed the coefficient of 12 on the basis of experience for similar situations in sediment-transporting flows. Richardson and others (1991, p. B-8) however, used 11.5 instead of 12.

Laursen's clear-water abutment-scour equation is applicable to abutments at relief bridges; however, if there is sufficient evidence to suggest that bedload transport will occur, Laursen's live-bed abutment-scour equation can be applied.

Abutment projects into the channel, overbank flow present

Laursen's equations 42, 44, and 45 can be used to calculate live-bed and clear-water scour when the abutment projects into the main channel and overbank flow is present. The abutment length for this situation should be determined from the following equation (Richardson and others, 1991, p. B-13):

$$l_{ae} = \frac{Q_e}{y_o V_o} \quad , \tag{46}$$

where Q_e is discharge obstructed by the embankment.

Abutment set back from main channel

Laursen (1962, p. 174) stated, "The effect of setting the abutment back from the normal bank of the stream is difficult to assess. In the laboratory experiments no measurable effect could be noted." If the abutment is set back more than 2.75 times the depth of scour, y_{ca}, Laursen's equations can be used to compute the abutment scour by evaluating the variables on the basis of the flow on the overbank being obstructed by the abutment (Richardson and others, 1991, p. B-14). Typically the overbank flow will not be transporting bed material, and Laursen's clear-water abutment-scour equation should be applied. If there is sufficient evidence to suggest that bedload transport will occur on the overbank, however, Laursen's live-bed abutment-scour equation can be applied.

Abutment set at edge of main channel

When there is no bedload transport on the overbank, the scour for a vertical-wall abutment set at the edge of the main channel can be computed from the following equation proposed by Laursen (1980) (Richardson and others, 1991, p. B-16):

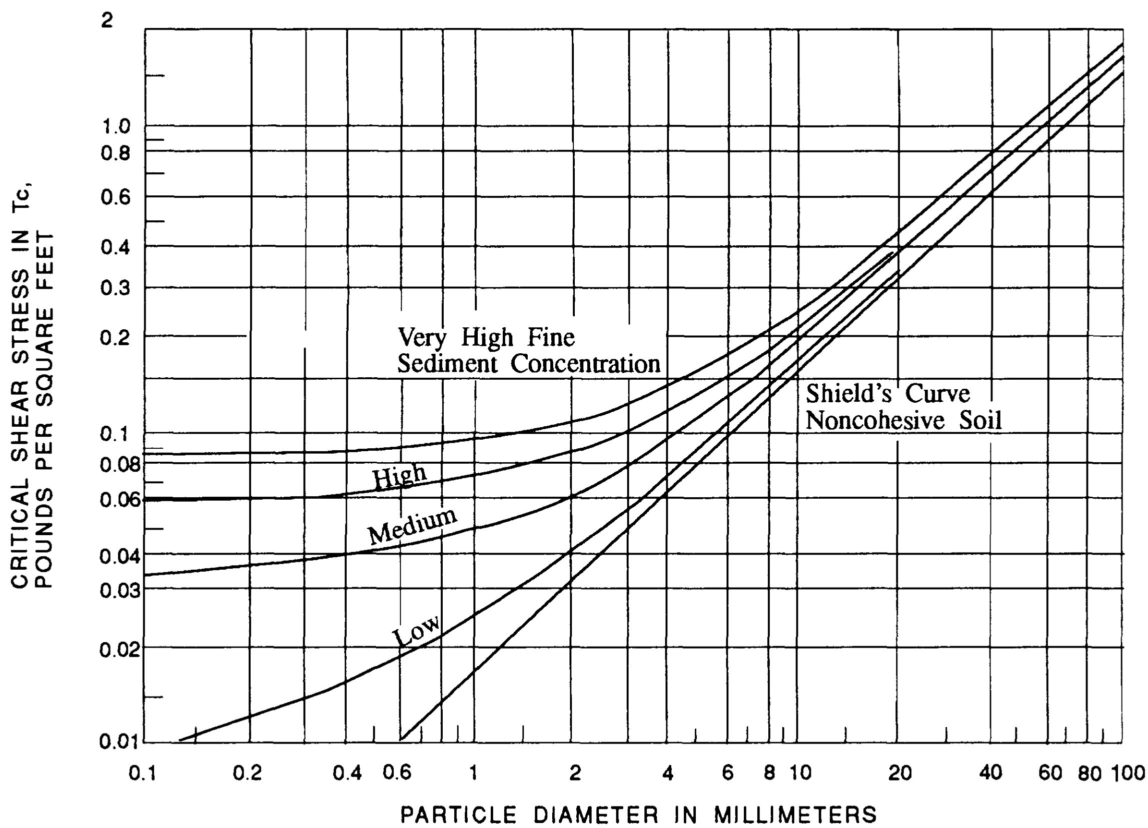


Figure 26. Critical shear stress as a function of bed-material size and suspended fine sediment. [From Richardson and others, 1991, p. B-11]

$$\frac{Q_e}{q_{mc}y_o} = 2.75\frac{y_{ca}}{y_o}\left(\left(\frac{y_{ca}}{4.1y_o}+1\right)^{7/6}-1\right), \quad (47)$$

where q_{mc} is discharge per unit width in the main channel.

An iterative solution is required to solve equation 47 for the depth of scour, y_{ca}.

Long abutments

Scour data collected at rock dikes on the Mississippi River indicate that equilibrium scour depths for large abutment length to depth of flow ratios (l_{at}/y_o>25) can be estimated by the following equation (Richardson and others, 1991, p. B-18):

$$y_{sa} = 4F_o^{1/3}y_o. \quad (48)$$

Abutments skewed to the stream

When abutments are skewed to the direction of flow in the stream, the scour at the abutment angled downstream is reduced because of the streamlining effect of the angle. Conversely, the scour at the abutment angled upstream is increased. The abutment-scour depths computed by use of equations 39, 41, 42, 44, 45, 47, and 48 should be corrected by use of figure 27, which is patterned after work by Ahmad (1953) (Richardson and others, 1991, p. B-18).

Froehlich's live-bed equation

Froehlich (1989) used multiple linear regression on 164 clear-water and 170 live-bed laboratory measurements of the maximum depth of local scour at model abutments to develop clear-water and live-bed abutment-scour equations. Because Froehlich's clear-water scour equation requires the standard deviation of the bed-material size distribution (which was not readily available at the selected sites) and because the equation is not currently recommended (Richardson and others, 1991, p. 48), this equation was not evaluated in this study. Froehlich's live-bed abutment-scour regression equation is as follows:

$$\frac{y_{sa}}{y_{oa}} = 2.27K_{sa}K_\theta\left(\frac{l}{y_{oa}}\right)^{0.43}F_a^{0.61}, \quad (49)$$

where K_{sa} is a coefficient based on the geometry of the abutment (1.0 for a vertical abutment that has square or rounded corners and a vertical embankment, 0.82 for a vertical abutment that has wingwalls and a sloped embankment, and 0.55 for a spill-through abutment and a sloped embankment);

K_θ is a coefficient based on the inclination of an approach roadway embankment to the direction of the flow,

$$K_\theta = \left(\frac{\theta}{90}\right)^{0.13};$$

l is length of an abutment, defined as, A_e/y_{oa};

A_e is cross-sectional area of the flow obstructed by the embankment;

y_{oa} is depth of flow at the abutment;

F_a is Froude number of the flow, defined as,

$$F_a = \frac{\left(\frac{Q_e}{A_e}\right)}{\sqrt{gy_o}}; \text{ and}$$

θ is angle of inclination of an embankment to the flow, in degrees; $\theta < 90°$ if the embankment points downstream.

Equation 49 is a minimum least-squares regression equation that is fit to the data. For design purposes, however, it is desirable to have the maximum scour which could be expected. Analysis of the data showed that when a value

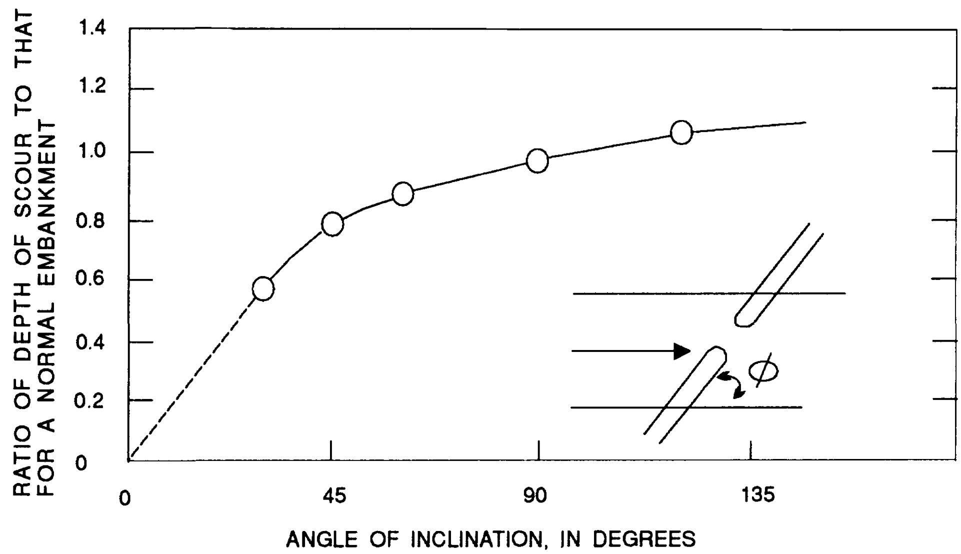


Figure 27. Scour-estimate adjustment for skew. [Modified from Richardson and others, 1991, p. B-19].

equal to the depth of flow at the abutment was added, the computed scour equaled or exceeded observed scour for 98 percent of the values. Therefore, solving equation 49 for depth of scour, y_{sa}, and including the factor of safety yields

$$y_{sa} = 2.27 K_{sa} K_{\theta} \left(\frac{l}{y_{oa}}\right)^{0.43} F_a^{0.61} y_{oa} + y_{oa} , \qquad (50)$$

which is recommended for all abutment configurations (Richardson and others, 1991, p. B-9).

Estimation of Hydrologic Conditions

Current guidance from the FHWA (Richardson and others, 1991, p. 23) suggests that "bridges should be designed to withstand the effects of scour resulting from a superflood (a flood exceeding the 100-year flood) with little risk of failing." The recommended design procedure is based on the scour resulting from either a 100-year peak discharge or a lesser discharge, if evidence shows that more scour would result from the lesser discharge. After the design is complete, a superflood equal to a 500-year peak discharge, or 1.7 times the 100-year peak discharge if the 500-year peak discharge cannot be estimated, is used to ensure that a factor of safety of 1.0 is maintained under ultimate load conditions with scour resulting from this superflood. Therefore, both the 100-year and 500-year peak discharges are needed to evaluate the safety of bridges by use of the published equations to predict potential scour.

Peak-flow data were not available for the study sites. Therefore, peak discharges for 10-, 25-, 50-, and 100-year return periods were determined by means of techniques presented in "Coordinated Discharges of Selected Streams in Indiana" (Indiana Department of Natural Resources, 1990). The 500-year peak discharge was estimated from the 10-, 25-, 50-, and 100-year peak discharges by use of linear regression on Pearson Type III, zero-skew plotting positions of the log-transformed discharges. The four discharges were plotted, and the shape of a line formed by these four points was analyzed. If the line appeared straight, or if the points were scattered and no definite curve could be delineated, a linear regression of all four data points was done to obtain an estimate of the 500-year peak discharge. If a curve were evident, the 10- and 25-year peak discharges were removed, and a linear regression with only the

50- and 100-year peak discharges was used to obtain an estimate of the 500-year peak discharge.

A historical peak discharge, the maximum flood that occurred during the life of the bridge, was used to evaluate the ability of selected published equations to reproduce measured historical scour; however, no historical discharge records were available for the study sites. Therefore, historical peak discharges were estimated from nearby USGS streamflow-gaging stations. At least two streamflow-gaging stations with similar hydrologic conditions were identified for each site. Data from streamflow-gaging stations upstream and downstream from the study sites were used, if available. The historical peak flows for each of the identified gaging stations were reviewed. The historical peak discharges for each gaging station were plotted against the drainage area using log transformations. The historical peak discharge at the study site then was computed as the discharge per unit drainage area for the study site (estimated from the plotted data) multiplied by the drainage area of the study site. Flow regulation by flood-control projects was accounted for in the selection of gaging stations. The peak discharge and drainage area for each site are presented in table 3.

Although the duration of a flood may affect the depth of scour, especially for cohesive materials, the selected equations are based on the assumption that the flood discharge is maintained for a sufficient period to allow equilibrium sediment transport through the scour holes. Therefore, the durations of the various floods were not assessed in this study because duration is not used in any of the selected published equations.

Estimation of Hydraulic Conditions

All of the bridge-pier-, abutment-, and contraction-scour equations require various hydraulic characteristics as input. Because measurements of the required hydraulic characteristics were not available for historical, 100-year, or 500-year peak discharges, estimates were made. A Water-Surface Profile computation model (WSPRO), developed by the USGS for the FHWA (Shearman, 1990; Shearman and others, 1986), was used to estimate the hydraulic conditions at the study sites for the required peak discharges.

Cross-section data and roughness coefficients were obtained from field surveys of each of the sites and input into the model. Slope-conveyance computations were used for the downstream boundary conditions. Historical water-surface profiles provided by the Indiana Department of Natural Resources (IDNR) were used to estimate water-surface slopes. Water-surface elevations computed by WSPRO for discharges represented by the IDNR water-surface profiles were compared to those profiles to verify the model. At U.S. Route 24 over Tippecanoe River, historical water-surface profiles were used to verify the model, but the selected floods were modeled without consideration for potential backwater effects from Lake Freeman.

Where applicable, the bridge routines available in WSPRO were used to estimate the hydraulic conditions at the bridge. Two sites, S.R. 32 over Buck Creek and U.S. Route 231 over the Kankakee River, did not have sufficient contraction through the bridge opening to warrant use of the bridge routines. Levee failure and overtopping are common along the Kankakee River; therefore, the entire discharge often is not maintained between the levees. For this study, however, the assumption was made that the entire flow was maintained by the levees, which will provide the worst-case hydraulic conditions for scour computations. The water-surface elevations computed by use of WSPRO were below the tops of the levees at the study site. Two additional sites, S.R. 258 over East Fork White River and State Road 358 over White River, are characterized by very complex geometry with multiple relief bridges and (or) road grades that became submerged to such a level that the available bridge and weir hydraulic routines were not applicable. These sites were modeled by use of composite cross sections constructed to represent, as much as possible, the complex geometries of the sites in a manner consistent with the limitations of WSPRO.

Although WSPRO is a one-dimensional model, 20 equal-conveyance tubes are computed to provide velocity and discharge distributions across a cross section. These velocity and

Table 3. *Hydrologic characteristics of study sites*

[U.S., United States; S.R., State Road; I, Interstate; Q, discharge]

				Estimated peak discharge in cubic feet per second, for given recurrence interval					Historical peak[1]	
Highway	River	Nearest city	Drainage area (square miles)	Q_{10}	Q_{25}	Q_{50}	Q_{100}	Q_{500}	Discharge (cubic feet per second)	Date
U.S. 24	Tippecanoe	Monticello, Ind.	1,768	17,00	19,500	22,300	25,000	31,500	18,900	February 1985
S.R. 32	Buck Creek	Yorktown, Ind.	100	3,800	5,000	6,000	7,000	9,500	5,000	April 1964
U.S. 41	White	Hazleton, Ind.	11,305	115,000	143,000	163,500	186,000	241,500	133,400	May 1961
I-74	Big Blue	Shelbyville, Ind.	314	10,800	13,300	15,200	17,300	22,500	14,700	March 1963
I-74	Whitewater	Harrison, Oh.	1,344	42,300	48,300	54,500	62,200	81,300	67,200	March 1963
U.S. 231	East Fork White	Haysville, Ind.	5,558	75,000	87,800	101,000	116,000	153,500	109,200	January 1937
U.S. 231	Kankakee	Hebron, Ind.	1,445	5,200	5,900	6,300	6,800	7,800	5,600	March 1985
S.R. 258	White	Seymour, Ind.	2,347	59,000	74,000	85,000	98,000	127,000	47,000	February 1982
S.R. 331	Tippecanoe	Old Tip Town, Ind.	390	2,650	3,100	3,550	3,850	4,500	1,650	October 1986
S.R. 358	White	Edwardsport, Ind.	5,013	68,000	81,000	97,000	110,000	141,900	45,600	February 1985

[1] Historical peak discharge is based on the life of the bridge, not on the period of record.

discharge distributions were used to determine the approach velocities and discharge conveyed through subsections of the bridge and approach cross sections as required by the scour equations. Corrections were made for all cross sections that were not oriented perpendicular to the flow.

The live-bed contraction-scour equation requires the following hydraulic characteristics at both the bridge and approach sections: slope of the energy grade line, average depth of flow, width of flow over which sediment is transported, and discharge conveyed over the specified width. The slope of the energy grade line was computed as the difference in elevation of the energy grade lines at the bridge and approach sections divided by the effective flow length between the two sections. The bottom width of the main channel was used for the width over which sediment is transported. The discharge conveyed over this bottom width was computed from the discharge distributions. The average depth of flow was computed as the cross-sectional area of the flow conveyed over the bottom width divided by the bottom width.

The clear-water contraction-scour equation was applied to the overbank areas within the bridge opening. This equation requires the width, depth, and discharge of the flow through the overbank area as input. The width of the overbank area was taken to be the distance from the top of the bank to the toe of the abutment. The discharge conveyed through the specified width was computed from the discharge distributions. The average depth of flow was computed as the cross-sectional area of the flow conveyed over the specified width divided by the specified width.

The various pier-scour equations require the following hydraulic characteristics: total discharge, depth of approach flow, approach velocity, and angle of attack. Total discharge was taken to be the total discharge conveyed through a given cross section. Depth of approach flow was computed as the difference from the water surface to the bed at the centerline of the pier in the bridge section. The approach velocity was taken to be the velocity at the centerline of the pier in the bridge section. If the velocity in adjacent flow tubes were greater than the velocity of the flow tube containing that pier, the velocity was increased to reflect a velocity typical of the adjacent tubes. The angle of attack was assumed to be 0°, except at U.S. Route 231 over East Fork White River near Haysville, Ind., where substantial evidence was available to indicate an angle of attack of 25°.

The abutment-scour equations require the following hydraulic characteristics: discharge, depth of flow, and width of the main channel at the approach section; discharge cross-sectional area and depth of flow blocked by the abutment; and the length of the abutment measured perpendicular to the flow. The discharge, depth of flow, and width of the main channel at the approach section were computed for the live-bed contraction-scour equation as described previously. The discharge blocked by the abutment was computed at the approach section. The station at the toe of the abutment in the bridge section was transferred, parallel to the direction of flow, to the approach section. The discharge conveyed by a subsection extending from this station to the edge of water was computed from the discharge distributions. The cross-sectional area of the discharge blocked by the abutment also was computed. The length of the abutment was computed as the distance between the toe of the abutment and the edge of the water. The depth of flow blocked by the abutment was computed as the area of flow blocked by the abutment divided by the length of the abutment.

Comparison of Computed to Measured Historical Scour

The comparison between computed and measured historical scour should be viewed in light of the assumptions necessary to achieve comparable data. First, it is assumed that the historical scour measured by use of geophysical techniques is associated with the peak historical discharge. In an ideal situation, such as the laboratory, this would be a valid assumption; however, this assumption is suspect for the field. In the field, debris accumulations, ice jams, and other anomalies affect the depth of scour occurring at a given discharge. No data are available to document the conditions at the bridge at the time the measured scour occurred. It is possible, however, that the scour measured was associated with a lesser discharge and debris or ice accumulations. Second, the contraction-scour and pier-scour equations are combined to yield a computed bed elevation. The measurements made by use of geophysical techniques resulted in an estimated minimum streambed elevation in the vicinity of the piers. Because the extent of the scour holes could not be delineated from these data, it was not possible to separate contraction scour from local scour. Therefore, only total scour estimates could be compared directly with the geophysical measurements. The contraction scour computed by use of equations 2 and 5 was combined with the local scour resulting from each pier-scour equation and the surveyed bed elevation to obtain a computed bed elevation. This computed bed elevation was compared to the estimated historical bed elevation to evaluate the various equations. In this process, inaccuracies inherent in the contraction-scour equations are transferred to the pier-scour equations; therefore, the accuracy of the pier-scour equations could not be evaluated separately from the contraction-scour equations.

All of the pier-scour equations discussed herein were applied to each of the bridges for the hydraulic conditions that were estimated for the historical peak discharge. No grain-size information was available for the study sites; a single characteristic grain size for each site was estimated from the class descriptions indicated on boring logs that were available. Long-term scour was assumed to be zero. A plane bed was assumed at all piers.

The hydraulic variables estimated from WSPRO and the estimated grain size and angle of attack for each pier is shown at the top of tables 4—13. The contraction scour computed from Laursen's equations and the local pier scour computed from each of the selected pier-scour equations are shown near the center of tables 4—13. The computed bed elevation, shown near the bottom of tables 4—13, was computed by subtracting the contraction scour, pier scour, and approach depth from the water-surface elevation. The estimated historical bed elevation resulting from the geophysical measurements is shown at the bottom of tables 4—13. A summary of the differences between computed and historical bed elevation at the nose of the pier, which is where the theory assumes maximum scour will occur, is shown in table 14.

At U.S. Route 24 over Tippecanoe River, all of the equations computed scour that was greater than the estimates from the geophysical techniques (tables 4 and 14). Field investigations indicated a cobble and boulder layer at an elevation of about 601 ft, and there was no evidence to indicate that the scour had penetrated this layer. The median grain size was estimated to be 4 mm, which is a fine gravel. Therefore, one might expect equations based on sand beds to predict too much scour: for this situation, many of the equations predicted scour 10 ft greater than was estimated from the field data. Only Blench-Inglis II, Inglis-Lacey, and the Froehlich equations account for the grain size of the bed material; although these equations predicted bed elevations that were closest to the estimated historical elevations, they too over-predicted by 2 ft and greater. The Inglis-Lacey equation predicted excessive scour at pier 5, which was in relatively shallow water, and the Chitale and Ahmad equations predicted excessive scour for the piers in the main channel.

The channel upstream from the bridge on S.R. 32 over Buck Creek is approximately the same width as the bridge opening; however, contraction-scour computations predicted about 2 ft of deposition rather than scour in the main channel (table 5). The Laursen, Shen, and CSU equations predicted bed elevations that were within 2 ft of the bed elevation estimated from the geophysical data. The Inglis-Lacey equation predicted deposition at pier 2, and the Chitale and Ahmad equations predicted excessive scour for both piers.

The Blench-Inglis I and II, Inglis-Poona I and II, Larras, and Froehlich pier-scour equations predicted bed elevations that were within 4 ft of the estimated historical bed elevations at U.S. Route 41 over White River (tables 6 and 14). Again, the Inglis-Lacey equation predicted excessive local scour at piers in relatively shallow water, and the Ahmad equation predicted excessive scour for the piers in the main channel.

The contraction-scour equations predicted about 1 ft of deposition in the main channel and 3 to 4 ft of scour in the overbank areas at I-74 over Big Blue River (table 7). All of the local scour equations predicted in bed elevations higher than those estimated from the geophysical data,

Table 4. *Historical pier scour at U.S. Route 24 over Tippecanoe River at Monticello, Ind.*

[ft^3/s, cubic feet per second; all elevations refer to feet above sea level; ft/s, feet per second; mm, millimeters; --, no data or computations]

Hydraulic characteristic or equation used	Pier number			
	5	4	3	2
Total discharge, in ft^3/s	18,900	18,900	18,900	18,900
Water-surface elevation	617.2	617.2	617.2	617.2
Approach depth, in feet	3.9	14.0	12.3	0
Approach velocity, in ft/s	2.8	6.7	6.3	0
Angle of attack, in degrees	0	0	0	0
Estimated grain size, in mm	4.0	4.0	4.0	4.0
Computed depth of contraction scour, in feet				
Laursen	--	2.6	2.6	--
Computed depth of pier scour, in feet				
Ahmad	5.0	23.2	20.5	--
Blench-Inglis I	3.6	6.3	6.2	--
Blench-Inglis II	2.5	5.5	5.5	--
Chitale	3.2	14.6	12.9	--
CSU	5.0	9.8	9.4	--
Froehlich	1.7	3.6	3.5	--
Inglis-Lacey	12.6	2.5	4.2	--
Inglis-Poona I	4.5	13.1	12.2	--
Inglis-Poona II	3.2	6.1	5.9	--
Larras	4.7	5.4	5.4	--
Laursen	4.3	9.1	8.5	--
Shen	4.2	8.0	7.7	--
Shen-Maza	6.2	12.5	12.0	--
Computed elevation of bed at nose of pier				
Ahmad	608.3	577.4	581.8	--
Blench-Inglis I	609.7	594.3	596.1	--
Blench-Inglis II	610.8	595.1	596.8	--
Chitale	610.1	586.0	589.4	--
CSU	608.3	590.8	592.9	--
Froehlich	611.6	597.0	598.8	--
Inglis-Lacey	600.7	598.1	598.1	--
Inglis-Poona I	608.8	587.5	590.1	--
Inglis-Poona II	610.1	594.5	596.4	--
Larras	608.6	595.2	596.9	--
Laursen	609.0	591.5	593.8	--
Shen	609.1	592.6	594.6	--
Shen-Maza	607.1	588.1	590.3	--
Estimated historical elevation of bed from geophysical measurements				
At nose of pier	--	601	601	601
Maximum depth	--	601	601	601

Table 5. *Historical pier scour at State Road 32 over Buck Creek at Yorktown, Ind.*

[ft^3/s, cubic feet per second; all elevations refer to feet above sea level; ft/s, feet per second; mm, millimeters; --, no data or computations]

Hydraulic characteristic or equation used	Pier number 2	Pier number 3
Total discharge, in ft^3/s	5,000	5,000
Water-surface elevation	900.7	900.7
Approach depth, in feet	11.5	10.1
Approach velocity, in ft/s	6.5	6.2
Angle of attack, in degrees	0	0
Estimated grain size, in mm	4.0	4.0
Computed depth of contraction scour, in feet		
Laursen	-1.7	-1.7
Computed depth of pier scour, in feet		
Ahmad	20.5	18.3
Blench-Inglis I	4.1	4.0
Blench-Inglis II	4.0	4.0
Chitale	12.8	11.4
CSU	6.9	6.6
Froehlich	2.5	2.4
Inglis-Lacey	-.9	.5
Inglis-Poona I	10.1	9.6
Inglis-Poona II	4.0	3.9
Larras	3.8	3.8
Laursen	6.5	6.0
Shen	5.8	5.6
Shen-Maza	8.9	8.6
Computed elevation of bed at nose of pier		
Ahmad	870.4	874.0
Blench-Inglis I	886.8	888.3
Blench-Inglis II	886.9	888.3
Chitale	878.1	880.9
CSU	884.0	885.7
Froehlich	888.4	889.9
Inglis-Lacey	891.8	891.8
Inglis-Poona I	880.8	882.7
Inglis-Poona II	886.9	888.4
Larras	887.1	888.5
Laursen	884.4	886.3
Shen	885.1	886.7
Shen-Maza	882.0	883.7
Estimated historical elevation of bed from geophysical measurements		
At nose of pier	886	886
Maximum depth	886	886

Table 6. *Historical pier scour at U.S. Route 41 over White River near Hazleton, Ind.*

[ft^3/s, cubic feet per second; all elevations refer to feet above sea level; ft/s, feet per second; mm, millimeters; --, no data or computations]

Hydraulic characteristic or equation used	Pier number 1	2	3	4	5	6	7	8
Total discharge, in ft^3/s	133,400	133,400	133,400	133,400	133,400	133,400	133,400	133,400
Water-surface elevation	410.9	410.9	410.9	410.9	410.9	410.9	410.9	410.9
Approach depth, in feet	10.7	11.1	9.9	10.7	30.2	29.8	29.4	13.4
Approach velocity, in ft/s	2.7	2.7	2.6	3.1	5.7	5.6	5.7	1.9
Angle of attack, in degrees	0	0	0	0	0	0	0	0
Estimated grain size, in mm	.5	.5	.5	.5	.5	.5	.5	.5
Computed depth of contraction scour, in feet								
Laursen	4.3	4.3	4.3	4.3	6.9	6.9	6.9	3.3
Computed depth of pier scour, in feet								
Ahmad	6.3	6.3	5.8	7.9	25.6	24.9	25.4	2.2
Blench-Inglis I	5.0	5.0	4.9	5.0	3.9	3.9	4.0	5.1
Blench-Inglis II	2.5	2.4	2.6	3.5	2.1	2.0	2.5	-1.0
Chitale	3.7	3.6	3.4	4.8	15.8	15.3	15.7	.7
CSU	5.5	5.5	5.3	5.8	8.7	8.6	8.6	4.8
Froehlich	2.8	2.8	2.7	2.8	4.7	4.7	4.7	2.8
Inglis-Lacey	34.1	33.7	34.9	34.1	14.6	15.0	15.4	31.4
Inglis-Poona I	3.2	3.0	3.1	4.2	5.0	4.9	5.3	-.4
Inglis-Poona II	4.7	4.8	4.6	4.7	4.5	4.5	4.6	5.0
Larras	4.5	4.5	4.5	4.5	4.5	4.5	4.5	4.5
Laursen	7.0	7.1	6.7	7.0	11.8	11.7	11.7	7.8
Shen	3.9	3.9	3.8	4.3	6.2	6.1	6.2	3.1
Shen-Maza	5.8	5.8	5.6	6.3	9.5	9.4	9.5	1.2
Computed elevation of bed at nose of pier								
Ahmad	389.6	389.2	390.9	388.0	348.2	349.3	349.2	392.0
Blench-Inglis I	390.9	390.5	391.8	390.9	369.9	370.3	370.6	389.1
Blench-Inglis II	393.4	393.1	394.1	392.4	371.7	372.2	372.1	395.2
Chitale	392.2	391.9	393.3	391.1	358.0	358.9	358.9	393.5
CSU	390.4	390.0	391.4	390.1	365.1	365.6	366.0	389.4
Froehlich	393.1	392.7	394.0	393.1	369.1	369.5	369.9	391.4
Inglis-Lacey	361.8	361.8	361.8	361.8	359.2	359.2	359.2	362.8
Inglis-Poona I	392.7	392.5	393.6	391.7	368.8	369.3	369.3	394.6
Inglis-Poona II	391.2	390.7	392.1	391.2	369.3	369.7	370.0	389.2
Larras	391.4	391.0	392.2	391.4	369.3	369.7	370.1	389.7
Laursen	388.9	388.4	390.0	388.9	362.0	362.5	362.9	386.4
Shen	392.0	391.6	392.9	391.6	367.6	368.1	368.4	391.1
Shen-Maza	390.1	389.7	391.1	389.6	364.3	364.8	365.1	393.0
Estimated historical elevation of bed from geophysical measurements								
At nose of pier	--	--	--	--	--	373	372	--
Maximum depth	--	--	--	--	369	373	372	--

Table 6. *Historical pier scour at U.S. Route 41 over White River near Hazleton, Ind.*--Continued

Hydraulic characteristic or equation used	Pier number 9	10	11	12	13	14	15
Total discharge, in ft^3/s	133,400	133,400	133,400	133,400	133,400	133,400	133,400
Water-surface elevation	410.9	410.9	410.9	410.9	410.9	410.9	410.9
Approach depth, in feet	7.4	8.5	9.4	10.5	11.6	12.6	11.1
Approach velocity, in ft/s	1.9	2.4	2.4	2.8	2.8	2.9	2.8
Angle of attack, in degrees	0	0	0	0	0	0	0
Estimated grain size, in mm	.5	.5	.5	.5	.5	.5	.5
Computed depth of contraction scour, in feet							
Laursen	3.3	3.3	3.3	3.3	3.3	3.3	3.3
Computed depth of pier scour, in feet							
Ahmad	3.1	5.0	5.0	6.7	6.7	7.2	6.7
Blench-Inglis I	4.5	4.7	4.8	4.9	5.0	5.1	5.0
Blench-Inglis II	1.8	2.6	2.3	2.8	2.4	2.3	2.6
Chitale	1.7	2.9	2.8	3.9	3.9	4.2	3.9
CSU	4.5	5.0	5.1	5.5	5.6	5.8	5.6
Froehlich	2.4	2.5	2.7	2.8	3.0	3.0	2.8
Inglis-Lacey	37.4	36.3	35.4	34.3	33.2	32.2	33.7
Inglis-Poona I	2.1	3.1	2.8	3.5	3.1	3.1	3.3
Inglis-Poona II	4.2	4.4	4.6	4.7	4.9	4.9	4.8
Larras	4.5	4.5	4.5	4.5	4.5	4.5	4.5
Laursen	5.8	6.2	6.6	6.9	7.3	7.6	7.1
Shen	3.1	3.6	3.6	4.0	4.0	4.1	4.0
Shen-Maza	1.2	2.0	2.0	5.9	5.9	6.1	5.9
Computed elevation of bed at nose of pier							
Ahmad	397.1	394.1	393.2	390.4	389.3	387.8	389.8
Blench-Inglis I	395.7	394.4	393.4	392.2	391.0	389.9	391.5
Blench-Inglis II	398.4	396.5	395.9	394.3	393.6	392.7	393.9
Chitale	398.5	396.2	395.4	393.2	392.1	390.8	392.6
CSU	395.7	394.1	393.1	391.6	390.4	389.2	390.9
Froehlich	397.8	396.6	395.5	394.3	393.0	392.0	393.7
Inglis-Lacey	362.8	362.8	362.8	362.8	362.8	362.8	362.8
Inglis-Poona I	398.1	396.0	395.4	393.6	392.9	391.9	393.2
Inglis-Poona II	396.0	394.7	393.6	392.4	391.1	390.1	391.7
Larras	395.7	394.6	393.7	392.6	391.5	390.5	392.0
Laursen	394.4	392.9	391.6	390.2	388.7	387.4	389.4
Shen	397.1	395.5	394.6	393.1	392.0	390.9	392.5
Shen-Maza	399.0	397.1	396.2	391.2	390.1	388.9	390.6
Estimated historical elevation of bed from geophysical measurements							
At nose of pier	--	--	--	--	--	--	--
Maximum depth	--	--	--	--	--	--	--

Table 7. *Historical pier scour at I-74 over Big Blue River near Shelbyville, Ind.*

[ft^3/s, cubic feet per second; all elevations refer to feet above sea level; ft/s, feet per second; mm, millimeters; --, no data or computations]

Hydraulic characteristic or equation used	Pier number 2	3	4	5	6	7
Total discharge, in ft^3/s	14,700	14,700	14,700	14,700	14,700	14,700
Water-surface elevation	764.9	764.9	764.9	764.9	764.9	764.9
Approach depth, in feet	5.3	7.3	13.6	14.9	7.0	2.0
Approach velocity, in ft/s	2.3	3.1	5.6	5.8	1.9	1.9
Angle of attack, in degrees	0	0	0	0	0	0
Estimated grain size, in mm	.5	.5	.5	.5	.5	.5
Computed depth of contraction scour, in feet						
Laursen	4.2	4.2	-1.3	-1.3	3.2	3.2
Computed depth of pier scour, in feet						
Ahmad	4.2	7.1	18.8	20.3	3.1	2.4
Blench-Inglis I	3.3	3.6	3.8	3.7	3.6	2.1
Blench-Inglis II	2.7	3.6	6.4	6.4	1.3	2.5
Chitale	2.6	4.5	11.9	12.9	1.7	1.5
CSU	3.8	4.6	6.4	6.6	3.7	3.1
Froehlich	1.7	2.1	3.0	3.2	1.9	1.3
Inglis-Lacey	16.2	14.2	7.9	6.6	14.5	19.5
Inglis-Poona I	3.0	4.1	8.0	8.1	1.7	2.5
Inglis-Poona II	3.1	3.4	3.9	3.8	3.4	1.9
Larras	3.6	3.6	3.6	3.6	3.6	3.6
Laursen	4.2	5.0	6.8	7.2	4.9	2.6
Shen	3.0	3.6	5.1	5.2	2.6	2.6
Shen-Maza	4.3	5.2	7.8	7.9	1.2	1.2
Computed elevation of bed at nose of pier						
Ahmad	751.2	746.3	733.8	731.0	751.6	757.3
Blench-Inglis I	752.1	749.8	748.8	747.6	751.1	757.6
Blench-Inglis II	752.7	749.8	746.2	744.9	753.4	757.2
Chitale	752.8	748.9	740.7	738.4	753.0	758.2
CSU	751.6	748.8	746.2	744.7	751.0	756.6
Froehlich	753.7	751.3	749.6	748.1	752.8	758.4
Inglis-Lacey	739.2	739.2	744.7	744.7	740.2	740.2
Inglis-Poona I	752.4	749.3	744.6	743.2	753.0	757.2
Inglis-Poona II	752.3	750.0	748.7	747.5	751.3	757.8
Larras	751.8	749.8	749.0	747.7	751.1	756.1
Laursen	751.2	748.4	745.8	744.1	749.8	757.1
Shen	752.4	749.8	747.5	746.1	752.1	757.1
Shen-Maza	751.1	748.2	744.8	743.4	753.5	758.5
Estimated historical elevation of bed from geophysical measurements						
At nose of pier	--	--	738–744	738–744	--	--
Maximum depth	--	--	738	738	--	--

Table 8. *Historical pier scour at I-74 over Whitewater River near Harrison, Oh.*

[ft^3/s, cubic feet per second; all elevations refer to feet above sea level; ft/s, feet per second; mm, millimeters; --, no data or computations]

Hydraulic characteristic or equation used	Pier number						
	16	15	14	13	12	11	10
Total discharge, in ft^3/s	67,200	67,200	67,200	67,200	67,200	67,200	67,200
Water-surface elevation	533.1	533.1	533.1	533.1	533.1	533.1	533.1
Approach depth, in feet	0	18.0	15.9	11.5	9.6	0	0
Approach velocity, in ft/s	0	9.3	8.8	3.9	2.5	0	0
Angle of attack, in degrees	0	0	0	0	0	0	0
Estimated grain size, in mm	.5	.5	.5	.5	.5	.5	.5
Computed depth of contraction scour, in feet							
Laursen	--	5.7	5.7	8.0	8.0	--	--
Computed depth of pier scour, in feet							
Ahmad	--	36.8	32.7	11.3	5.4	--	--
Blench-Inglis I	--	4.2	4.3	4.4	4.3	--	--
Blench-Inglis II	--	12.6	12.1	4.3	2.0	--	--
Chitale	--	22.3	19.8	7.0	3.1	--	--
CSU	--	9.0	8.6	5.8	4.7	--	--
Froehlich	--	4.0	3.8	2.8	2.4	--	--
Inglis-Lacey	--	17.6	19.7	24.1	26.0	--	--
Inglis-Poona I	--	15.5	14.6	5.3	2.5	--	--
Inglis-Poona II	--	4.4	4.4	4.3	4.1	--	--
Larras	--	4.0	4.0	4.0	4.0	--	--
Laursen	--	8.4	7.9	6.7	6.1	--	--
Shen	--	7.6	7.4	4.4	3.4	--	--
Shen-Maza	--	11.9	11.5	6.6	4.9	--	--
Computed elevation of bed at nose of pier							
Ahmad	--	472.6	478.8	502.3	510.1	--	--
Blench-Inglis I	--	505.2	507.2	509.2	511.2	--	--
Blench-Inglis II	--	496.8	499.4	509.3	513.5	--	--
Chitale	--	487.1	491.7	506.6	512.4	--	--
CSU	--	500.4	502.9	507.8	510.8	--	--
Froehlich	--	505.4	507.7	510.8	513.1	--	--
Inglis-Lacey	--	491.8	491.8	489.5	489.5	--	--
Inglis-Poona I	--	493.9	496.9	508.3	513.0	--	--
Inglis-Poona II	--	505.0	507.1	509.3	511.4	--	--
Larras	--	505.4	507.5	509.6	511.5	--	--
Laursen	--	501.0	503.6	506.9	509.4	--	--
Shen	--	501.8	504.1	509.2	512.1	--	--
Shen-Maza	--	497.5	500.0	507.0	510.6	--	--
Estimated historical elevation of bed from geophysical measurements							
At nose of pier	--	492	509	--	--	--	--
Maximum depth	--	490	509	--	--	--	--

Table 9. *Historical pier scour at U.S. Route 231 over Kankakee River near Hebron, Ind.*

[ft^3/s, cubic feet per second; all elevations refer to feet above sea level; ft/s, feet per second; mm, millimeters; --, no data or computations]

Hydraulic characteristic or equation used	Pier number 2	3	4	5
Total discharge, in ft^3/s	5,600	5,600	5,600	5,600
Water-surface elevation	647.5	647.5	647.5	647.5
Approach depth, in feet	8.5	9.1	9.9	9.9
Approach velocity, in ft/s	3.5	3.7	3.8	3.8
Angle of attack, in degrees	0	0	0	0
Estimated grain size, in mm	1	1	1	1
Computed depth of contraction scour, in feet				
Laursen	0.6	0.6	0.6	0.6
Computed depth of pier scour, in feet				
Ahmad	8.8	9.7	10.3	10.3
Blench-Inglis I	2.1	2.1	2.0	2.0
Blench-Inglis II	1.4	1.5	1.3	1.3
Chitale	5.5	6.1	6.5	6.5
CSU	3.4	3.5	3.6	3.6
Froehlich	1.6	1.6	1.6	1.6
Inglis-Lacey	5.4	4.8	4.0	4.0
Inglis-Poona I	3.2	3.3	3.3	3.3
Inglis-Poona II	2.2	2.2	2.1	2.1
Larras	2.4	2.4	2.4	2.4
Laursen	4.1	4.2	4.4	4.4
Shen	2.7	2.8	2.8	2.8
Shen-Maza	3.9	4.0	4.1	4.1
Computed elevation of bed at nose of pier				
Ahmad	629.6	628.1	626.7	626.7
Blench-Inglis I	636.3	635.7	635.0	635.0
Blench-Inglis II	637.0	636.3	635.7	635.7
Chitale	632.9	631.7	630.5	630.5
CSU	635.0	634.3	633.4	633.4
Froehlich	636.8	636.2	635.4	635.4
Inglis-Lacey	633.0	633.0	633.0	633.0
Inglis-Poona I	635.2	634.5	633.7	633.7
Inglis-Poona II	636.2	635.6	634.9	634.9
Larras	636.0	635.4	634.6	634.6
Laursen	634.3	633.6	632.6	632.6
Shen	635.7	635.0	634.2	634.2
Shen-Maza	634.5	633.8	632.9	632.9
Estimated historical elevation of bed from geophysical measurements				
At nose of pier	--	629	--	--
Maximum depth	633	629	631	631

Table 10. *Historical pier scour at U.S. Route 231 over East Fork White River near Haysville, Ind.*

[ft^3/s, cubic feet per second; all elevations refer to feet above sea level; ft/s, feet per second; mm, millimeters; --, no data or computations]

Hydraulic characteristic or equation used	Pier number 2	Pier number 3	Pier number 4
Total discharge, in ft^3/s	109,200	109,200	109,200
Water-surface elevation	452.0	452.0	452.0
Approach depth, in feet	32.0	39.1	16.6
Approach velocity, in ft/s	6.7	7.9	4.2
Angle of attack, in degrees	25	25	25
Estimated grain size, in mm	.2	.2	.2
Computed depth of contraction scour, in feet			
Laursen	10.0	13.7	23.1
Computed depth of pier scour, in feet			
Ahmad	32.6	43.3	13.9
Blench-Inglis I	7.8	7.2	7.8
Blench-Inglis II	13.3	15.2	9.2
Chitale	20.4	27.3	8.6
CSU	25.5	28.3	19.0
Froehlich	14.8	16.6	10.6
Inglis-Lacey	16.8	9.7	32.2
Inglis-Poona I	11.6	13.7	7.7
Inglis-Poona II	8.1	7.8	7.4
Larras	6.4	6.4	6.4
Laursen	34.0	37.9	24.3
Shen	9.1	10.1	6.8
Shen-Maza	14.4	16.1	10.5
Computed elevation of bed at nose of pier			
Ahmad	377.4	355.9	398.4
Blench-Inglis I	402.2	392.0	404.5
Blench-Inglis II	396.7	384.0	403.1
Chitale	389.6	371.9	403.7
CSU	384.5	370.9	393.3
Froehlich	395.2	382.6	401.7
Inglis-Lacey	393.2	389.5	380.1
Inglis-Poona I	398.4	385.5	404.6
Inglis-Poona II	401.9	391.4	404.9
Larras	403.6	392.8	405.9
Laursen	376.0	361.3	388.0
Shen	400.9	389.1	405.5
Shen-Maza	395.6	383.1	401.8
Estimated historical elevation of bed from geophysical measurements			
At nose of pier	--	--	--
Maximum depth	402	402	--

Table 11. *Historical pier scour at State Road 258 over East Fork White River near Seymour, Ind.*

[ft^3/s, cubic feet per second; all elevations refer to feet above sea level; ft/s, feet per second; mm, millimeters; --, no data or computations]

Hydraulic characteristic or equation used	Pier number 7	6	5	4	3	2
Total discharge, in ft^3/s	47,000	47,000	47,000	47,000	47,000	47,000
Water-surface elevation	564.8	564.8	564.8	564.8	564.8	564.8
Approach depth, in feet	2.1	7.8	15.6	15.8	5.1	1.7
Approach velocity, in ft/s	3.0	5.0	6.0	6.6	1.6	1.6
Angle of attack, in degrees	0	0	0	0	0	0
Estimated grain size, in mm	1.0	1.0	1.0	1.0	1.0	1.0
Computed depth of contraction scour, in feet						
Laursen	-0.1	-0.1	10.1	10.1	-0.7	-0.7
Computed depth of pier scour, in feet						
Ahmad	4.0	12.9	21.6	24.1	2.2	1.8
Blench-Inglis I	1.8	2.7	2.1	2.1	2.6	1.7
Blench-Inglis II	2.7	4.2	3.1	3.9	.4	1.5
Chitale	2.5	8.1	13.7	15.3	1.2	1.1
CSU	3.0	4.5	5.4	5.6	2.6	2.3
Froehlich	.9	1.9	2.4	2.5	1.3	.8
Inglis-Lacey	26.1	20.4	12.6	12.4	23.1	26.5
Inglis-Poona I	3.3	6.3	6.7	7.8	1.1	1.8
Inglis-Poona II	1.7	2.7	2.4	2.4	2.4	1.5
Larras	2.8	2.8	2.8	2.8	2.8	2.8
Laursen	2.2	4.4	6.2	6.2	3.5	2.0
Shen	2.8	3.9	4.3	4.6	1.9	1.9
Shen-Maza	4.1	5.7	6.5	6.9	.9	.9
Computed elevation of bed at nose of pier						
Ahmad	558.8	544.2	517.5	514.8	558.2	562.0
Blench-Inglis I	561.0	554.4	537.0	536.8	557.8	562.1
Blench-Inglis II	560.1	552.9	536.0	535.0	560.0	562.3
Chitale	560.3	549.0	525.4	523.6	559.2	562.7
CSU	559.8	552.6	533.7	533.3	557.8	561.5
Froehlich	561.9	555.2	536.7	536.4	559.1	563.0
Inglis-Lacey	536.7	536.7	526.5	526.5	537.3	537.3
Inglis-Poona I	559.5	550.8	532.4	531.1	559.3	562.0
Inglis-Poona II	561.1	554.4	536.7	536.5	558.0	562.3
Larras	560.0	554.3	536.3	536.1	557.6	561.0
Laursen	560.6	552.7	532.9	532.7	556.9	561.8
Shen	560.0	553.2	534.8	534.3	558.5	561.9
Shen-Maza	558.7	551.4	532.6	532.0	559.5	562.9
Estimated historical elevation of bed from geophysical measurements						
At nose of pier	--	--	542	--	--	--
Maximum depth	--	--	542	542	--	--

Table 12. *Historical pier scour at State Road 331 over Tippecanoe River at Old Tip Town, Ind.*

[ft^3/s, cubic feet per second; all elevations refer to feet above sea level; ft/s, feet per second; mm, millimeters; --, no data or computations]

Hydraulic characteristic or equation used	Pier number 2	Pier number 3
Total discharge, in ft^3/s	1,650	1,650
Water-surface elevation	769.0	769.0
Approach depth, in feet	7.1	7.2
Approach velocity, in ft/s	2.2	2.2
Angle of attack, in degrees	0	0
Estimated grain size, in mm	1.0	1.0
Computed depth of contraction scour, in feet		
Laursen	1.0	1.0
Computed depth of pier scour, in feet		
Ahmad	4.2	4.2
Blench-Inglis I	2.2	2.2
Blench-Inglis II	.1	.1
Chitale	2.4	2.4
CSU	2.7	2.7
Froehlich	1.3	1.3
Inglis-Lacey	2.1	2.0
Inglis-Poona I	1.2	1.2
Inglis-Poona II	2.2	2.2
Larras	2.4	2.4
Laursen	3.7	3.8
Shen	2.0	2.0
Shen-Maza	2.9	2.9
Computed elevation of bed at nose of pier		
Ahmad	756.7	756.6
Blench-Inglis I	758.7	758.6
Blench-Inglis II	760.8	760.7
Chitale	758.5	758.4
CSU	758.2	758.1
Froehlich	759.6	759.5
Inglis-Lacey	758.8	758.8
Inglis-Poona I	759.7	759.6
Inglis-Poona II	758.7	758.6
Larras	758.5	758.4
Laursen	757.2	757.0
Shen	758.9	758.8
Shen-Maza	758.0	757.9
Estimated historical elevation of bed from geophysical measurements		
At nose of pier	753	751
Maximum depth	753	751

Table 13. *Historical pier scour at State Road 358 over White River near Edwardsport, Ind.*

[ft^3/s, cubic feet per second; all elevations refer to feet above sea level; ft/s, feet per second; mm, millimeters; --, no data or computations]

Hydraulic characteristic or equation used	Pier number 5	4	3	2
Total discharge, in ft^3/s	45,600	45,600	45,600	45,600
Water-surface elevation	452.3	452.3	452.3	452.3
Approach depth, in feet	5.3	19.1	18.3	26.6
Approach velocity, in ft/s	2.4	8.3	8.5	5.5
Angle of attack, in degrees	0	0	0	0
Estimated grain size, in mm	.4	.4	.4	.4
Computed depth of contraction scour, in feet				
Laursen	4.9	-0.6	-0.6	-0.6
Computed depth of pier scour, in feet				
Ahmad	4.5	33.7	33.8	23.5
Blench-Inglis I	3.3	3.3	3.4	2.2
Blench-Inglis II	3.1	10.5	11.0	1.8
Chitale	2.8	21.0	21.0	14.5
CSU	3.9	7.9	8.0	6.9
Froehlich	1.7	3.6	3.6	3.8
Inglis-Lacey	27.2	13.4	14.2	5.9
Inglis-Poona I	3.2	12.5	13.0	3.7
Inglis-Poona II	3.1	3.6	3.7	2.9
Larras	3.6	3.6	3.6	3.6
Laursen	4.2	8.1	7.9	9.6
Shen	3.0	6.5	6.6	5.1
Shen-Maza	4.4	10.1	10.3	7.7
Computed elevation of bed at nose of pier				
Ahmad	437.6	400.1	400.8	402.8
Blench-Inglis I	438.8	430.5	431.2	424.1
Blench-Inglis II	439.0	423.3	423.6	424.5
Chitale	439.3	412.8	413.6	411.8
CSU	438.2	425.9	426.6	419.4
Froehlich	440.4	430.2	431.0	422.5
Inglis-Lacey	414.9	420.4	420.4	420.4
Inglis-Poona I	438.9	421.3	421.6	422.6
Inglis-Poona II	439.0	430.2	430.9	423.4
Larras	438.5	430.2	431.0	422.7
Laursen	437.9	425.7	426.7	416.7
Shen	439.1	427.3	428.0	421.2
Shen-Maza	437.7	423.7	424.3	418.6
Estimated historical elevation of bed from geophysical measurements				
At nose of pier	--	--	--	421
Maximum depth	--	--	--	420

Table 14. *Comparison of computed and historical bed elevations*

[U.S., United States; S.R., State Road; I, Interstate; F.S., factor of safety]

Site, magnitude, or category	Pier	Difference between the computed bed elevation and measured historic bed elevation (computed - historic), in feet, for given equation													
		Ahmad	Blench-Inglis I	Blench-Inglis II	Chitale	CSU	Froehlich	Froehlich with F.S.	Inglis-Lacey	Inglis-Poona I	Inglis-Poona II	Larras	Laursen	Shen	Shen-Maza
U.S. Route 24 over	4	-23.6	-6.7	-5.9	-15.0	-10.2	-4.0	-10.0	-2.9	-13.5	-6.5	-5.8	-9.5	-8.4	-12.9
Tippecanoe River	3	-19.2	-4.9	-4.2	-11.6	-8.1	-2.2	-8.2	-2.9	-10.9	-4.6	-4.1	-7.2	-6.4	-10.7
S.R. 32 over	2	-15.6	.8	.9	-7.9	-2.0	2.4	-1.3	5.8	-5.2	.9	1.1	-1.6	-.9	-4.0
Buck Creek River	3	-12.0	2.3	2.3	-5.1	-.3	3.9	.2	5.8	-3.3	2.4	2.5	.3	.7	-2.3
U.S. 41 over	6	-23.7	-2.7	-.8	-14.1	-7.4	-3.5	-8.2	-13.8	-3.7	-3.3	-3.3	-10.5	-4.9	-8.2
White River	7	-22.8	-1.4	.1	-13.1	-6	-2.1	-6.8	-12.8	-2.7	-2.0	-1.9	-9.1	-3.6	-6.9
I-74 over	4	-4.2	10.8	8.2	2.7	8.2	11.6	8.1	6.7	6.6	10.7	11.0	7.8	9.5	6.8
Big Blue River	5	-7.0	9.6	6.9	.4	6.7	10.1	6.6	6.7	5.2	9.5	9.7	6.1	8.1	5.4
I-74 over	15	-19.4	13.2	4.8	-4.9	8.4	13.4	9.4	-.2	1.9	13.0	13.4	9.0	9.8	5.5
Whitewater River	14	-30.2	-1.8	-9.6	-17.3	-6.1	-1.3	-5.3	-17.2	-12.1	-1.9	-1.5	-5.4	-4.9	-9.0
U.S. 231 over Kankakee River	3	-.9	6.7	7.3	2.7	5.3	7.2	5.2	4.0	5.5	6.6	6.4	4.6	6.0	4.8
S.R. 258 over E. Fork White River	5	-24.5	-5.0	-6.0	-16.6	-8.3	-5.3	-7.8	-15.5	-9.6	-5.3	-5.7	-9.1	-7.2	-9.4
S.R. 331 over	2	3.7	5.7	7.8	5.5	5.2	6.6	4.6	5.8	6.7	5.7	5.5	4.2	5.9	5.0
Tippecanoe River	3	5.6	7.6	9.7	7.4	7.1	8.5	6.5	7.8	8.6	7.6	7.4	6.0	7.8	6.9
S.R. 358 over White River	2	-18.2	3.1	3.5	-9.2	-1.6	1.5	-1.0	-.6	1.6	2.4	1.7	-4.3	.2	-2.4
		Number of occurrences													
Greater than 10 feet		10	2	0	6	1	3	1	4	3	2	2	1	0	2
10 to 5 feet		2	6	8	5	11	4	10	6	7	6	6	8	9	9
Less than 5 feet		3	7	7	4	3	8	4	5	5	7	7	5	6	4
Scour underestimated		2	9	10	5	6	9	7	7	7	9	9	7	8	6
Scour overestimated		13	6	5	10	9	6	8	8	8	6	6	8	7	9

except for the Ahmad equation, which appeared to have predicted excessive scour. Therefore, the primary problem at this site may be the contraction-scour equations. The Inglis-Lacey equation again predicted excessive scour in shallow water.

The WSPRO model computed very similar hydraulic conditions at piers 14 and 15 of I-74 over Whitewater River; however, the estimated historical scour differed by more than 15 ft for these two piers (table 8). Because of the similarity of the hydraulic conditions, no single equation matched the bed elevations at both piers. All of the equations overestimated the scour at pier 14, although the Froehlich equation predicted the most reasonable bed elevation (table 14). Conversely, most of the equations underestimated the scour at pier 15, although the Inglis-Lacey equation predicted the computed bed elevation nearest to the historical elevation. On the basis of field observations and measurements, the scour at pier 15 is believed to be the result of a debris accumulation, a condition that is not reflected in the hydraulic parameters used in the scour equations. The maximum scour at pier 15 actually occurred 25 ft upstream from the pier.

At U.S. Route 231 over Kankakee River, all of the equations, with the exception of the Ahmad equation, underestimated the historical scour (tables 9 and 14). The Ahmad and Chitale equations were the only equations that predicted bed elevations within 3 ft of the estimated historical bed elevation.

At U.S. Route 231 over East Fork White River, results from the contraction-scour equations again appear to be suspect. Although the main-channel contraction scour of 13.7 ft is large, this is a sand-bed channel, and the historical flood was in excess of the 50-year flood (table 10). The 10 and 23 ft of contraction scour computed for the left and right overbank seems excessive, however. At pier 2, the Blench-Inglis I, Shen, Larras, and Froehlich pier-scour equations predicted bed elevations that were within 2 ft of the estimated historical bed elevation (table 10); at pier 3, however, almost all of the equations overpredicted the scour by at least 10 ft.

The problems associated with estimating the hydraulic conditions for the complex geometry at S. R. 258 over East Fork White River were discussed previously. All of the equations overestimated scour, and resulting bed elevations were at least 5 ft lower than the estimated historical bed elevations (tables 11 and 14); this discrepancy may be due to the poor estimate of hydraulic conditions.

All of the equations underestimated the scour at S.R. 331 over Tippecanoe River. Because all of the equations underestimated the scour, the estimated contraction scour or hydraulics could be inaccurate. No explanation for the consistent underestimation is obvious from the data collected for this study.

The problems associated with estimating the hydraulic conditions for the complex geometry at S.R. 358 over White River (discussed previously) may be the cause of the small amount of deposition predicted when the contraction-scour equations were applied in the main channel. Use of the Froehlich, Inglis-Lacey, Inglis-Poona I, Shen, Larras, and CSU equations resulted in bed elevations that were within 2 ft of the estimated historical bed elevation (tables 13 and 14). The Inglis-Lacey equation predicted excessive local scour at piers in relatively shallow water, and the Chitale and Ahmad equations predicted excessive scour for piers in the main channel.

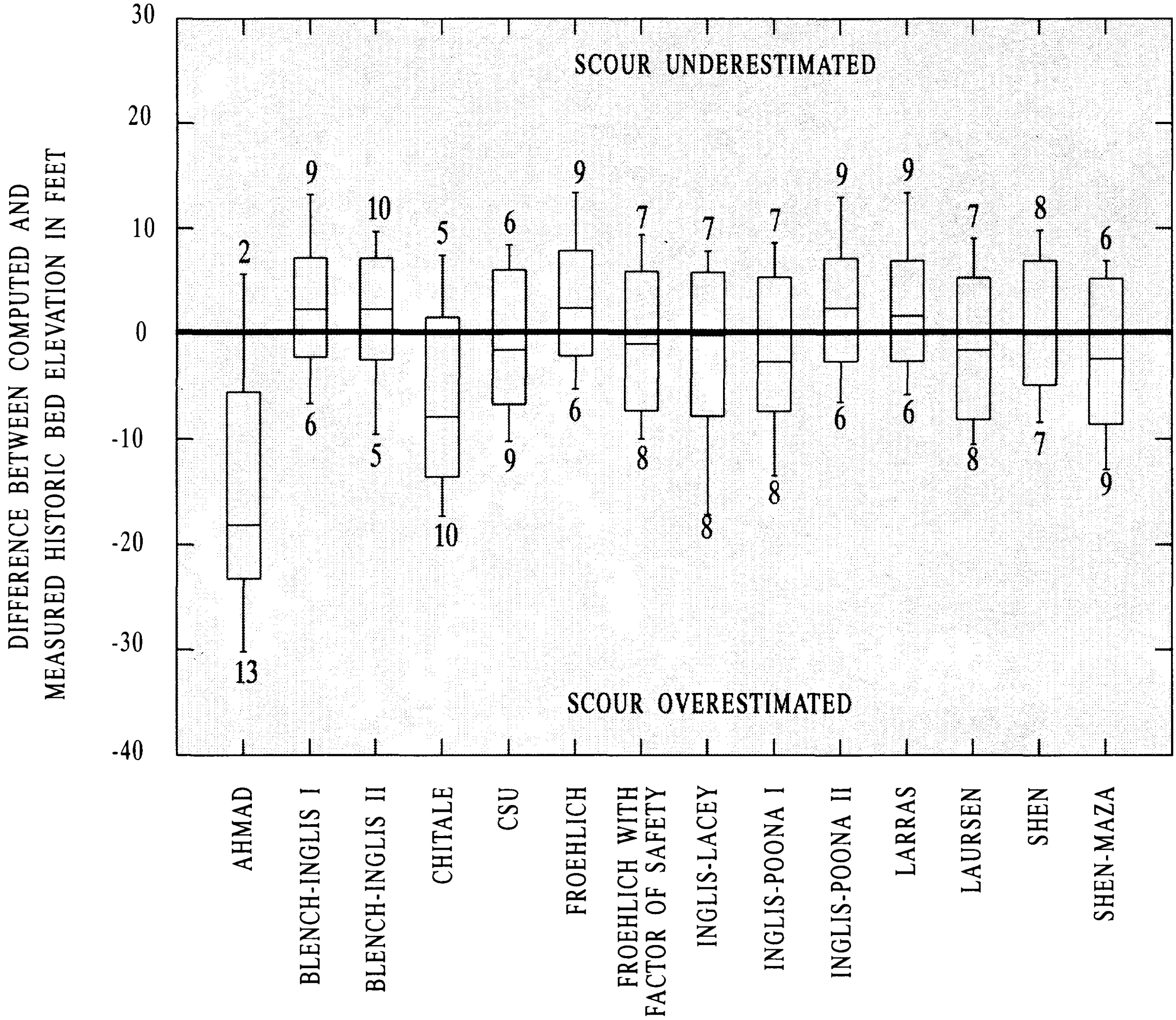
Only the Inglis-Lacey, Chitale, and Ahmad pier-scour equations commonly produced results that were grossly different from the historical data. The Inglis-Lacey equation consistently predicted excessive scour when applied to piers in the shallow overbank areas. This poor performance in shallow areas is not due to the difference between clear-water and live-bed scour but rather to the inclusion of the total discharge rather than velocity or unit discharge as a variable in the equation. The Chitale equation commonly predicted excessive scour at the piers in the main channel. This equation, however, is based on model experiments for one bridge and uses only the Froude number and depth of flow as variables; the size and configuration of the pier is not considered. The Ahmad equation also predicted excessive scour at piers in the main channel. The problems associated with this equation are: (1) the equation is not dimensionally homogeneous, and (2) no guidance was provided for selection of the coefficient, K. Because K is a function of boundary geometry, abutment shape, width of piers, shape of piers, and the angle of the approach flow, it should differ for each bridge. No formula was provided to determine which K to use, only a range of 1.7 to 2.0, from which 1.8 was selected and applied uniformly to all the sites. Although no data were available to evaluate the contraction-scour equation, a few of the contraction-scour computations predicted what seem to be excessive scour, especially in clear-water situations.

For bridge design, it is desirable to use an equation that estimates the depth of scour accurately but when in error tends to overestimate the depth of scour. Because this is a very small data set, one must be careful when drawing conclusions. Table 14 groups the magnitude of the difference between computed and historical bed elevations into three categories (differences greater than 10 ft, differences from 10 to 5 ft inclusive, and differences less than 5 ft) and shows how often each equation overpredicted or underpredicted the total scour indicated by the historical data. These same data are also displayed using a box plot in figure 28. Given the general belief that laboratory equations overestimate scour in the field, it is surprising that approximately half of the computations underestimated the scour measured by use of geophysical techniques. The Froehlich pier-scour equation, including a factor of safety for design purposes, also failed to estimate sufficient scour at approximately half of the sites. Based on the results shown in figure 28, it can be seen that no equation accurately predicted the historical scour at all the study sites. Therefore, the preferred design equation would be the equation that provides the best combination of accuracy and safety. The FHWA procedures (Laursen's contraction-scour equation combined with the CSU pier-scour equation) provided a combination of accuracy and safety, required by design equations, equal to or better than the other equations evaluated.

Computed Depths of Scour for 100-Year and 500-Year Discharges

The potential scour resulting from the 100-year and 500-year discharges were computed for each of the sites in accordance with procedures outlined in Richardson and others (1991). Equation 2 or 5 was used to compute the appropriate contraction scour, and the CSU equation (eq. 19) was used to compute the local pier scour. Both the Froehlich abutment-scour equation (eq. 50) and the alternative method of computing abutment scour by use of the other equations discussed herein (eqs. 39, 41, 42, 44, 45, 46, 47, and 48) were used. The Froehlich abutment-scour equation typically predicted smaller estimates of abutment scour than did the alternate method, except for long abutments. For long abutments, the alternative method involves an equation developed from empirical analysis of dikes in the Mississippi River and consistently predicted scour depths less than those predicted by the Froehlich abutment-scour equation. Because the Froehlich abutment-scour equation is currently the primary method recommended by the FHWA (Richardson and others, 1991, p. 47), this equation (eq. 50) was used to compute the potential scour at the abutments of the selected sites.

The results of the WSPRO model and scour computations are shown in tables 15–34. Figures 29–38 are graphical representations of the surveyed beds and the computed beds resulting from the (A) 100-year and (B) 500-year peak discharges. The graphical representation of the computed bed, which is based on an assumed angle of repose for the bed material of 30^0 (Richardson and others, 1991, p. 57), shows overlapping scour holes, which influence the ultimate depth of scour. At I-74 over the Big Blue River (fig. 32), abutment scour overlaps with the local scour of the adjacent pier. At S.R. 258 over East Fork White River (fig. 36), pier 6 is on the bank of the main channel and is between the areas of clear-water and live-bed contraction scour. Because of deep contraction scour in the main channel, the graphical representation of the computed bed is significantly deeper at pier 6 than was anticipated from the data in tables 28–29. The scour at pier 3 of S.R. 331 over Tippecanoe River (fig. 37) also is affected by the large scour hole at the adjacent abutment. Scour



EXPLANATION

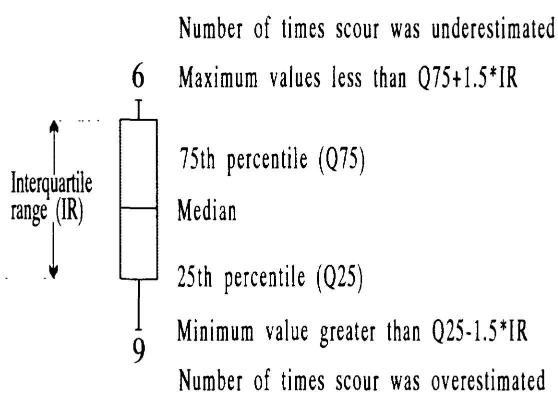


Figure 28. Summary of differences between the computed and measured historical bed elevations for selected equations.

at both abutments of S.R. 358 over White River (fig. 38) is great enough to overlap with the local scour at adjacent piers.

The effect of overlapping scour holes is unknown at this time, but it is anticipated that this overlap could result in a deepening of scour (Richardson and others, 1991, p. 58). Because the equations for computing local abutment scour are often very conservative, the FHWA recommends abutment foundations be set to standards of the American Association of State Highway and Transportation Officials and that protection of the abutments be provided by use of rock riprap in accordance to design procedures outlined in Brown and Clyde (1989) (Richardson and others, 1991, p. 47). If appropriate protection of the abutments is provided, abutment scour need not be calculated (Richardson and others, 1991, p. 70).

Reliability of Scour Equations for Assessing Scour Potential

Based on the comparison of the recommended pier-scour equation (CSU pier-scour equation) to the historical data, the potential scour predicted from the 100-year and 500-year peak discharges are likely to be conservative estimates. At S.R. 331 over Tippecanoe River and at I-74 over Big Blue River, however, the FHWA procedure (Laursen's contraction-scour equation combined with the CSU pier-scour equation) underestimated the historical scour by more than 5 ft; therefore, the potential scour may be underestimated. At U.S. Route 231 over East Fork White River, the recommended procedure overestimated the historical scour by 15 to 20 ft (table 10), and at S.R. 258 over East Fork White River, the recommended procedure overestimated the historical scour by about 10 ft (table 11). At both sites, contraction scour in excess of 10 ft was predicted; therefore, additional data and sediment-transport modeling at these sites are required to verify the accuracy of the potential-scour computations. Computed abutment scour at about one-half of the sites seems to be excessive.

Table 15. *Potential scour resulting from the 100-year peak discharge at U.S. Route 24 over Tippecanoe River at Monticello, Ind.*

[ft^3/s, cubic feet per second; all elevations refer to feet above sea level; ft/s, feet per second; mm, millimeters; --, no data or computations]

Pier-scour characteristic	Pier number 5	4	3	2
Total discharge, in ft^3/s	25,000	25,000	25,000	25,000
Water-surface elevation	618.8	618.8	618.8	618.8
Approach depth, in feet	5.5	15.6	14.2	0
Approach velocity, in ft/s	3.4	7.7	7.1	0
Angle of attack, in degrees	0	0	0	0
Estimated grain size, in mm	4.0	4.0	4.0	4.0
Computed depth of scour or elevation				
Contraction scour, in feet	--	2.9	2.9	--
Local scour, in feet	5.7	10.6	10.1	--
Correction for bedforms, in feet	.6	1.1	1.0	--
Computed elevation	607.0	588.6	590.6	--

Abutment-scour characteristic	Abutment Left	Right
Total discharge, in ft^3/s	25,000	25,000
Water-surface elevation	618.8	618.8
Abutment location	Set back	Set back
Overbank flow	No	No
Bedload condition	Clear water	Clear water
Abutment type	Spill through	Spill through
Discharge blocked, in ft^3/s	0	0
Length, in feet	0	0
Approach depth, in feet	0	0
Approach velocity, in ft/s	0	0
Angle of abutment, in degrees	0	0
Estimated grain size, in mm	4.0	4.0
Computed depth of scour or elevation		
Contraction scour, in feet	0	0
Local scour, in feet	0	0
Correction for bedforms, in feet	0	0
Computed elevation	--	--

Table 16. *Potential scour resulting from the 500-year peak discharge at U.S. Route 24 over Tippecanoe River at Monticello, Ind.*

[ft^3/s, cubic feet per second; all elevations refer to feet above sea level; ft/s, feet per second; mm, millimeters; --, no data or computations]

Pier-scour characteristic	Pier number 5	4	3	2
Total discharge, in ft^3/s	31,500	31,500	31,500	31,500
Water-surface elevation	620.2	620.2	620.2	620.2
Approach depth, in feet	6.9	17.0	15.3	1.1
Approach velocity, in ft/s	4.1	8.6	8.2	4.3
Angle of attack, in degrees	0	0	0	0
Estimated grain size, in mm	4.0	4.0	4.0	4.0
Computed depth of scour or elevation				
Contraction scour, in feet	--	3.3	3.3	--
Local scour, in feet	6.4	11.2	10.9	5.1
Correction for bedforms, in feet	.6	1.1	1.1	.5
Computed elevation	606.3	587.6	589.6	613.5

Abutment-scour characteristic	Abutment Left	Right
Total discharge, in ft^3/s	31,500	31,500
Water-surface elevation	620.2	620.2
Abutment location	Set back	Set back
Overbank flow	No	No
Bedload condition	Clear water	Clear water
Abutment type	Spill through	Spill through
Discharge blocked, in ft^3/s	0	0
Length, in feet	0	0
Approach depth, in feet	0	0
Approach velocity, in ft/s	0	0
Angle of abutment, in degrees	0	0
Estimated grain size, in mm	4.0	4.0
Computed depth of scour or elevation		
Contraction scour, in feet	0	0
Local scour, in feet	0	0
Correction for bedforms, in feet	0	0
Computed elevation	--	--

Table 17. *Potential scour resulting from the 100-year peak discharge at State Road 32 over Buck Creek at Yorktown, Ind.*

[ft^3/s, cubic feet per second; all elevations refer to feet above sea level; ft/s, feet per second; mm, millimeters; --, no data or computations]

Pier-scour characteristic	Pier number	
	3	2
Total discharge, in ft^3/s	7,000	7,000
Water-surface elevation	902.1	902.1
Approach depth, in feet	12.9	11.5
Approach velocity, in ft/s	8.0	7.7
Angle of attack, in degrees	0	0
Estimated grain size, in mm	4.0	4.0
Computed depth of scour or elevation		
Contraction scour, in feet	-1.7	-1.7
Local scour, in feet	7.7	7.4
Correction for bedforms, in feet	.8	.7
Computed elevation	882.4	884.2

Abutment-scour characteristic	Abutment	
	Left	Right
Total discharge, in ft^3/s	7,000	7,000
Water-surface elevation	902.1	902.1
Abutment location	Set back	Set back
Overbank flow	Yes	Yes
Bedload condition	Clear water	Clear water
Abutment type	Wingwall	Wingwall
Discharge blocked, in ft^3/s	0	0
Length, in feet	0	0
Approach depth, in feet	0	0
Approach velocity, in ft/s	0	0
Angle of abutment, in degrees	0	0
Estimated grain size, in mm	4.0	4.0
Computed depth of scour or elevation		
Contraction scour, in feet	1.1	1.1
Local scour, in feet	0	0
Correction for bedforms, in feet	0	0
Computed elevation	891.0	891.9

Table 18. *Potential scour resulting from the 500-year peak discharge at State Road 32 over Buck Creek at Yorktown, Ind.*

[ft^3/s, cubic feet per second; all elevations refer to feet above sea level; ft/s, feet per second; mm, millimeters; --, no data or computations]

Pier-scour characteristic	Pier number 3	Pier number 2
Total discharge, in ft^3/s	9,500	9,500
Water-surface elevation	903.5	903.5
Approach depth, in feet	14.3	12.9
Approach velocity, in ft/s	9.6	9.2
Angle of attack, in degrees	0	0
Estimated grain size, in mm	4.0	4.0
Computed depth of scour or elevation		
Contraction scour, in feet	-1.3	-1.3
Local scour, in feet	8.4	8.1
Correction for bedforms, in feet	.8	.8
Computed elevation	881.3	883.0

Abutment-scour characteristic	Abutment Left	Abutment Right
Total discharge, in ft^3/s	9,500	9,500
Water-surface elevation	903.5	903.5
Abutment location	Set back	Set back
Overbank flow	Yes	Yes
Bedload condition	Clear water	Clear water
Abutment type	Wingwall	Wingwall
Discharge blocked, in ft^3/s	0	18
Length, in feet	0	20.0
Approach depth, in feet	0	.5
Approach velocity, in ft/s	0	1.9
Angle of abutment, in degrees	0	0
Estimated grain size, in mm	4.0	4.0
Computed depth of scour or elevation		
Contraction scour, in feet	3.0	1.2
Local scour, in feet	0	3.3
Correction for bedforms, in feet	0	0
Computed elevation	889.1	888.5

Table 19. *Potential scour resulting from the 100-year peak discharge at U.S. Route 41 over White River near Hazleton, Ind.*

[ft^3/s, cubic feet per second; all elevations refer to feet above sea level; ft/s, feet per second; mm, millimeters; --, no data or computations]

Pier-scour characteristic	Pier number							
	1	2	3	4	5	6	7	8
Total discharge, in ft^3/s	186,000	186,000	186,000	186,000	186,000	186,000	186,000	186,000
Water-surface elevation	414.1	414.1	414.1	414.1	414.1	414.1	414.1	414.1
Approach depth, in feet	13.9	14.3	13.1	13.9	33.4	33.0	32.6	16.6
Approach velocity, in ft/s	3.3	3.3	3.3	3.3	6.6	6.2	6.5	2.8
Angle of attack, in degrees	0	0	0	0	0	0	0	0
Estimated grain size, in mm	.5	.5	.5	.5	.5	.5	.5	.5
Computed depth of scour or elevation								
Contraction scour, in feet	8.2	8.2	8.2	8.2	10.5	10.5	10.5	7.8
Local scour, in feet	6.2	6.2	6.1	6.2	9.4	9.1	9.3	5.9
Correction for bedforms, in feet	.6	.6	.6	.6	.9	.9	.9	.6
Computed elevation	385.2	384.8	386.1	385.2	359.9	360.6	360.8	383.2

Abutment-scour characteristic	Abutment
	Left
Total discharge, in ft^3/s	186,000
Water-surface elevation	414.1
Abutment location	Set back
Overbank flow	Yes
Bedload condition	Clear water
Abutment type	Spill through
Discharge blocked, in ft^3/s	1,210
Length, in feet	216.4
Approach depth, in feet	3.7
Approach velocity, in ft/s	1.5
Angle of abutment, in degrees	0
Estimated grain size, in mm	Concrete protection
Computed depth of scour or elevation	
Contraction scour, in feet	0
Local scour, in feet	0
Correction for bedforms, in feet	0
Computed elevation	--

Table 19. *Potential scour resulting from the 100-year peak discharge at U.S. Route 41 over White River near Hazleton, Ind.*--Continued

Pier-scour characteristic	Pier number						
	9	10	11	12	13	14	15
Total discharge, in ft^3/s	186,000	186,000	186,000	186,000	186,000	186,000	186,000
Water-surface elevation	414.1	414.1	414.1	414.1	414.1	414.1	414.1
Approach depth, in feet	10.6	11.7	12.6	13.7	14.8	15.8	14.3
Approach velocity, in ft/s	2.8	3.1	3.2	3.2	3.4	3.6	3.4
Angle of attack, in degrees	0	0	0	0	0	0	0
Estimated grain size, in mm	.5	.5	.5	.5	.5	.5	.5
Computed depth of scour or elevation							
Contraction scour, in feet	7.8	7.8	7.8	7.8	7.8	7.8	7.8
Local scour, in feet	5.6	5.9	6.0	6.1	6.3	6.5	6.3
Correction for bedforms, in feet	.6	.6	.6	.6	.6	.6	.6
Computed elevation	389.5	388.1	387.1	385.9	384.6	383.4	385.1

Abutment-scour characteristic	Abutment
	Right
Total discharge, in ft^3/s	186,000
Water-surface elevation	414.1
Abutment location	Set back
Overbank flow	Yes
Bed-load condition	Clear water
Abutment type	Spill through
Discharge blocked, in ft^3/s	59,440
Length, in feet	2,421.0
Approach depth, in feet	12.5
Approach velocity, in ft/s	2.0
Angle of abutment, in degrees	0
Estimated grain size, in mm	Concrete protection
Computed depth of scour or elevation	
Contraction scour, in feet	0
Local scour, in feet	0
Correction for bedforms, in feet	0
Computed elevation	--

Table 20. *Potential scour resulting from the 500-year peak discharge at U.S. Route 41 over White River near Hazleton, Ind.*

[ft^3/s, cubic feet per second; all elevations refer to feet above sea level; ft/s, feet per second; mm, millimeters; --, no data or computations]

Pier-scour characteristic	Pier number							
	1	2	3	4	5	6	7	8
Total discharge, in ft^3/s	241,500	241,500	241,500	241,500	241,500	241,500	241,500	241,500
Water-surface elevation	417.1	417.1	417.1	417.1	417.1	417.1	417.1	417.1
Approach depth, in feet	16.9	17.3	16.1	16.9	36.4	36.0	35.6	19.6
Approach velocity, in ft/s	3.7	3.9	3.8	3.8	7.1	6.9	6.9	4.7
Angle of attack, in degrees	0	0	0	0	0	0	0	0
Estimated grain size, in mm	.5	.5	.5	.5	.5	.5	.5	.5
	Computed depth of scour or elevation							
Contraction scour, in feet	13.1	13.1	13.1	13.1	12.4	12.4	12.4	12.9
Local scour, in feet	6.7	6.8	6.7	6.7	9.8	9.6	9.6	7.5
Correction for bedforms, in feet	.7	.7	.7	.7	1.0	1.0	1.0	.7
Computed elevation	379.7	379.2	380.5	379.7	357.5	358.1	358.5	376.4

Abutment-scour characteristic	Abutment
	Left
Total discharge, in ft^3/s	241,500
Water-surface elevation	417.1
Abutment location	Set back
Overbank flow	Yes
Bedload condition	Clear water
Abutment type	Spill through
Discharge blocked, in ft^3/s	2,370
Length, in feet	280.0
Approach depth, in feet	5.6
Approach velocity, in ft/s	1.5
Angle of abutment, in degrees	0
Estimated grain size, in mm	Concrete protection
	Computed depth of scour or elevation
Contraction scour, in feet	0
Local scour, in feet	0
Correction for bedforms, in feet	0
Computed elevation	--

Table 20. *Potential scour resulting from the 500-year peak discharge at U.S. Route 41 over White River near Hazleton, Ind.*--Continued

Pier-scour characteristic	Pier number						
	9	10	11	12	13	14	15
Total discharge, in ft^3/s	241,500	241,500	241,500	241,500	241,500	241,500	241,500
Water-surface elevation	417.1	417.1	417.1	417.1	417.1	417.1	417.1
Approach depth, in feet	13.6	14.7	15.6	16.7	17.8	18.8	17.3
Approach velocity, in ft/s	3.4	3.6	3.6	3.8	4.1	4.2	3.8
Angle of attack, in degrees	0	0	0	0	0	0	0
Estimated grain size, in mm	.5	.5	.5	.5	.5	.5	.5
	Computed depth of scour or elevation						
Contraction scour, in feet	12.9	12.9	12.9	12.9	12.9	12.9	12.9
Local scour, in feet	6.2	6.5	6.5	6.7	7.0	7.1	6.8
Correction for bedforms, in feet	.6	.6	.6	.7	.7	.7	.7
Computed elevation	383.8	382.4	381.5	380.1	378.7	377.6	379.4

Abutment-scour characteristic	Abutment
	Right
Total discharge, in ft^3/s	241,500
Water-surface elevation	417.1
Abutment location	Set back
Overbank flow	Yes
Bedload condition	Clear water
Abutment type	Spill through
Discharge blocked, in ft^3/s	83,230
Length, in feet	2,525.0
Approach depth, in feet	15.1
Approach velocity, in ft/s	2.2
Angle of abutment, in degrees	0
Estimated grain size, in mm	Concrete protection
	Computed depth of scour or elevation
Contraction scour, in feet	0
Local scour, in feet	0
Correction for bedforms, in feet	0
Computed elevation	--

Table 21. *Potential scour resulting from the 100-year peak discharge at I-74 over Big Blue River near Shelbyville, Ind.*

[ft^3/s, cubic feet per second; all elevations refer to feet above sea level; ft/s, feet per second; mm, millimeters; --, no data or computations]

Pier-scour characteristic	Pier number					
	2	3	4	5	6	7
Total discharge, in ft^3/s	17,300	17,300	17,300	17,300	17,300	17,300
Water-surface elevation	765.7	765.7	765.7	765.7	765.7	765.7
Approach depth, in feet	6.1	8.1	14.4	15.7	7.8	2.8
Approach velocity, in ft/s	2.6	3.4	5.8	6.2	2.1	2.1
Angle of attack, in degrees	0	0	0	0	0	0
Estimated grain size, in mm	.5	.5	.5	.5	.5	.5
Computed depth of scour or elevation						
Contraction scour, in feet	5.6	5.6	0	0	4.5	4.5
Local scour, in feet	4.1	4.8	6.5	6.8	3.9	3.4
Correction for bedforms, in feet	.4	.5	.6	.7	.4	.3
Computed elevation	749.5	746.7	744.2	742.5	749.1	754.7

Abutment-scour characteristic	Abutment	
	Left	Right
Characteristics		
Total discharge, in ft^3/s	17,300	17,300
Water-surface elevation	765.7	765.7
Abutment location	Set back	Set back
Overbank flow	Yes	Yes
Bedload condition	Clear water	Clear water
Abutment type	Spill through	Spill through
Discharge blocked, in ft^3/s	1,835	3,504
Length, in feet	513.0	734.7
Approach depth, in feet	4.3	3.6
Approach velocity, in ft/s	.8	1.3
Angle of abutment, in degrees	15	-15
Estimated grain size, in mm	.5	.5
Computed depth of scour or elevation		
Contraction scour, in feet	5.6	4.5
Local scour, in feet	12.8	15.7
Correction for bedforms, in feet	0	0
Computed elevation	741.2	743.2

Table 22. *Potential scour resulting from the 500-year peak discharge at I-74 over Big Blue River near Shelbyville, Ind.*

[ft^3/s, cubic feet per second; all elevations refer to feet above sea level; ft/s, feet per second; mm, millimeters; --, no data or computations]

Pier-scour characteristic	Pier number					
	2	3	4	5	6	7
Total discharge, in ft^3/s	22,500	22,500	22,500	22,500	22,500	22,500
Water-surface elevation	767.1	767.1	767.1	767.1	767.1	767.1
Approach depth, in feet	7.5	9.5	15.8	17.1	9.2	4.2
Approach velocity, in ft/s	3.0	4.2	6.7	7.0	4.1	2.6
Angle of attack, in degrees	0	0	0	0	0	0
Estimated grain size, in mm	.5	.5	.5	.5	.5	.5
	Computed depth of scour or elevation					
Contraction scour, in feet	8.6	8.6	1.9	1.9	7.2	7.2
Local scour, in feet	4.5	5.4	7.0	7.2	5.3	3.9
Correction for bedforms, in feet	.5	.5	.7	.7	.5	.4
Computed elevation	746.0	743.1	741.7	740.2	744.9	751.4

Abutment-scour characteristic	Abutment	
	Left	Right
Total discharge, in ft^3/s	22,500	22,500
Water-surface elevation	767.1	767.1
Abutment location	Set back	Set back
Overbank flow	Yes	Yes
Bedload condition	Clear water	Clear water
Abutment type	Spill through	Spill through
Discharge blocked, in ft^3/s	3,087	6,165
Length, in feet	537.7	783.5
Approach depth, in feet	5.7	5.0
Approach velocity, in ft/s	1.0	1.6
Angle of abutment, in degrees	15	-15
Estimated grain size, in mm	.5	.5
	Computed depth of scour or elevation	
Contraction scour, in feet	8.6	7.2
Local scour, in feet	16.2	20.0
Correction for bedforms, in feet	0	0
Computed elevation	734.5	734.8

Table 23. *Potential scour resulting from the 100-year peak discharge at I-74 over Whitewater River near Harrison, Oh.*

[ft^3/s, cubic feet per second; all elevations refer to feet above sea level; ft/s, feet per second; mm, millimeters; --, no data or computations]

Pier-scour characteristic	Pier number						
	16	15	14	13	12	11	10
Total discharge, in ft^3/s	62,200	62,200	62,200	62,200	62,200	62,200	62,200
Water-surface elevation	532.7	532.7	532.7	532.7	532.7	532.7	532.7
Approach depth, in feet	0	17.6	15.5	11.1	9.2	0	0
Approach velocity, in ft/s	0	8.9	8.4	3.8	2.4	0	0
Angle of attack, in degrees	0	0	0	0	0	0	0
Estimated grain size, in mm	.5	.5	.5	.5	.5	.5	.5
	Computed depth of scour or elevation						
Contraction scour, in feet	--	4.9	4.9	7.0	7.0	--	--
Local scour, in feet	--	8.8	8.4	5.7	4.6	--	--
Correction for bedforms, in feet	--	.9	.8	.6	.5	--	--
Computed elevation	--	500.5	503.1	508.3	511.4	--	--

Abutment-scour characteristic	Abutment	
	Left	Right
Total discharge, in ft^3/s	62,200	62,200
Water-surface elevation	532.7	532.7
Abutment location	Set back	Set back
Overbank flow	Yes	Yes
Bedload condition	Clear water	Clear water
Abutment type	Spill through	Spill through
Discharge blocked, in ft^3/s	0	0
Length, in feet	0	0
Approach depth, in feet	0	0
Approach velocity, in ft/s	0	0
Angle of abutment, in degrees	0	0
Estimated grain size, in mm	.5	.5
	Computed depth of scour or elevation	
Contraction scour, in feet	1.7	0
Local scour, in feet	0	0
Correction for bedforms, in feet	0	0
Computed elevation	494.1	--

Table 24. *Potential scour resulting from the 500-year peak discharge at I-74 over Whitewater River near Harrison, Oh.*

[ft^3/s, cubic feet per second; all elevations refer to feet above sea level; ft/s, feet per second; mm, millimeters; --, no data or computations]

Pier-scour characteristic	Pier number						
	16	15	14	13	12	11	10
Total discharge, in ft^3/s	81,300	81,300	81,300	81,300	81,300	81,300	81,300
Water-surface elevation	534.2	534.2	534.2	534.2	534.2	534.2	534.2
Approach depth, in feet	0	19.1	17.0	12.6	10.7	0	0
Approach velocity, in ft/s	0	10.3	9.9	4.4	2.8	0	0
Angle of attack, in degrees	0	0	0	0	0	0	0
Estimated grain size, in mm	.5	.5	.5	.5	.5	.5	.5
Computed depth of scour or elevation							
Contraction scour, in feet	0	8.4	8.4	11.1	11.1	0	0
Local scour, in feet	0	9.5	9.2	6.2	5.0	0	0
Correction for bedforms, in feet	0	.9	.9	.6	.5	0	0
Computed elevation	--	496.3	498.7	503.7	506.9	--	--

Abutment-scour characteristic	Abutment	
	Left	Right
Total discharge, in ft^3/s	81,300	81,300
Water-surface elevation	534.2	534.2
Abutment location	Set back	Set back
Overbank flow	Yes	Yes
Bedload condition	Clear water	Clear water
Abutment type	Spill through	Spill through
Discharge blocked, in ft^3/s	0	0
Length, in feet	0	0
Approach depth, in feet	0	0
Approach velocity, in ft/s	0	0
Angle of abutment, in degrees	0	0
Estimated grain size, in mm	.5	.5
Computed depth of scour or elevation		
Contraction scour, in feet	4.1	0
Local scour, in feet	0	0
Correction for bedforms, in feet	0	0
Computed elevation	491.7	--

Table 25. *Potential scour resulting from the 100-year peak discharge at U.S. Route 231 over Kankakee River near Hebron, Ind.*

[ft^3/s, cubic feet per second; all elevations refer to feet above sea level; ft/s, feet per second; mm, millimeters; --, no data or computations]

Pier-scour characteristic	Pier number			
	2	3	4	5
Total discharge, in ft^3/s	6,800	6,800	6,800	6,800
Water-surface elevation	648.7	648.7	648.7	648.7
Approach depth, in feet	9.7	10.3	11.1	11.1
Approach velocity, in ft/s	3.8	4.0	4.1	4.0
Angle of attack, in degrees	0	0	0	0
Estimated grain size, in mm	1.0	1.0	1.0	1.0
Computed depth of scour or elevation				
Contraction scour, in feet	0.6	0.6	0.6	0.6
Local scour, in feet	3.6	3.7	3.8	3.7
Correction for bedforms, in feet	.4	.4	.4	.4
Computed elevation	634.4	633.7	632.8	632.9

Abutment-scour characteristic	Abutment	
	Left	Right
Total discharge, in ft^3/s	6,800	6,800
Water-surface elevation	648.7	648.7
Abutment location	Edge of channel	Edge of channel
Overbank flow	No	No
Bedload condition	Live bed	Live bed
Abutment type	Spill through	Spill through
Discharge blocked, in ft^3/s	0	0
Length, in feet	0	0
Approach depth, in feet	0	0
Approach velocity, in ft/s	0	0
Angle of abutment, in degrees	48	-48
Estimated grain size, in mm	1.0	1.0
Computed depth of scour or elevation		
Contraction scour, in feet	0	0
Local scour, in feet	0	0
Correction for bedforms, in feet	0	0
Computed elevation	--	--

Table 26. *Potential scour resulting from the 500-year peak discharge at U.S. Route 231 over Kankakee River near Hebron, Ind.*

[ft^3/s, cubic feet per second; all elevations refer to feet above sea level; ft/s, feet per second; mm, millimeters; --, no data or computations]

Pier-scour characteristic	Pier number			
	2	3	4	5
Total discharge, in ft^3/s	7,800	7,800	7,800	7,800
Water-surface elevation	649.6	649.6	649.6	649.6
Approach depth, in feet	10.6	11.2	12.0	12.0
Approach velocity, in ft/s	4.0	4.2	4.3	4.3
Angle of attack, in degrees	0	0	0	0
Estimated grain size, in mm	1.0	1.0	1.0	1.0
Computed depth of scour or elevation				
Contraction scour, in feet	0.4	0.4	0.4	0.4
Local scour, in feet	3.7	3.8	3.9	3.9
Correction for bedforms, in feet	.4	.4	.4	.4
Computed elevation	634.5	633.8	632.9	632.9

Abutment-scour characteristic	Abutment	
	Left	Right
Total discharge, in ft^3/s	7,800	7,800
Water-surface elevation	649.6	649.6
Abutment location	Edge of channel	Edge of channel
Overbank flow	No	No
Bedload condition	Live bed	Live bed
Abutment type	Spill through	Spill through
Discharge blocked, in ft^3/s	0	0
Length, in feet	0	0
Approach depth, in feet	0	0
Approach velocity, in ft/s	0	0
Angle of abutment, in degrees	48	-48
Estimated grain size, in mm	1.0	1.0
Computed depth of scour or elevation		
Contraction scour, in feet	0	0
Local scour, in feet	0	0
Correction for bedforms, in feet	0	0
Computed elevation	--	--

Table 27. *Potential scour resulting from the 100-year peak discharge at U.S. Route 231 over East Fork White River near Haysville, Ind.*

[ft^3/s, cubic feet per second; all elevations refer to feet above sea level; ft/s, feet per second; mm, millimeters; --, no data or computations]

Pier-scour characteristic	Pier number 2	3	4
Total discharge, in ft^3/s	116,000	116,000	116,000
Water-surface elevation	452.7	452.7	452.7
Approach depth, in feet	32.7	39.8	17.3
Approach velocity, in ft/s	7.0	8.2	4.4
Angle of attack, in degrees	25	25	25
Estimated grain size, in mm	.2	.2	.2
Computed depth of scour or elevation			
Contraction scour, in feet	11.1	15.8	25.7
Local scour, in feet	26.1	28.8	19.5
Correction for bedforms, in feet	2.6	2.9	2.0
Computed elevation	380.2	365.4	388.2

Abutment-scour characteristic	Abutment Left	Abutment Right
Total discharge, in ft^3/s	116,000	116,000
Water-surface elevation	452.7	452.7
Abutment location	Set back	Set back
Overbank flow	Yes	Yes
Bedload condition	Clear water	Clear water
Abutment type	Spill through	Spill through
Discharge blocked, in ft^3/s	12,230	27,100
Length, in feet	489.6	2,358.6
Approach depth, in feet	12.4	7.7
Approach velocity, in ft/s	2.0	1.5
Angle of abutment, in degrees	25	-25
Estimated grain size, in mm	.2	.2
Computed depth of scour or elevation		
Contraction scour, in feet	11.1	25.7
Local scour, in feet	31.6	33.4
Correction for bedforms, in feet	0	0
Computed elevation	400.6	376.1

Table 28. *Potential scour resulting from the 500-year peak discharge at U.S. Route 231 over East Fork White River near Haysville, Ind.*

[ft^3/s, cubic feet per second; all elevations refer to feet above sea level; ft/s, feet per second; mm, millimeters; --, no data or computations]

Pier-scour characteristic	Pier number		
	2	3	4
Total discharge, in ft^3/s	153,500	153,500	153,500
Water-surface elevation	455.9	455.9	455.9
Approach depth, in feet	35.9	43.0	20.5
Approach velocity, in ft/s	7.5	9.8	5.6
Angle of attack, in degrees	25	25	25
Estimated grain size, in mm	.2	.2	.2
Computed depth of scour or elevation			
Contraction scour, in feet	18.3	28.0	37.2
Local scour, in feet	27.2	31.5	22.2
Correction for bedforms, in feet	2.7	3.1	2.2
Computed elevation	371.8	350.3	373.8

Abutment-scour characteristic	Abutment	
	Left	Right
Total discharge, in ft^3/s	153,500	153,500
Water-surface elevation	455.9	455.9
Abutment location	Set back	Set back
Overbank flow	Yes	Yes
Bedload condition	Clear water	Clear water
Abutment type	Spill through	Spill through
Discharge blocked, in ft^3/s	18,180	49,530
Length, in feet	495.3	2,373.2
Approach depth, in feet	16.1	11.5
Approach velocity, in ft/s	2.3	1.8
Angle of abutment, in degrees	25	-25
Estimated grain size, in mm	.2	.2
Computed depth of scour or elevation		
Contraction scour, in feet	18.3	37.2
Local scour, in feet	38.3	43.7
Correction for bedforms, in feet	0	0
Computed elevation	386.2	353.8

Table 29. *Potential scour resulting from the 100-year peak discharge at State Road 258 over East Fork White River near Seymour, Ind.*

[ft^3/s, cubic feet per second; all elevations refer to feet above sea level; ft/s, feet per second; mm, millimeters; --, no data or computations]

Pier-scour characteristic	Pier number					
	7	6	5	4	3	2
Total discharge, in ft^3/s	98,000	98,000	98,000	98,000	98,000	98,000
Water-surface elevation	566.4	566.4	566.4	566.4	566.4	566.4
Approach depth, in feet	3.7	9.4	17.2	17.4	6.7	3.3
Approach velocity, in ft/s	3.3	6.5	7.9	7.2	2.8	2.8
Angle of attack, in degrees	0	0	0	0	0	0
Estimated grain size, in mm	1.0	1.0	1.0	1.0	1.0	1.0
Computed depth of scour or elevation						
Contraction scour, in feet	-0.8	-0.8	13.7	13.7	1.4	1.4
Local scour, in feet	3.4	5.2	6.1	5.9	3.5	3.1
Correction for bedforms, in feet	.3	.5	.6	.6	.4	.3
Computed elevation	559.8	552.1	528.8	528.8	554.4	558.3

Abutment-scour characteristic	Abutment	
	Left	Right
Total discharge, in ft^3/s	98,000	98,000
Water-surface elevation	566.4	566.4
Abutment location	Set back	Set back
Overbank flow	Yes	Yes
Bedload condition	Clear water	Clear water
Abutment type	Spill through	Spill through
Discharge blocked, in ft^3/s	1,940	1,270
Length, in feet	342.0	272.1
Approach depth, in feet	3.0	3.9
Approach velocity, in ft/s	1.9	1.2
Angle of abutment, in degrees	15	-15
Estimated grain size, in mm	1.0	1.0
Computed depth of scour or elevation		
Contraction scour, in feet	-0.8	1.4
Local scour, in feet	13.7	11.5
Correction for bedforms, in feet	0	0
Computed elevation	549.1	551.4

Table 30. *Potential scour resulting from the 500-year peak discharge at State Road 258 over East Fork White River near Seymour, Ind.*

[ft^3/s, cubic feet per second; all elevations refer to feet above sea level; ft/s, feet per second; mm, millimeters; --, no data or computations]

Pier-scour characteristic	Pier number					
	7	6	5	4	3	2
Total discharge, in ft^3/s	127,000	127,000	127,000	127,000	127,000	127,000
Water-surface elevation	567.2	567.2	567.2	567.2	567.2	567.2
Approach depth, in feet	4.5	10.2	18.0	18.2	7.5	4.1
Approach velocity, in ft/s	3.3	7.0	7.8	7.0	3.5	3.5
Angle of attack, in degrees	0	0	0	0	0	0
Estimated grain size, in mm	1.0	1.0	1.0	1.0	1.0	1.0
	Computed depth of scour or elevation					
Contraction scour, in feet	-1.1	-1.1	18.3	18.3	3.1	3.1
Local scour, in feet	3.5	5.4	6.1	5.9	3.9	3.6
Correction for bedforms, in feet	.4	.5	.6	.6	.4	.4
Computed elevation	559.9	552.2	524.2	524.2	552.3	556.0

Abutment-scour characteristic	Abutment	
	Left	Right
Total discharge, in ft^3/s	127,000	127,000
Water-surface elevation	567.2	567.2
Abutment location	Set back	Set back
Overbank flow	Yes	Yes
Bedload condition	Clear water	Clear water
Abutment type	Spill through	Spill through
Discharge blocked, in ft^3/s	1,800	1,570
Length, in feet	242.0	202.1
Approach depth, in feet	3.4	4.1
Approach velocity, in ft/s	2.2	1.9
Angle of abutment, in degrees	15	-15
Estimated grain size, in mm	1.0	1.0
	Computed depth of scour or elevation	
Contraction scour, in feet	-1.1	3.1
Local scour, in feet	13.8	13.0
Correction for bedforms, in feet	0	0
Computed elevation	549.3	548.1

Table 31. *Potential scour resulting from the 100-year peak discharge at State Road 331 over Tippecanoe River at Old Tip Town, Ind.*

[ft^3/s, cubic feet per second; all elevations refer to feet above sea level; ft/s, feet per second; mm, millimeters; --, no data or computations]

Pier-scour characteristic	Pier number 2	Pier number 3
Total discharge, in ft^3/s	3,850	3,850
Water-surface elevation	771.6	771.6
Approach depth, in feet	9.7	9.8
Approach velocity, in ft/s	3.4	3.4
Angle of attack, in degrees	0	0
Estimated grain size, in mm	1.0	1.0
Computed depth of scour or elevation		
Contraction scour, in feet	2.1	2.1
Local scour, in feet	3.4	3.4
Correction for bedforms, in feet	.3	.3
Computed elevation	756.1	756.0

Abutment-scour characteristic	Abutment Left	Abutment Right
Total discharge, in ft^3/s	3,850	3,850
Water-surface elevation	771.6	771.6
Abutment location	Edge of channel	In channel
Overbank flow	Yes	Yes
Bedload condition	Live bed	Live bed
Abutment type	Spill through	Spill through
Discharge blocked, in ft^3/s	145	1,589
Length, in feet	147.5	116.5
Approach depth, in feet	1.6	5.8
Approach velocity, in ft/s	.6	2.3
Angle of abutment, in degrees	41	-41
Estimated grain size, in mm	1.0	1.0
Computed depth of scour or elevation		
Contraction scour, in feet	2.1	2.1
Local scour, in feet	4.9	14.1
Correction for bedforms, in feet	0	0
Computed elevation	755.1	745.2

Table 32. *Potential scour resulting from the 500-year peak discharge at State Road 331 over Tippecanoe River at Old Tip Town, Ind.*

[ft^3/s, cubic feet per second; all elevations refer to feet above sea level; ft/s, feet per second; mm, millimeters; --, no data or computations]

Pier-scour characteristic	Pier number 2	Pier number 3
Total discharge, in ft^3/s	4,500	4,500
Water-surface elevation	772.2	772.2
Approach depth, in feet	10.3	10.4
Approach velocity, in ft/s	3.7	3.8
Angle of attack, in degrees	0	0
Estimated grain size, in mm	1.0	1.0
Computed depth of scour or elevation		
Contraction scour, in feet	2.6	2.6
Local scour, in feet	3.6	3.6
Correction for bedforms, in feet	.4	.4
Computed elevation	755.3	755.2

Abutment-scour characteristic	Abutment Left	Abutment Right
Total discharge, in ft^3/s	4,500	4,500
Water-surface elevation	772.2	772.2
Abutment location	Edge of channel	In channel
Overbank flow	Yes	Yes
Bedload condition	Live bed	Live bed
Abutment type	Spill through	Spill through
Discharge blocked, in ft^3/s	203	1,825
Length, in feet	332.3	122.6
Approach depth, in feet	2.0	6.1
Approach velocity, in ft/s	.6	2.4
Angle of abutment, in degrees	41	-41
Estimated grain size, in mm	1.0	1.0
Computed depth of scour or elevation		
Contraction scour, in feet	2.6	2.6
Local scour, in feet	5.6	14.9
Correction for bedforms, in feet	0	0
Computed elevation	753.8	743.8

Table 33. *Potential scour resulting from the 100-year peak discharge at State Road 358 over White River near Edwardsport, Ind.*

[ft^3/s, cubic feet per second; all elevations refer to feet above sea level; ft/s, feet per second; mm, millimeters; --, no data or computations]

Pier-scour characteristic	Pier number			
	5	4	3	2
Total discharge, in ft^3/s	110,000	110,000	110,000	110,000
Water-surface elevation	457.3	457.3	457.3	457.3
Approach depth, in feet	10.3	24.1	23.3	31.6
Approach velocity, in ft/s	5.5	8.2	10.0	7.2
Angle of attack, in degrees	0	0	0	0
Estimated grain size, in mm	.4	.4	.4	.4
Computed depth of scour or elevation				
Contraction scour, in feet	21.3	-0.5	-0.5	-0.5
Local scour, in feet	6.1	8.1	8.8	8.0
Correction for bedforms, in feet	.6	.8	.9	.8
Computed elevation	419.0	424.8	424.8	417.4

Abutment-scour characteristic	Abutment	
	Left	Right
Total discharge, in ft^3/s	110,000	110,000
Water-surface elevation	457.3	457.3
Abutment location	Set back	Edge of channel
Overbank flow	Yes	Yes
Bedload condition	Clear water	Live bed
Abutment type	Spill through	Spill through
Discharge blocked, in ft^3/s	3,400	6,510
Length, in feet	292.0	241.0
Approach depth, in feet	7.3	6.2
Approach velocity, in ft/s	1.6	4.4
Angle of abutment, in degrees	0	0
Estimated grain size, in mm	.4	.4
Computed depth of scour or elevation		
Contraction scour, in feet	21.3	-0.5
Local scour, in feet	18.5	24.4
Correction for bedforms, in feet	0	0
Computed elevation	407.6	398.8

Table 34. *Potential scour resulting from the 500-year peak discharge at State Road 358 over White River near Edwardsport, Ind.*

[ft^3/s, cubic feet per second; all elevations refer to feet above sea level; ft/s, feet per second; mm, millimeters; --, no data or computations]

Pier-scour characteristic	Pier number			
	5	4	3	2
Total discharge, in ft^3/s	141,900	141,900	141,900	141,900
Water-surface elevation	459.0	459.0	459.0	459.0
Approach depth, in feet	12.0	25.8	25.0	33.3
Approach velocity, in ft/s	7.0	9.5	10.9	7.7
Angle of attack, in degrees	0	0	0	0
Estimated grain size, in mm	.4	.4	.4	.4
Computed depth of scour or elevation				
Contraction scour, in feet	27.0	-1.0	-1.0	-1.0
Local scour, in feet	6.9	8.7	9.2	8.3
Correction for bedforms, in feet	.7	.9	.9	.8
Computed elevation	412.4	424.6	424.9	417.6

Abutment-scour characteristic	Abutment	
	Left	Right
Total discharge, in ft^3/s	141,900	141,900
Water-surface elevation	459.0	459.0
Abutment location	Set back	Edge of channel
Overbank flow	Yes	Yes
Bedload condition	Clear water	Live bed
Abutment type	Spill through	Spill through
Discharge blocked, in ft^3/s	5,630	9,500
Length, in feet	292.0	254.7
Approach depth, in feet	9.0	7.5
Approach velocity, in ft/s	2.1	5.0
Angle of abutment, in degrees	0	0
Estimated grain size, in mm	.4	.4
Computed depth of scour or elevation		
Contraction scour, in feet	27.0	-1.0
Local scour, in feet	23.2	28.8
Correction for bedforms, in feet	0	0
Computed elevation	397.1	394.3

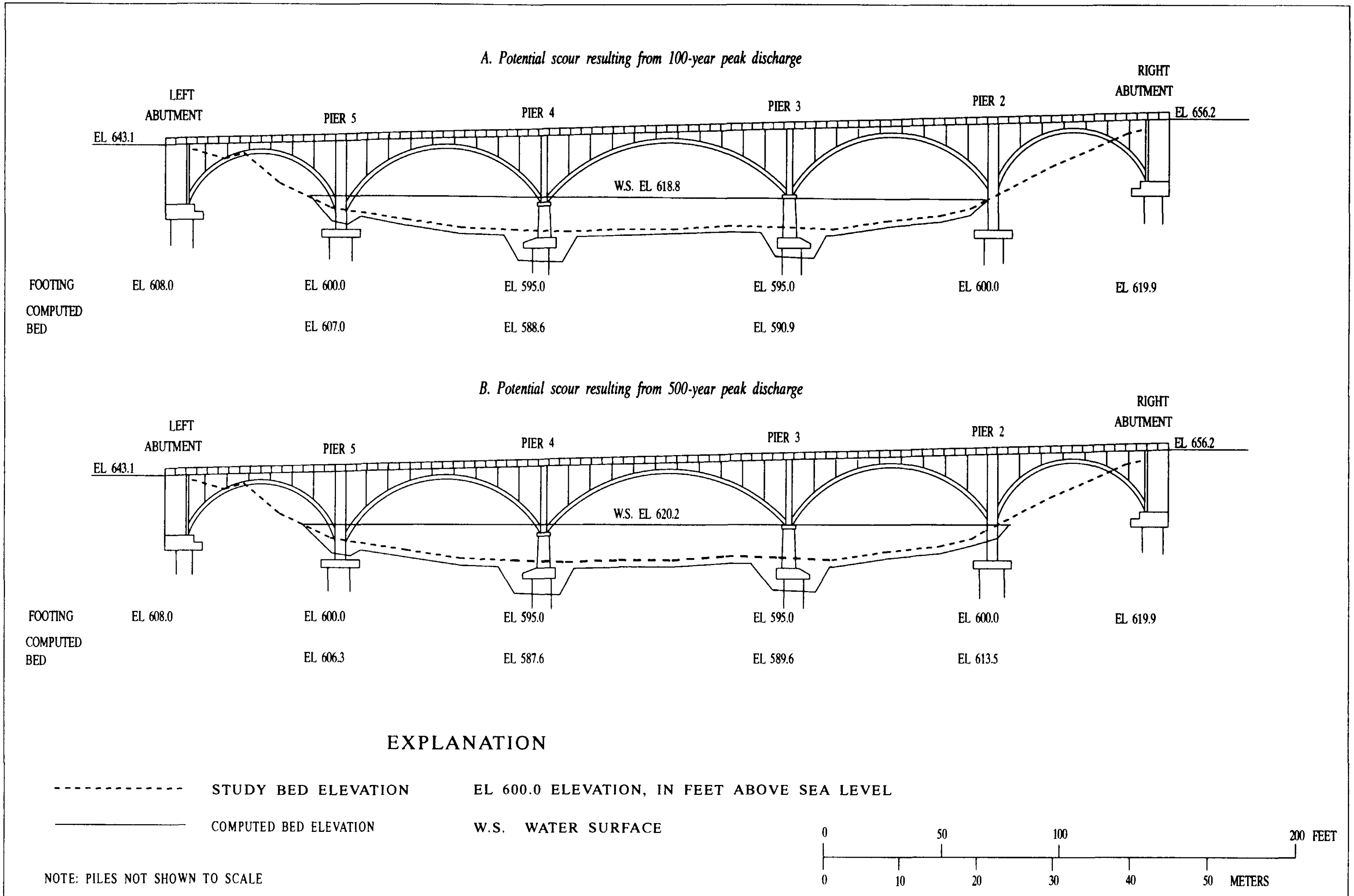


Figure 29. Potential scour at U.S. Route 24 over Tippecanoe River at Monticello, Indiana: (A) 100-year peak discharge, and (B) 500-year discharge.

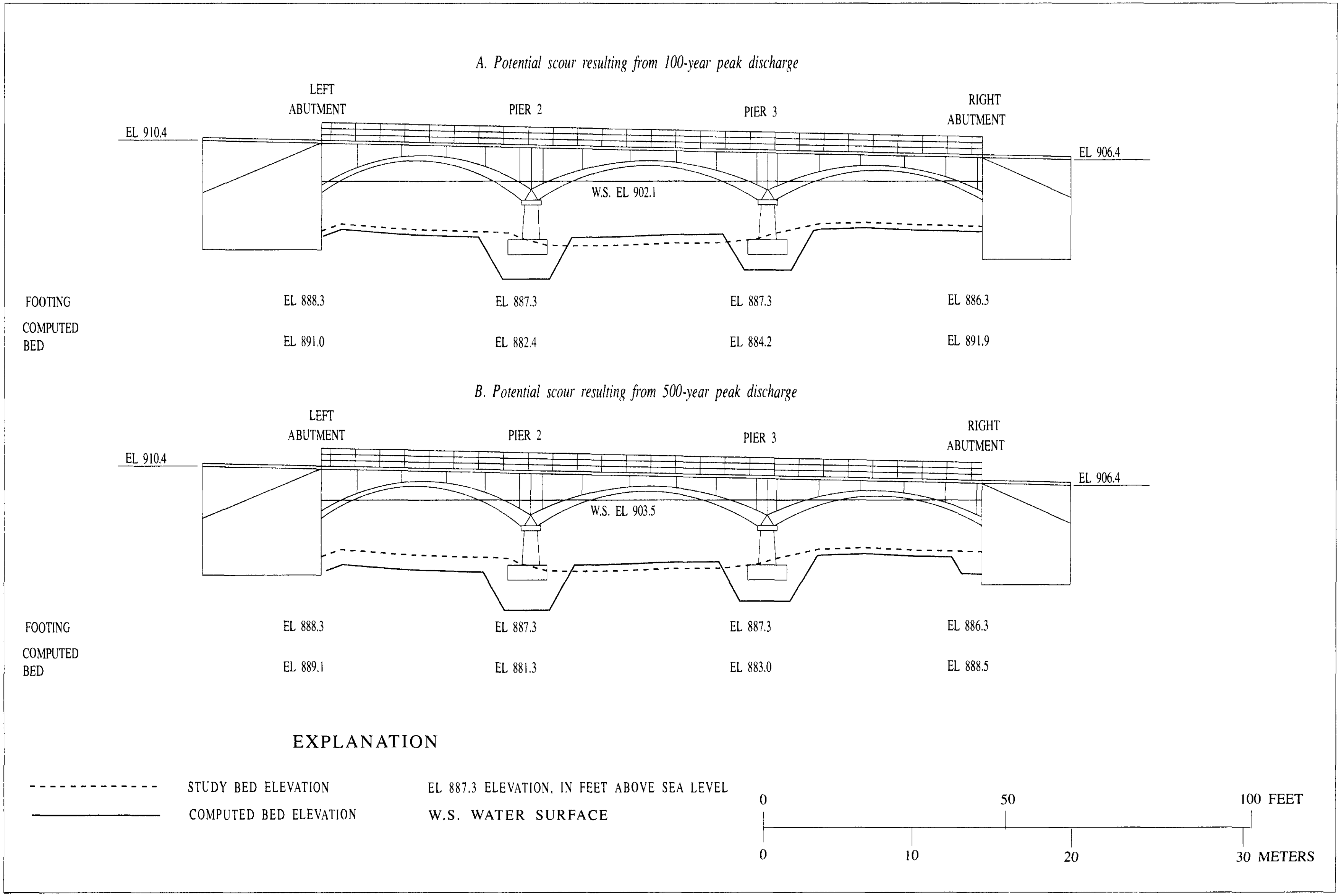


Figure 30. Potential scour at State Road 32 over Buck Creek at Yorktown, Indiana.

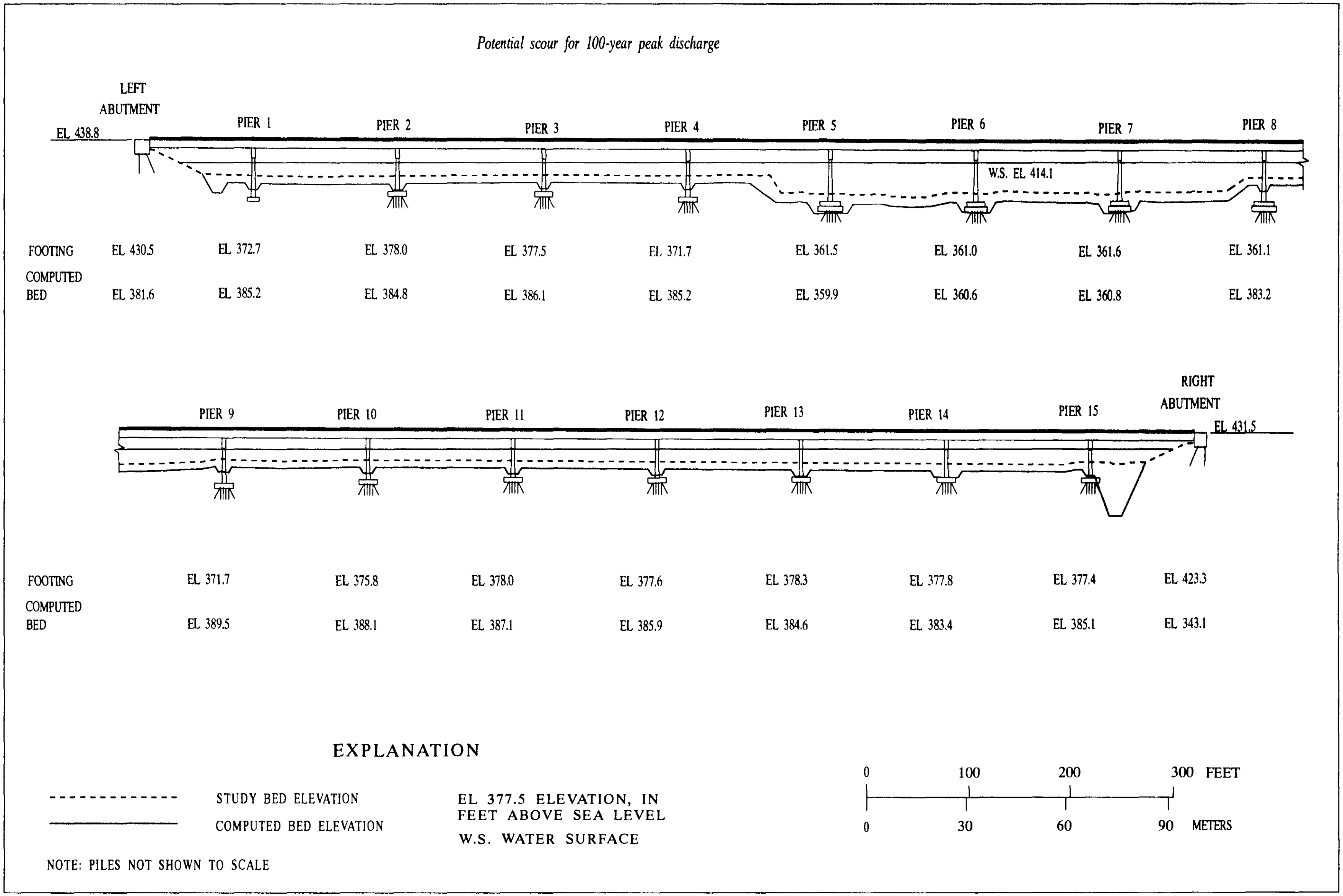


Figure 31(a). Potential scour at U.S. Route 41 over White River near Hazelton, Indiana.

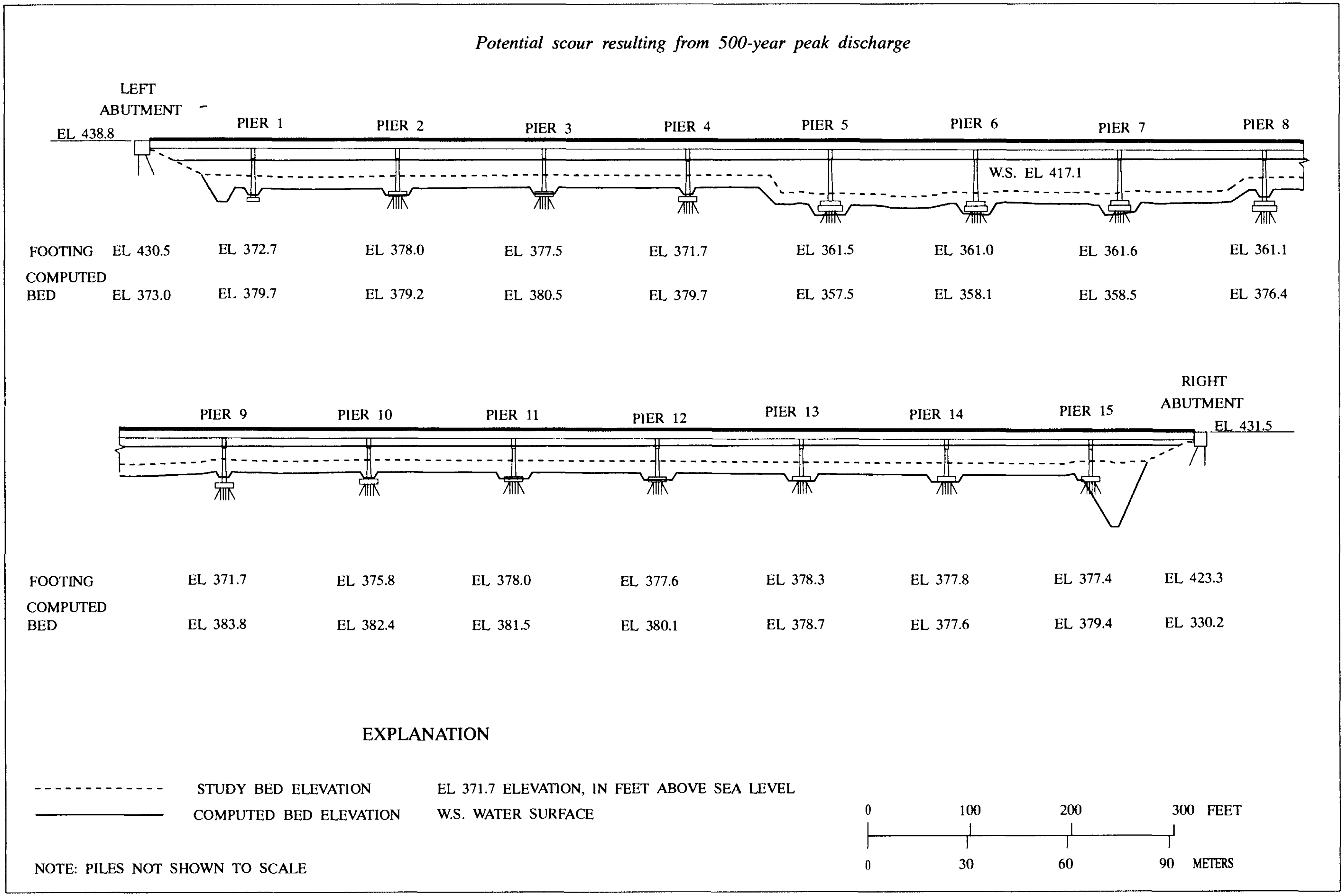


Figure 31(b). Potential scour at U.S. Route 41 over White River near Hazleton, Indiana.

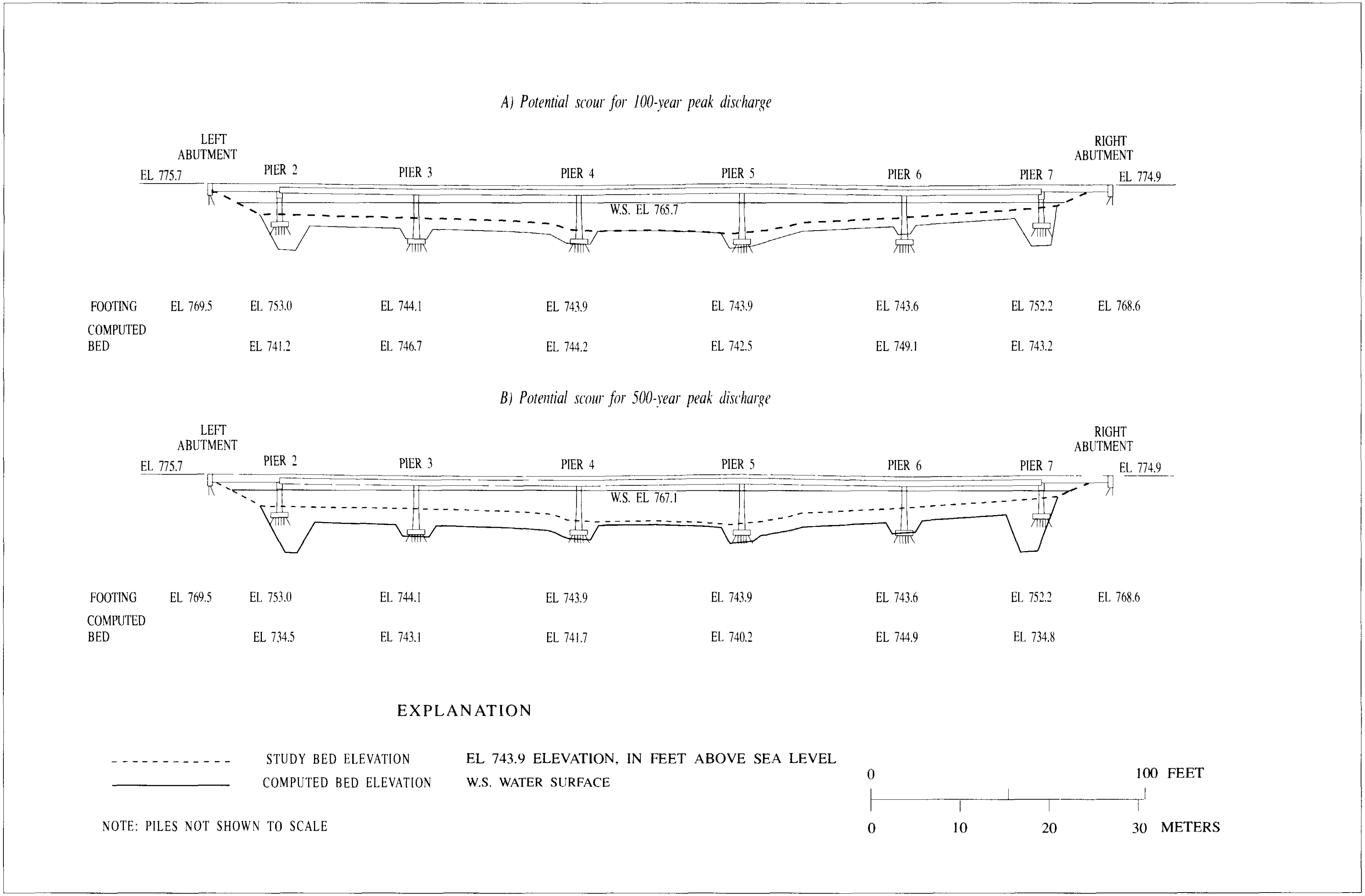


Figure 32. Potential scour at I-74 over Big Blue River near Shelbyville, Indiana: (A) 100-year peak discharge, and (B) 500-year peak discharge.

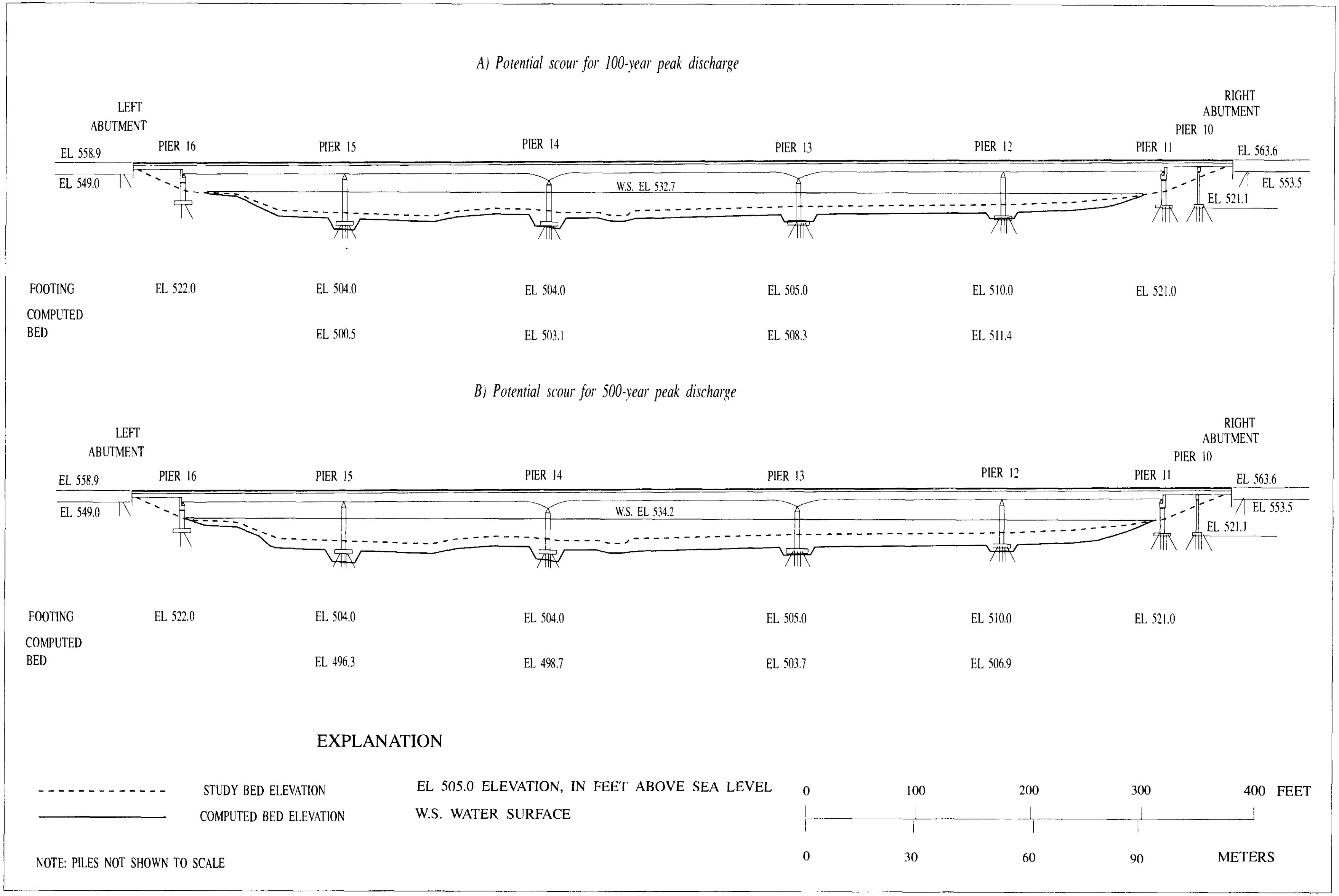


Figure 33. Potential scour at I-74 over Whitewater River near Harrison, Ohio: (A) 100-year peak discharge, and (B) 500-year peak dsicharge.

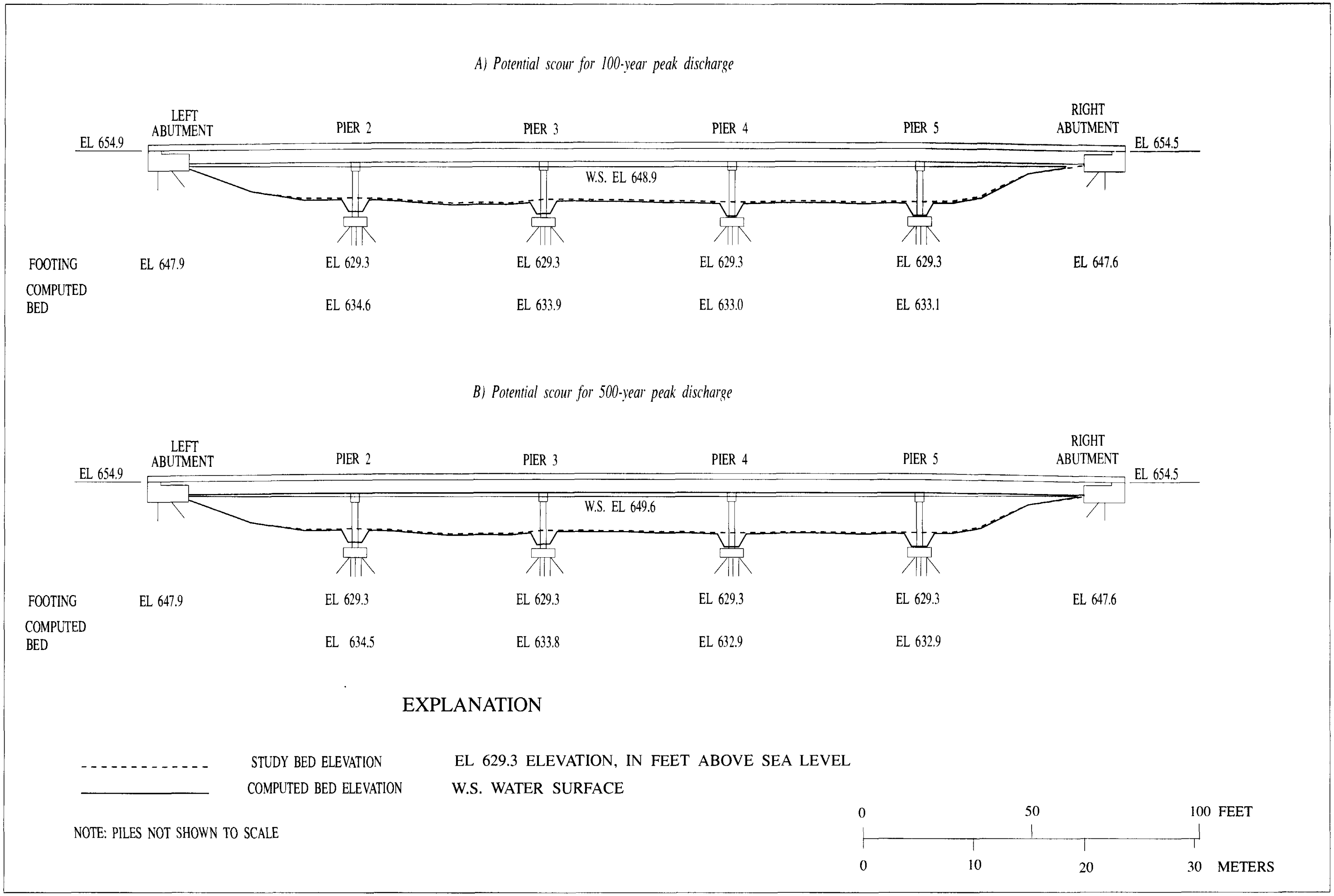


Figure 34. Potential scour at U.S. Route 231 over Kankakee River near Hebron, Indiana: (A) 100-year peak discharge, and (B) 500-year peak discharge.

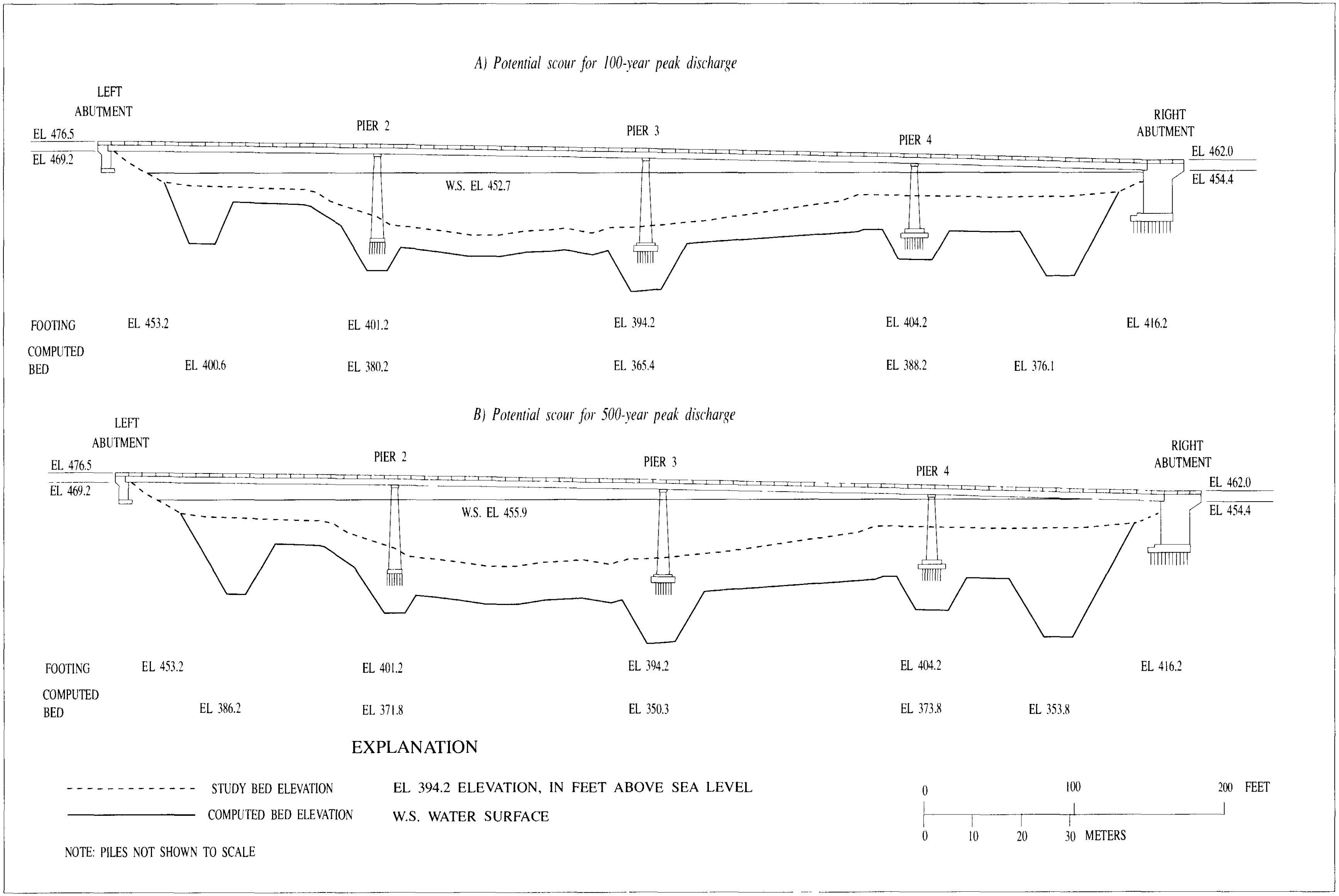


Figure 35. Potential scour at U.S. Route 231 over East Fork White River near Haysville, Indiana: (A) 100-year peak discharge and (B) 500-year peak discharge.

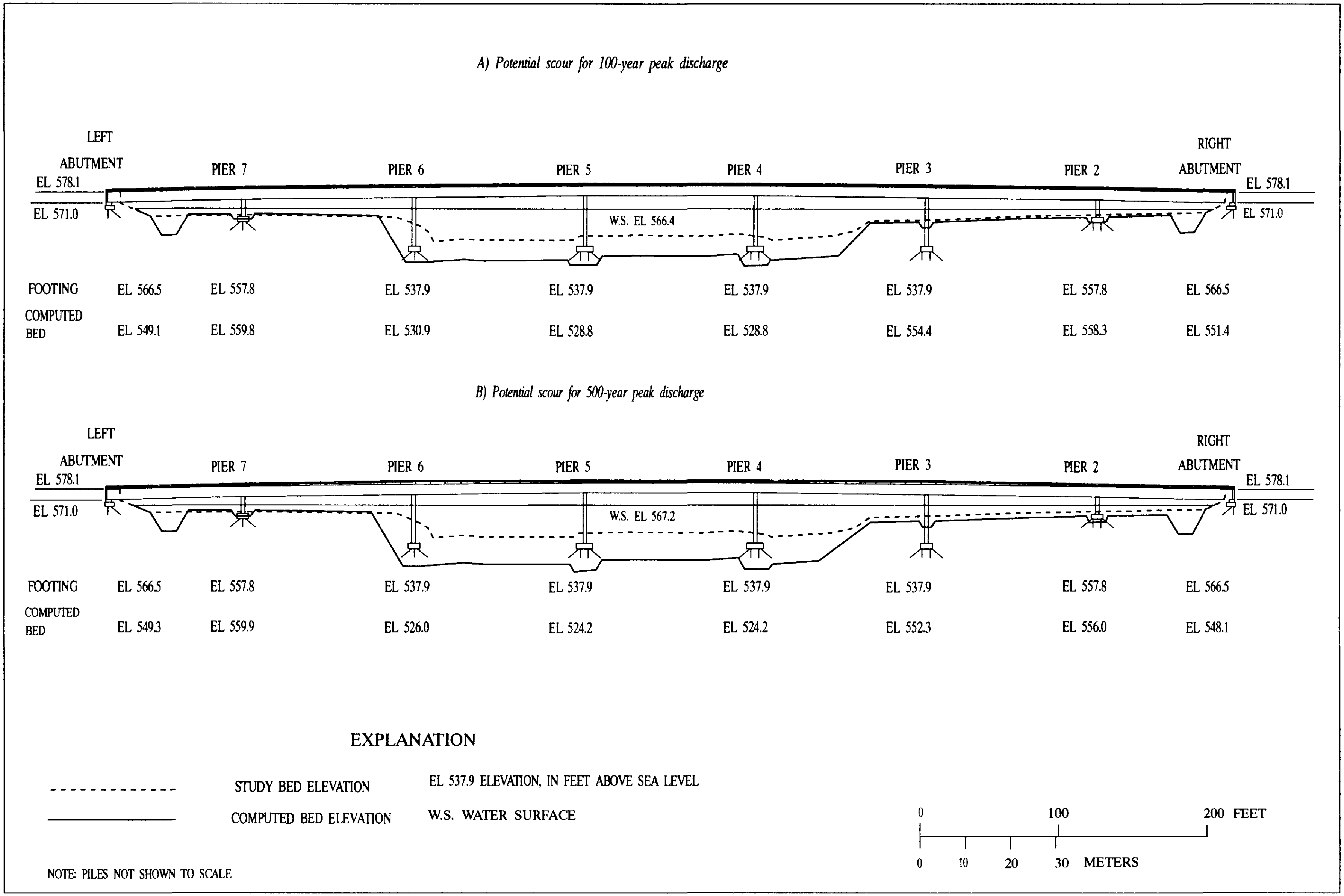


Figure 36. Potential scour at State Road 258 over East Fork White River near Seymour, Indiana: (A) 100-year peak discharge and (B) 500-year peak discharge.

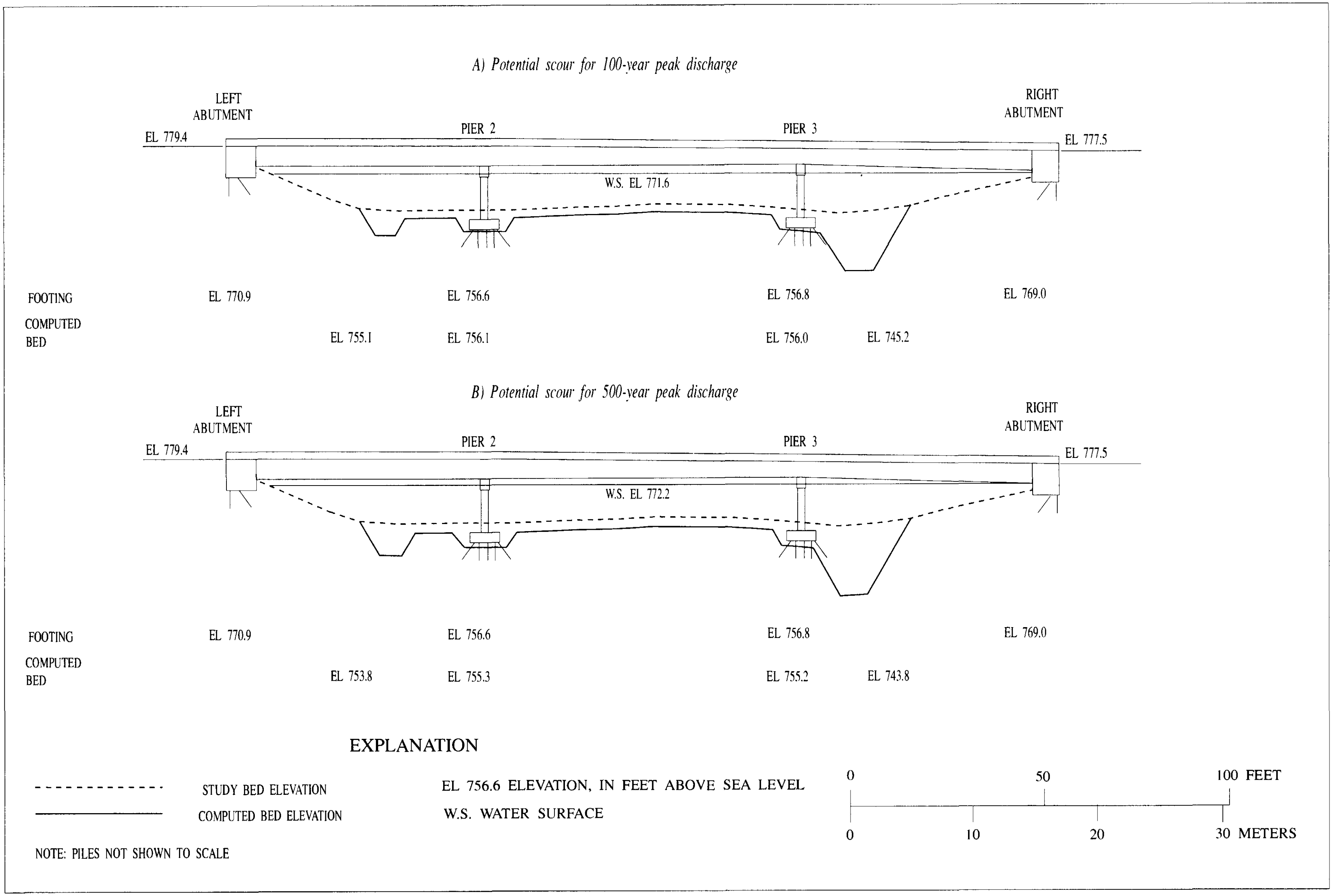


Figure 37. Potential scour at State Road 331 over Tippecanoe River at Old Tip Town, Indiana: (A) 100-year peak discharge and (B) 500-year peak discharge.

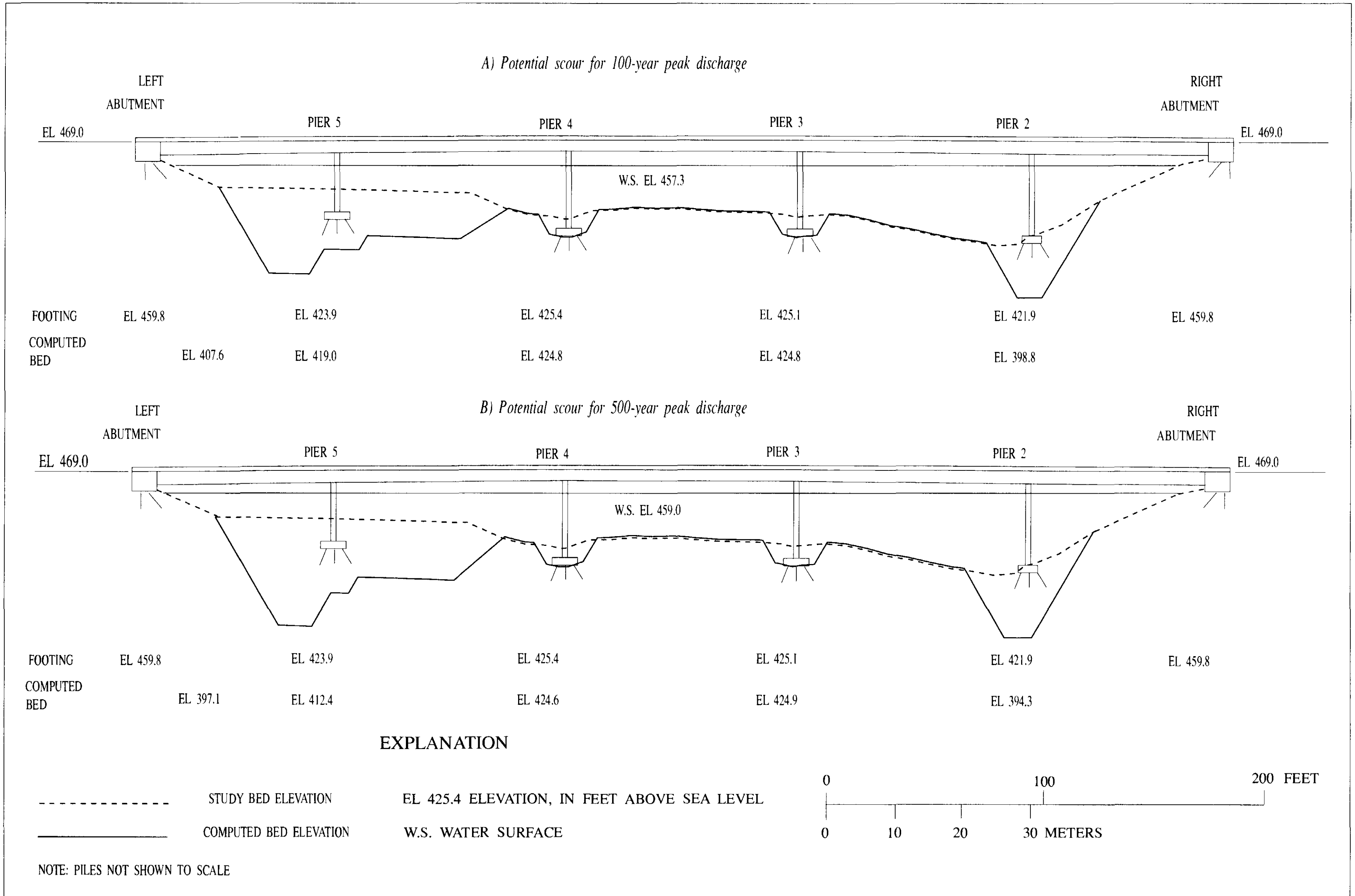


Figure 38. Potential scour at State Road 358 over White River near Edwardsport, Indiana: (A) 100-year peak discharge, and (B) 500-year peak discharge

SUMMARY AND CONCLUSIONS

Scour around bridges is a serious problem on many rivers; bridge failure commonly is attributed to undermining of piers or abutments by scour. This study evaluated 10 bridge sites to determine the feasibility of using geophysical techniques to measure the maximum historic scour, present estimates of potential scour resulting from the 100- and 500-year floods, and assess the reliability of 13 scour equations for application in Indiana.

Geophysical techniques consisting of GPR and a tuned transducer were used to survey 10 bridge openings to locate evidence of scour holes that may have refilled. The GPR was used successfully on gravel bars and in water less than 4 ft deep. In water depths greater than 4 ft, however, the signal was attenuated in the water column because of the high specific conductance of the water. The tuned transducer was used with a 3.5- to 7-kHz and a 14 kHz transducer suspended 6 to 12 in. below the water surface. This equipment was usable in water depths greater than 5 ft. Side echo, debris, point reflections from cobbles and boulders, and multiple reflections obscured data from both the GPR and the tuned transducer. From the 10 sites surveyed, results at 9 sites were considered adequate for determination of the approximate location and depth of subsurface interfaces indicating scour holes; however, the record was not of sufficient resolution to map the lateral extent of the refilled scour hole.

The historical data collected by use of geophysical techniques were used to evaluate the applicability of 13 pier-scour equations. This comparison was made by assuming that the historical scour measured by use of geophysical techniques was associated with the peak historical discharge. In an ideal situation, such as under laboratory conditions, this would be a valid assumption; however, this assumption is suspect for field conditions. In the field, debris accumulations, ice jams, and other anomalies affect the depth of scour occurring at a given discharge, making it possible that the scour measured was associated with a lesser discharge in combination with debris or ice accumulations. Because the historical data contained only estimates of total scour, the contraction- and pier-scour equations were used to compute bed elevations used for the evaluation. Inaccuracies inherent in the contraction-scour computations were uniformly transferred to all of the pier-scour equations, and the accuracy of the contraction-scour equation could not be separated from the pier-scour equations.

Only the Inglis-Lacey, Chitale, and Ahmad pier-scour equations commonly produced results that were different from the historical data. Although no data were available to evaluate the contraction-scour equation, a few of the contraction-scour computations resulted in what appears to be excessive scour, especially in clear-water conditions. The evaluation of the pier-scour equations failed to identify an equation that accurately predicted the historical scour at all of the study sites; however, on the basis of the data presented herein, the FHWA procedures (Laursen's contraction-scour equation combined with the CSU pier-scour equation) provided a combination of accuracy and safety, required by design equations, equal to or better than the other equations evaluated. Additional data, especially real-time data, are needed to verify this evaluation.

The potential scour resulting from the 100-year and 500-year peak discharges were computed by use of the FHWA recommended procedures. At S.R. 331 over Tippecanoe River and at I-74 over Big Blue River, the FHWA procedure underestimated the historical scour by more than 5 ft; therefore, potential scour also may be underestimated. At U.S. Route 231 over East Fork White River, however, the FHWA procedure overestimated the historical scour by 15 to 20 ft, and at S.R. 258 over East Fork White River, the recommended procedure overestimated the historical scour by about 10 ft. At both sites, contraction scour in excess of 10 ft was computed. Additional data and sediment-transport modeling at these sites are required to verify the accuracy of the potential-scour computations. Computed abutment scour at about half of the sites seems to be excessive; however, current FHWA guidance recommends setting abutment foundations to standards of American Association of State Highway and Transportation Officials and that abutment protection be provided by use of riprap. If appropriate protection of the abutments is provided, abutment scour need not be calculated.

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