SCOUR ASSESSMENTS AND SEDIMENT-TRANSPORT SIMULATION FOR SELECTED BRIDGE SITES IN SOUTH DAKOTA

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CONVERSION FACTORS AND VERTICAL DATUM

Multiply	By	To obtain
cubic foot per second (ft^3/s)	0.02832	cubic meter per second
foot (ft)	0.3048	meter
foot squared (ft^2)	0.0929	meter squared
foot squared per second (ft^2/s)	0.0929	meter squared per second
foot per second (ft/s)	0.3048	meter per second
foot per second squared (ft/s^2)	0.3048	meter per second squared
inch (in.)	25.4	millimeter
mile (mi)	1.609	kilometer
square foot (ft^2)	0.0929	meter squared
square mile (mi^2)	2.590	square 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 general adjustment of the first-order level nets of the United States and Canada, formerly called Sea Level Datum of 1929.

Water year: Water year is the 12-month period from October 1 through September 30. The actual water year is designated by the calendar year in which it ends (includes 9 of 12 months).

Scour Assessments and Sediment-Transport Simulation for Selected Bridge Sites in South Dakota

By Colin A. Niehus

ABSTRACT

Scour at bridges is a major concern in the design of new bridges and in the evaluation of structural stability of existing bridges. Equations for estimating pier, contraction, and abutment scour have been developed from numerous laboratory studies using sand-bed flumes, but little verification of these scour equations has been done for actual rivers with various bed conditions. This report describes the results of reconnaissance and detailed scour assessments and a sediment-transport simulation for selected bridge sites in South Dakota.

Reconnaissance scour assessments were done during 1991 for 32 bridge sites. The reconnaissance assessments for each bridge site included compilation of general and structural data, field inspection to record and measure pertinent scour variables, and evaluation of scour susceptibility using various scour-index forms. Observed pier scour at the 32 sites ranged from 0 to 7 feet, observed contraction scour ranged from 0 to 4 feet, and observed abutment scour ranged from 0 to 10 feet.

Thirteen bridge sites having high potential for scour were selected for detailed assessments, which were accomplished during 1992-95. These detailed assessments included prediction of scour depths for 2-, 100-, and 500-year flows using selected published scour equations; measurement of scour during high flows; comparison of measured and predicted scour; and identification of which scour equations best predict actual scour.

The medians of predicted pier-scour depth at each of the 13 bridge sites (using 13 scour equations) ranged from 2.4 to 6.8 feet for the 2-year flows and ranged from 3.4 to 13.3 feet for the 500-year flows. The maximum pier scour measured during high flows ranged from 0 to 8.5 feet. Statistical comparison (Spearman rank correlation) of predicted pier-scour depths (using flow data collected during scour measurements) indicate that the Laursen, Shen (method b), Colorado State University, and Blench (method b) equations correlate closer with measured scour than do the other prediction equations. The predicted pier-scour depths using the Varzeliotis and Carstens equations have weak statistical relations with measured scour depths. Medians of predicted pier-scour depth from the Shen (method a), Chitale, Bata, and Carstens equations are statistically equal to the median of measured pier-scour depths, based on the Wilcoxon signed-ranks test.

The medians of contraction scour depth at each of the 13 bridge sites (using one equation) ranged from -0.1 foot for the 2-year flows to 23.2 feet for the 500-year flows. The maximum contraction scour measured during high flows ranged from 0 to 3.0 feet. The contraction-scour prediction equation substantially overestimated the scour depths in almost all comparisons with the measured scour depths. A significant reason for this discrepancy is due to the wide flood plain (as wide as 5,000 feet) at most of the bridge sites that were investigated. One possible way to reduce this effect for bridge design is to make a decision

on what is the effective approach section and thereby limit the size of the bridge flow approach width.

The medians of abutment-scour depth at each of the 13 bridge sites (using five equations) ranged from 8.2 to 16.5 feet for the 2-year flows and ranged from 5.7 to 41 feet for the 500-year flows. The maximum abutment scour measured during high flows ranged from 0 to 4.0 feet. The abutment-scour prediction equations also substantially overestimated the scour depths in almost all comparisons with the measured scour depths. The Liu and others (live bed) equation predicted abutment-scour depths substantially lower than the other four abutment-scour equations and closer to the actual measured scour depths. However, this equation at times predicted greater scour depths for 2-year flows than it did for 500-year flows, making its use highly questionable. Again, limiting the bridge flow approach width would produce more reasonable predicted abutment scour.

During 1994-95, the Bri-Stars sediment-transport model was run for the White River near Presho bridge site to better understand the sediment and hydraulic processes at this site. The transport simulation was run with the 2-year flow event (9,860 cubic feet per second) and with the flow event just below road overtopping (28,500 cubic feet per second). The results for the 9,860-cubic feet per second simulation show aggradation at the bridge section of 1.5 feet across the flood plain and about 0.5 foot in the main channel. The total predicted sediment load for the 2-year flow event at cross sections located within about 1,400 feet of the bridge section (upstream and downstream) ranges from 159 to 391 tons. The results for the 28,500-cubic feet per second simulation show degradation at and upstream of the bridge section. The degradation at the bridge section is about 1.5 to 2.0 feet across most of the section. The thalweg profiles show degradation at and upstream of the bridge section and both aggradation and degradation downstream of the bridge section. The total predicted sediment load for the 28,500-cubic feet per second flow event at cross sections located within about 1,400 feet of the bridge section (upstream and downstream) ranges from 169 to 1,618 tons. A series of model simulations was performed to analyze the sensitivity of the transport simulation to various parameters. The sensitivity results for the 9,860-cubic feet per second simulation show that the active-layer thickness, sediment equation, roughness coefficient, and sediment size are the most sensitive parameters. The sediment inflow, the number of stream tubes, and the number of time steps are the least sensitive parameters. The sensitivity results for the 28,500-cubic feet per second simulation show that the active-layer thickness, sediment equation, number of stream tubes, roughness coefficient, and sediment size are the most sensitive parameters. The sediment inflow and number of time steps are the least sensitive parameters.

INTRODUCTION

Scour at bridges is the most common cause of bridge failure (Butch, 1991). Consequently, an understanding of bridge scour and methods for estimating scour are vital for the design of new bridges and the maintenance of existing bridges. Equations have been developed for estimating scour, but they are based primarily on laboratory tests with sand-bed flumes. Little verification of these scour equations has been done on actual rivers with various bed materials. The equations generally tend to overestimate scour depth on silt- and clay-bed streams, partly because the equations were developed for sand-bed streams.

Total scour at bridges is made up of three components: (1) general scour, (2) contraction scour, and (3) local scour. General scour involves geomorphological processes that cause degradation and/or aggradation of the stream or river, separate from any effects of the bridge. Degradation and aggradation are the long-term adjustments of the streams and rivers to past disturbances such as construction of bridges, construction of dams, changes in land use in watersheds, changes in the alignment of streams or rivers, and changes in available sediment load. Contraction scour is the general lowering of the channel section due to flow acceleration through the channel constriction caused by the bridge. Contraction scour can occur when the bridge abutments are constructed in the main channel or when the bridge is constructed in the flood plain of the river or stream. The stream or river tends to scour the channel bottom to increase the flow area and consequently decrease the flow velocity. Local scour is the localized erosion around obstructions in the flow. Local scour at bridges includes pier and abutment scour. Scour at piers is caused by the pileup of water on the upstream face of the pier and the resultant vortices that remove materials from the base region of the pier structure. The downstream side of the pier undergoes scour due to vortices in the wake region. Abutment scour is caused by vortices formed where the flow accelerates around the structure.

In 1991, the U.S. Geological Survey (USGS), in cooperation with the South Dakota Department of Transportation (SDDOT), began a 5-year study of bridge scour in South Dakota. The project was part of a national cooperative effort among the States, the Federal Highway Administration (FHWA), and the USGS to analyze scour potential at existing bridge sites. The FHWA has established a requirement that all State highway agencies evaluate the bridges on the Federal Aid System for susceptibility to scour-related failure.

Purpose and Scope

The purpose of this report is to summarize: (1) reconnaissance scour assessments of 32 selected bridge sites in South Dakota; (2) detailed scour analyses of 13 of the 32 selected bridge sites, including comparison of predicted and measured scour; and (3) sediment-transport simulation for one bridge site. The reconnaissance scour assessments were done during 1991 and involved the compilation of data (pertinent to scour) available for each of the 32 selected bridge sites, and field visits to each of the sites to inspect, measure, and record variables important to bridge scour. Thirteen of the 32 bridge sites were chosen for detailed assessments, which were accomplished during 1992-95. These detailed assessments included computation of 2-, 100-, and 500-year flood flows; computation of scour depths for the 2-, 100-, and 500-year flood flows; scour and flow measurement during high flows; and comparison of measured scour and predicted scour. During 1994-95, sediment transport was simulated for one bridge site using the Bri-Stars (Bridge Stream Tube Model for Alluvial River Simulation) model in order to better understand the sediment and hydraulic processes.

Acknowledgments

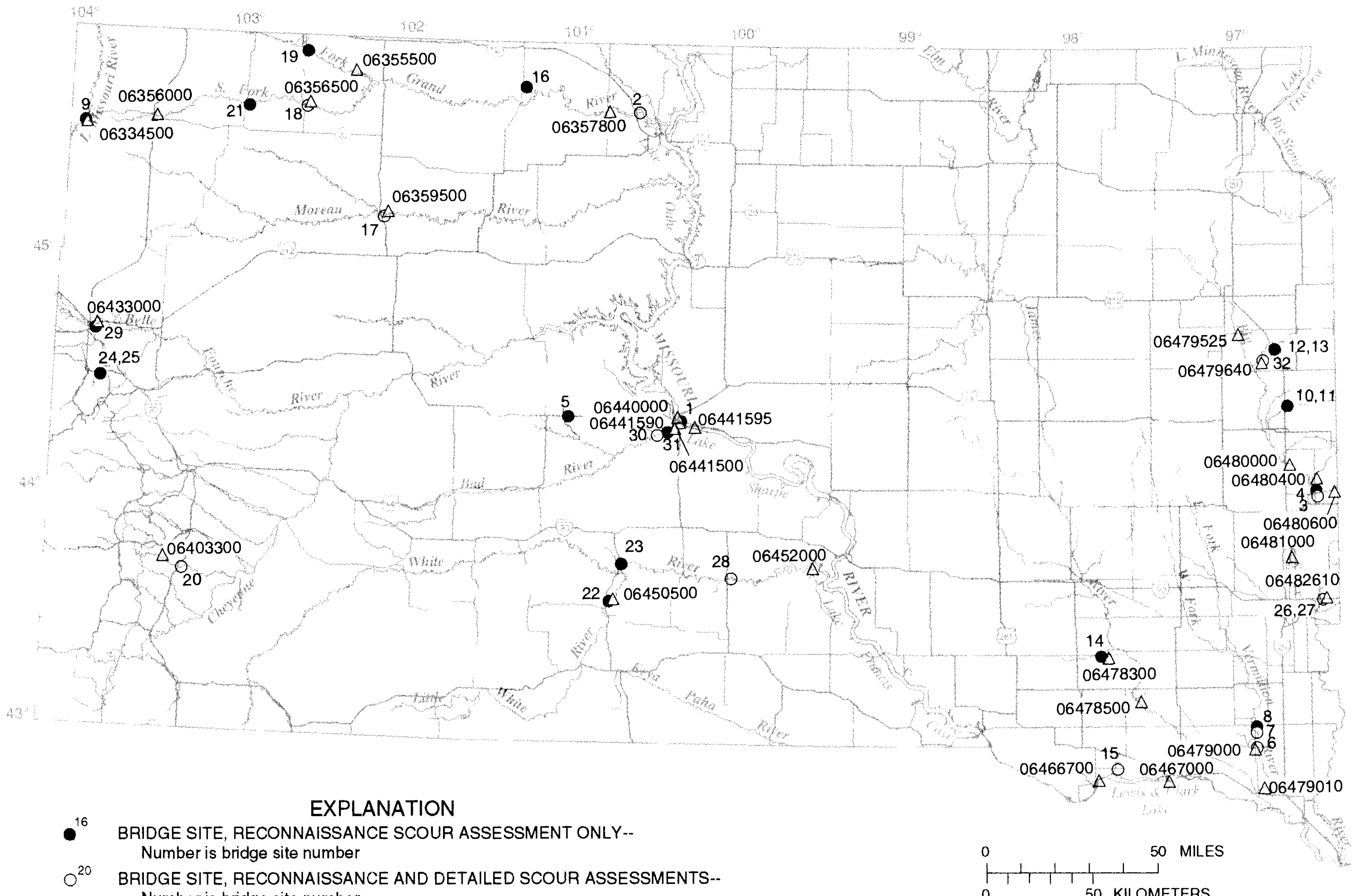
The author thanks the SDDOT for providing bridge site plans, inspection reports, boring logs, and general assistance for the bridge sites that were investigated. The author also appreciates the assistance given by the FHWA in reviewing some of the work that was done.

RECONNAISSANCE SCOUR ASSESSMENTS FOR SELECTED BRIDGE SITES

Thirty-one bridge sites having high scour potential were originally selected by the SDDOT for reconnaissance scour assessments. One additional bridge site was subsequently added at the request of the FHWA. These scour assessments were done during 1991 and involved compilation of information and structural data available for each of the selected bridge sites and field visits to inspect, measure, and record variables considered to be important to bridge scour. Required field data and methods used in the investigation are described by Davidian (1984), Shearman and others (1986), Arcement and Schneider (1989), Shearman (1990), and Richardson and others (1991).

Description of Bridge Sites and Tabulation of Reconnaissance Scour Assessments

The 32 bridge sites selected for reconnaissance scour assessment are presented in figure 1. Some of the rivers and streams on which the bridge sites are located include the North Fork Grand River, South Fork Grand River, Grand River, Moreau River, Redwater River, Little White River, White River, Vermillion River, Split Rock Creek, and Big Sioux River. Major rivers not having any bridge sites assessed for scour in this study include the Missouri River, which in South Dakota is mostly a system of reservoirs, and the James River located in east-central South Dakota. Construction plans and inspection reports for each bridge site were obtained from SDDOT and used in determining scour potential.



EXPLANATION

● 16 BRIDGE SITE, RECONNAISSANCE SCOUR ASSESSMENT ONLY--
Number is bridge site number

○ 20 BRIDGE SITE, RECONNAISSANCE AND DETAILED SCOUR ASSESSMENTS--
Number is bridge site number

△ 06356000 GAGING STATION-- Number is station identification number

Figure 1. Locations of selected bridge sites (see table 1) and U.S. Geological Survey gaging stations.

The lengths of the bridges that were studied range from 42 to 556 ft. Most of the selected bridges have piers on either spread footings or pilings. The abutment types include spill-through or vertical abutments. Spill-through abutments are characterized by sloped embankments that channel the water through the bridge opening and protect the concrete bridge abutment structures from scour. Scour at spill-through abutments is about 50 percent of that of vertical wall abutments (Richardson and others, 1991). Vertical abutments have no protecting sloping embankments at the abutment structure. Many are characterized by a vertical concrete wall that intersects the main channel of the stream. Pier shapes at the inspected bridge sites include round, square, rectangular, pointed, and octagon (see Richardson and others, 1991, for illustration of common pier shapes). Some of the sites also have sets of piers, one pier upstream and one pier downstream, at a particular location under the bridge. Some of these sites have the two piers connected by concrete, forming a web. The flood-plain widths at the bridge sites range from 150 to 5,000 ft, and grass is the predominant flood-plain cover. Silt and clay are the predominant bed materials of the streams or rivers. Potential debris accumulation at the bridge sites was common.

A cross section was defined at each bridge site to help determine if there was existing scour at the site. Scour was measured by determining the difference between a reference line and the existing bed. The reference line was drawn on the cross section at locations to represent pre-scour conditions and was drawn to remove any apparent scour holes caused by pier and abutment scour. Contraction scour was measured by first making a determination of where the bed was before any contraction scour took place and measuring the difference. The mount of pier and abutment scour was removed before the contraction-scour measurement was completed. Historical cross-sectional data, when available, were used to help determine the amount of scour. The cross section was defined by using a sounding weight attached to a measuring tape and measuring down from the bridge deck. The channel also was inspected by wading the stream or river where possible and measuring scour using the water surface as a reference point. This was especially necessary where the piers were inset under the bridge. Observed pier scour ranged from 0 to 7 ft, observed contraction scour ranged from 0 to 4 ft, and observed abutment scour ranged from 0 to 10 ft.

Clear-water and live-bed scour are two particular conditions of scour (Richardson and others, 1991). Clear-water scour occurs where there is no movement of the bed material of the stream upstream of the bridge crossing. Typical clear-water situations include flat low-gradient streams during low flow and vegetated channels. Most of the streams investigated that had observed scour holes meet these conditions, especially considering the heavily vegetated flood plains. Also, most of the banks at and near the bridge sites were in good condition, with almost 100-percent vegetative cover. Live-bed scour occurs when the bed material upstream of the bridge crossing is moving.

Data obtained for the 31 original bridge sites, plus the added bridge site on Hidewood Creek near Estelline (site 32), were grouped into index and structural data, which are shown in tables 1 and 2, respectively. The channel data for the 32 bridge sites, including a listing of the scour observed at each bridge site, are summarized in table 3. Estimates of the Manning "n," using Arcement and Schneider (1989) and experience as guidelines, also are included in table 3. Additional scour-assessment information collected for each of the sites is included in the Supplement Information section at the end of this report.

Data for streamflow- and/or stage-gaging stations located at or near the bridge-scour study sites are summarized in table 4. The locations of these gaging stations are shown in figure 1. The data from these stations were used in the hydrologic and hydraulic analyses of the selected bridge sites.

Selection of Bridge Sites for Detailed Scour Assessments

Three previously developed scour-assessment forms were used to assess scour potential at the 32 bridge sites and to help select bridge sites for detailed scour assessments. These forms include a checklist used in New York for bridge-site selections (fig. 2), an observed-scour index form used in Tennessee (fig. 3), and a potential-scour index form used in Tennessee (fig. 4).

A checklist used in New York was used to take into account bridge-site parameters that were ideal for bridge-scour measurements. This checklist assigns

Table 1. Summary of index data for selected bridge sites

[dms, degrees, minutes, and seconds; ---, no data or not applicable]

Site number (fig. 1)	River/stream	Nearest town	Bridge structure number	Highway number	Highway log mile[1]	Latitude (dms)	Longitude (dms)
1	Capitol Lake outlet	Pierre	33-113-123	Capitol Ave.	---	442152	1002035
2	Grand River	Mobridge	16-665-200	US 12	173.40	453954	1003813
3	Big Sioux River	Flandreau	51-150-099	SD 13	108.13	440307	963513
4	Big Sioux River	Flandreau	51-150-082	SD 13	109.93	440440	963512
5	Frozen Man Creek	Hayes	59-078-279	frontage of US 14	---	442219	1010105
6	Vermillion River overflow	Wakonda	14-100-062	SD 19	19.22	425943	965747
7	Vermillion River	Centerville	14-100-019	SD 19	23.50	430327	965748
8	Vermillion River	Centerville	14-100-001	SD 19	25.25	430458	965748
9	Little Missouri River	Camp Crook	32-043-278	SD 20	4.26	453253	1035817
10	East Branch North Deer Creek	Brookings	06-185-074	I29N	141.45	442610	964522
11	East Branch North Deer Creek	Brookings	06-184-074	I29S	141.45	442610	964523
12	Hidewood Creek	Clear Lake	20-027-207	I29S	159.16	444042	964947
13	Hidewood Creek	Clear Lake	20-028-207	I29N	159.16	444042	964946
14	North Branch Dry Creek	Parkston	34-125-080	SD 44	355.11	432305	975159
15	Snatch Creek	Springfield	05-198-180	SD 52	320.87	425430	974633
16	Hump Creek	McIntosh	16-329-127	SD 65	220.95	454554	1011947
17	Moreau River	Faith	53-392-521	SD 73	190.11	451152	1020923
18	South Fork Grand River	Bison	53-149-209	SD 75	221.82	453856	1023835
19	North Fork Grand River	Lodgepole	53-150-046	SD 75	238.75	455302	1023908
20	French Creek	Fairburn	17-400-131	SD 79	48.00	434047	1031532
21	South Fork Grand River	Buffalo	32-517-215	SD 79	210.59	453824	1025950
22	Little White River	White River	48-250-185	US 83	46.83	433605	1004458
23	Horse Creek	White River	38-192-284	US 83	58.86	434526	1004055
24	False Bottom Creek	Spearfish	41-126-087	I90W	15.33	442835	1034745
25	False Bottom Creek	Spearfish	41-126-088	I90E	15.33	442834	1034745
26	Split Rock Creek	Brandon	50-284-166	I90E	407.01	433630	963347
27	Split Rock Creek	Brandon	50-284-165	I90W	407.01	433631	963347
28	White River	Presho	43-160-339	US 183	61.53	434217	1000227
29	Redwater River	Belle Fourche	10-105-376	US 212	15.07	444002	1035021
30	Powell Creek	Fort Pierre	59-339-327	214	180.73	441811	1002912
31	Willow Creek	Fort Pierre	59-374-317	214	184.79	441903	1002507
32	Hidewood Creek	Estelline	---	---	---	443751	965327

[1]Distance, in miles, from a specified starting point on the numbered highway.

Table 2. Summary of structural data for selected bridge sites

[---, no data]

Site number (fig. 1)	Bridge length (feet)	Number of pier sets	Pier shapes	Pier width (inches)	Pier footings	Abutment type	Type of slope protection	Number of spur dikes
1	68	2	Rectangular (arch)	---	Spread	Vertical	Concrete apron and riprap	None
2	556	4	Octagon	48	Piling	Spill-through	Riprap at bridge	None
3	436	3	Octagon with web	36	Piling	Spill-through	Riprap at bridge	None
4	297	3	Pointed octagon with web	36	Piling	Spill-through	Riprap	None
5	140	5	2-square with web 3-octagon	20.5 15.5	---	Vertical	None	None
6	88	4	Square	16	Piling	Vertical	None	None
7	146	3	Square	26	Piling	Vertical	Riprap	None
8	122	3	Square	20	Piling	Vertical	Riprap	None
9	330	3	Octagon with web	1-22 2-39	Piling	Spill-through	Riprap at left	None
10	152	4	Round	24	Piling	Spill-through	None	None
11	152	4	Round	24	Piling	Spill-through	None	None
12	233	2	Round	33	Piling	Spill-through	Riprap	None
13	233	2	Round	33	Piling	Spill-through	Riprap	None
14	86	2	Square	22	Spread	Spill-through	Riprap	None
15	72	1	Square	20	Piling	Vertical	None	None
16	174	4	Square	24	Spread	Spill-through	None	None
17	378	6	3-square 3-pointed with web	30 36	Spread	Spill-through	None	None
18	234	2	Rounded with web	24	Spread	Spill-through	Riprap	None
19	342	4	Pointed with web	36	Spread	Spill-through	Riprap on left	None
20	117	4	Square	20	Spread	Spill-through	None	None
21	459	6	Octagon with web	36	5-spread 1-piling	Spill-through	None	None
22	314	4	Octagon with web	36	Spread	Spill-through	None	None
23	163	4	Square	22	Spread	Spill-through	None	None
24	106	2	Round	24	Spread	Spill-through	Riprap	None
25	106	2	Round	24	Spread	Spill-through	Riprap	None
26	330	4	Octagon	33	3-spread 1-piling	Spill-through	None	None
27	337	4	Octagon	33	3-spread 1-piling	Spill-through	Riprap upstream left	None
28	433	4	3-octagon with web 1-other	39 ---	Spread Piling	Spill-through	None	3
29	384	11	Mostly octagon	mostly 12-24	Piling	Vertical	None	None
30	42	0	---	---	---	Vertical	None	None
31	122	2	Square	27	Piling	Spill-through	Spur	None
32	72	1	"I" beam driven into ground	10	Piling	Spill-through	Riprap	None

Table 3. Summary of channel and observed-scour data for selected bridge sites

[u.s., upstream; d.s., downstream; esp., especially; ---, no remark; %, percent; <, less than]

Site number (fig. 1)	Estimated Manning "n" at bridge	Estimated Manning "n" of stream	Remarks on stream condition	Bank cover	Bank condition	Bed material	Stream approach angle	Flood-plain width (feet)	Type of flood-plain cover	Pier scour (feet)	Contraction scour (feet)	Abutment scour (feet)	Potential for debris accumulation
1	---	---	u.s.: lake	Riprap	Good	Silt & clay	Perpendicular	u.s.:lake (700) d.s.: 200	u.s.: grass d.s.: grass with trees	0	0	0	None.
2	0.032	0.034	---	90% grass & weeds	Good	Sand & silt/clay	Perpendicular	4,200	Grass, weeds, & dead trees	0 - 1	0	0	Some potential for trees at piers.
3	.030	.033	---	100% grass/weeds & trees	Good	Silt & clay	Flow approaches at a large angle	2,500	Grass with some trees	0 - 7	2	0	Some potential for trees at piers.
4	.036	.035	---	90% grass/weeds & trees	Some erosion esp. on u.s. right	Silt & clay	Flow approaches at a large angle	2,200	Grass with some trees	0 - 2.5	0	0 - 3	Some potential for small tree at piers.
5	.040	.036	In lake	100% grass/weeds & small trees	Good	Silt & clay	Flow approaches at a large angle	300	Grass/weeds & small trees	0 - 1.5	0	0	Large potential for blockage.
6	.032	.035	No u.s. channel	u.s.: crops d.s.: 100% grass & weeds	Good	Silt & clay	Temporary lake u.s.	2,000	u.s.: crops d.s.:grass & weeds	1 - 1.5	3	0	Large potential for blockage.
7	.033	.034	---	90% grass & trees	Some erosion	Silt & clay	Perpendicular at bridge but close u.s. bend	5,000	Grass with some trees	0 - 1	1	0	Large potential for tree blockage.
8	.034	.036	Degrading (u.s. & d.s. close to bend)	100% grass/weeds & trees	Some erosion because of bends	Silt & clay	Flow approaches at a large angle	4,600	Grass & small trees	0	4	0	Large potential for blockage.
9	.038	.037	Degrading	80% grass & small trees	Sloughing on left	Sand & silt/clay	Flow approaches at an angle (bridge on bend)	1,200	Grass & small trees	0 - 2	0	5	None.
10	.036	.036	---	100% grass & weeds	Good	Silt & clay	Flow approaches at an angle	1,000	Grass	0	4	0	None.
11	.036	.036	---	100% grass & weeds	Good	Silt & clay	Flow approaches at an angle	1,000	Grass	0	4	0	None.
12	.038	.036	---	100% grass & weeds	Good	Silt & clay	Perpendicular	2,200	Grass	0	0	0	None.

Table 3. Summary of channel and observed-scour data for selected bridge sites—Continued

Site number (fig. 1)	Estimated Manning "n" at bridge	Estimated Manning "n" of stream	Remarks on stream condition	Bank cover	Bank condition	Bed material	Stream approach angle	Flood-plain width (feet)	Type of flood-plain cover	Pier scour (feet)	Contraction scour (feet)	Abutment scour (feet)	Potential for debris accumulation
13	0.038	0.036	---	100% grass & weeds	Good	Silt & clay	Perpendicular	2,200	Grass	0	0	0	None.
14	.038	.035	---	100% grass/weeds & some trees	Some erosion d.s.	Silt & clay	Perpendicular	250	Grass	0	0	0	Some potential for blockage.
15	.035	.035	---	50-100% grass& small trees	Some erosion on u.s. side	Silt & clay	Perpendicular	350	Grass	0	2 (exposed footing caps)	0	Large potential for tree blockage.
16	.034	.034	Some degradation	90% grass & weeds	Some erosion	Sand & silt/clay	Perpendicular (bend close u.s.)	500	Grass & some small trees	3	0	0	Some potential for debris at pier.
17	.035	.036	---	80% grass & weeds	Some erosion esp. on left	Sand & silt/clay	Flow at an angle	700	Grass	0 - 2	0	0	Some potential for small trees at piers.
18	.036	.036	----	100% grass & weeds	Good	Silt and clay	Perpendicular	1,500	Grass	1.5 - 2	0	0 - 1	None.
19	.038	.036	---	100% grass & weeds	Good	Silt & clay	Flow approaches at a slight angle	1,000	Grass	0 - 4	0	0	None.
20	.036	.04	---	100% grass & large trees	Good	Sand, silt/clay, & gravel	On meander (flow towards left bank)	250	Grass	0 - 3	2	0	Potential for large tree blockage.
21	.038	.037	---	100% grass	Sloughing on right u.s.	Sand & silt/clay	Low flow approaches at a moderate angle	1,200	Grass	0 -4	0	0 - 2	None.
22	.030	.038	Stable (shale bottom)	0-25% u.s. 100% rest	Sloughing on left u.s.	Silt & clay	Perpendicular	600	Grass & trees	0 -2	0	0 - 2	Some potential for debris at piers.
23	.035	.035	---	100% grass & weeds	Good	Silt & clay	Flow at moderate angle	250	Grass & weeds	0 -1.5	0	0	Some potential for debris at piers.
24	0.039	0.039	Bed armored with 0.5-1 foot rocks	80% grass	Good	Gravel & cobble/ boulder	Perpendicular (channelized)	170	Grass & some small trees	0	0	0	None.

Table 3. Summary of channel and observed-scour data for selected bridge sites—Continued

Site number (fig. 1)	Estimated Manning "n" at bridge	Estimated Manning "n" of stream	Remarks on stream condition	Bank cover	Bank condition	Bed material	Stream approach angle	Flood-plain width (feet)	Type of flood-plain cover	Pier scour (feet)	Contraction scour (feet)	Abutment scour (feet)	Potential for debris accumulation
25	.039	.039	Bed armored with 0.5-1 foot rocks	80% grass	Good	Gravel & cobble/ boulder	Perpendicular (channelized)	170	Grass & some small trees	0	0	0	None.
26	.035	.036	---	100% grass & weeds	Some erosion	Silt & clay	Flow approaches at a moderate angle	700	Crops & grass with some trees	0 - 1.5	1	0	Some potential for debris at piers.
27	.035	.037	---	100% grass & weeds	Some erosion	Silt & clay	Flow approaches at a moderate angle	700	Crops & grass with some trees	0 - 2.5	1	0	Some potential for debris at piers.
28	.035	.037	Degrading on left side because of meander	90% grass & weeds	Some sloughing on u.s. left bank	Sand with some clay	On meander (flow towards left bank)	3,300	Grass with some trees	0 - 2	0	0	None.
29	.040	.039	---	100% grass/weeds & trees	Good	Silt & clay	Flow approaches at a slight angle	390	Grass, brush, & trees	0 -1	0	0	Large potential for blockage.
30	.030	.035	Degrading	< 5% u.s. < 25% d.s. grass & weeds	Both u.s. & d.s. have an erosion problem	Silt & clay	Perpendicular at the bridge but close to u.s. bend	150	None to grass & weeds	---	3	0	None.
31	.035	.037	Degrading	u.s.: 10% grass/weeds d.s.: 100% trees, brush, & grass	Sloughing u.s. & at the bridge	Silt & clay	Flow approaches at a moderate angle	1,400	Grass/weeds & small trees	0	0	5 - 10	Some potential for debris at piers.
32	.030	.032	---	u.s.: 100% grass d.s.: crops	---	Silt & clay	Flow approaches on a slight angle	1,300	u.s.: grass d.s.: crops	2	2	0	None.

Table 4. Summary of data for U.S. Geological Survey streamflow- and/or stage-gaging stations located near selected bridge sites

[mi^2; square miles; ft^3/s, cubic feet per second; --, no data]

Station number (fig. 1)	Station name	Drainage area (mi^2)	Period of record used for flow calculations (water year)	Peak flow (ft^3/s)	Minimum daily flow (ft^3/s)	Annual mean flow (ft^3/s)
06334500	Little Missouri River at Camp Crook, SD	1,970	1904-05, 1957-93	9,420	0.00	121
06355500	North Fork Grand near White Butte, SD	1,190	1967-93	6,710	.00	41.5
06356000	South Fork Grand at Buffalo, SD	148	1956-93	2,780	.00	8.53
06356500	South Fork Grand River near Cash, SD	1,350	1947-93	27,000	.00	51.3
06357800	Grand River at Little Eagle, SD	5,370	1959-93	31,000	.00	230
06359500	Moreau River near Faith, SD	2,660	1944-93	26,000	.00	132
06403300	French Creek above Fairburn, SD	105	1983-93	329	.02	6.5
06433000	Redwater River above Belle Fourche, SD	920	1946-93	16,400	.00	131
06440000	Missouri River at Pierre, SD	--	--	--	--	--
06441500	Bad River at Fort Pierre, SD	3,107	1929-93	43,800	.00	147
06441590	Missouri River at La Framboise Island, at Pierre, SD	--	--	--	--	--
06441595	Missouri River at Farm Island, near Pierre, SD	--	--	--	--	--
06450500	Little White River below White River, SD	1,570, [2]1,310	1950-93	13,700	7.0	128
06452000	White River near Oacoma, SD	10,200	1929-93	51,900	.00	530
06466700	Lewis and Clark Lake at Springfield, SD	--	[1]1968-93	--	--	--
06467000	Lewis and Clark Lake near Yankton, SD	279,500	[1]1956-93	--	--	--
06478300	Dry Creek near Parkston, SD	97.2	--	[4]4,210	.00	--
06478500	James River near Scotland, SD	20,653, [2]16,505	1929-93	29,400	.00	438
06479000	Vermillion River near Wakonda, SD	2,170, [2]1,676	1946-83, [3]1989-93	[5]17,000	.00	125
06479010	Vermillion River near Vermillion, SD	2,302, [2]1,808	1984-93	21,400	3.6	408
06479525	Big Sioux River near Castlewood, SD	1,997, [2]570	1977-93	2,250	.00	64.2
06479640	Hidewood Creek near Estelline, SD	164	1969-85	[6]17,300	.00	25.8
06480000	Big Sioux River near Brookings, SD	3,898, [2]2,419	1954-93	33,900	.00	241
06480400	Spring Creek near Flandreau, SD	63.2	1983-93	4,480	.00	21.1
06480650	Flandreau Creek above Flandreau, SD	100	1982-91	2,650	.00	34.9
06481000	Big Sioux River near Dell Rapids, SD	4,483, [2]3,004	1949-93	41,300	.00	350
06482610	Split Rock Creek at Corson, SD	464	1966-89	[7]18,900	0.00	100

[1]Stage gage only.
[2]Contributing drainage only.
[3]Crest-stage gage partial-record station (peaks only obtained).
[4]Peak occurred in 1960.
[5]Peak occurred in 1984.
[6]Peak occurred in 1992.
[7]Peak occurred in 1993.

Rating Item	+	0	-
Is bridge accessible at high flow? Yes (+); No (-)			
Is streambed composed of bedrock or clay? No (+); Yes (-)			
Distance from bridge deck to streambed (in feet)? Less than 40 (+); 40 to 80 (0); more than 80 (-)			
Is sustained high flow likely during a flood? Yes (+); No (-)			
Can scour be measured safely at this bridge? Yes (+); No (-)			
Are there any other factors that would prevent scour from being measured at this site? No (+); Yes (-)			
Is scour likely to occur at one or more piers? Yes (+); No (0)			/////
Is scour likely to occur at more than one pier? Yes (+); No (0)			/////
Is scour likely to occur at one or more bridge abutments? Yes (+); No (0)			/////
Can pier be reached by a sounding weight lowered from the bridge? Yes (+); No (0)			/////
Does the bridge constrict high flows significantly? Yes (+); No (0)			/////
Shape of pier nose: square or round (+); sharp (0)			/////
Angle at which flow approaches piers (in degrees): 0 to 5 (+); more than 5 (0)			/////
Are pier footings exposed? No (+); Yes or don't know (0)			/////
Has riprap been placed around one or more piers? N0 (+); Yes or don't know (0)			/////
Is debris lodged on one or more piers? No (+); Yes (0)			/////
Is a gaging station located nearby (within view of the bridge)? Yes (+); No (0)			/////
Is boat access available nearby? Yes (+); No (0)			/////
Does the bridge have trusses? No (+). Yes (0)			/////
Will traffic lane need to be closed to make measurements? No (+); Yes (0)			/////
Totals (+, 0, and -)			

Figure 2. A checklist used in New York for bridge-site selection (Butch,1991).

Variables, diagnostic characteristics, and assigned values for calculation of observed-scour index

[Observed-scour index equals sum of assigned values]

1. Pier and Abutment Scour (local; sum for all)

If pier:	none	observed	footing exposed	piling exposed
	0	1	2	3
If bent:	none	observed	moderate	severe
	0	1	2	3

2. Failed Riprap at Bridge (sum of both values)

left	right
1	1

3. Bed Riprap Moved?

yes	no
1	0

4. Blowhole Observed?

yes	no
3	0

5. Mass Wasting at Pier (calculated for each pier)

yes	no
3	0

Figure 3. An observed-scour index form used in Tennessee (B.A. Bryan, USGS, written commun., 1992).

Variables, diagnostic characteristics, and assigned values for calculation of potential-scour index

[Potential-scour index equals sum of assigned values]

1. Bed Material

bedrock	boulder/ cobble	gravel	sand	unknown alluvium	silt/ clay
0	1	2	3	3.5	4

2. Bed Protection

yes	no	(with)	1 blank protected	2 blank protected
0	1		2	3

3. Stage of Channel Evolution

I	II	III	IV	V	VI
0	1	2	4	3	0

4. Percent of Channel Constriction

0-5	6-25	26-50	51-75	76-100
0	1	2	3	4

5. Number of Piers in Channel

0	1-2	>2
0	1	2

6. Percent of Blockage: horizontal (6), vertical (7), total (8)

0-5	6-25	26-50	51-75	76-100	
0	1	2	3	4	(values to be divided by 3)

9. Bank Erosion for each Bank

none	fluvial	mass-wasting
0	1	2

10. Meander Impact Point from Bridge (in feet)

0-25	26-50	51-100	>100
3	2	1	0

11. Pier Skew for each Pier (sum for all piers in channel)

yes	no
1	0

12. Mass Wasting at Pier (calculated for each pier)

yes	no
3	0

13. High-flow Angle of Approach (in degrees)

0-10	11-25	26-40	41-60	61-90
0	1	2	2.5	3

Figure 4. A potential-scour index form used in Tennessee (B.A. Bryan, USGS, written commun., 1992).

high scour potential to bridge sites that: (1) were likely to scour at the piers and abutments; (2) were likely to have sustained high flows during a flood; (3) had highly scourable bottom materials; and (4) had substantial contraction of the flow through the bridge structure during high flows. The checklist also gave high ratings to sites where it was feasible to measure scour. These feasibility factors included distance from bridge deck to stream bottom, accessibility of bridge site during high flows, safety considerations, presence of slope and bed protection, presence of or potential for debris accumulation, and presence of bridge trusses.

An observed-scour index form used in Tennessee rated sites high that had observable scour, had riprap that had been displaced by high flows, and had definite erosion of banks at the sites. A potential-scour index form used in Tennessee rated sites high that had a high potential for scour from future high flows. Factors contributing to high ratings included highly scourable bottom materials, no bed protection, high contraction of the flow by the bridge structure, large number of piers, bridges that were close to or in meanders, large skewness of flow, and where bank erosion was taking place. Silt and clay streams, which are common in South Dakota, were given the highest ratings.

A summary of the results of scour assessments using these forms, as well as the final selections of the bridge sites for more detailed assessments, are presented in table 5. The numbers on the forms are arbitrary and intended to provide measures of relative differences only between sites.

Thirteen bridge sites that were considered to have high scour potential based on the rankings shown on the scour-assessment forms were selected for detailed scour assessments (fig. 1 and table 5). They are site 2 (Grand River near Mobridge), site 3 (Big Sioux River near Flandreau), site 6 (Vermillion River Overflow near Wakonda), site 7 (Vermillion River near Centerville), site 15 (Snatch Creek near Springfield), site 17 (Moreau River near Faith), site 18 (South Fork Grand River near Bison), site 20 (French Creek near Fairburn), sites 26 and 27 (Split Rock Creek near Brandon), site 28 (White River near Presho), site 30 (Powell Creek near Fort Pierre), and site 32 (Hidewood Creek near Estelline).

Sites were selected to ensure that a wide range of bridge lengths, bridge types, stream types (small and large), drainage areas, flow alignments, pier types, and presence or absence of piers were included. Some sites having high rankings were dropped from the final selection list because they were very similar to another site that already had been selected for further study. For example, site 8 was dropped because of its similarity to the selected site 7. Sites where there was a problem with a lateral shift of the stream also were avoided; this is why sites 9 and 31 were not selected. Site 30 was chosen because of it uniqueness of having no piers. Sites also were not selected if there was a high degree of slope or bed protection, making scour unlikely. This was one reason why sites 22, 24, and 25 were not selected.

DETAILED SCOUR ASSESSMENTS FOR SELECTED BRIDGE SITES

Detailed scour assessments for 13 sites were accomplished during 1992-95 and included determination of 2-, 100-, and 500-year recurrence flows; basic hydraulic analysis; determination of scour depths for the 2-, 100-, and 500-year flows; measurement of scour and flow during high flows; comparison of results using measured and predicted scour; and comparison of results using different scour equations. Because site 32 (Hidewood Creek near Estelline) was selected for detailed assessment at a later date, it was not studied in as much detail as the other 12 bridge sites. Individual bridge-site reports containing the basic hydrologic, hydraulic, and scour-prediction-equation results for the 2-, 100-, and 500-year flows were prepared for each of the 12 bridge sites and were provided to SDDOT in October 1993. This section of the report summarizes those individual bridge-site reports.

Flood Hydrology

Annual peak flows having 2-, 100-, and 500-year recurrence intervals were used to calculate scour depth at 12 selected bridge sites (table 6). A historic peak flow was used to calculate scour depth at one site (site 32). Three methods were used to determine the 2-, 100-, and 500-year peak flows at the 12 sites. These methods include: (1) Log-Pearson Type III analyses of recorded and historic peak flows

Table 5. Summary of results using various forms to select bridge sites for detailed scour assessments
[T, tie; ---, not rated]

Site number (fig. 1)	Selection number using a checklist from New York (ranking)	Observed-scour index number using a form from Tennessee (ranking)	Potential-scour index number using a form from Tennessee (ranking)	Combined observed- & potential-scour index numbers using forms from Tennessee (ranking)	Bridge-study sites selected for further analyses	Comments
1	7 (26T)	0 (28T)	14 (13T)	14 (20T)		
2	11 (15T)	6 (2T)	14 (13T)	20 (7T)	Yes	Part of Oahe Reservoir at times.
3	17 (2)	4 (7T)	14 (13T)	18 (12T)	Yes	In pool created by downstream dam.
4	12 (13T)	1 (23T)	15 (9T)	16 (17T)		
5	7 (26T)	2 (16T)	16.2 (7)	18.2 (11)		Part of Hayes Lake.
6	13 (4T)	8 (1)	11 (23T)	19 (9T)	Yes	Relief bridge.
7	15 (3)	6 (2T)	17 (6)	23 (5)	Yes	
8	13 (4T)	6 (2T)	14 (13T)	20 (7T)		Similar to site 7.
9	12 (13T)	5 (5)	19 (4)	24 (2T)		Lateral shift of stream.
10	10 (17T)	2 (16T)	11 (23T)	13 (23T)		
11	10 (17T)	2 (16T)	11 (23T)	13 (23T)		
12	8 (24T)	0 (28T)	13 (18T)	13 (23T)		
13	8 (24T)	0 (28T)	13 (18T)	13 (23T)		
14	6 (30T)	0 (28T)	14 (13T)	14 (20T)		
15	13 (4T)	4.5 (6)	11.7 (21T)	16.2 (16)	Yes	Representative of small drainage areas.
16	9 (23)	2 (16T)	11 (23T)	13 (23T)		
17	13 (4T)	3 (13T)	11 (23T)	14 (20T)	Yes	
18	18 (1)	3 (13T)	15 (9T)	18 (12T)	Yes	
19	11 (15T)	2 (16T)	16 (8)	18 (12T)		
20	13 (4T)	4 (7T)	11.7 (21T)	15.7 (19)	Yes	
21	10 (17T)	4 (7T)	15 (9T)	19 (9T)		Large skewness of flow possible.
22	10 (17T)	1 (23T)	9 (28T)	10 (28T)		Bed is bedrock.
23	6 (30T)	1 (23T)	9 (28T)	10 (28T)		
24	7 (26T)	1 (23T)	7 (30T)	8 (30T)		Piers are riprapped.
25	7 (26T)	1 (23T)	7 (30T)	8 (30T)		Piers are riprapped.
26	13 (4T)	4 (7T)	20 (2T)	24 (2T)	Yes	
27	13 (4T)	4 (7T)	20 (2T)	24 (2T)	Yes	
28	13 (4T)	2 (16T)	23 (1)	25 (1)	Yes	
29	10 (17T)	2 (16T)	15 (9T)	17 (15)		Large potential for blockage during flood.
30	13 (4T)	4 (7T)	12 (20)	16 (17T)	Yes	Representative of small bridge without piers.
31	10 (17T)	3 (13T)	18 (5)	21 (6)		Lateral shift of stream.
32	---	---	---	---	Yes	Selected at request of FHWA.

Table 6. Summary of 2-, 100-, and 500-year predicted peak flows at selected bridge sites

[mi^2, square miles; ft^3/s, cubic feet per second; --, not applicable; ---, not performed]

Site number (fig. 1)	Stream	Location	Drainage area (mi^2)	Predicted peak discharges 2-year (ft^3/s)	100-year (ft^3/s)	500-year (ft^3/s)	Method used to determine peak	Period used to determine peak using Log-Pearson Type III
2	Grand River	near Mobridge	5,470	5,370	36,100	53,300	Log-Pearson Type III[2].	1951-90
3	Big Sioux River	near Flandreau	4,096	2,320	31,300	53,100	Log-Pearson Type III[2].	1954-89
6	Vermillion River overflow	near Wakonda	[1]2,170	[1]1,200	[1]22,600	[1]46,400	Log-Pearson Type III.	1958-90
7	Vermillion River	near Centerville	1,992	1,150	21,700	44,500	Log-Pearson Type III[2].	1958-90
15	Snatch Creek	near Springfield	44	82	1,930	3,280	Regression equations.	--
17	Moreau River	near Faith	2,660	3,870	36,900	58,200	Log-Pearson Type III.	1944-90
18	South Fork Grand River	near Bison	1,350	1,440	17,300	32,700	Log-Pearson Type III.	1946-90
20	French Creek	near Fairburn	130	81	1,010	1,910	Log-Pearson Type III[2].	1982-90
26	Split Rock Creek	near Brandon	466	2,200	22,500	39,200	Log-Pearson Type III[2].	1966-89
27	Split Rock Creek	near Brandon	466	2,200	22,500	39,200	Log-Pearson Type III[2].	1966-89
28	White River	near Presho	9,343	9,860	48,000	71,800	Log-Pearson Type III[2].	1929-90
30	Powell Creek	near Fort Pierre	13	208	2,860	4,860	Regression equations.	--
32	Hidewood Creek	near Estelline	---	---	---	---	---	--

[1]These values are for the Vermillion River (the bridge-study site is a relief bridge on the flood plain).
[2]Adjusted using drainage-area ratio.

at three sites having streamflow-gaging stations; (2) use of a drainage-area ratio adjustment to transfer results of Log Pearson Type III analyses from nearby gaged sites on the same stream to seven sites; and (3) use of regression equations to compute 2- and 100-year peak flows, followed by use of a constant multiplier to compute 500-year peak flows from the 100-year peak flows (Federal Highway Administration, 1988) at two sites.

The Log-Pearson Type III procedures that were used are recommended by the Interagency Advisory Committee on Water Data (1981). These procedures use the magnitudes of the annual peak flows at a streamflow-gaging station (systematic data) and historic data. These magnitudes are assumed to be independent random variables that follow a Log-Pearson Type III probability distribution. The procedures also detect and adjust for low outliers, high outliers, and historic peak flows.

At site 6 (Vermillion River near Wakonda, station 06479000), the annual peak flows for the period 1958 through 1990 were used to compute the 2-, 100-, and 500-year peak flows. The 2-, 100-, and 500-year flows were calculated for the entire Vermillion River, not just the overflow direct drainage, because the site 6 overflow bridge receives high flows when the Vermillion River overflows its main channel and levees. At site 17 (Moreau River near Faith, station 06359500), the annual peak flows for the period 1944 through 1990 were used to compute the 2-, 100-, and 500-year peak flows. For site 18 (South Fork Grand River near Cash, station 06356500), the annual peak flows for the period 1946 through 1990 were used to compute the 2-, 100-, and 500-year peak flows. The 2-, 100-, and 500-year flows were not computed for site 32, the Hidewood Creek near Estelline bridge site, because of the lateness of the site's selection.

Peak flows for sites 2, 3, 7, 20, 26, 27, and 28 were determined by multiplying the Log-Pearson Type III results for nearby streamflow-gaging stations times the square root of the drainage-area ratio for the nearby stations and the bridge sites. The equation is shown as follows:

$$Qsite = Qstation\sqrt{(Asite)/(Astation)}$$

where

Qsite = predicted peak flow at the site;

Qstation = Log-Pearson Type III peak flow results (either 2-, 100-, or 500-year);

Asite = drainage area upstream of the site; and

Astation = drainage area upstream of the streamflow-gaging station.

The Grand River at Little Eagle station (06357800) is 14 mi upstream of site 2, which has 100 mi^2 of additional drainage area. The Big Sioux River near Brookings station (06480000) is 22 mi upstream of site 3, which has 200 additional mi^2 of drainage area. The Vermillion River near Wakonda station (06479000) is 6 mi downstream of site 7, which has 178 mi^2 less drainage area. The French Creek near Fairburn station (06403300) is 7 mi upstream of site 20, which has 25 mi^2 of additional drainage area. The Split Rock Creek at Corson station (06482610) is 1 mi upstream of sites 26 and 27, which have 2 mi^2 of additional drainage area. The White River near Oacoma station (06452000) is 55 mi downstream of site 28, which has 857 mi^2 less drainage area.

Two- and 100-year peak flows for sites 15 (Snatch Creek near Springfield) and 30 (Powell Creek near Pierre) were determined by using USGS regression equations (Becker, 1980) because peak-flow data are not available for these streams. The regression equations are considered to be applicable for basins having drainage areas that range from 0.05 to 100 mi^2. Parameters used in the equations are contributing drainage area, main channel slope, and soil-infiltration index. The 500-year peak flow was estimated by multiplying the 100-year peak flow by 1.7 (Federal Highway Administration, 1988).

Hydraulic and Bed-Material Analyses

A basic hydraulic analysis was completed for each of the 13 selected bridge sites. The computer model WSPRO (Water Surface Profile Program (Shearman, 1990)), was used to conduct step-backwater analyses for the 2-year, 100-year, and 500-year peak flows at 12 of the bridge sites. Starting water-surface elevations were determined using existing flow data, or by trial and error if no flow data existed. Final starting water-surface elevations at the most downstream station were selected for the cases where there was convergence at the exit sections. All cross-section data required for WSPRO input were collected in the field. This included surveying the approach, bridge, and exit sections. For site 32, a 17,300-ft^3/s flow (from slope-area analysis) that occurred on June 16, 1992, was used for hydraulic analysis.

A summary of the WSPRO results is presented in table 7. The data displayed for the bridge and approach sections include water-surface elevations, flow velocities, flow areas, and head losses through the bridges. The bridge section was located at the downstream edge of the bridge, and the approach section generally was located one bridge length upstream of the bridge. Flow velocities at the bridges ranged from 0.95 to 4.98 ft/s for the 2-year peak flows, 5.85 to 14.18 ft/s for the 100-year peak flows, and 6.71 to 16.51 ft/s for the 500-year peak flows. Flow velocities at the approaches to the bridges ranged from 1.26 to 4.18 ft/s for the 2-year peak flows, 0.46 to 7.46 ft/s for the 100-year peak flows, and 0.73 to 8.46 ft/s for the 500-year peak flows. Total losses through the bridges ranged from 0.03 to 0.59 ft for the 2-year peak flows, 0.23 to 4.38 ft for the 100-year peak flows, and 0.60 to 5.78 ft for the 500-year peak flows. Site 32 results are for the flow that occurred on June 16, 1992. The flow velocity at the bridge was 9.33 ft/s, and the flow velocity at the approach was 2.27 ft/s. WSPRO results could not be determined for some of the flows at the sites because model convergence was not attained.

Bed-material analyses were completed at all sites except site 32 to assess general channel stability and armoring potential. Large equipment such as drilling rigs or backhoes was not used for logistical and economic reasons. Instead, sampling was done at locations where bed material was exposed, judged to generally be representative of streambed material, and could readily be removed with a hand shovel and bucket without risking loss of fine materials. A standard sieve analysis was performed at the USGS sediment laboratory in Iowa City, Iowa, for bed material considered appropriate for the general stability and armoring potential analysis. Core log information supplied from the SDDOT was also used to characterize the bed material. The previously mentioned bridge-site reports contain a more detailed bed-material discussion for each of the sites.

Table 7. Summary of WSPRO results for various peak flows at selected bridge sites

[WSPRO, computer model for Water-Surface Profile Computations; ft, feet; ft/s, feet per second; ft^2, square feet; ft^3/s, cubic feet per second; ---, undetermined. Bridge section is located at the downstream edge of the bridge]

		Bridge section					Approach section		
Site number (fig. 1)	Recurrence interval (years)	Water-surface elevation (ft)	Flow velocity (ft/s)	Flow area (ft^2)	Friction head loss (ft)	Other head losses (ft)	Water-surface elevation (ft)	Flow velocity (ft/s)	Flow area (ft^2)
2	2	1,602.05	4.01	1,338	0.54	0.05	1,602.97	1.87	2,866
	100	1,605.76	14.18	2,546	1.41	2.97	1,612.16	2.28	15,858
	500	1,607.73	16.51	3,229	1.20	4.58	1,616.19	2.47	21,556
3	2	1,527.88	.95	2,441	.04	0	1,527.93	1.26	1,836
	100	1,533.04	7.46	4,196	.36	.49	1,534.78	2.48	12,621
	500	1,534.07	11.63	4,566	.63	1.58	1,538.31	2.93	18,144
6	100	1,163.31	9.78	582	.09	2.09	1,165.69	.46	16,379
	500	1,163.93	9.45	635	.13	1.93	1,166.49	.73	18,470
7	2	1,178.52	1.38	833	.03	0	1,178.48	2.68	429
	100	1,186.26	10.23	1,939	.26	2.27	1,189.38	.85	25,621
	500	1,186.26	11.82	1,939	.32	3.40	1,190.60	1.42	31,261
15	100	1,272.57	5.85	330	.19	.04	1,273.18	3.62	533
	500	1,273.57	8.09	405	.20	.40	1,274.89	3.44	953
17	2	2,249.39	3.28	1,179	.28	0	2,249.67	3.98	973
	100	2,262.84	7.18	5,139	.40	.39	2,264.24	3.71	9,942
	500	2,262.84	11.33	5,139	.61	1.82	2,266.78	4.40	13,241
18	2	2,428.19	2.72	529	.28	0	2,428.51	3.69	390
	100	2,436.68	8.49	2,038	.25	.49	2,437.95	5.05	3,425
	500	2,438.46	13.62	2,401	.40	2.79	2,443.86	3.83	8,544
20	500	3,397.74	9.11	210	---	---	3,401.58	1.85	1,034
26 & 27	2	1,309.91	3.18	691	.40	.01	1,310.38	2.69	819
	100	1,318.33	10.25	2,196	.64	.79	1,320.50	7.46	3,014
	500	1,319.95	15.45	2,544	.96	2.13	1,324.92	8.46	4,643
28	2	1,578.13	4.98	1,981	.25	.01	1,578.43	4.18	2,359
	100	1,584.84	7.25	4,446	.58	1.53	1,587.02	1.73	27,788
	500	1,586.52	6.71	4,863	.54	1.27	1,588.49	2.23	32,201
30	2	1,490.56	3.80	55	.23	0	1,490.99	3.81	55
	100	1,497.75	8.12	352	.18	.17	1,498.79	3.68	778
	500	1,497.57	14.11	345	.31	1.41	1,501.41	3.44	1,412
32	(1)	27.40	9.33	1,129	---	---	30.37	2.27	7,503

[1]Flow of 17,300 ft^3/s that occurred on June 16, 1992.

Determination of Scour Depths Using Scour-Prediction Equations

Estimations of the scour depths for selected flows at the 12 bridge sites, and the 17,300-ft^3/s flow for site 32, were determined using published scour-prediction equations (Jarrett and Boyle, 1986; Richardson and others, 1991). These scour-prediction equations included 13 pier equations, 1 contraction equation, and 5 abutment equations. Data necessary for prediction of pier scour include approach velocity, approach depth, pier width, Froude number, and bed-material size. Data necessary for prediction of contraction scour include approach velocity in the main channel, depth in the contracted section, bottom width of the bridge opening, flow in the approach channel, and flow in the contracted section. Data necessary for prediction of abutment scour include abutment shape, flow angle, abutment length, Froude number, flow obstructed by abutment, and depth at the abutment. The scour-prediction equations are presented in the following sections.

Pier-Scour Equations

1. Laursen equation—1956 and 1958 (Jarrett and Boyle, 1986):

$$D = 1.5b^{0.7}H^{0.3} \tag{1}$$

where

D = scour depth measured from mean bed elevation, in feet;
b = width of the pier, in feet; and
H = flow depth (stage), in feet.

2. Shen and others equations—1969 (Jarrett and Boyle, 1986):

$$\frac{D_e}{b} = 11.0F_p^2 \tag{2a}$$

$$\frac{D_e}{b} = 3.4F_p^{0.67} \tag{2b}$$

where

D_e = scour depth at equilibrium measured from mean bed elevation, in feet;
b = width of the pier, in feet;
F_p = pier Froude number = $V/(gb^*)^{0.5}$, where V = flow velocity, in feet per second; g = acceleration of gravity, in feet per second squared; and
b^* = width of the pier projected on a plane normal to undisturbed flow, in feet.

3. Colorado State University equation (Richardson, Harrison, and Davis, 1991):

$$\frac{y_s}{y_1} = 2.0K_1K_2\left(\frac{a}{y_1}\right)^{0.65}Fr_1^{0.43} \tag{3}$$

where

y_s = scour depth, in feet;
y_1 = flow depth just upstream of the pier, in feet;
K_1 = correction for pier nose shape;
K_2 = correction for angle of attack of flow;
a = pier width, in feet; and
Fr_1 = Froude number = $V_1/(gy_1)^{0.5}$, where V_1 = flow velocity, in feet per second; g = acceleration of gravity, in feet per second squared.

4. Blench equations—1960 (Jarrett and Boyle, 1986):

$$\frac{D^*}{H} = 1.8(b/H)^{0.25} \tag{4a}$$

$$D^* = 1.8(d_r) \tag{4b}$$

where

D^* = scour depth measured from the water surface, in feet;
H = flow depth at the pier, in feet;
b = width of pier, in feet; and
d_r = regime depth = $(q^2/F_b)^{0.33}$, in feet, where q = VH, in feet squared per second; V = approach velocity, in feet per second; F_b = bed factor = V^2/H, in feet per second squared.

5. Inglis-Poona equations—1949 (Jarrett and Boyle, 1986):

$$\frac{D^*_{max}}{b} = 1.7\,(q^{0.67}b)^{0.78} \qquad (5a)$$

$$\frac{D^*_{max}}{b} = 1.73\,(H/b)^{0.78} \qquad (5b)$$

where

D^*_{max} = maximum scour depth measured from the water surface, in feet;

b = width of pier, in feet;

$q = VH$, in feet squared per second, where V = approach velocity, in feet per second; and H = flow depth at the pier, in feet.

6. Chitale equation—1962 (Jarrett and Boyle, 1986):

$$\frac{D}{H} = 6.65F - 5.49F^2 - 0.51 \qquad (6)$$

where

D = scour depth measured from the water surface, in feet;

H = flow depth at the pier, in feet;

F = Froude number = $V/(gH)^{0.5}$, where V = velocity at the pier, in feet per second; and g = acceleration of gravity, in feet per second squared.

7. Bata equation—1960 (Jarrett and Boyle, 1986):

$$\frac{D}{H} = 10\left(\frac{V^2}{gH} - \frac{3d}{H}\right) \qquad (7)$$

where

D = scour depth measured from the water surface, in feet;

H = flow depth at the pier, in feet;

V = approach velocity, in feet per second;

d = diameter of bed material, in feet; and

g = acceleration of gravity, in feet per second squared.

8. Varzeliotis equation—1960 (Jarrett and Boyle, 1986):

$$\frac{D^*_{max}}{b} = 1.43\,(q^{0.67}/b)^{0.72} \qquad (8)$$

where

D^*_{max} = maximum scour depth measured from the water surface, in feet;

b = width of pier, in feet;

$q = VH$, in feet squared per second, where V = approach velocity, in feet per second; and H =flow depth at the pier, in feet.

9. Carstens equation—1966 (Jarrett and Boyle, 1986):

$$\frac{D_e}{b} = 0.546\,[\,(N_s^2 - 1.64)/(N_s^2 - 5.02)\,]^{0.83} \qquad (9)$$

where

D_e = scour depth at equilibrium measured from mean bed elevation, in feet;

b = width of pier, in feet;

N_s = the sediment number = $V/[\,(s-1)\,gd_m]^{0.5}$, where V = approach velocity, in feet per second; s = specific gravity of the sand = 2.65; g = acceleration of gravity, in feet per second squared; and d_m = mean diameter of bed material, in feet.

10. Breusers equation—1964 (Jarrett and Boyle, 1986):

$$D_{max\,e} = 1.4b^* \qquad (10)$$

where

$D_{max\,e}$ = maximum scour depth measured from the water surface, in feet, and

b^* = width of the pier projected on a plane normal to undisturbed flow, in feet.

Contraction-Scour Equation

11. Laursen equation—1960 (Richardson, Harrison, and Davis, 1991):

$$\frac{y_2}{y_1} = \left(\frac{Q_{mc2}}{Q_{mc1}}\right)^{6/7}\left(\frac{W_{c1}}{W_{c2}}\right)^{K_1} \qquad (11a)$$

$$y_s = y_2 - y_1 \qquad (11b)$$

where

y_2 = average depth in the contracted section, in feet;
y_1 = average depth in the main channel at the approach section, in feet;
Q_{mc2} = flow in the contracted channel, in cubic feet per second;
Q_{mc1} = flow in the approach channel that is transporting sediment, in cubic feet per second;
W_{c1} = bottom width of the main channel at the approach section, in feet;
W_{c2} = bottom width of the bridge opening, in feet;
y_s = average scour depth, in feet; and
K_1 = an exponent determined as follows:

V_{*c}/w	K_1	Mode of bed material transport
<0.50	0.59	Mostly contact bed material
<0.50 to 2.0	0.64	Some suspended bed material
>2.0	0.69	Mostly suspended bed material

V_{*c} = $(gy_1S_1)^{0.5}$, shear velocity, in feet per second, where g = acceleration of gravity, in feet per second squared; and S_1 = slope of energy grade line of main channel; and
w = fall velocity of D_{50} of bed material, in feet per second.

Abutment-Scour Equations

12. Froehlich (live bed) equation—1989 (Richardson, Harrison, and Davis, 1991):

$$\frac{y_s}{y_a} = 2.27K_1K_2\,(a'/y_a)^{0.43}Fr_e^{0.61} + 1 \qquad (12)$$

where

y_s = scour depth, in feet;
y_a = depth of flood-plain flow at the abutment, in feet;
K_1 = coefficient for abutment shape;
K_2 = coefficient for angle of embankment to flow;
a' = the length of abutment projected normal to flow, in feet, = A_e/y_a, where A_e = the flow area of the approach cross section obstructed by the embankment, in feet squared;
Fr_e = Froude number of approach flow upstream of the abutment = $V_e/(gy_a)^{0.5}$, where $V_e = Q_e/A_e$, in feet per second; Q_e = the flow obstructed by the abutment and approach embankment, in cubic feet per second; and g = acceleration of gravity, in feet per second squared.

13. Froehlich (clear water) equation—1989 (Richardson, Harrison, and Davis, 1991):

$$\frac{y_s}{y_1} = 0.78K_1K_2\left(\frac{a'}{y_1}\right)^{0.63}Fr_e^{1.16}\left(\frac{y_1}{D_{50}}\right)^{0.43}G^{-1.87} + 1 \qquad (13)$$

where

y_s = scour depth, in feet;
y_1 = depth of flow at the abutment, in feet;
K_1 = coefficient for abutment shape determined as follows:

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

K_2 = coefficient for angle of embankment to flow;
a' = length of abutment projected normal to flow, in feet, = A_e/y_1, where A_e = flow area of the approach cross section obstructed by the embankment, in feet squared;
Fr_e = Froude number of approach flow upstream of the abutment = $V_e/(gy_1)^{0.5}$, where $V_e = Q_e/A_e$, in feet per second; Q_e = flow obstructed by the abutment and approach embankment, in cubic feet per second; and g = acceleration of gravity, in feet per second squared

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

D_{50}, D_{84}, D_{16} = grain sizes of the bed material. The subscript indicates the percent of bed material finer than the indicated size.

14. Federal Highway Administration equation, when $a/y_1 > 25$—1990 (Richardson, Harrison, and Davis, 1991):

$$\frac{y_s}{y_1} = 4Fr_1^{0.33} \qquad (14)$$

where

y_s = scour depth, in feet;

y_1 = upstream flow depth, in feet; and

a = abutment and embankment length, in feet;

Fr_1 = Froude number of approach flow upstream of the abutment $Fr_1 = \frac{v_1}{(gy_1)^{0.5}}$, where v_1 = upstream velocity, in feet per second; and g = acceleration of gravity, in feet per second squared.

15. Liu and others (live bed) equation—1961 (Richardson, Harrison, and Davis, 1991):

$$\frac{y_s}{y_1} = 1.1\left(\frac{a}{y_1}\right)^{0.40} Fr_1^{0.33} \qquad (15)$$

where

y_s = equilibrium depth of scour, in feet;

y_1 = average upstream flow depth in the main channel, in feet;

a = abutment and embankment length, in feet;

Fr_1 = upstream Froude number $Fr_1 = \frac{v_1}{(gy_1)^{0.5}}$, where v_1 = upstream velocity, in feet per second; and g = acceleration of gravity, in feet per second squared.

16. Laursen (live bed) equation—1980 (Richardson, Harrison, and Davis, 1991):

$$\frac{y_s}{y_1} = 1.5z\left(\frac{a}{y_1}\right)^{0.48} \qquad (16)$$

where

y_s = scour depth, in feet;

y_1 = upstream flow depth, in feet;

z = coefficient for abutment shape; and

a = abutment length, in feet.

Predicted Scour Depths

Predicted scour depths at the 13 selected bridge sites for the 2-, 100-, and 500-year flows are presented in table 8 for pier scour and in table 9 for contraction and abutment scour. Left and right are determined in table 9 by looking in a downstream direction. Most of the scour-prediction equations were written for sand-bed channels, which are more easily scoured than the typical silt and clay channels at the 13 selected bridge sites. Computed scour depths using the published equations also tend to be conservative due to the inclusion of a factor of safety.

Boxplots showing the distribution of predicted pier-scour depths at selected bridge sites for the 2-, 100-, and 500-year flows are presented in figure 5. No scour computations were done at some of the bridge sites because WSPRO model convergence was not attained, and no corresponding box plots are shown for these sites. Boxplots graphically summarize data and show whether the data are symmetrically distributed or skewed. In a boxplot diagram, the box represents the interquartile range (25th to 75th percentile) with the horizontal line within the box representing the median. A step is 1.5 times the interquartile range. Data points between one and two steps from the box in either direction are called "outside points." Data points farther than two steps beyond the box are called "far outside points." The boxplots for pier scour for the 2-year flows indicate that calculated pier-scour depths are evenly distributed, with the exception of far outliers from the Inglis-Poona (5a) equation (see table 8). The medians for eight sites range from 2.4 to 6.8 ft. The boxplots for pier scour for the 100- and 500-year flows indicate that the distributions of calculated scour depths are skewed to the right (larger values). The medians for the 100- and 500-year flows range from 4.4 to 10.9 ft and from 3.4 to 13.3 ft, respectively. The interquartile ranges for the 100- and 500-year flows are relatively large compared to the interquartile ranges for 2-year flow. The boxplots for the 100- and 500-year flows also show some far outliers. The far outliers are from the Inglis-Poona (5a) equation (table 8). Because the data sets are bounded on the low side by 0, right skewness for the data distribution is expected.

Table 8. Summary of predicted pier-scour depths for various peak flows at selected bridge sites

[---, no piers; see text for actual equations; ft^3/s, cubic feet per second]

Site number (fig. 1)	Recurrence interval (years)	Pier scour depth (feet) determined using local-scour equation developed by indicated investigator (equation number in parentheses)												
		Laursen	Shen		Colorado State University	Blench		Inglis-Poona		Chitale	Bata	Varzeliotis	Carstens	Breusers
		(1)	(2a)	(2b)	(3)	(4a)	(4b)	(5a)	(5b)	(6)	(7)	(8)	(9)	(10)
2	2	6.9	5.7	6.9	5.8	3.8	5.1	104.2	3.6	6.0	5.2	3.8	2.2	5.6
	100	8.7	35.5	12.7	9.5	4.4	11.1	253.1	4.4	19.7	323	9.1	2.2	5.6
	500	9.3	45.2	13.7	10.3	4.4	13.8	301.4	4.4	24.7	41.1	9.8	2.2	5.6
3	2	7.3	.6	2.6	3.3	3.1	11.9	41.2	3.2	-2.1	-.6	-6.8	.6	4.2
	100	8.0	26.1	9.4	7.8	3.3	16.6	161.4	3.3	23.1	23.1	4.2	2.5	4.2
	500	8.3	51.0	11.8	9.1	3.2	18.6	206.6	3.3	31.7	45.7	7.0	2.0	4.2
6	100	3.7	35.5	6.1	4.6	1.1	8.1	21.7	1.2	15.1	32.2	4.4	.7	1.9
	500	3.7	39.1	6.3	4.8	1.0	8.5	22.9	1.2	15.9	35.5	4.6	.7	1.9
7	2	4.7	1.0	2.6	2.9	2.4	5.8	18.4	2.4	1.0	.8	-.9	1.9	3.1
	100	6.1	48.4	9.4	7.4	1.7	13.9	94.9	2.0	25.2	43.9	6.0	1.2	3.1
	500	6.3	64.1	10.4	8.0	1.4	14.9	106.6	1.8	27.8	58.2	7.2	1.2	3.1
15	100	3.8	16.6	5.5	4.4	1.9	5.4	25.7	1.9	9.4	14.6	3.8	.9	2.4
	500	4.1	27.7	6.6	5.0	1.8	6.4	32.9	1.8	11.9	25.0	5.1	.9	2.4
17	2	6.6	5.2	5.5	4.5	3.3	8.5	73.4	3.3	6.9	4.5	1.6	2.2	4.2
	100	8.4	31.5	10.0	7.5	3.1	19.6	184.0	3.2	27.5	28.4	3.1	1.7	4.2
	500	8.6	78.9	13.7	9.2	2.9	21.0	248.3	3.1	38.4	71.6	8.7	1.7	4.2
18	2	4.4	4.4	4.0	3.4	2.2	5.8	24.8	2.2	5.4	3.7	1.1	1.5	2.8
	100	5.6	35.5	8.0	5.9	1.4	12.9	67.9	1.7	22.0	32.0	4.2	1.1	2.8
	500	6.0	63.2	9.7	6.9	.5	16.3	90.0	1.0	30.0	57.1	5.8	1.1	2.8
20	500	3.3	39.8	7.4	4.9	1.8	3.2	26.3	1.7	3.4	35.7	6.2	1.1	2.3
26 & 27	2	5.7	5.2	5.2	4.6	3.0	6.6	55.0	3.0	6.3	4.1	10.2	3.5	3.9
	100	7.3	46.0	10.9	8.2	2.9	14.2	154.4	3.0	25.3	41.2	25.0	3.1	3.9
	500	7.7	83.1	13.3	9.6	3.1	16.8	198.2	3.0	31.6	75.0	31.2	1.8	3.9
28	2	6.8	9.6	7.1	6.0	3.6	7.9	99.9	3.5	9.7	8.7	3.5	1.8	4.6
	100	8.2	25.3	9.8	7.9	3.5	14.8	169.5	3.4	20.1	22.9	5.1	1.8	4.6
	500	8.4	25.3	9.8	8.0	3.6	15.8	176.1	3.5	21.1	22.9	4.6	1.8	4.6
30	2	---	---	---	---	---	---	---	---	---	---	---	---	---
	100	---	---	---	---	---	---	---	---	---	---	---	---	---
	500	---	---	---	---	---	---	---	---	---	---	---	---	---
32	(1)	2.8	30.8	4.1	3.4	-1.7	11.4	0.6	-1.2	19.3	27.8	0.1	0.4	1.1

[1]Flow of 17,300 ft^3/s that occurred on June 16, 1992.

Table 9. Summary of predicted contraction and abutment-scour depths for various peak flows at selected bridge sites

[---, no results, see text for actual equations; ft^3/s, cubic feet per second]

Site number (fig. 1)	Recurrence interval (years)	Contraction scour: Laursen (11)	Abutment scour: Froehlich (live bed) (12) Left	Froehlich (live bed) (12) Right	Froehlich (clear water) (13) Left	Froehlich (clear water) (13) Right	Federal Highway Administration (14) Left	Federal Highway Administration (14) Right	Liu and others (live bed) (15) Left	Liu and others (live bed) (15) Right	Laursen (16) Left	Laursen (16) Right
2	2	2.1	7.1	14.0	22.0	194.6	4.9	4.9	5.2	6.4	27.4	35.4
	100	25.9	25.9	32.5	308.2	568.8	22.3	22.3	2.1	2.6	67.2	86.7
	500	35.9	31.9	38.8	398.4	685.9	29.5	29.5	1.8	2.2	78.7	101.6
3	2	.9	8.3	8.6	5.7	6.1	8.2	8.2	1.5	1.5	17.8	17.8
	100	12.8	21.4	38.8	11.7	12.1	17.5	17.5	3.1	3.5	70.3	82.6
	500	17.8	28.9	45.5	14.1	14.5	24.7	24.7	2.7	3.0	84.4	95.7
6	100	39.5	26.2	17.5	13.0	10.1	6.7	6.7	4.5	2.4	120.7	58.3
	500	45.4	33.3	20.7	15.9	11.3	8.8	8.8	4.8	2.5	131.0	58.9
7	2	.0	---	---	---	---	---	---	---	---	---	---
	100	62.1	40.3	22.5	45.2	19.5	8.6	8.6	6.0	2.6	156.0	56.1
	500	71.9	43.0	39.0	48.3	51.0	10.0	10.0	5.3	2.2	172.9	62.2
15	100	1.3	10.8	10.9	7.3	6.1	9.1	9.1	3.4	2.4	22.0	14.9
	500	8.0	15.7	15.1	9.2	8.0	8.7	8.7	3.6	3.0	23.5	18.5
17	2	-.5	16.5	---	10.9	---	18.3	---	1.8	---	21.1	---
	100	14.7	41.0	35.2	29.8	25.7	18.9	18.9	3.0	4.0	53.2	75.7
	500	26.7	46.9	42.8	34.3	30.5	25.6	25.6	2.7	3.6	62.0	88.2
18	2	-.3	---	9.4	---	4.2	---	9.2	---	2.7	---	16.9
	100	4.4	22.0	26.8	14.7	13.5	14.5	14.5	4.1	3.8	40.4	37.4
	500	23.2	34.6	34.7	19.4	18.0	23.0	23.0	2.5	3.0	56.8	70.5
20	500	6.2	---	11.3	---	3.9	---	5.7	---	4.8	---	31.4
26 & 27	2	---	---	---	---	---	---	---	---	---	---	---
	100	.8	21.8	11.6	18.1	9.9	29.7	29.7	.9	.4	14.0	5.5
	500	3.5	27.5	21.0	21.6	14.0	32.0	32.0	1.7	2.2	27.4	38.0
28	2	-0.1	---	14.8	---	16.1	---	7.5	---	8.0	---	41.8
	100	36.1	20.0	29.8	22.4	74.4	15.2	15.2	0.6	4.3	14.9	165.8
	500	42.0	20.9	31.2	22.3	72.0	16.4	16.4	.5	3.8	15.1	178.3
30	2	-.1	---	---	---	---	---	---	---	---	---	---
	100	.4	13.1	17.2	9.3	13.1	9.4	9.4	3.8	1.6	26.4	9.4
	500	11.8	23.7	18.1	17.3	13.4	14.3	14.3	2.8	1.3	35.9	13.5
32	(1)	33.8	34.2	37.2	25.5	28.0	14.3	14.3	3.2	3.7	60.1	70.3

[1]Flow of 17,300 ft^3/s that occurred on June 16, 1992.

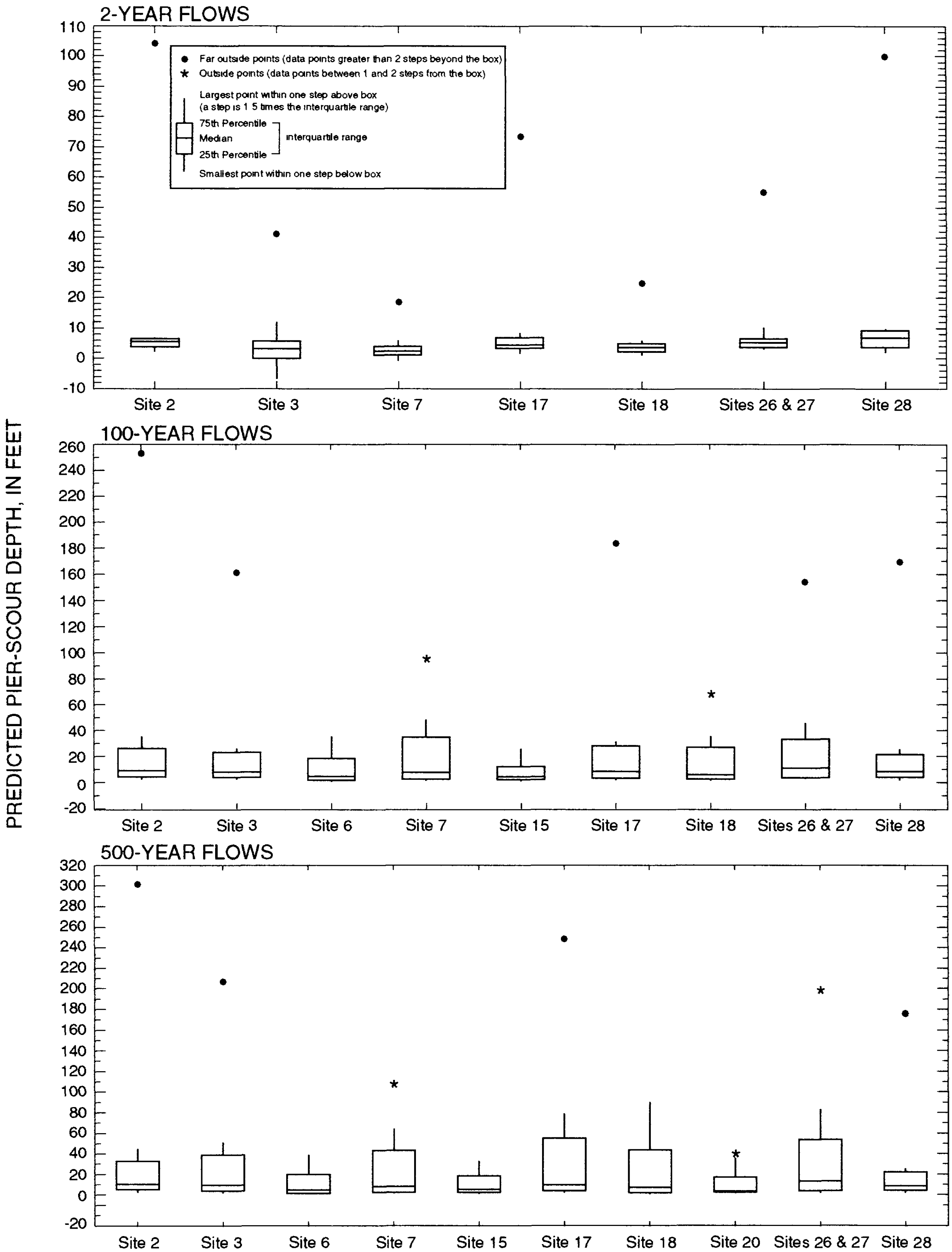


Figure 5. Distribution of predicted pier-scour depths using published equations for various flows at selected bridge sites.

Only the Laursen (11) equation was used to estimate scour depths for contraction scour. The medians of contraction scour for the selected sites for the 2-, 100-, and 500-year flows are -0.1, 13.75, and 23.2 ft, respectively.

Boxplots showing the distribution of predicted abutment scour depths using five different equations at each selected bridge site for the 2-, 100-, and 500-year flows are presented in figure 6. The boxplots indicate that the distribution of abutment scour for the 2-year flows for sites 3 and 18 are about normal, whereas the distribution of abutment scour for sites 2 and 28 are skewed to the right and for site 17 is skewed to the left. The medians for these selected sites range from 8.2 to 16.5 ft. The boxplots indicate that the distribution of abutment scour for the 100- and 500-year flows are approximately normal for about one-half of the sites and that the distribution is skewed to the right for most of the other sites. The medians for selected sites for the 100- and 500-year flows range from 9.1 to 29.2 ft and from 5.7 to 41 ft, respectively. The interquartile ranges are greater than 20 feet for most of the flows. Data in table 9 show that the outliers in the boxplots for 100- and 500-year flows are from the Froehlich clear-water (13) and Laursen (16) equations.

Measured Scour Depths During High-Flow Conditions

Wire-weight and crest-stage gages were installed at each selected bridge site where a streamflow-gaging station did not already exist. If possible, scour-detection devices were installed on the upstream and downstream edges of piers in the main channel at the sites. These devices, which were especially useful to measure pier scour where the piers were inset under the bridges, consisted of a 1.25-in.-diameter schedule-80 pipe with a metal base located within a 2-in.-diameter galvanized pipe. The device worked by allowing the inside pipe to drop when scour lowered the bed at the edge of the pier. The top part of the inside pipe was marked with a 0.5-ft scale that could be read to measure scour if the pipe had dropped during or after a high flow. These devices had limited success because, at many of the bridge sites, debris tended to jam the inside pipe and lock it in place. The device would work best on sand-bed streams with no susceptibility to debris.

Scour was measured during high flows using fathometers and by sounding. Both methods were used for quality-assurance purposes. Eagle Mach I™ fathometers with paper printouts and associated 8-degree transducers were used to measure scour, which was measured on the upstream side of the bridge, unless debris was a problem. A downstream cross section also was usually defined. Sounding was done using standard streamflow-gaging procedures (Rantz and others, 1982). Depths were measured by suspension of a sounding weight from a cable during high flow or by wading and using a wading rod during lower flows.

If possible, velocity also was measured at the upstream edge of the bridge. An attempt was made to measure velocity and depth as close as possible to the upstream edge of the piers. Flow was computed using standard streamflow-gaging procedures.

Scour was measured during high-flow events at 10 of the 13 selected bridge sites. A summary of the scour measurements for the pier, contraction, and abutment scour at these bridges is presented in table 10. Left and right are determined in table 10 by looking in a downstream direction. The reported scour depth is the scour depth interpreted from the cross sections defined near the peak of the flow event. Scour was measured as the difference between a reference line and the existing bed. The reference line was drawn on the cross section at locations to represent pre-scour conditions and was drawn to remove any apparent scour holes caused by previous pier and abutment scour. Contraction scour was measured by first making a determination of where the bed was before any contraction scour took place and measuring the difference. Historical cross-sectional data, when available, were used to help determine the amount of scour. Some of the depths reported may possibly include scour that took place during previous high-flow events and the previous scour hole did not refill. In other instances, debris protected the channel at the piers and abutments and reduced the scour.

Scour was only measured at the sites if the flow was about equal to or greater than the theoretical 2-year flow. Some sites did not meet this criteria and therefore did not have any scour measurements made.

Cross sections at selected bridges for dates when scour was measured are presented in figures 7-13. In these figures, left and right banks are determined by looking in a downstream direction.

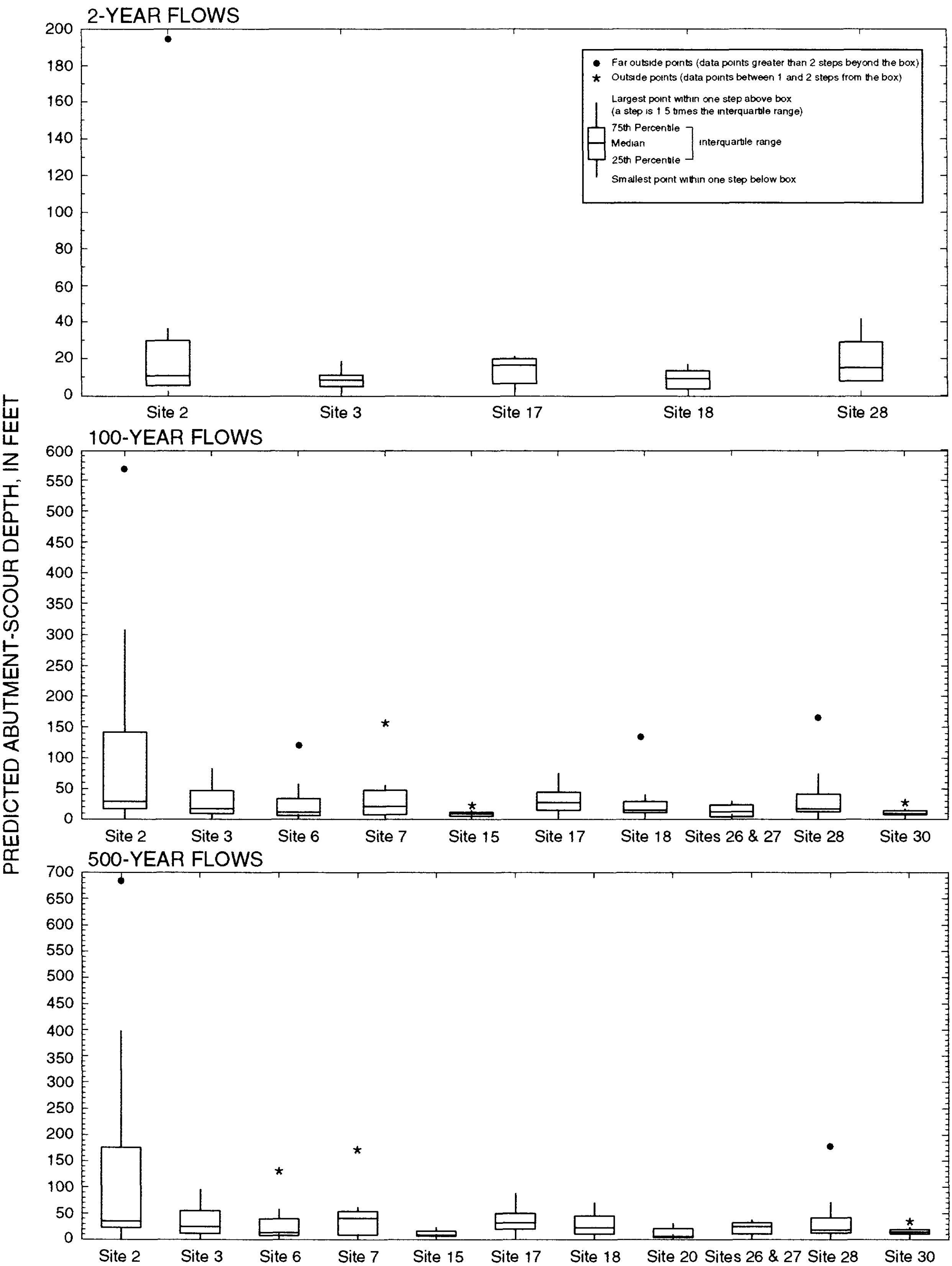


Figure 6. Distribution of predicted abutment-scour depths using published equations for various flows at selected bridge sites.

Table 10. Summary of scour measurements at selected bridge sites

[ft^3/s, cubic feet per second; ---, no data; u.s., upstream; d.s., downstream; -, aggradation; +, scour was greater than value shown]

Site number (fig. 1)	Date	Discharge (ft^3/s)	Pier scour[1], in feet, with horizontal location[2] in parentheses						Contraction scour, in feet	Abutment scour, in feet		Maximum bed change at a point	Notes
			Pier A	Pier B	Pier C	Pier D	Pier E	Pier F		Left bank	Right bank		
2	3-9-93	3,950	---	0.5 u.s. (217)	2.0 u.s. (337)	---	---	---	2.5 u.s.	0.3 u.s.	0.6 u.s.	4.0 u.s.	
3	6-22-92	4,346	0.2 u.s. 0.2 d.s. (98)	1.4 u.s. 7.6 d.s. (219)	0.2 u.s. 0.5 d.s. (338)	---	---	---	0.8 u.s. 1.0 d.s.	1.2 u.s. 1.4 d.s.	0.4 u.s. 1.0 d.s.	1.2 u.s. 1.8 d.s.	
	3-30-93	9,090	0 u.s. 0 d.s.	1.2 u.s. 8.5 d.s.	0.1 u.s. 2.0 d.s.	---	---	---	1.0 u.s. 1.0 d.s.	1.4 u.s. 1.8 d.s.	0 u.s. 1.5 d.s.	1.4 u.s. 1.8 d.s.	
	7-7-93	7,774	0.6 u.s. 0 d.s.	2.0 u.s. 7.0 d.s.	1.0 u.s. 1.8 d.s.	---	---	---	2.0 u.s. 3.0 d.s.	1.6 u.s. 2.0 d.s.	3.5 u.s. 4.0 d.s.	7.7 u.s. 6.5 d.s.	(3)
6	3-29-93	1,120	1.0 u.s. (1855)	1.0 u.s. (1838)	1.2 u.s. (1820)	0.6 u.s. (1801)	---	---	-1.0 u.s.	1.1 u.s.	0.7 u.s.	-1.8 u.s.	(4)
	4-22-93	---	1.0 u.s.	2.4 u.s. 1.1 d.s.	1.6 u.s. .8 d.s.	0.3 u.s. 0 d.s.	---	---	0.5 u.s. 1.0 d.s.	1.5 u.s.	0.4 u.s. 1.3 d.s.	1.5 u.s. 1.5 d.s.	
	5-11-93	560	0.7 u.s.	1.5 u.s. 1.0 d.s.	1.4 u.s. .8 d.s.	2.0 u.s. 1.3 d.s.	---	---	-2.0 u.s. -0.8 d.s.	0.8 u.s.	1.3 u.s. 0.6 d.s.	-2.0 d.s.	(4)
	7-7-93	804	0.7 u.s. 0 d.s.	1.1 u.s. 0.5 d.s.	1.0 u.s. 0.6 d.s.	0 u.s. 0 d.s.	---	---	0 u.s. 0 d.s.	---	0.7 u.s.	1.0 u.s -0.5 d.s.	(4)
7	3-26-93	1,800	0.2 u.s. (39)	0.2 u.s. (74)	0 (100)	---	---	---	-0.5 u.s.	0.6 u.s.	0 u.s.	-1.6 u.s.	
	3-30-93	5,350	0 u.s. 0 d.s.	1.0 d.s.	0.8 d.s.	---	---	---	1.5+ u.s. 1.0+ d.s.	0.8 u.s. 0.2 d.s.	1.2 u.s. 1.8 d.s.	2.7 u.s. 2.7 d.s.	(4)
	4-1-93	4,900	0 d.s.	1.1 d.s.	---	---	---	---	---	-0.3 d.s.	2.1 d.s.	2.1 d.s.	
	5-11-93	4,790	0.5 u.s. 0.4 d.s.	1.0 u.s. 1.9 d.s.	1.7 u.s.	---	---	---	0.5 u.s. -1.0 d.s.	0.5 u.s. 0.4 d.s.	0.3 u.s. 2.7 d.s.	1.5.u.s. 2.7 d.s.	(4)
	7-8-93	8,540	0 u.s. 2.2 d.s.	2.2 u.s. 1.2 d.s.	1.5 u.s. 1.5 d.s.	---	---	---	0.5 u.s. 1.0 d.s.	0.7 u.s. 1.6 d.s.	1.6 u.s. 1.1 d.s.	1.6 u.s. 2.2 d.s.	
15	7-13-93	1,860	0.2 u.s. (38)	---	---	---	---	---	0 u.s.	0.3 u.s.	0.3 d.s.	0.3 u.s.	

Table 10. Summary of scour measurements at selected bridge sites—Continued

Site number (fig. 1)	Date	Discharge (ft^3/s)	Pier scour[1], in feet, with horizontal location[2] in parentheses						Contraction scour, in feet	Abutment scour, in feet		Maximum bed change at a point	Notes
			Pier A	Pier B	Pier C	Pier D	Pier E	Pier F		Left bank	Right bank		
17	6-11-93	1,800	---	---	3.5 u.s. 0.9 d.s. (177)	---	---	---	1.5 u.s. -0.6 d.s.	0.5 u.s. 0.9 d.s.	0 u.s. 1.0 d.s.	1.9 u.s. 1.0 d.s.	
	7-21-93	4,130	---	---	2.5 u.s. 1.2 d.s.	---	---	---	-0.4 u.s. 0.5 d.s.	0.2 u.s. 1.5 d.s.	1.0 u.s. 0.7 d.s.	1.0 u.s. 1.5 d.s.	
	7-27-93	5,100	---	---	2.3 u.s.	---	---	---	1.0 u.s.	0.5 u.s.	0.4 u.s.	0.8 u.s.	
26 & 27	3-29-93	4,600	---	0.5 u.s. 1.5 d.s. (105)	1.5 u.s. 1.3 d.s. (175)	---	---	---	0 u.s. 0.5 d.s.	0 u.s. 1.2 d.s.	0.8 u.s. 0 d.s.	1.2 u.s. 1.2 d.s.	
	5-8-93	14,700	---	1.0 u.s. 1.7 d.s.	2.0 u.s. 1.0 d.s.	---	---	---	2.0 u.s. 3.0 d.s.	0 u.s. 0 d.s.	2.0 u.s. 0.7 d.s.	3.5 u.s. 4.0 d.s.	(5)
28	5-8-93	7,040	6.5 u.s. 1.4 d.s. (128)	2.0 u.s. 4.0 d.s. (249)	---	---	---	---	1.5 u.s. 0 d.s.	2.0 u.s. 1.7 d.s.	4.0 u.s. 0.8 d.s.	5.0 u.s. 4.0 d.s.	
32	6-16-92	17,300	---	---	---	---	---	---	3.0	3.0 u.s.	3.0 u.s.	3.0 u.s.	

[1]Pier scour measured on upstream and downstream edge of bridge.
[2]Distance from left bridge abutment, in feet.
[3]Peak stage about 2 feet higher than stage at discharge shown.
[4]Debris made measuring scour difficult, especially on upstream side of bridge
[5]Peak stage about 3 feet higher than stage at discharge shown.

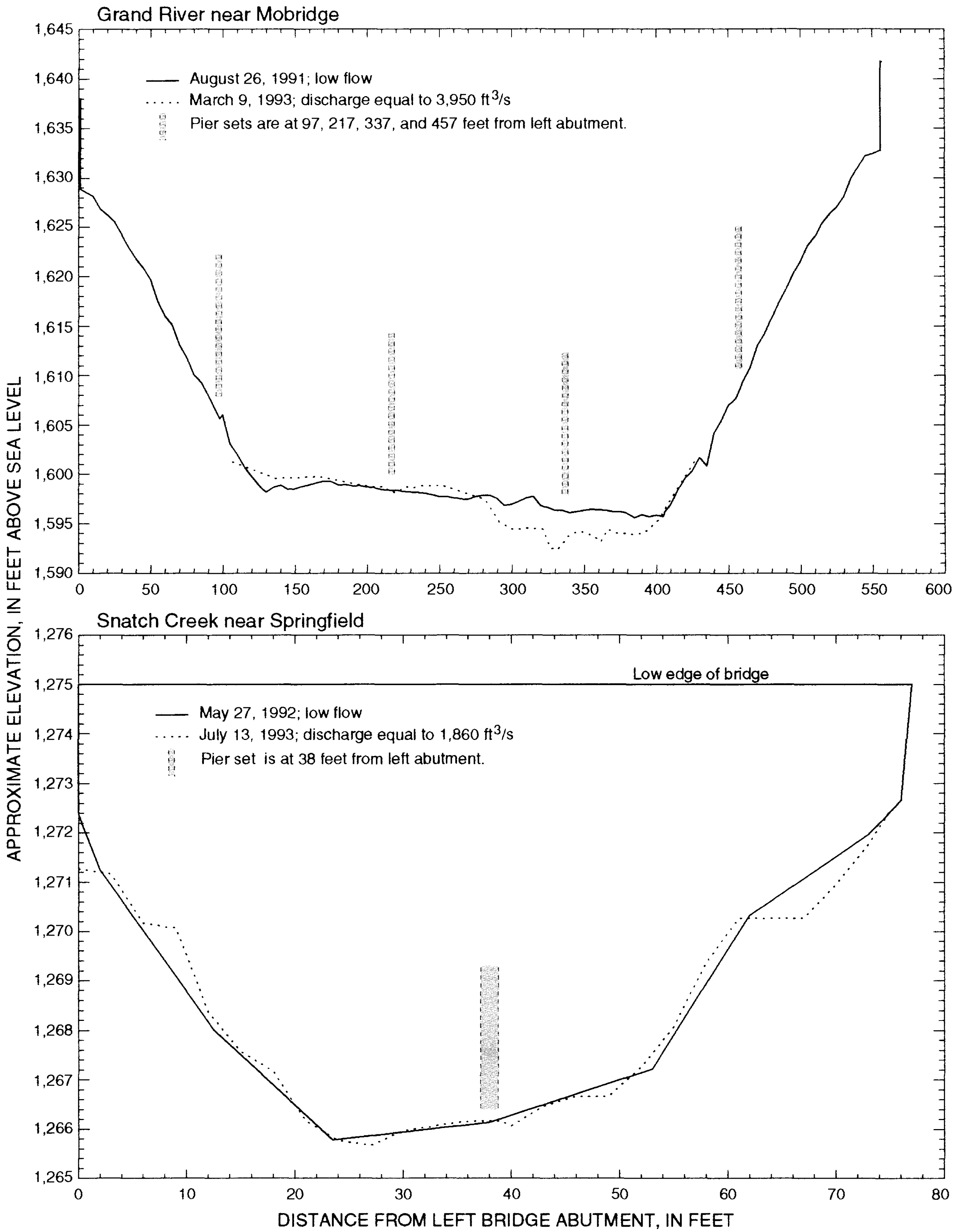


Figure 7. Upstream cross sections at the Grand River near Mobridge (site 2) and Snatch Creek near Springfield (site 15) bridge sites.

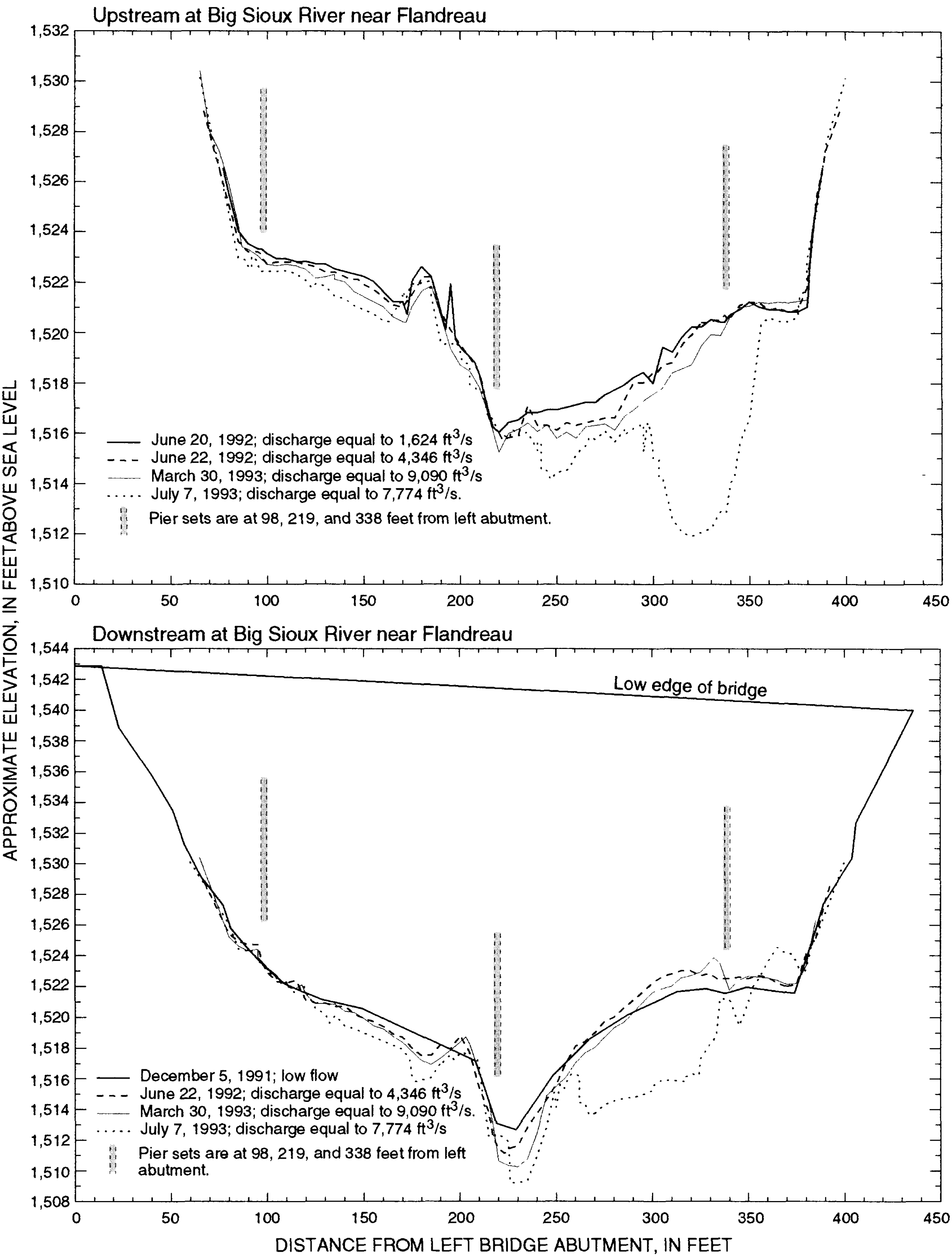


Figure 8. Upstream and downstream cross sections at the Big Sioux River near Flandreau (site 3) bridge site.

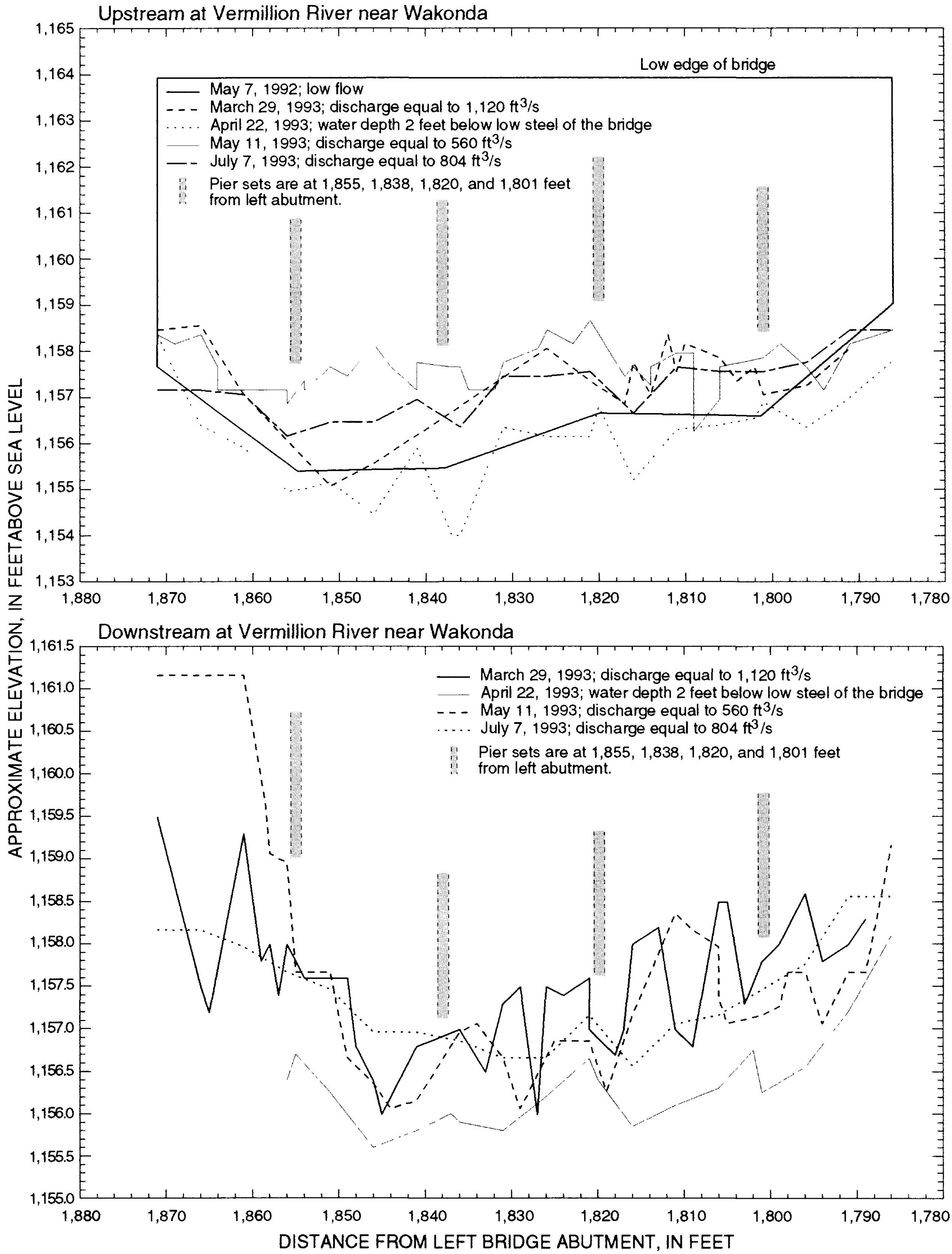


Figure 9. Upstream and downstream cross sections at overflow on the Vermillion River near Wakonda (site 6) bridge site.

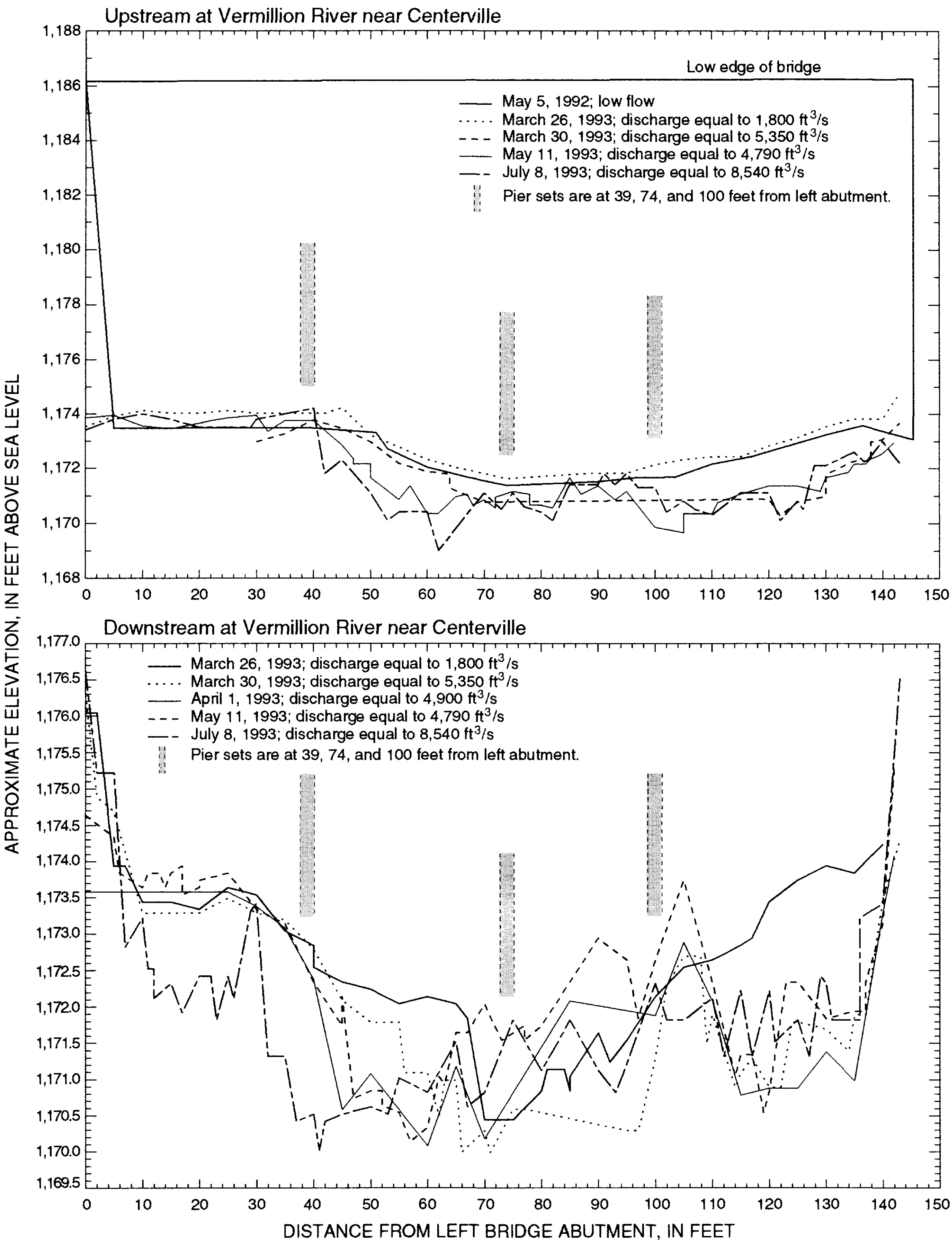


Figure 10. Upstream and downstream cross sections at the Vermillion River near Centerville (site 7) bridge site.

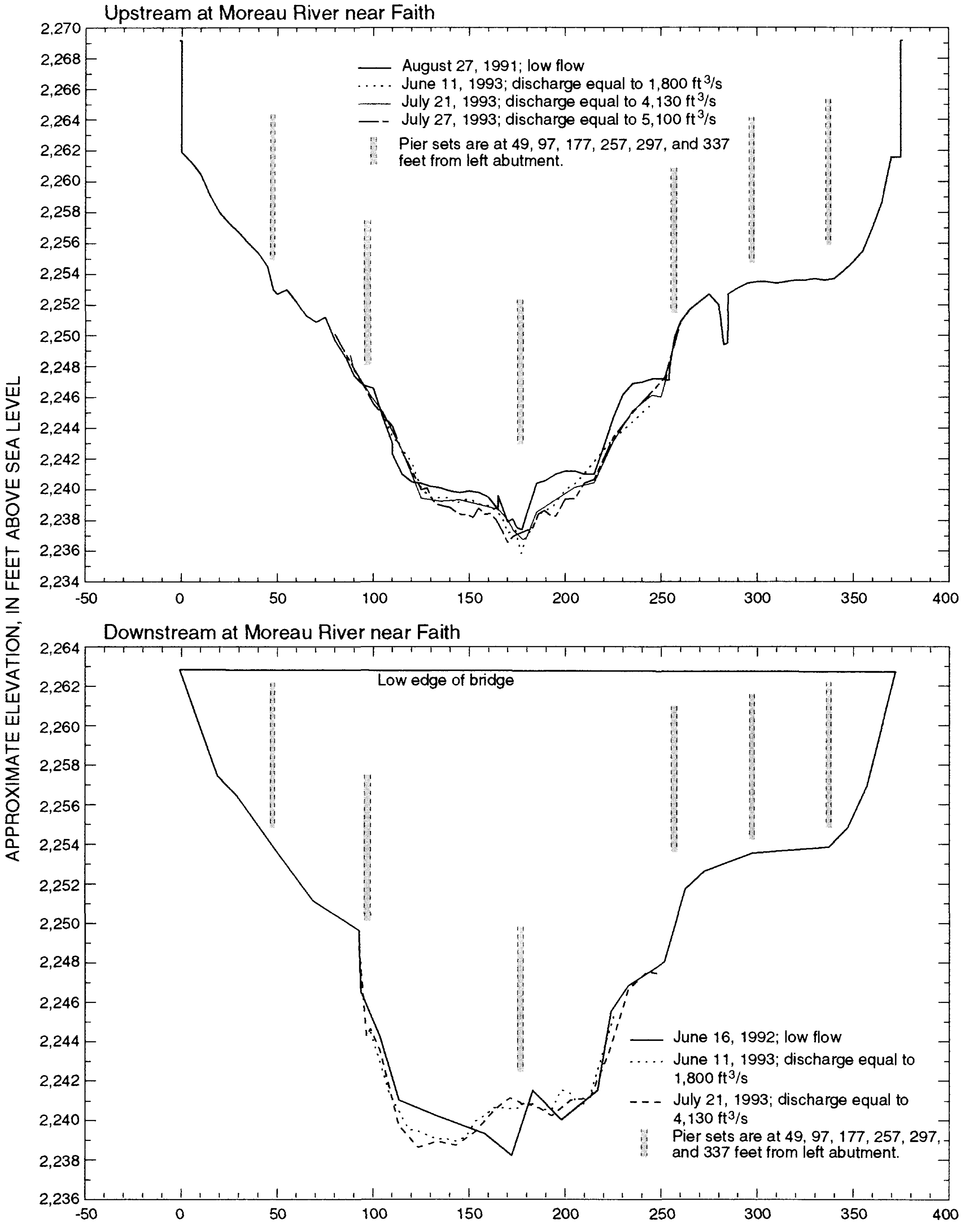


Figure 11. Upstream and downstream cross sections at the Moreau River near Faith (site 17) bridge site.

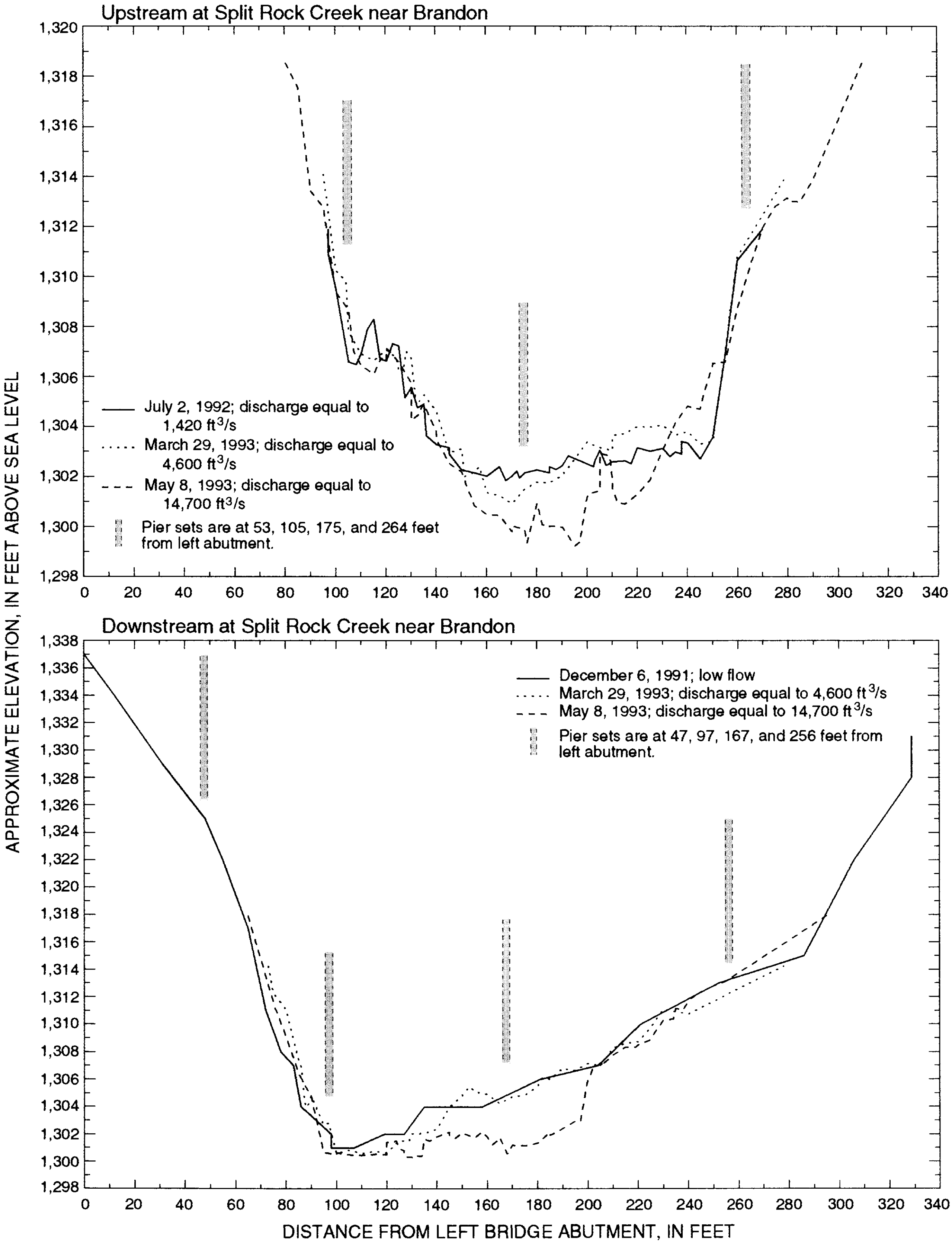


Figure 12. Upstream and downstream cross sections at the Split Rock Creek near Brandon (sites 26 and 27) bridge sites.

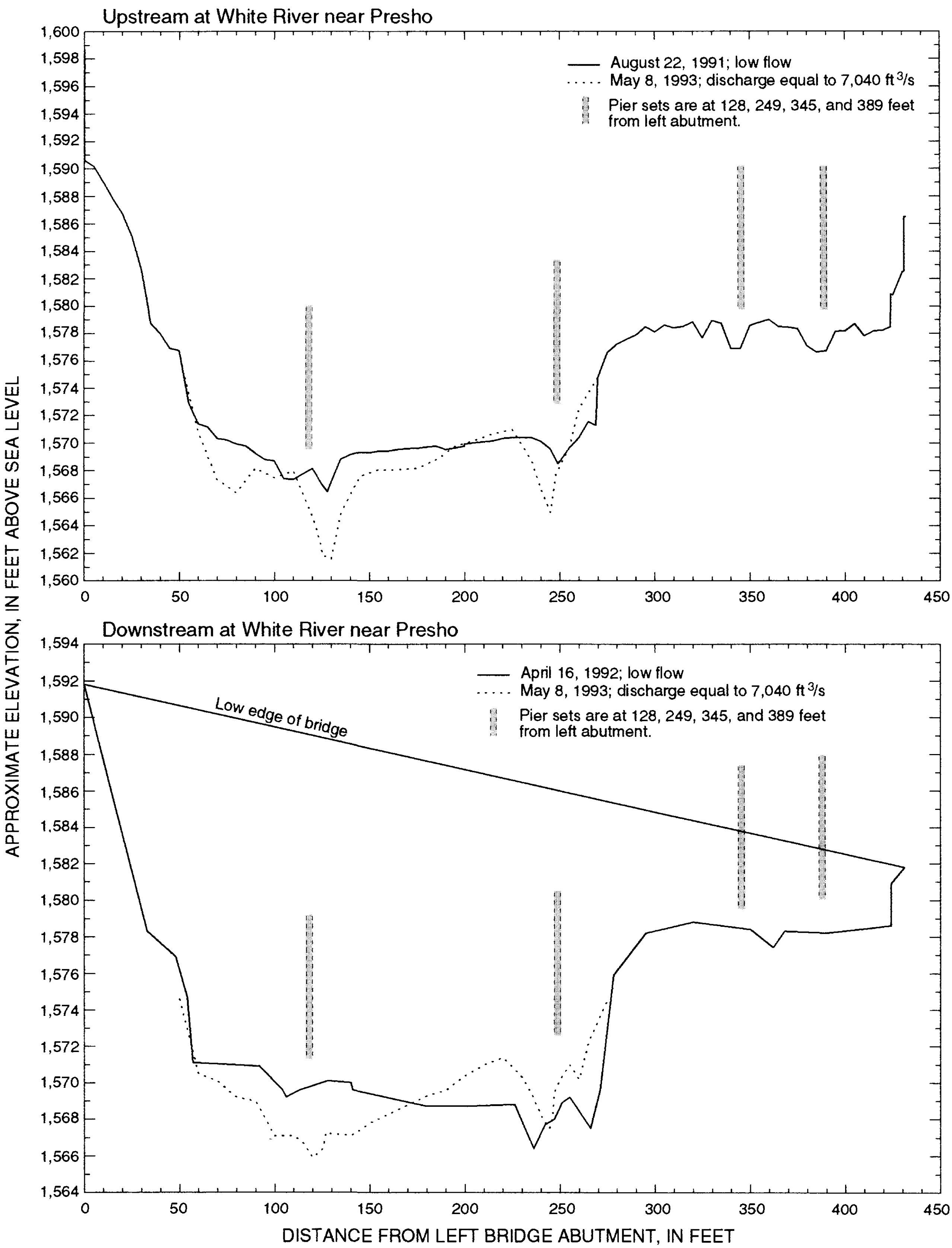


Figure 13. Upstream and downstream cross sections at the White River near Presho (site 28) bridge site

The maximum measured pier scour was 2.0 ft at sites 2, 26, and 27, 2.2 ft at site 7, 2.4 ft at site 6, 3.5 ft at site 17, 6.5 ft at site 28, and 8.5 ft at site 3. The maximum measured contraction scour was about 1.5 ft at sites 7, 17, and 28; 2.5 ft at site 2; and 3.0 ft at sites 3, 26, 27, and 32. The maximum measured abutment scour was 1.5 ft at sites 6 and 17, 2.0 ft at sites 26 and 27, 2.7 ft at site 7, 3.0 ft at site 32, and 4.0 ft at sites 3 and 28. The maximum bed change at a point along the section also is included in table 10. Sites 2, 26, 27, 28, and 3 all had a maximum measured bed change at a point of greater than or equal to 4.0 ft during a high flow event. The flow events during which scour was measured included near 100-year flows at sites 15, 26, and 27. Scour at sites 26 and 27 was measured on May 8, 1993, when the water level was 3 ft below the peak. Scour was measured at sites 3, 6, and 7 when the flow was well in excess of the 2-year flow. Scour was measured at site 32 after the peak flow of record subsided.

Comparison of Measured and Predicted Scour Depths

Flow data collected during high-flow events when scour was measured were used to calculate predicted pier-scour depths using 13 published equations. Data collected and used in these equations included peak flow magnitudes, water depths at the piers, and velocities upstream of the piers. These predicted scour depths were compared to the measured scour depths to make an assessment as to which prediction equations best reflect actual pier scour. A summary of these comparisons at selected bridge sites is presented in table 11 and figure 14.

The figures are presented to illustrate how predicted pier scour compares with measured pier scour. Points that plot close to a 1:1-slope reference line indicate a close relationship between the scour-prediction equation and the actual field-measured scour. Most of the points presented on these figures plot above this 1:1-slope reference line, indicating that the equations tend to overestimate the scour depths.

Laursen (1), Blench (4b), and Inglis-Poona (5a) equations overestimate the pier-scour depths for almost every point plotted. However, the Laursen (1) equation closer approximates the measured pier-scour depths than the other two equations, especially at the greater depths. The Inglis-Poona (5a) equation greatly overpredicts pier-scour depths.

The Shen (2b), Colorado State University (3), Blench (4a), Inglis-Poona (5b), and Breusers (10) equations also tend to overestimate the pier-scour depths. However, for a few of the larger measured pier-scour depths, they underestimate the scour depths.

The Shen (2a), Bata (7), and Carstens (9) equations produced many points that plot close to the 1:1-slope reference line, but also produced both over and underestimated pier-scour depths for some of the larger scour depths.

The Varzeliotis (8) equation underestimates the measured pier-scour depths in most cases. It predicts aggradation in most cases when scour occurred and was measured during a flood. The Chitale (6) equation exhibits little correlation with measured pier-scour depths.

The equations that best correlate with measured pier scour based on the above figures is highly subjective, but the Laursen (1), Shen (2b), Colorado State University (3), Blench (4a), Inglis-Poona (5b), and Breusers (10) equations appear to be more closely correlated with measured pier scour than the other equations.

Boxplots showing the distribution of the measured pier-scour depths during high flows and the corresponding predicted pier-scour depths using published equations at selected bridge sites are presented in figure 15. A boxplot for the Inglis-Poona (5a) equation was not done because it has a much higher range of values than for the other equations.

The boxplot for measured pier-scour depths shows that the distribution of data points is skewed to the right, with some far outliers. The median is 1.15 ft and the interquartile range is 1.625 ft. The boxplots for the predicted scour depths from the published equations are predominantly skewed to the right. The medians for these equations range from -3.25 ft using the Varzeliotis (8) equation to 7.8 ft using the Blench (4b) equation. The interquartiles range from 0.575 ft for the Carstens (9) equation to 41.975 ft for the Inglis-Poona (5a) equation. With the exception of the Blench (4b), the Chitale (6), and the Inglis-Poona (5a) equations, the interquartile ranges are all less than 3.5 ft.

Table 11. Summary of comparison of measured and predicted pier-scour depths at selected bridge sites

[The measured scour depth shown is taken from upstream or downstream side of the bridge. The depths and velocities used in the prediction equations are field-measured values. ft^3/s, cubic feet per second; ---, no data]

Site number	Discharge (ft^3/s)	Date of measurement	Measured scour depth (feet)[1]	Pier-scour depth (feet) determined using scour-prediction equation developed by indicated investigator (equation number in parentheses)												
				Laursen	Shen		Colorado State University	Blench		Inglis-Poona		Chitale	Bata	Varzeliotis	Carstens	Breusers
				(1)	(2a)	(2b)	(3)	(4a)	(4b)	(5a)	(5b)	(6)	(7)	(8)	(9)	(10)
2	3,950	3-9-93	2.0 (337)	7.8	8.5	7.9	6.6	4.3	7.6	141.3	4.1	9.0	7.8	4.1	2.2	5.6
3	4,346	6-22-92	0.2 (98)	5.5	.1	1.6	2.1	3.1	4.7	17.4	2.9	-1.4	-.5	-2.3	---	4.2
			7.6 (219)	7.0	.6	2.6	3.2	3.2	10.2	39.4	3.3	-1.4	-.1	-5.2	.1	4.2
			0.5 (338)	6.1	.1	1.6	2.2	3.3	6.6	19.4	3.2	-2.2	-.5	-4.0	.6	4.2
	9,090	3-30-93	0 (98)	5.9	1.0	3.1	3.4	3.2	6.0	37.9	3.1	1.1	.3	-.9	---	4.2
			8.5 (219)	7.5	.8	2.9	3.5	2.9	13.4	48.1	3.1	-1.7	.1	-7.6	---	4.2
			2.0 (338)	6.6	4.2	5.1	4.8	3.3	8.4	68.5	3.3	5.8	3.2	.6	2.4	4.2
	7,774	7-7-93	0.6 (98)	5.9	.2	1.9	2.4	3.2	5.8	22.8	3.1	-1.3	-.4	-2.7	.5	4.2
			7.0 (219)	7.5	.7	2.8	3.4	2.9	13.0	45.2	3.1	-2.0	.0	-7.4	---	4.2
			1.8 (338)	7.6	1.1	3.3	3.8	2.8	13.6	54.8	3.1	-.5	.4	-6.9	---	4.2
6	1,120	3-29-93	1.0 (1,855)	3.5	.3	1.3	1.7	1.1	6.7	.2	1.3	-1.1	.2	-4.1	1.3	1.9
			1.0 (1,838)	3.5	2.0	2.3	2.5	1.1	7.1	5.1	1.2	2.9	1.7	-2.1	.8	1.9
			1.2 (1,820)	3.5	.7	1.6	1.9	1.1	7.1	1.7	1.2	.0	.5	-3.7	.9	1.9
			0.6 (1,801)	3.5	1.2	2.0	2.2	1.2	6.6	3.6	1.3	1.6	1.1	-2.4	.8	1.9
	560	5-11-93	0.7 (1,855)	3.5	.2	1.1	1.5	1.1	7.0	-.9	1.3	-1.8	.1	-4.8	7.5	1.9
			1.5 (1,838)	3.4	.3	1.3	1.7	1.2	6.4	.4	1.3	-.9	.2	-3.8	1.3	1.9
			1.4 (1,820)	3.3	.3	1.3	1.6	1.3	5.7	.8	1.4	-.7	.2	-3.1	1.3	1.9
			2.0 (1,801)	3.3	.5	1.5	1.8	1.3	5.8	1.5	1.4	-.1	.4	-2.8	1.0	1.9
	804	7-7-93	0.7 (1,855)	3.3	.2	1.1	1.5	1.3	5.4	.0	1.4	-1.1	.1	-3.3	7.5	1.9
			1.1 (1,838)	3.2	.5	1.5	1.7	1.4	4.9	1.9	1.4	.1	.4	-2.1	1.0	1.9
			1.0 (1,820)	3.1	.7	1.6	1.8	1.4	4.6	2.6	1.5	.7	.5	-1.5	.9	1.9
			0 (1,801)	3.1	1.1	1.9	2.0	1.4	4.4	3.9	1.5	1.6	.9	-.8	.8	1.9

Table 11. Summary of comparison of measured and predicted pier-scour depths at selected bridge sites—Continued

Site number	Discharge (ft^3/s)	Date of measurement	Measured scour depth (feet)[1]	Laursen (1)	Shen (2a)	Shen (2b)	Colorado State University (3)	Blench (4a)	Blench (4b)	Inglis-Poona (5a)	Inglis-Poona (5b)	Chitale (6)	Bata (7)	Varzeliotis (8)	Carstens (9)	Breusers (10)
7	1,800	3-26-93	0.2 (39)	5.1	0.1	1.3	1.9	2.4	7.4	7.7	2.4	-2.7	0.0	-5.2	---	3.1
			0.2 (74)	5.4	.8	2.4	2.9	2.2	9.4	19.2	2.3	-.3	.6	-4.6	1.5	3.1
			0 (100)	5.3	1.0	2.6	3.0	2.3	8.7	20.9	2.4	.5	.8	-3.6	1.4	3.1
	5,350	3-30-93	0 (39)	5.6	2.9	3.7	3.9	2.1	10.0	32.7	2.3	4.2	2.5	-2.4	1.3	3.1
			1.0 (74)	5.9	1.7	3.0	3.5	1.6	12.4	28.3	2.0	1.4	1.4	-5.7	1.3	3.1
			0.8 (100)	5.9	.8	2.4	3.0	1.6	12.3	20.3	2.0	-1.3	.6	-7.3	1.5	3.1
	4,790	5-11-93	0.5 (39)	5.5	1.2	2.8	3.2	2.1	9.7	23.5	2.3	1.0	1.0	-4.0	1.3	3.1
			1.9 (74)	5.8	2.7	3.6	3.9	1.8	11.8	33.6	2.0	3.7	2.4	-4.0	1.3	3.1
			1.7 (100)	5.9	2.7	3.6	3.9	1.6	12.4	34.1	2.0	3.7	2.4	-4.5	1.3	3.1
	8,540	7-8-93	2.2 (39)	6.0	4.7	4.3	4.4	1.5	12.8	42.4	1.9	6.9	4.2	-3.2	1.2	3.1
			2.2 (74)	6.1	5.5	4.5	4.6	1.4	13.6	45.8	1.8	7.9	4.9	-3.3	1.2	3.1
			1.5 (100)	6.1	4.7	4.3	4.5	1.3	14.0	43.7	1.7	6.9	4.2	-4.2	1.2	3.1
15	1,860	7-13-93	0.2 (38)	4.2	6.0	4.0	3.7	1.7	7.0	20.0	1.8	7.1	5.3	.7	1.0	2.4
17	1,800	6-11-93	3.5 (177)	6.4	4.2	5.1	4.3	3.3	7.7	65.8	3.3	5.7	3.6	1.0	---	4.2
	4,130	7-21-93	2.5 (177)	6.7	8.5	6.5	5.1	3.3	9.2	88.3	3.3	9.7	7.6	2.2	1.7	4.2
	5,100	7-27-93	2.3 (177)	7.0	7.5	6.2	5.1	3.2	10.4	90.1	3.3	9.5	6.7	1.1	1.7	4.2
28	7,040	5-8/93	6.5 (128)	7.4	16.3	8.5	6.9	3.5	10.4	132.3	3.6	14.4	14.8	4.4	---	4.6
			4.0 (249)	6.8	3.3	5.0	4.7	3.6	7.8	72.7	3.5	4.7	3.0	.5	1.8	4.6

Pier-scour depth (feet) determined using scour-prediction equation developed by indicated investigator (equation number in parentheses)

[1]Number in parentheses is distance from left abutment of bridge, in feet.

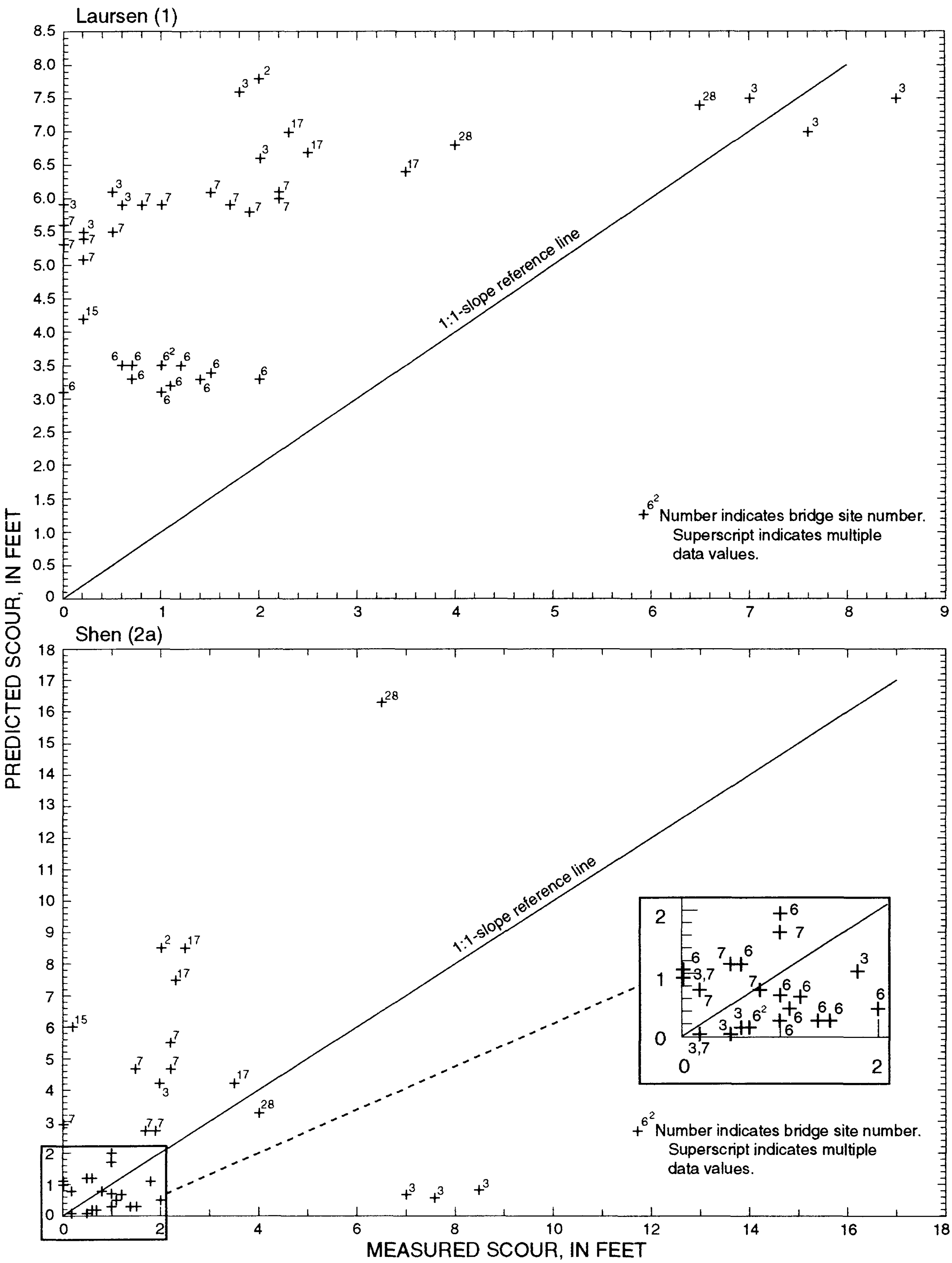


Figure 14. Measured versus predicted pier-scour depths using published equations at selected bridge sites.

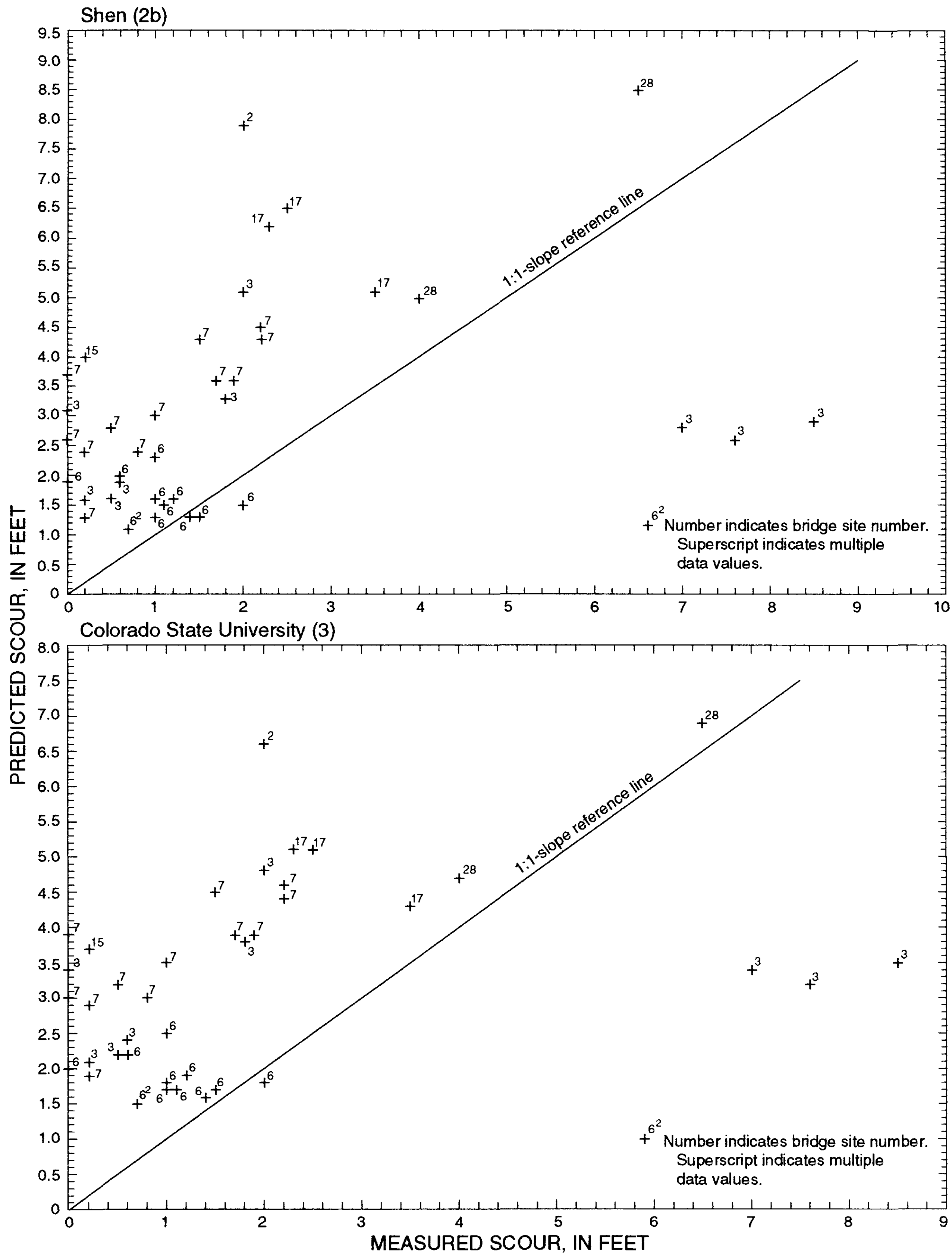


Figure 14. Measured versus predicted pier-scour depths using published equations at selected bridge sites.--Continued

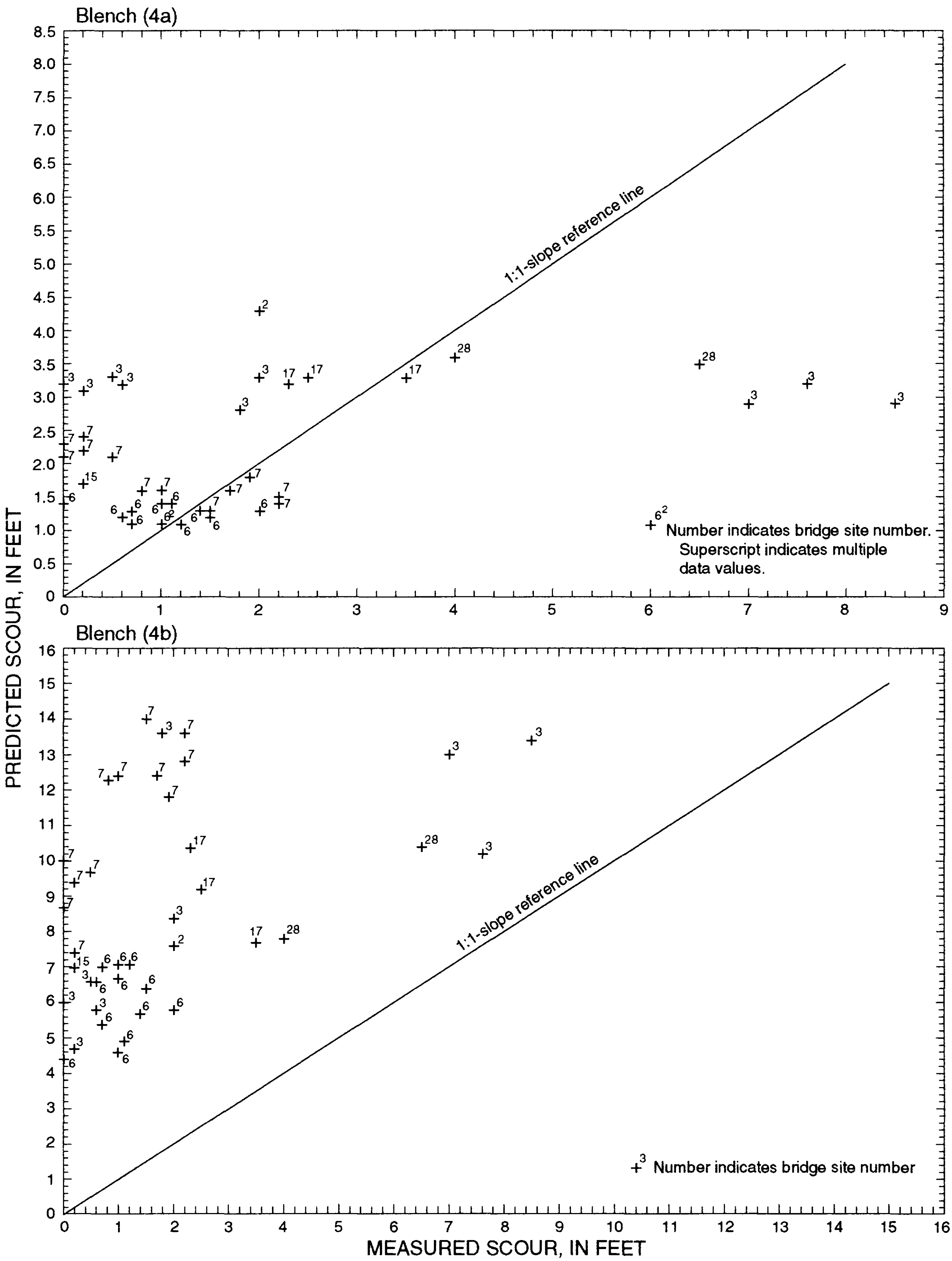


Figure 14. Measured versus predicted pier-scour depths using published equations at selected bridge sites.--Continued

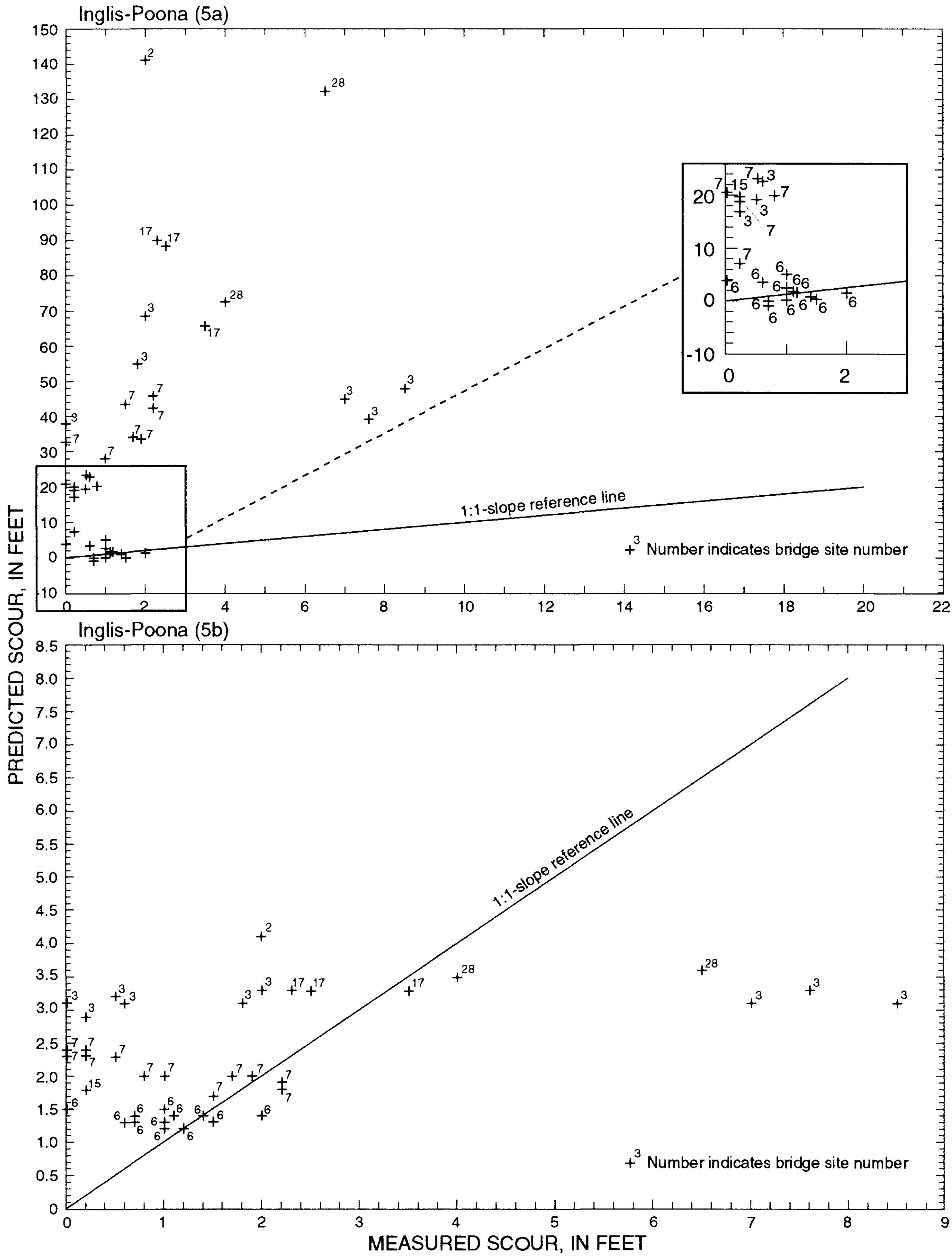


Figure 14. Measured versus predicted pier-scour depths using published equations at selected bridge sites.--Continued

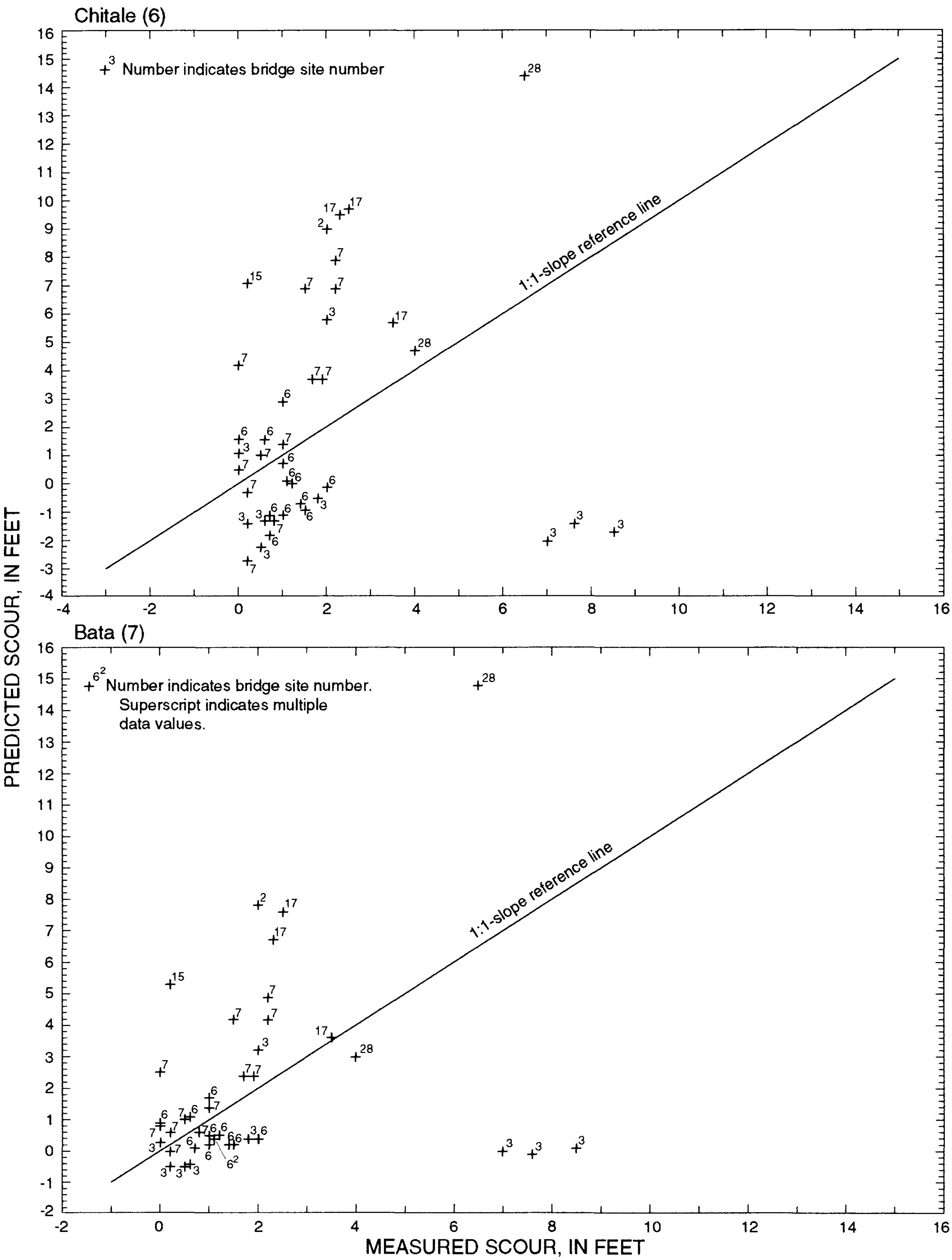


Figure 14. Measured versus predicted pier-scour depths using published equations at selected bridge sites.--Continued

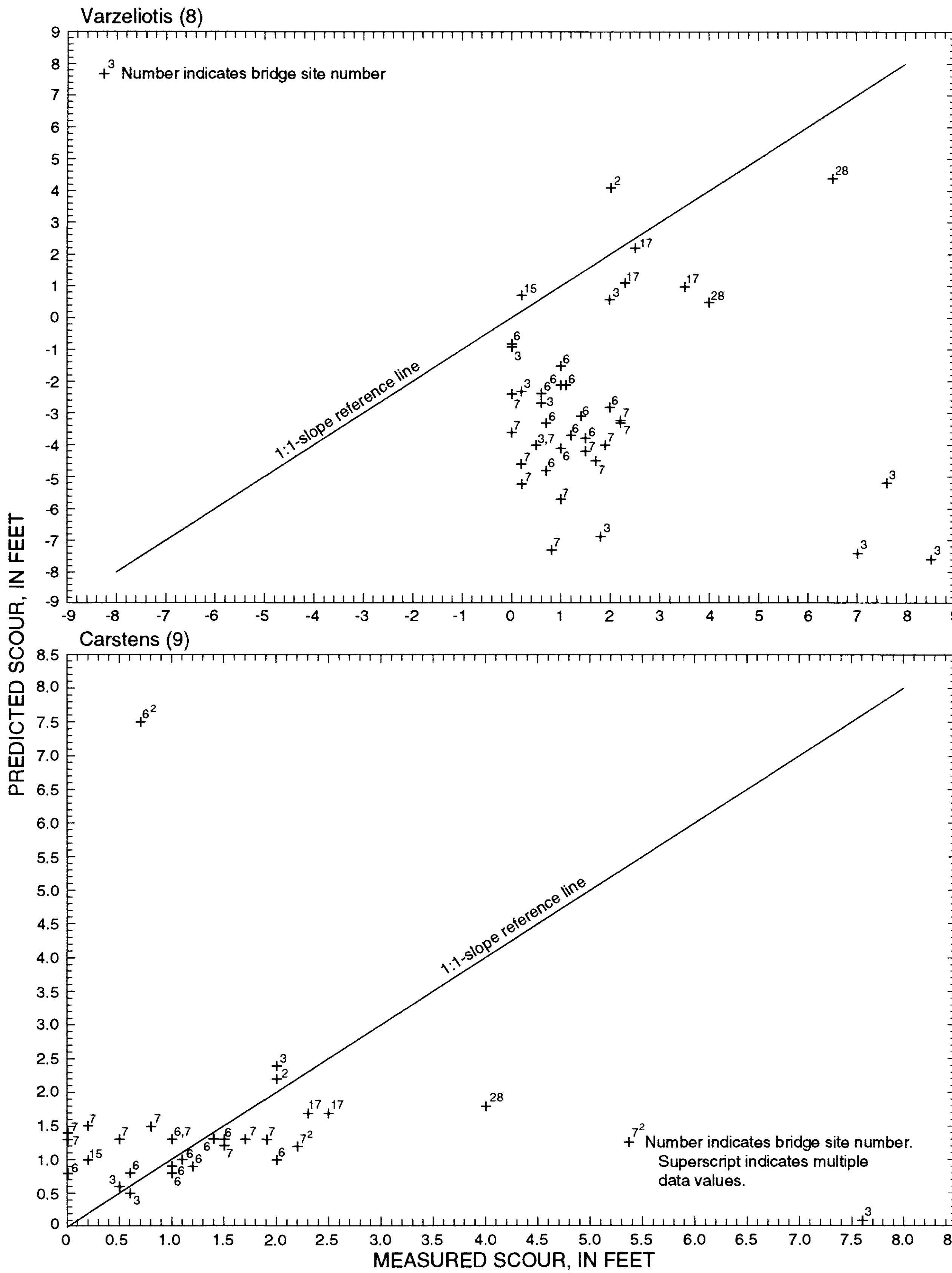


Figure 14. Measured versus predicted pier-scour depths using published equations at selected bridge sites.--Continued

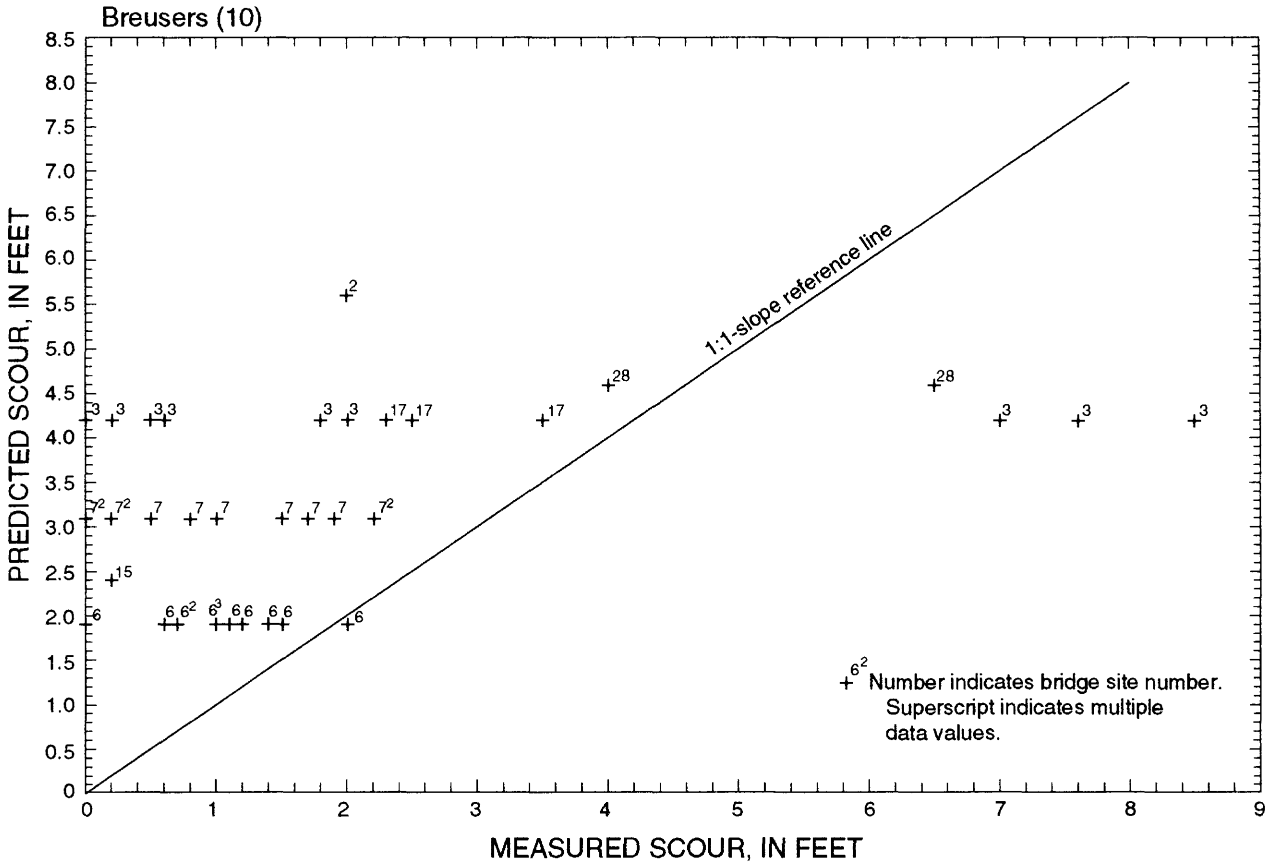


Figure 14. Measured versus predicted pier-scour depths using published equations at selected bridge sites.--Continued

The Wilcoxon signed-ranks test was used to determine if there were statistical differences between the medians of the measured and predicted pier-scour depths. This test is nonparametric so the data do not have to be normally distributed. The null hypothesis for each comparison was that the median of the measured scour depths is equal to the median of the predicted scour depths for that particular equation. A two-tail test at the 0.05 level of significance indicated that the Shen (2a), Chitale (6), Bata (7), and Carstens (9) equations are the only medians of measured pier-scour depths statistically equal (the hypothesis is accepted) to the median of predicted pier-scour depths. The Blench (4a) equation also was very close to meeting the test. The "p values" for the Wilcoxon signed-ranks tests were as follows: Laursen (1): 0.0000; Shen (2a): 0.1126; Shen (2b): 0.00001; Colorado State University (3): 0.0001; Blench (4a): 0.0451; Blench (4b): 0.0000; Inglis-Poona (5a): 0.0000; Inglis-Poona (5b): 0.0100; Chitale (6): 0.7116; Bata (7): 0.6047; Varzeliotis (8): 0.0000; Carstens (9): 0.8151; and Breusers (10): 0.0001.

The results of Spearman rank correlation analyses between the measured and predicted scour depths are presented in table 12. The rank correlation coefficient is used for nonparametric data and measures the monotonic association between two samples (whether one sample increases or decreases with the other sample even when the relation between the samples is not linear). The strongest relation between the measured and predicted pier-scour depths was for depths estimated using the Laursen (1) and Inglis-Poona (5a) equations, which had Spearman rank correlation coefficients of 0.60 and 0.57, respectively. However, the Inglis-Poona (5a) equation, based on the data collected, vastly overestimates pier-scour depths. The Shen (2b), Colorado State University (3), and Blench (4b) equations also have relatively strong relations between measured and predicted pier-scour depths. The Varzeliotis (8) and Carstens (9) equations had weak relations between the measured and predicted depths, with rank correlation coefficients of 0.03 and 0.17, respectively.

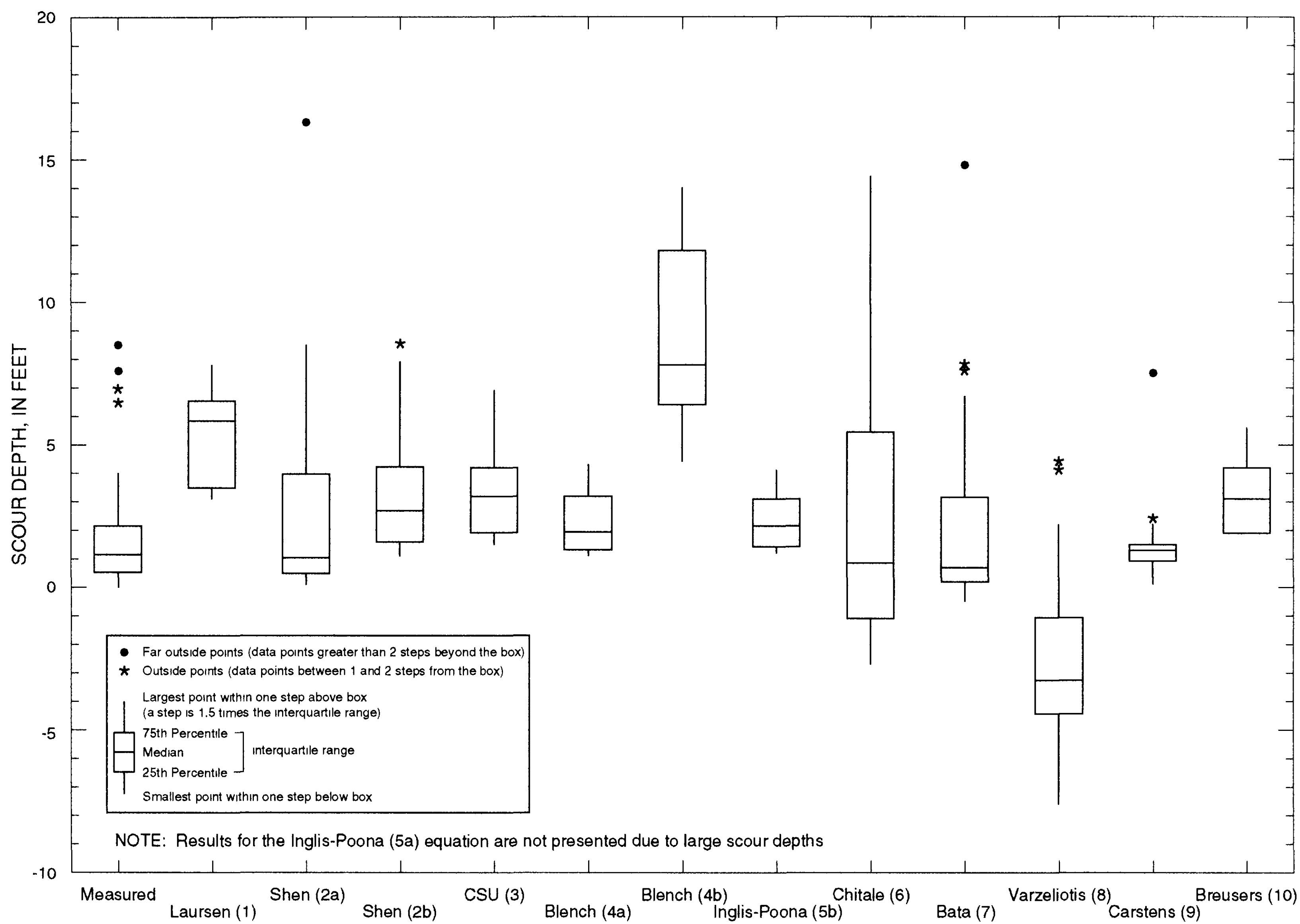


Figure 15. Distribution of measured pier-scour depths during high flows and predicted pier-scour depths using published equations at selected bridge sites.

Table 12. Spearman rank correlation analyses between measured and predicted pier-scour depth at selected bridge sites

	Spearman rank correlation coefficient for scour depths estimated using indicated equation												
	Laursen	Shen		Colorado State University	Blench		Inglis-Poona		Chitale	Bata	Varzeliotis	Carstens	Breusers
	(1)	(2a)	(2b)	(3)	(4a)	(4b)	(5a)	(5b)	(6)	(7)	(8)	(9)	(10)
Measured	0.60	0.38	0.47	0.49	0.27	0.49	0.57	0.35	0.27	0.29	0.03	0.17	0.40
Laursen (1)		.50	.76	.81	.76	.71	.92	.83	.26	.30	-0.03	.25	.91
Shen (2a)			.92	.88	.31	.54	.74	.35	.93	.95	.48	.24	.36
Shen (2b)				.99	.61	.63	.93	.65	.80	.81	.39	.25	.68
Colorado State University(3)					.63	.69	.95	.69	.74	.76	.31	.26	.73
Blench (4a)						.23	.77	.98	.19	.17	.30	.24	.94
Blench (4b)							.67	.35	.28	.40	-.42	.24	.44
Inglis-Poona (5a)								.83	.56	.57	.24	.20	.86
Inglis-Poona (5b)									.20	.20	.22	.29	.96
Chitale (6)										.96	.67	.20	.19
Bata (7)											.54	.28	.18
Varzeliotis (8)												.03	.15
Carstens (9)													.24
Breusers (10)													

The results of Spearman rank correlation analyses between the predicted scour depths using the published equations are also shown in table 12. There are significant relations among many of the published equations, with the main exception being the Varzeliotis (8) and Carstens (9) equations where the correlation coefficients are low. The Laursen (1), Shen (2b), Colorado State University (3), and Inglis-Poona (5a) equations exhibit strong relations with most of the other equations.

Scour data collected during high-flow events when scour was measured also was used to calculate predicted contraction- and abutment-scour depths using published equations. Only one equation (Laursen) was used to predict contraction-scour depths. Five equations were used to predict abutment-scour depths.

The contraction-scour prediction equation substantially overestimated the scour depths in almost all comparisons with the measured scour depths (tables 9 and 10). One reason for the large predicted scour depths obtained using the contraction equation, as compared to the measured scour depths, is the presence of wide flood plains (as wide as 5,000 ft) at most of the investigated bridge sites. One way to reduce this effect for bridge design is to make a decision on what is the effective approach section and thereby limit the size of the bridge flow approach width.

The abutment-scour prediction equations also substantially overestimated the scour depths in almost all comparisons with the measured scour depths (tables 9 and 10). For example, 1.8 ft (maximum) of abutment scour was measured at site 3 during a flow of 9,090 ft^3/s, as compared to a range of 1.5 to 17.8 ft (using the 2-year recurrence flows) scour depth predicted by the equations. At site 6, 0.7 ft (maximum) of abutment scour was measured during a flow of 11,500 ft^3/s, as compared to a range of 2.4 to 58.3 ft (using 100-year flows) scour depth predicted by the equations. At site 7, 1.6 ft (maximum) of abutment scour was measured during a flow of 8,540 ft^3/s, as compared to a range of 6.0 to 156.0 ft (using 100-year flows) scour depth predicted by the equations. At site 15 during a flow with nearly 100-year recurrence interval, 0.3 ft (maximum) of abutment scour was measured, as compared to a range of 3.4 to 22.0 ft scour depth predicted by the equations. At sites 26 and 27 during a flow with approximately a 100-year recurrence interval, 2.0 ft (maximum) of abutment scour was measured, as compared to a range of 0.4 to 29.7 ft scour depth predicted by the equations. At site 32, 3.0 ft (maximum) of abutment depth was measured during a flow of 17,300 ft^3/s, as compared to a range of 3.7 to 70.3 ft scour depth predicted by the equations.

The Liu and others (live bed) equation predicted abutment-scour depths substantially lower than the other four abutment-scour equations and closer to the actual measured scour depths. However, this equation at times predicted higher scour depths for 2-year flows than it did for 500-year flows, making its use highly questionable. One reason for the large predicted scour depths for the abutment-scour equations, as compared to the measured scour depths, is the presence of wide flood plains at most of the investigated bridge sites. This wide flood plain produced very large left and right overbank areas at most bridge sites, and there is a significant correlation between large overbank areas and large predicted scour depths based on scour measured during high flows. Again, one way to reduce this effect for bridge design is to make a decision on what is the effective approach section and thereby limit the size of the bridge flow approach width. Limiting this approach width to less than three or four times the bridge opening will reduce the predicted abutment-scour depth and result in better agreement with the measured scour depths for the bridge sites investigated in this study.

SEDIMENT-TRANSPORT SIMULATION FOR THE WHITE RIVER NEAR PRESHO BRIDGE SITE

Sediment-transport simulation using Bri-Stars (Bridge Stream Tube Model for Alluvial River Simulation) (Molinas, 1990) was performed during 1994-95 for site 28, the White River near Presho bridge, in order to better understand the sediment and hydraulic processes at the site. The 2-year flow event (9,860 ft^3/s) and the flow event just below road overtopping (28,500 ft^3/s) were simulated with the Bri-Stars model. The flows for the 100- and 500-year events could not be simulated because of Bri-Stars model limitations relating to road overflow. This site also was modeled because it posed special analytical problems due to the bridge's location on the extreme northern edge of the flood plain and the large potential for sediment load in the river.

Description and Limitations of the Transport Simulation

The Bri-Stars model uses the concept of stream tubes to simulate streambed variations (Molinas, 1990). The stream tubes allow lateral and longitudinal variation of hydraulic and sediment conditions. Although the program can also simulate streambed widening (minimization component), this option was not used. The program is semi two-dimensional. The bed in each stream tube is allowed to degrade or aggrade depending on flow conditions. Backwater computations are used to describe flow conditions in the stream. Sediment routing is performed by satisfying the sediment continuity equation, which is given as:

$$\frac{\partial Q_s}{\partial x} + n\frac{\partial A_d}{\partial t} + \frac{\partial A_s}{\partial t} - q_s = 0$$

where n is the volume of sediment in a unit bed layer volume, or one minus porosity. In the program, n is set equal to a commonly used value of 0.6. A_d is the volume of sediment deposition per unit length (x), A_s is the volume of sediment in suspension at the cross section per unit length (x), Q_s is the volumetric sediment discharge, q_s is the lateral sediment inflow, and t is time.

The streambed is divided into two layers, active and inactive. The active layer is the upper layer of the bed where degradation is simulated. The thickness of this layer must be minimized for the model to produce reasonable results. The inactive layer is located beneath the active layer. Sediment-size distribution in the program is addressed by the use of preselected size groups. The model can address pier and abutment scour, but this option was not used because scour had already been calculated using the published scour equations.

The relative sensitivity of the model is high for: roughness coefficients, sediment inflow, water inflow, cross-section geometry, active-layer thickness, sediment-transport equation, pier-scour equation, time-step duration, and roughness equation (Molinas, 1990). The relative sensitivity of the model is medium or low for: variation of bed elevation, sediment-size distribution, water temperature, coefficient of losses, number of stream tubes, number of time iterations, and stream power-minimization parameters (Molinas, 1990).

Transport-Simulation Input

Selected input data to the Bri-Stars model included channel-geometry data upstream and downstream of the point of interest, channel-roughness and loss-coefficient data, water-surface profiles, and sediment data. A summary of selected model input data for the White River near Presho bridge site is presented in table 13.

Ten cross sections were used in the model. The three farthest upstream and two farthest downstream cross sections were interpolated using USGS 1:24,000 7.5-minute quadrangle maps and the other five cross sections were surveyed. The extreme upstream cross sections were necessary to allow the simulated sediment load to stabilize and reach equilibrium before reaching the cross sections near the bridge site. Thus, minimal initial sediment load data were necessary for the model runs. The extreme downstream cross sections were necessary to allow the water-surface profile to converge before reaching the cross sections near the bridge site. The cross sections used in the model included the bridge approach (station 8070), bridge (station 7720), and the exit (station 7320) sections used in previous WSPRO runs. The cross sections were spaced as evenly as possible, with the stationing from downstream to upstream, even though the simulation progressed from upstream to downstream. Only the most definitive points of the surveyed cross sections were used, due to the limitation of 27 points per model cross section. Upstream cross sections were added to the model input to allow the sediment load of the model to stabilize and attain equilibrium before reaching the cross sections near the bridge. Some initial sediment load was also input to avoid extreme degradation in the upstream cross sections.

The hydraulic head losses through the bridge were modeled using coefficient of losses at the bridge cross section. These losses were calibrated using WSPRO analysis, even though the model has an option for an independent WSPRO run.

As previously discussed, the potential active-layer thickness is the upper layer of the stream bed where degradation is simulated. As recommended by Molinas (1990), this active-layer thickness per time-step interval was kept equal to or less than 0.25 ft per 10-minute time-step interval and preferably at about 0.10 ft per 10-minute time step. If this thickness is computed by the program, very unreasonable results can be anticipated.

Table 13. Summary of transport-simulation input data for the White River near Presho bridge site

[ft., feet; mm., millimeters; ft^3/s, cubic feet per second; appr., bridge approach section; bridg., bridge section; exit, bridge exit section]

Cross-section number	Station	Thalweg elevation (ft)	Cross-section width (ft)	Manning "n" range	Sediment-size fractions 0.0625 - 0.5 (mm)	0.5 - 1.0 (mm)	1.0 - 2.0 (mm)	Surveyed cross section (yes, no)
1	18000	1,576.3	3,400	0.04	0.83	0.13	0.04	no
2	15500	1,574.3	3,400	0.04	.83	.13	.04	no
3	12940	1,572.3	3,350	0.035 - 0.05	.83	.13	.04	no
4	8560	1,568.4	1,300	0.035 - 0.05	.83	.13	.04	yes
5	8070	1,568.3	1,075	0.035 - 0.055	.83	.13	.04	yes (appr.)
6	[1]7720	1,566.4	431	0.035 - 0.045	.83	.13	.04	yes (bridg.)
7	7320	1,568.0	1,490	0.035 - 0.05	.83	.13	.04	yes (exit)
8	6360	1,566.6	2,640	0.038 - 0.065	.83	.13	.04	yes
9	2760	1,563.9	3,460	0.038 - 0.065	.83	.13	.04	no
10	1000	1,563.0	3,890	0.038 - 0.065	.83	.13	.04	no

[1]Bridge section.

Discharge (ft^3/s)	Number of time steps	Length of time steps (days)	Number of stream tubes	Initial sediment load (tons)	Maximum active layer (ft)	Sediment equation
9,860	48	0.0069	3	1,000	0.16	Ackers and White
28,500	48	.0069	3	10,000	.16	Ackers and White

Three stream tubes were used, even though two stream tubes are usually sufficient for reasonable results. If the number of stream tubes is set high, the model takes much more time to run and may experience convergence problems. The number of time steps was set at 48, with a time-step duration of 10 minutes, resulting in an 8-hour simulation. Again, these values were recommended by Molinas (1990).

The model reached equilibrium faster with an initial input load at the upstream cross section. The Ackers and White sediment equation (Molinas, 1990) was used, which is more of a fine-grained sediment equation than the other Bri-Stars options. The sediment ranges used included 0.0625 to 0.5 mm (millimeters), 0.5 to 1.0 mm, and 1.0 to 2.0 mm. These ranges were determined by analyzing the standard sieve results from a sample collected near the bridge site. The sieve analysis was performed at the USGS sediment laboratory in Iowa City, Iowa.

Transport-Simulation Results

The 2-year flow event (9,860 ft^3/s) and the flow event just below road overtopping (28,500 ft^3/s) were simulated. The model was calibrated by keeping the active-layer thickness to a minimum, applying the WSPRO results to the water-surface profiles computed by the model, comparing cross-section changes to cross sections collected during high-flow events (disregarding abutment and pier-scour effects, because they were not included in the model), and evaluating the reasonableness of the final results. The results for stations 8560, 8070 (bridge approach section), 7720 (bridge section), 7320 (bridge exit section), and 6360 for these two flow events are summarized in table 14, which includes the results for time steps 1, 12, 24, 36, and 48. Information summarized for each station and selected time step include water-surface elevation, average velocity, thalweg elevation (lowest point in the cross section), total sediment load, and sediment load per stream tube.

Table 14. Summary of transport-simulation results for the White River near Presho bridge site

[ft^3/s, cubic feet per second; ft/s., feet per second; ft, feet; appr., bridge approach; bridg., bridge; exit, bridge exit]

Discharge (ft^3/s)	Cross-section number	Station	Time step number	Water-surface elevation	Average velocity (ft/s)	Thalweg elevation (ft)	Total sediment load (tons)	Sediment load per stream tube (tons) No. 1	No. 2	No. 3
9,860	4	8560	1	1,578.81	4.54	1,568.4	1,923.1	270.7	363.3	1,289.1
			12	1,578.94	4.11	1,568.3	660.8	150.8	132.8	377.2
			24	1,579.02	3.92	1,568.2	452.8	136.4	95.0	221.4
			36	1,579.06	3.81	1,568.2	380.3	130.5	89.1	160.7
			48	1,579.11	3.72	1,568.1	391.4	126.3	135.5	129.6
9,860	5	8070 (appr.)	1	1,578.47	4.13	1,568.3	623.2	254.1	279.8	89.3
			12	1,578.53	4.73	1,568.5	446.1	151.8	99.6	194.7
			24	1,578.58	4.80	1,568.7	433.2	138.1	99.7	195.4
			36	1,578.62	4.76	1,568.8	395.8	132.7	94.8	168.3
			48	1,578.65	4.71	1,568.9	372.5	129.6	88.0	154.9
9,860	6	7720 (bridg.)	1	1,578.08	4.94	1,566.4	225.5	63.4	85.2	76.9
			12	1,578.06	5.30	1,566.5	267.1	36.5	104.2	126.4
			24	1,578.05	5.50	1,566.6	292.9	39.6	117.8	135.5
			36	1,578.04	5.64	1,566.8	217.1	41.2	50.9	125.0
			48	1,578.03	5.75	1,566.8	268.6	44.0	100.0	124.6
9,860	7	7320 (exit)	1	1,577.94	3.96	1,568.0	255.6	79.9	96.8	78.9
			12	1,577.92	3.97	1,568.0	258.7	79.7	89.2	89.8
			24	1,577.91	3.98	1,568.1	218.3	83.5	45.1	89.7
			36	1,577.90	3.99	1,568.1	246.1	83.8	77.7	84.6
			48	1,577.89	4.01	1,568.1	214.3	87.5	45.1	81.7
9,860	8	6360	1	1,577.48	2.41	1,566.6	158.0	82.4	66.1	9.5
			12	1,577.45	2.44	1,566.6	165.2	75.4	78.6	11.2
			24	1,577.43	2.47	1,566.6	159.1	70.8	75.9	12.4
			36	1,577.41	2.49	1,566.6	158.5	72.0	73.3	13.2
			48	1,577.39	2.52	1,566.6	159.0	73.5	71.7	13.8
28,500	4	8560	1	1,584.47	3.09	1,568.4	791.9	490.8	207.2	93.9
			12	1,584.28	3.40	1,569.5	1,449.1	900.1	364.8	184.2
			24	1,584.23	3.57	1,569.6	1,644.3	1,035.2	373.3	235.8
			36	1,584.23	3.64	1,569.6	1,656.7	1,047.9	358.1	250.7
			48	1,584.26	3.68	1,569.7	1,618.4	1,010.5	351.6	256.3
28,500	5	8070 (appr.)	1	1,584.14	3.56	1,568.3	984.5	518.3	385.2	81.0
			12	1,583.93	3.65	1,568.2	1,207.2	713.7	389.8	103.7
			24	1,583.83	3.75	1,568.2	1,375.2	850.7	392.7	131.8
			36	1,583.80	3.82	1,568.2	1,459.0	942.6	367.1	149.3
			48	1,583.81	3.85	1,568.1	1,471.2	959.0	355.4	156.8

Table 14. Summary of transport-simulation results for the White River near Presho bridge site—Continued

Discharge (ft^3/s)	Cross-section number	Station	Time step number	Water-surface elevation	Average velocity (ft/s)	Thalweg elevation (ft)	Total sediment load (tons)	Sediment load per stream tube (tons)		
								No. 1	No. 2	No. 3
28,500	6	7720 (bridg.)	1	1,582.85	7.36	1,566.4	1,693.3	656.7	505.5	531.1
			12	1,582.79	7.06	1,566.0	1,650.8	807.3	491.7	351.8
			24	1,582.80	6.84	1,565.7	1,668.6	865.8	486.1	316.7
			36	1,582.82	6.73	1,565.4	1,622.8	855.8	452.6	314.4
			48	1,582.83	6.74	1,565.2	1,587.5	825.8	437.7	324.0
28,500	7	7320 (exit)	1	1,582.89	4.27	1,568.0	1,488.1	592.5	753.5	142.1
			12	1,582.77	4.40	1,568.1	1,346.8	689.8	489.4	167.6
			24	1,582.72	4.49	1,568.2	1,448.9	784.3	488.1	176.5
			36	1,582.73	4.53	1,568.2	1,492.0	852.8	454.0	185.2
			48	1,582.73	4.57	1,568.2	1,479.3	848.6	437.6	193.1
28,500	8	6360	1	1,582.76	1.99	1,566.6	94.9	14.5	54.2	26.2
			12	1,582.62	2.06	1,566.6	126.2	19.6	73.5	33.1
			24	1,582.57	2.11	1,566.7	144.9	23.6	82.7	38.6
			36	1,582.56	2.14	1,566.8	157.3	26.4	88.2	42.7
			48	1,582.56	2.17	1,566.8	169.1	28.9	93.6	46.6

The results for the 9,860-ft^3/s run show aggradation at and near the bridge site (fig. 16). The aggradation at the bridge section is 1.5 ft across much of the flood plain and about 0.5 ft in the channel. These results do not show the localized effects of pier and abutment scour at the bridge section that could negate this small amount of aggradation. The approach section has aggradation of about 0.5 ft across the entire section. The aggradation is much less downstream of the bridge section. Cross-section and thalweg water-surface profile plots are shown in figure 16. The total sediment loads at the end of the simulation for stations 8560, 8070, 7720, 7320, and 6360 range from 159 to 391 tons. By time step 24, the total sediment loads are fairly stable. For example, at station 8070, the total sediment loads range from 433 to 372 tons from time step 24 to time step 48.

The results for the 28,500-ft^3/s run show degradation at and upstream of the bridge section (fig. 17). The degradation at the bridge section is about 1.5 to 2.0 ft for most of the section. Again, these cross-section results do not show the localized effects of pier and abutment scour that could increase the degradation effect. The approach section has aggradation of about 2.0 ft across the left side of the channel section and degradation of less than 0.5 ft on the extreme right side of the channel section. The flood plain of the approach section has little change. The thalweg profiles show degradation at and some distance upstream of the bridge section, with aggradation and degradation downstream of the bridge section (fig. 17). The total sediment loads for stations 8560, 8070, 7720, and 7320 range from 1,471 to 1,618 tons. The total sediment load at the end of the simulation for station 6360 is only 169 tons. Station 6360 is at a section downstream of the bridge section where the width of the flood plain is about twice the width at station 7320, the exit section of the bridge. This increased width causes a much smaller velocity leading to a much smaller sediment load. By time step 24, the total sediment loads are again quite stable. For example, at station 7320, the total sediment loads range from 1,449 to 1,492 tons from time step 24 to time step 48.

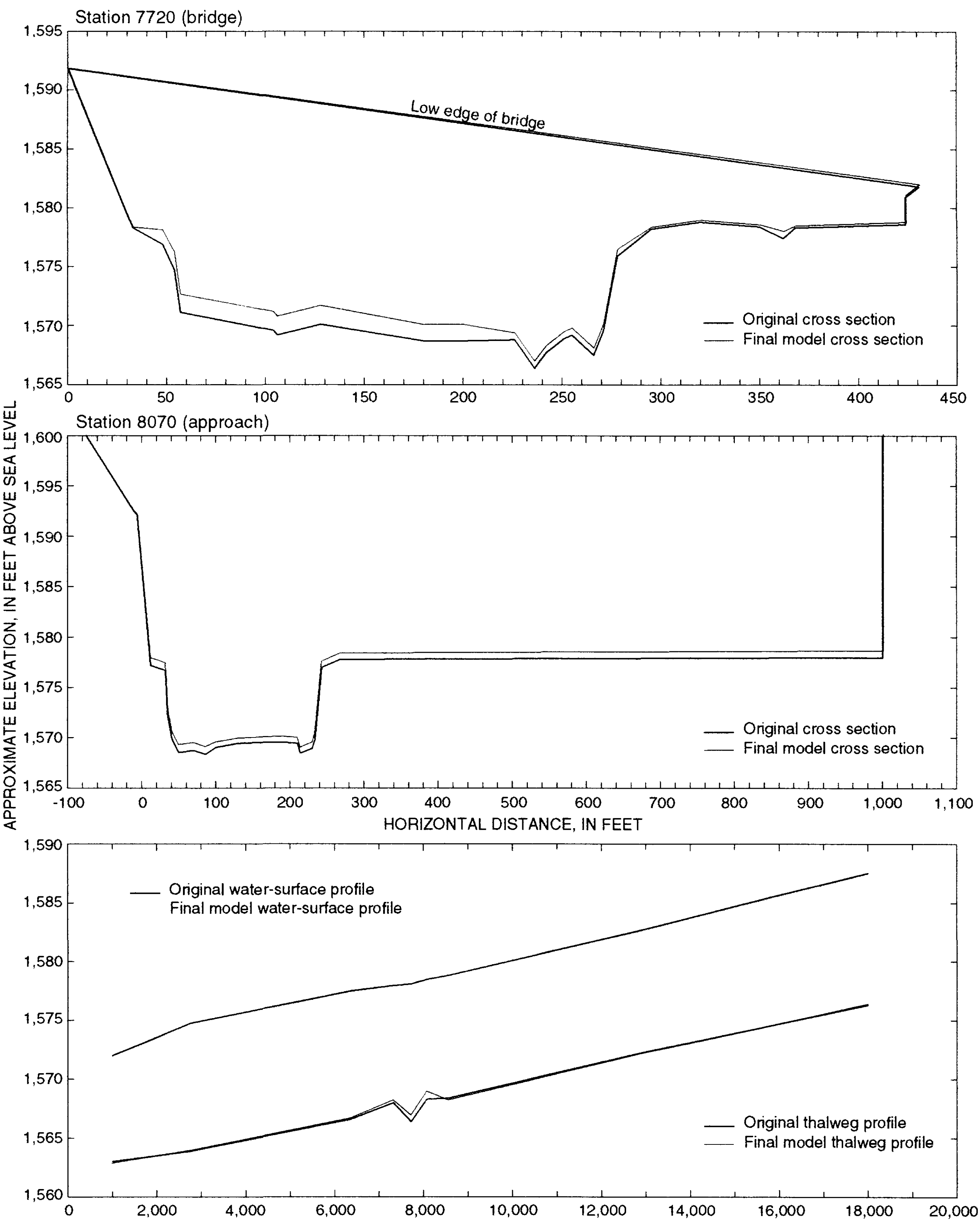


Figure 16. Simulated cross sections and water-surface and thalweg profiles from a discharge of 9,860 ft^3/s for the White River near Presho bridge site.

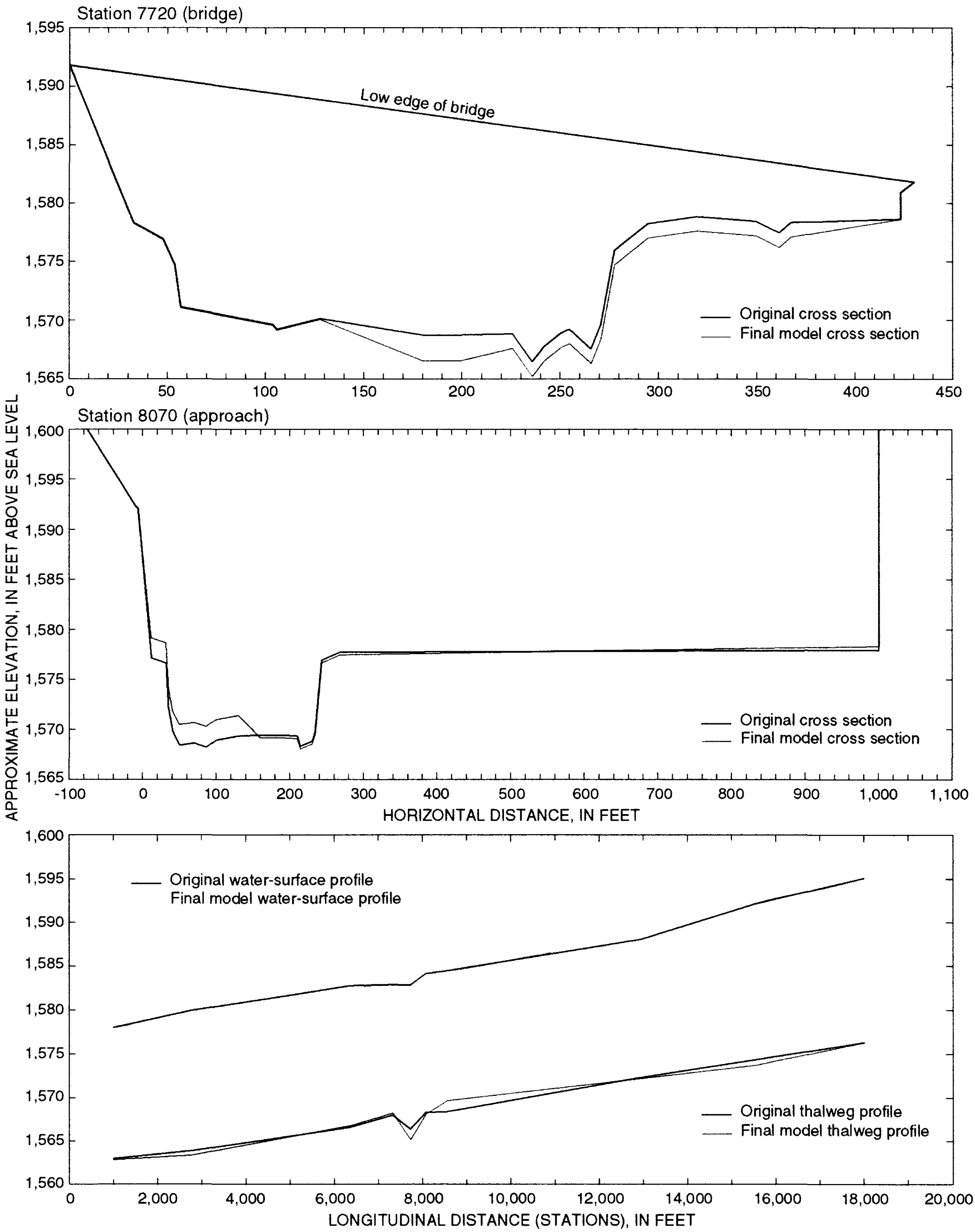


Figure 17. Simulated cross sections and water-surface and thalweg profiles from a discharge of 28,500 ft^3/s for the White River near Presho bridge site.

Sensitivity Analyses

A series of model runs was performed to analyze the sensitivity of the transport simulation to various input parameters. Because some of the input parameters have been interpolated or are based on judgement and experience, it is important to evaluate the effects of the variation of input parameter values on the results of the simulations. The parameters evaluated include active-layer thickness, sediment inflow, sediment equation, number of stream tubes, roughness coefficient, number of time steps, and sediment size. A summary of the thalweg and water-surface elevations and total sediment load for these model-sensitivity runs for stations 8070 and 7720 are presented in table 15. Plots of the thalweg and water surface profiles for selected parameter variations are presented in figures 18 and 19.

The sensitivity results for the 9,860-ft^3/s simulation show that the active-layer thickness, sediment equation, roughness coefficient, and sediment size are the most sensitive parameters. The sediment inflow and the number of stream tubes and time steps are the least sensitive parameters.

When the potential active-layer thickness factor was increased from 3 to 25, the total simulated sediment load rose about 7,000 tons at station 8070 (bridge approach section) and about 7,300 tons at station 7720 (bridge section). A comparison of the corresponding thalweg profiles indicated much more degradation using the higher thickness factor, especially downstream of the bridge section. When the sediment equation was changed to the Yang method (Molinas, 1990), the total sediment load didn't change much at stations 8070 and 7720, but a comparison of the corresponding thalweg profiles indicated much more degradation upstream of the bridge section. When the sediment equation was changed to the Englund and Hansen method (Molinas, 1990), the total sediment load didn't change much at station 8070, but at station 7720 the total sediment load increased about 200 tons. A comparison of the corresponding thalweg and water-surface profiles from the original section indicated increased degradation upstream and downstream of the bridge section and a large change in the water-surface profile upstream of the bridge section. When the roughness coefficients were increased by a factor of 2, the total sediment load decreased by about 300 tons at station 8070 and decreased by about 200 tons at station 7720. When the sediment sizes were increased by a factor of 5, the total sediment load decreased about 300 tons at station 8070 and decreased over 200 tons at station 7720. When the sediment sizes were decreased by a factor of one-fifth, the total sediment load decreased almost to zero at stations 8070 and 7720.

When the number of stream tubes was increased from 3 to 5, the total simulated sediment load remained fairly constant at stations 8070 and 7720. When the number of stream tubes was decreased from 3 to 1, the total sediment load increased about 50 tons at station 8070 and increased about 25 tons at station 7720. A comparison of the corresponding thalweg profiles indicated very little change. Analyses of varying the number of time steps and sediment inflow show similar results.

The sensitivity results for the 28,500-ft^3/s simulation show that the active-layer thickness, sediment equation, number of stream tubes, roughness coefficient, and sediment size are the most sensitive parameters. The sediment inflow and number of time steps are the least sensitive parameters.

When the potential active-layer thickness factor was decreased from 14 to 1, the total simulated sediment load decreased about 1,200 tons at stations 8070 and 7720. A comparison of the corresponding thalweg profiles indicated much less degradation on the upstream end using the lower thickness factor. When the sediment equation was changed to the Yang method, the total sediment load decreased about 300 tons at station 8070 and about 600 tons at station 7720. A comparison of the corresponding thalweg profiles indicated more aggradation immediately upstream of the bridge section and more degradation on the upstream end. When the sediment equation was changed to the Englund and Hansen method, the total sediment load increased about 700 tons at station 8070 and about 900 tons at station 7720. A comparison of the corresponding thalweg profiles indicated less degradation at the bridge section. When the roughness coefficients were increased by a factor of 2, the total sediment load decreased about 850 tons at station 8070 and about 350 tons at station 7720. When the roughness coefficients were decreased by a factor of one-half, the total sediment load increased about 2,400 tons at station 8070 and about 1,700 tons at station 7720. When the sediment sizes were increased by a factor of 5, the total sediment load

Table 15. Summary of transport-simulation sensitivity for the White River near Presho bridge site

[ft^3/s, cubic feet per second; ft, feet; =, equals]

Discharge (ft^3/s)	Station number	Station	Water-surface elevation (ft)	Thalweg elevation (ft)	Total sediment load (tons)	Change
9,860	5	8070	1,578.65	1,568.9	372.5	None.
			1,578.92	1,568.8	7,295.0	Potential active-layer thickness factor increased from 3 to 25.
			1,578.60	1,568.7	270.8	Potential active-layer thickness factor decreased from 3 to 1.
			1,578.65	1,568.9	262.7	Sediment inflow increased by a factor of 5.
			1,578.65	1,568.9	374.1	Sediment inflow decreased from 1,000 to 0 tons.
			1,579.19	1,568.7	429.0	Sediment equation changed to Yang method.
			1,580.44	1,569.2	436.7	Sediment equation changed to Englund and Hansen method.
			1,578.74	1,568.9	332.1	Number of stream tubes increased from 3 to 5.
			1,578.81	1,568.8	422.8	Number of stream tubes decreased from 3 to 1.
			1,581.89	1,568.4	87.4	Roughness coefficients increased by a factor of 2.
			1,575.77	1,570.0	372.2	Roughness coefficients decreased by a factor of one-half.
			1,578.71	1,568.8	347.5	Number of time steps increased from 48 to 96.
			1,578.58	1,568.7	433.2	Number of time steps decreased from 48 to 24.
			1,578.49	1,568.5	59.6	Sediment sizes increased by a factor of 5.
			1,579.47	1,568.3	0.3	Sediment sizes decreased by a factor of one-fifth.
9,860	6	7720	1,578.03	1,566.8	268.6	None.
			1,578.06	1,565.8	7,575.0	Potential active-layer thickness factor increased from 3 to 25.
			1,578.04	1,566.8	58.0	Potential active-layer thickness factor decreased from 3 to 1.
			1,578.03	1,566.8	262.9	Sediment inflow increased by a factor of 5.
			1,578.03	1,566.8	270.0	Sediment inflow decreased from 1,000 to 0 tons.
			1,577.97	1,567.2	269.1	Sediment equation changed to Yang method.
			1,577.90	1,567.7	474.5	Sediment equation changed to Englund and Hansen method.
			1,578.02	1,567.0	259.0	Number of stream tubes increased from 3 to 5.
			1,578.06	1,567.8	295.3	Number of stream tubes decreased from 3 to 1.
			1,581.50	1,566.3	97.1	Roughness coefficients increased by a factor of 2.
			1,575.19	1,566.9	349.0	Roughness coefficients decreased by a factor of one-half.
			1,577.99	1,566.9	288.4	Number of time steps increased from 48 to 96.
			1,578.05	1,566.6	292.9	Number of time steps decreased from 48 to 24.
			1,578.10	1,566.5	37.1	Sediment sizes increased by a factor of 5.
			1,578.08	1,566.4	0.3	Sediment sizes decreased by a factor of one-fifth.

Table 15. Summary of transport-simulation sensitivity for the White River near Presho bridge site—Continued

Discharge (ft^3/s)	Station number	Station	Water-surface elevation (ft)	Thalweg elevation (ft)	Total sediment load (tons)	Change
28,500	5	8070	1,583.81	1,568.1	1,471.2	None.
			1,583.80	1,568.2	1,770.7	Potential active-layer thickness factor increased from 14 to 25 (time step = 34).
			1,584.05	1,568.2	332.1	Potential active-layer thickness factor decreased from 14 to 1.
			1,583.81	1,568.1	1,475.1	Sediment inflow increased by a factor of 5.
			1,583.81	1,568.1	1,471.7	Sediment inflow decreased from 10,000 to 0 tons.
			1,583.82	1,568.9	1,155.1	Sediment equation changed to Yang method.
			1,583.86	1,568.2	2,207.5	Sediment equation changed to Englund and Hansen method (time step = 10).
			1,583.68	1,567.8	1,460.5	Number of stream tubes increased from 3 to 5.
			1,583.79	1,568.6	961.5	Number of stream tubes decreased from 3 to 1.
			1,587.27	1,568.2	629.4	Roughness coefficients increased by a factor of 2 (time step = 2).
			1,578.85	1,567.1	3,831.8	Roughness coefficients decreased by a factor of one-half.
			1,583.81	1,568.1	1,470.5	Number of time steps increased from 48 to 96 (time step = 49).
			1,583.83	1,568.2	1,375.2	Number of time steps decreased from 48 to 24.
			1,584.00	1,568.2	52.6	Sediment sizes increased by a factor of 5.
			1,584.14	1,568.3	3.0	Sediment sizes decreased by a factor of one-fifth.
28,500	6	7720	1,582.83	1,565.2	1,587.5	None.
			1,582.76	1,565.4	1,856.4	Potential active-layer thickness factor increased from 14 to 25 (time step = 34).
			1,582.81	1,566.2	392.8	Potential active-layer thickness factor decreased from 14 to 1.
			1,582.83	1,565.2	1,557.0	Sediment inflow increased by a factor of 5.
			1,582.83	1,565.2	1,584.6	Sediment inflow decreased from 10,000 to 0 tons.
			1,582.63	1,565.6	1,011.6	Sediment equation changed to Yang method.
			1,582.65	1,566.0	2,528.3	Sediment equation changed to Englund and Hansen method (time step = 10).
			1,582.68	1,564.4	1,446.9	Number of stream tubes increased from 3 to 5.
			1,582.96	1,565.0	1,147.2	Number of stream tubes decreased from 3 to 1.
			1,596.29	1,566.3	1,237.6	Roughness coefficients increased by a factor of 2 (time step = 2).
			1,578.87	1,563.5	3,272.2	Roughness coefficients decreased by a factor of one-half.
			1,582.83	1,565.1	1,581.5	Number of time steps increased from 48 to 96 (time step = 49).
			1,582.80	1,565.7	1,565.7	Number of time steps decreased from 48 to 24.
			1,582.90	1,566.1	156.2	Sediment sizes increased by a factor of 5.
			1,582.85	1,566.4	1.7	Sediment sizes decreased by a factor of one-fifth.

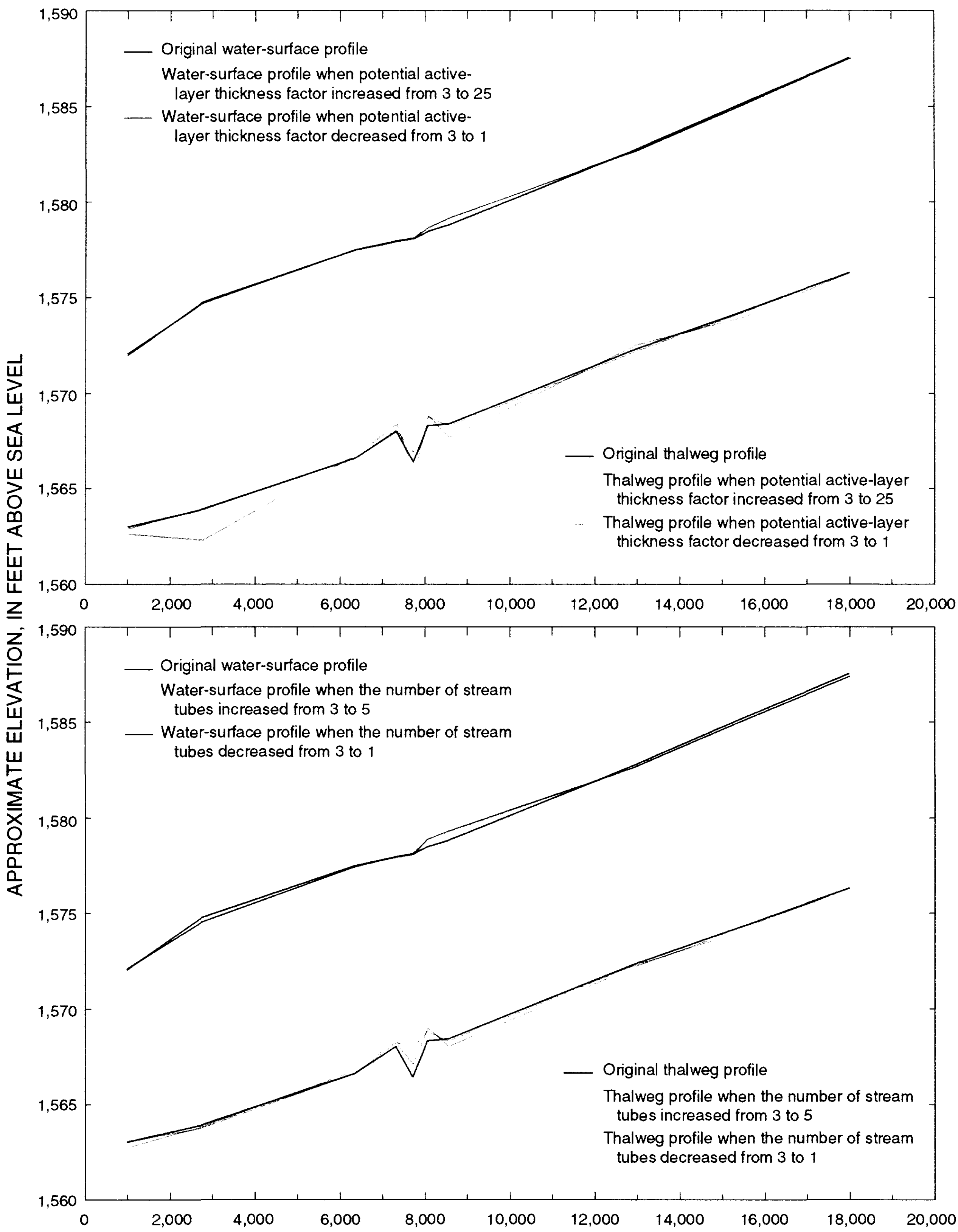


Figure 18. Bri-Stars model sensitivity comparisons of water-surface and thalweg profiles for a discharge of 9,860 ft^3/s at the White River near Presho bridge site.

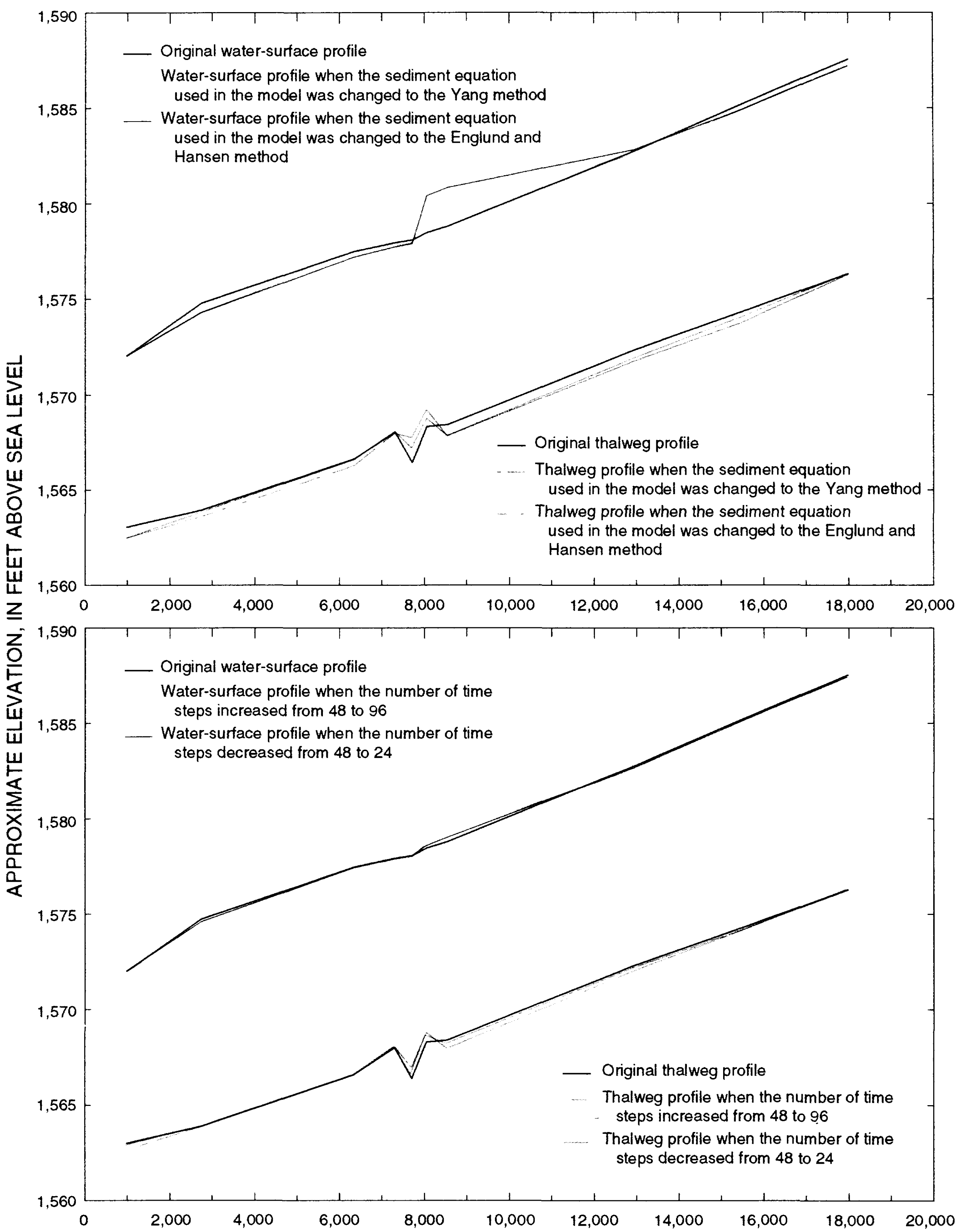


Figure 18. Bri-Stars model sensitivity comparisons of water-surface and thalweg profiles for a discharge of 9,860 ft^3/s at the White River near Presho bridge site.--Continued

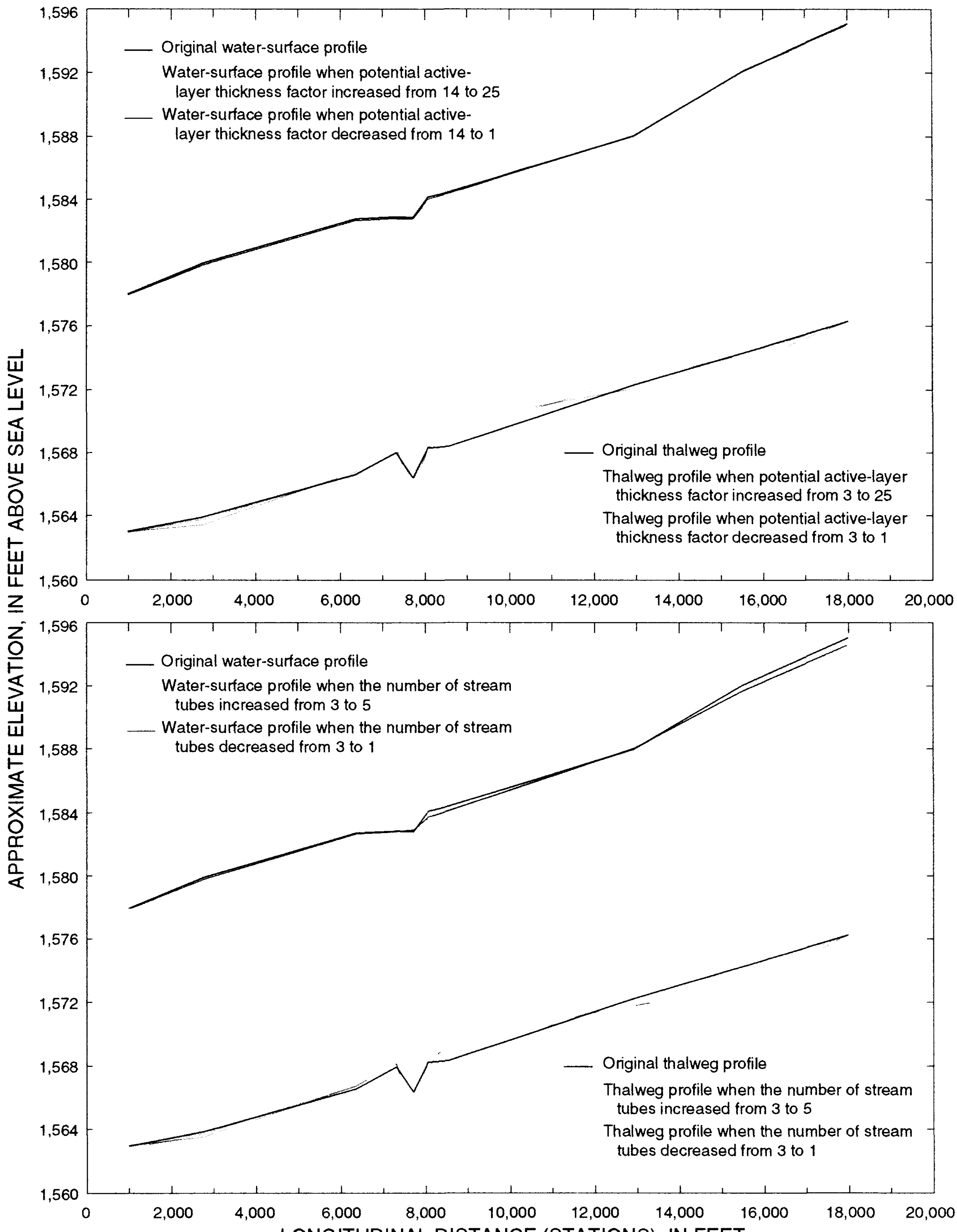


Figure 19. Bri-Stars model sensitivity comparisons of water-surface and thalweg profiles for a discharge of 28,500 ft^3/s at the White River near Presho bridge site.

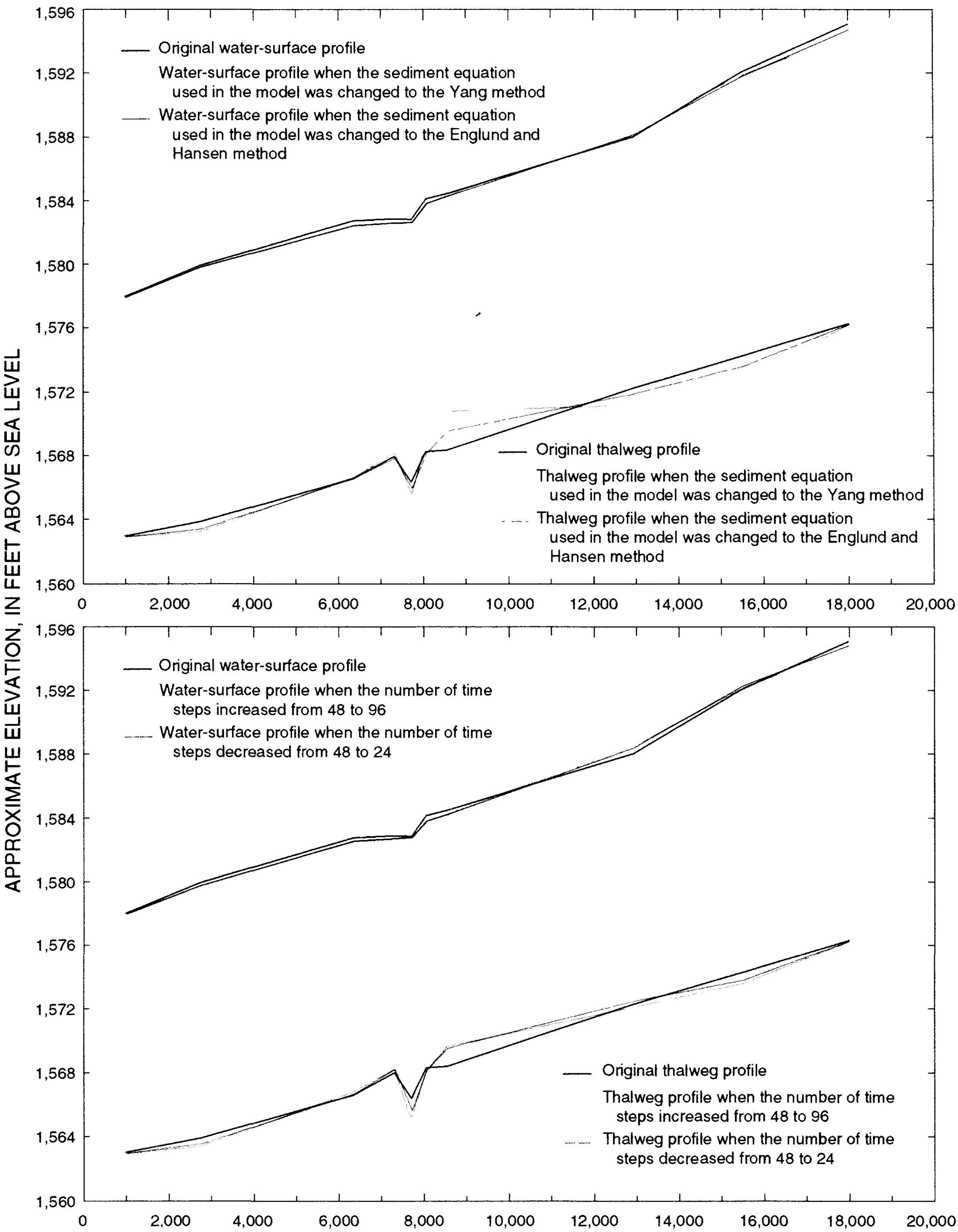


Figure 19. Bri-Stars model sensitivity comparisons of water-surface and thalweg profiles for a discharge of 28,500 ft^3/s at the White River near Presho bridge site.--Continued

decreased about 1,400 tons at stations 8070 and 7720. When the sediment sizes were decreased by a factor of one-fifth, the total sediment load decreased almost to zero at stations 8070 and 7720. When the number of stream tubes was decreased from 3 to 1, the total sediment load decreased about 500 tons at stations 8070 and 7720. A comparison of the corresponding thalweg profiles indicated more degradation on the upstream end.

When the number of time steps was decreased from 48 to 24, the total simulated sediment load remained relatively stable at stations 8070 and 7720. A comparison of the corresponding thalweg profiles indicated little change also. Although the model results indicate that sediment load and water-surface elevations changed substantially with changes in active-layer thickness, sediment equation, number of stream tubes, roughness coefficient, and sediment size, the thalweg elevation of the bridge section showed little change. The ability of the model to predict scour depth at bridges thus appears to be insensitive to changes in model parameters. Analyses of varying the sediment inflow show similar results.

SUMMARY

Scour at bridges is a major concern in the design of new bridges and in the evaluation of structural stability of existing bridges. Equations for estimating pier, contraction, and abutment scour have been developed from numerous laboratory studies using sand-bed flumes, but little verification of these scour equations has been done for actual rivers with various bed conditions. Scour assessments and a sediment-transport simulation were performed for selected bridge sites in South Dakota. This included reconnaissance scour assessments for 32 bridge sites; detailed scour assessments for 13 of the 32 bridge sites, including comparison of predicted and measured scour; and sediment-transport simulation for one bridge site.

The reconnaissance scour assessments were done during 1991 and included compilation of available data (pertinent to scour) for each of the 32 bridge sites and field visits to each site to inspect, measure, and record variables thought to be important to bridge scour. The lengths of the bridges range from 42 to 556 ft. The most common abutment type was a spill-through abutment. The flood-plain widths at the bridge sites range from 150 to 5,000 ft, with grass being the predominant cover. Silt and clay are the most common bed materials of the streams. Many of the bridge sites had large potential for debris accumulation. Cross sections were defined at the bridge sites to aid in the determination of the depth of scour at the sites. Observed pier scour at the sites ranged from 0 to 7 ft, observed contraction scour ranged from 0 to 4 ft, and observed abutment scour ranged from 0 to 10 ft.

The 32 selected bridge sites were evaluated for scour susceptibility using a checklist used in New York for bridge-site selections, an observed-scour index form used in Tennessee, and a potential-scour index form used in Tennessee. The checklist was used to take into account bridge-site parameters that were ideal for bridge-scour measurements. The observed-scour index form rated sites as having high scour potential if they had observable scour, had riprap that had been displaced by high flows, and had definite erosion of banks at the sites. The potential-scour index form rated sites high that had a high potential for scour from future high flows.

Thirteen bridge sites having high scour potential were selected for detailed scour assessments, which were accomplished during 1992-95. These detailed assessments included calculation of scour depths for the theoretical 2-, 100-, and 500-year recurrence flows using selected published scour equations; scour measurements during high flows; comparison of measured and predicted scour; and identification of which scour equations best predict actual scour.

The medians of predicted pier-scour depth using the scour equations range from 2.4 to 6.8 ft for the 2-year flows, 4.4 to 10.9 ft for the 100-year flows, and from 3.4 to 13.3 ft for the 500-year flows. The maximum measured pier scour was 2.0 ft at the Grand River near Mobridge site (site 2) and the Split Rock Creek near Brandon sites (sites 26 and 27), 2.2 ft at the Vermillion River near Centerville site (site 7), 2.4 ft at the Vermillion River overflow near Wakonda site (site 6), 3.5 ft at the Moreau River near Faith site (site 17), 6.5 ft at the White River near Presho site (site 28), and 8.5 ft at the Big Sioux River near Flandreau site (site 3). The flows during which scour was measured included near 100-year flows at the Snatch Creek near Springfield (site 15) and the Split Rock Creek near Brandon (sites 26 and 27) sites.

Flow data collected during high flows when scour was measured were used to predict pier-scour depths using 13 published equations. Using plots of predicted pier scour versus measured pier scour, the Laursen (1), Shen (2b), Colorado State University (3), Blench (4a), Inglis-Poona (5b), and Breusers (10) equations closer approximated the measured scour than the other prediction equations. The Wilcoxon signed-ranks test was used to determine if there were statistical differences between the medians of the measured and predicted pier-scour depths. A two-tail test at the 0.05 level of significance indicated that the Shen (2a), Chitale (6), Bata (7), and Carstens (9) equations are the only medians of measured pier-scour depths statistically equal to the median of predicted pier-scour depths.

Spearman rank correlation analyses were done between the measured and predicted pier-scour depths. The strongest relation between the measured and predicted pier-scour depths was for depths estimated using the Laursen (1) and Inglis-Poona (5a) equations, which had Spearman correlation coefficients of 0.60 and 0.57, respectively. However, the Inglis-Poona (5a) equation vastly overestimates pier-scour depths. The Shen (2b), Colorado State University (3), and Blench (4b) equations also have relatively strong relations between measured and predicted pier-scour depths. The Varzeliotis (8) and Carstens (9) equations had weak statistical relations between the measured and predicted pier-scour depths, with Spearman correlation coefficients of 0.03 and 0.17, respectively.

Spearman rank correlation analyses also were done between the predicted pier-scour depths using the published equations. There are significant relations among many of the published equations, with the main exception being the Varzeliotis (8) and Carstens (9) equations where the correlation coefficients are low. The Laursen (1), Shen (2b), Colorado State University (3), and Inglis-Poona (5a) equations exhibit strong relations with most of the other equations.

Only one equation (Laursen) was used to predict contraction-scour depths. The contraction-scour equation substantially overestimated the scour depths in almost all comparisons with the measured scour depths. One reason for the large predicted scour depths as compared to the measured scour depths is the presence of wide flood plains (as wide as 5,000 ft) at most of the investigated bridge sites. One way to reduce this effect for bridge design is to make a decision on what is the effective approach section and thereby limit the size of the bridge flow approach width.

The medians of predicted contraction scour are -0.1, 13.75, and 23.2 ft, for the 2-, 100-, and 500-year flows, respectively. The maximum measured contraction scour was about 1.5 ft at sites 7, 17, and 28, 2.5 ft at site 2, and 3.0 ft at sites 3, 26, 27, and 32.

Five equations were used to predict abutment-scour depths. The abutment-scour equations also substantially overestimated the scour depths in almost all comparisons with the measured scour depths. The Liu and others (live bed) equation predicted abutment-scour depths substantially lower than the other four abutment-scour equations and closer to the actual measured scour depths. However, this equation at times predicted higher scour depths for 2-year flows than it did for 500-year flows, making its use highly questionable. Again, one reason for the large predicted scour depths for the abutment-scour equations, as compared to the measured scour depths, is the presence of wide flood plains at most of the investigated bridge sites. Limiting the bridge approach section would produce more reasonable predicted abutment scour.

The medians of predicted abutment-scour depth range from 8.2 to 16.5 ft for the 2-year flows, 9.1 to 29.2 ft for the 100-year flows, and from 5.7 to 41 ft for the 500-year flows. The maximum measured abutment scour was 1.5 ft at sites 6 and 17, 2.0 ft at sites 26 and 27, 2.7 ft at site 7, 3.0 ft at site 32, and 4.0 ft at sites 3 and 28.

During 1994-95, the Bri-Stars sediment-transport model was run for site 28, the White River near Presho bridge, to better understand the sediment and hydraulic processes at the site. The 2-year flow event (9,860 ft^3/s) and the flow event just below road overtopping (28,500 ft^3/s) were simulated. This site also was modeled because it posed special analytical problems due to the bridge's location on the extreme northern edge of the flood plain and the large potential for sediment load in the river.

The transport-simulation results for the 9,860-ft^3/s run show aggradation at and near the bridge section. The simulated aggradation at the bridge section is 1.5 ft across much of the flood plain and about 0.5 ft in the channel. These results do not show the localized effects of pier and abutment scour that could increase the degradation effect. The approach section has aggradation of about 0.5 ft. across the entire section. The aggradation is much less downstream of the bridge section. The total simulated sediment loads for cross sections located within about 1,400 feet of the bridge (upstream and downstream) range from 159 to 391 tons.

The results for the 28,500-ft^3/s run show degradation at and upstream of the bridge section. The simulated degradation at the bridge section is about 1.5 to 2.0 ft for most of the section. These results do not show the localized effects of pier and abutment scour that could increase the degradation effect. The approach section has aggradation of about 2.0 ft across the left side of the channel section and degradation of less than 0.5 ft on the extreme right side of the channel section. The flood plain of the approach section has little change. The thalweg profiles show degradation at and some distance upstream of the bridge section, with aggradation and degradation downstream of the bridge section. The total simulated sediment loads for cross sections located within about 1,400 feet of the bridge section (upstream and downstream) range from 169 to 1,618 tons, but generally are between 1,471 and 1,618 tons.

A series of model runs was performed to analyze the sensitivity of the transport simulation to various input parameters. The sensitivity results for the 9,860-ft^3/s model run show that the active-layer thickness, sediment equation, roughness coefficient, and sediment size are the most sensitive parameters. The sediment inflow and the number of stream tubes and time steps are the least sensitive parameters. The sensitivity results for the 28,500-ft^3/s model run show that the active-layer thickness, sediment equation, number of stream tubes, roughness coefficient, and sediment size are the most sensitive parameters. The sediment inflow and number of time steps are the least sensitive parameters.

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SUPPLEMENTAL INFORMATION

Additional Scour-Assessment Information for the Bridge Sites

Site 1: Capitol Lake Outlet at Pierre

Site 1, the Capitol Lake Outlet bridge (33-113-123), is located on Capitol Avenue in Pierre in central South Dakota (T. 110 N., R. 79 W., sec. 4). This bridge is a three-span, metal-plate arch bridge, 68 ft in length, built in 1950. No streamflow-gaging station is located on Capitol Lake or near this site. The bridge has stoplogs on the upstream face across the arches that regulate the level of Capitol Lake. The two pier sets (continuous from front to back of bridge) formed from the arches are rectangular in shape and located on spread footings. The bridge opening is classified as a vertical abutment. The site is well protected by riprap and possibly a concrete apron on the upstream edge and riprap on the downstream edge of the bridge. The bed material is predominantly silt and clay, and the flood-plain cover is mainly grass. There is little potential for debris accumulation at the site.

During high and low flows, the stream approach angle is perpendicular to the upstream faces of the bridge pier sets and bridge opening. There is a large contraction of the flow because of the width of the upstream lake (estimated to be 700 ft) during high flows. Downstream, the flood-plain width was estimated to be 200 ft. However, because of the bridge acting as a control on the lake outflow and because of the good erosion protection upstream (concrete apron) and downstream (riprap), little scour was observed or can be anticipated to occur in the future at this site.

Site 2: Grand River near Mobridge

Site 2, the Grand River bridge (16-665-200), is located on US Highway No. 12, 5 mi west of the City of Wakpala in north-central South Dakota (T. 20 N., R. 28 E., sec. 26). This bridge is a 5-span, continuous-composite, steel-girder bridge, 556 ft in length, built in 1960. The bridge has four octagonal pier sets (two piers per set), 48 in. wide, located on piling. The bridge opening is classified as a spill-through abutment with 3:1 slope embankments. The site is well protected by riprap deposited around the entire wraparound embankments. The bed material of the stream is predominantly sand and silt/clay. The banks at the site are in good condition, with a 90-percent cover of grass and weeds. The flood-plain cover is mainly grass, weeds, and various dead trees (killed when Oahe Reservoir inundated the flood plain). There is some potential for debris accumulation at the site, but because of the large bridge opening it most likely would be confined locally at the pier sets.

The low-flow channel is confined on the south side of the bridge opening between the southernmost pier sets. During high flows, the stream approach angle is perpendicular to the upstream faces of the bridge pier sets and bridge opening. There is a substantial contraction of the flow with the flood-plain width estimated to be 4,200 ft upstream and downstream of the bridge. The bridge site at times also can be inundated by backwater from Oahe Reservoir, but at the time of the field assessment it was not. Little or no scour was observed at the site with the exception of about 1 ft of pier scour at some of the pier sets.

The Grand River at Little Eagle streamflow-gaging station (06357800) is located about 14 mi upstream of this site and has a drainage area of 5,370 mi^2. This station has been operated since 1958, with the largest recorded peak discharge being 31,000 ft^3/s on March 23, 1987. The annual mean discharge at this gaging station for the period 1958 through 1993 is 230 ft^3/s.

Site 3: Big Sioux River near Flandreau

Site 3 (51-150-099), over the Big Sioux River, is located on SD Highway No. 13, 0.3 mi north of the City of Flandreau in east-central South Dakota (T. 107 N., R. 48 W., sec. 22). This bridge is a four-span, continuous-composite, steel-girder bridge, 436 ft in length, built in 1964. The bridge has three octagonal pier sets with webs, 36 in. wide, located on piling. The bridge opening is classified as a spill-through abutment with 3:1 slope embankments. The site is well protected by riprap deposited around the entire wraparound embankments. The bed material of the stream is predominantly silt and clay. The banks at the site are in good condition, with a 100-percent cover of grass, weeds, and trees. The flood-plain cover is mainly grass with some trees. There is some potential for debris accumulation at the site (large number of trees upstream of the site), but because of the large bridge opening, it most likely would be confined locally at the pier sets.

There is a dam structure and a small bridge about 1 mi downstream of the site that affect the water level at the site. The dam structure maintains a minimum pool at the upstream bridge site. The small bridge, which is located immediately downstream of the dam, is undersized and very susceptible to debris accumulation during high flows, to the point of substantial blockage of the bridge opening.

During low to moderate flows, the stream approaches at an estimated angle of 45 degrees to the upstream faces of the bridge pier sets and bridge opening. At high flows, this angle is not as large. There is a substantial contraction of the flow with the flood-plain width estimated to be 2,500 ft upstream and downstream of the bridge. Substantial scour was observed at the site, with an estimated 7 ft of pier scour at one of the pier sets and an estimated 2 ft of contraction scour. No abutment scour was observed.

The Big Sioux River near Brookings streamflow-gaging station (06480000) is located about 22 mi upstream of this site and has a drainage area of 3,898 mi^2 (2,419 mi^2 contributing). This station has been operated since 1953, with the largest recorded peak discharge being 33,900 ft^3/s on April 9, 1969. The annual mean discharge at this gaging station for the period 1953 through 1993 is 241 ft^3/s. The Big Sioux River near Dell Rapids streamflow-gaging station (06481000) is located about 18 mi downstream of this site and has a drainage area of 4,483 mi^2 (3,004 mi^2 contributing). This station has been operated since 1948, with the largest recorded peak discharge being 41,300 ft^3/s on April 9, 1969. The annual mean discharge at this gaging station for the period 1948 through 1993 is 350 ft^3/s.

Site 4: Big Sioux River near Flandreau

Site 4 (51-150-082), over the Big Sioux River, is located on SD Highway No. 13, 2 mi north of the City of Flandreau in east-central South Dakota (T. 107 N., R. 48 W., sec. 15). This bridge is a four-span, continuous-composite, steel-girder bridge, 297 ft in length, built in 1964. The Big Sioux River near Brookings streamflow-gaging station (06480000) is located about 27 mi upstream of this site, and the Big Sioux River near Dell Rapids streamflow-gaging station (06481000) is located about 13 mi downstream of this site (gages were discussed previously under site 3). The bridge has three pointed octagonal pier sets with webs, 36 in. wide, located on piling. The bridge opening is classified as a spill-through abutment. The site has some riprap on both the left and right banks under the bridge. The bed material of the stream is predominantly silt and clay. Some erosion was observed on upstream right banks, but the bank is 90 percent covered by grass, weeds, and trees. The flood-plain cover is mainly grass with some trees. There is some potential for debris accumulation at the site (numerous trees on the bank slopes), but because of the large bridge opening, any accumulation probably would be confined locally at the pier sets.

For low flows, there is a control immediately upstream of the bridge. It probably is the remnants of an old bridge site. During low to high flows, the stream has an estimated approach angle of 40 degrees or more to the bridge opening. However, the pier sets and abutments are skewed parallel to the flow to minimize scour. There is a substantial contraction of the flow, as the flood-plain width is estimated to be 2,200 ft upstream and downstream of the bridge. Moderate scour was observed at the site, with an estimated 2.5 ft of pier scour at the central pier sets and an estimated 3 ft of abutment scour at one of the abutments. No contraction scour was observed at this site.

Site 5: Frozen Man Creek near Hayes

Site 5, the Frozen Man Creek bridge (59-078-279), is located on the frontage road north of US Highway No. 14, 0.3 mi east of the City of Hayes in central South Dakota (T. 5 N., R. 26 E., sec. 19). The bridge spans the upper extent of Hayes Lake, which is formed by a dam located about 1 mi downstream. This bridge is a six-span, simple steel-girder bridge, 140 ft in length, built in 1922 and widened in 1933. No streamflow-gaging stations are located near this site. The bridge has two square pier sets with webs and three octagonal pier sets (four piers per set), 20.5 and 15.5 in. wide, respectively. It is unknown whether the pier sets are located on spread footings or piling. The bridge opening is classified as a vertical abutment. The site has no slope protection under the bridge or on the embankments. The bed material of the stream is predominantly silt and clay. The banks at the site are in good condition with a 100-percent cover of grass, weeds, and small trees. The flood-plain cover is mainly grass, weeds, and small trees. There is large potential for debris accumulation at the site because of the large number of trees upstream, because of the large number of pier sets, and because of the relatively small bridge opening.

When full, or nearly full, Hayes Lake maintains a minimum pool at the site. During low to high flows, the stream approaches at an estimated angle of 45 degrees to the upstream faces of the bridge pier sets and bridge opening. There is some contraction of the flow, with the flood-plain width estimated to be 300 ft upstream and downstream of the bridge. Not much scour was observed at the site, with the observed scour estimated to range from 0 to 1.5 ft at the pier sets. No abutment or contraction scour was observed at this site.

Site 6: Vermillion River Overflow near Wakonda

Site 6, the Vermillion River overflow bridge (14-100-062), is located on SD Highway No. 19, 6 mi southeast of the City of Wakonda in southeast South Dakota (T. 94 N., R. 52 W., sec. 2). This bridge is a five-span, continuous-concrete bridge, 88 ft in length, built in 1938. The bridge is a relief bridge for the Vermillion River. The Vermillion River has levees in this area that are very susceptible to breakouts during floods, which then allows flood-plain flow through this relief bridge. The bridge has four square pier sets (four piers per set), 16 in. wide, located on piling. The bridge opening is classified as a vertical abutment. The site has no riprap protection under the bridge or on the embankments. The bed material of the stream is predominantly silt and clay. There is no defined channel upstream of the site. The downstream banks at the site are in good condition with a 100-percent cover of grass and weeds. The flood-plain cover is cropland upstream and grass and weeds downstream. There is large potential for debris accumulation at the site because of the cropland cover upstream, the large number of pier sets, and the small bridge opening at the site.

The only flows anticipated at the site occur when the Vermillion River breaks out of its banks or levees and into the flood plain. Inflow is perpendicular to the bridge opening during these flood-plain flows. At extremely large flows, the water will wash over the road south of the bridge site, reducing the flow through the relief bridge. There is a substantial contraction of the flow, with the flood-plain width estimated to be 2,000 ft upstream and downstream of the bridge. Moderate scour was observed at the site, with an estimated 1 to 1.5 ft of pier scour at the pier sets and an estimated 3 ft of contraction scour. No abutment scour was observed at this site.

The Vermillion River near Wakonda gaging station (06479000) is located at the main channel Vermillion River bridge at the site and has a drainage area of 2,170 mi^2 (1,676 mi^2 contributing). This station was operated from 1945 through 1983 as a continuous-record streamflow-gaging station and has been operated from 1989 through 1993 as a crest-stage partial-record station. The largest recorded peak discharge is 17,000 ft^3/s on June 23, 1984. The annual mean discharge at this gaging station for the period 1945 through 1983 is 125 ft^3/s. The Vermillion River near Vermillion streamflow-gaging station (06479010) is located about 16 mi downstream of this site and has a drainage area of 2,302 mi^2 (1,808 mi^2 contributing). This station has been operated since 1983, with the largest recorded peak discharge being 21,400 ft^3/s on June 23, 1984. The annual mean discharge at this gaging station for the period 1983 through 1993 is 408 ft^3/s.

Site 7: Vermillion River near Centerville

Site 7, the Vermillion River bridge (14-100-019), is located on SD Highway No. 19, 4 mi south of the City of Centerville in southeast South Dakota (T. 95 N., R. 52 W., sec. 10). This bridge is a four-span, simple steel-girder bridge, 146 ft in length, built in 1938. The Vermillion River near Wakonda gaging station (06479000) is located about 6 mi downstream of this site, and the Vermillion River near Vermillion streamflow-gaging station (06479010) is located about 22 mi downstream of this site (gages were discussed previously under site 6). The bridge has three square pier sets (two piers per set), 26 in. wide, located on piling. The bridge opening is classified as a vertical abutment. The site has riprap protection on the upstream banks. The bed material of the stream is predominantly silt and clay. Some erosion was observed in the banks at the bridge, but the bank cover is 90-percent grass and trees. The banks were eroding on the upstream left side and the downstream right side. The flood-plain cover is mainly grass with some trees. There is large potential for debris accumulation at the site because of the large number of trees upstream, because of the large number of pier sets, and because of the relatively small bridge opening.

The low-flow channel includes most of the bridge opening. At low to moderate flows, the stream approaches perpendicular to the upstream faces of the bridge pier sets and bridge opening. At high flows, there may be some flow parallel to the bridge opening causing some skew of the flow. Immediately downstream of the bridge, the channel is controlled by levees. There are 90-degree bends a few hundred feet upstream and downstream of the site. There is a substantial contraction of the flow, with the flood-plain width estimated to be 5,000 ft upstream and downstream of the bridge. Little scour was observed at the site, with an estimated 0 to 1 ft of pier scour observed at the pier sets and an estimated 1 ft of contraction scour. No abutment scour was observed at this site. However, the site has a large potential for scour because of the contracted flow and past history of scour at the site.

Site 8: Vermillion River near Centerville

Site 8, the Vermillion River bridge (14-100-001), is located on SD Highway No. 19, 2 mi south of the City of Centerville in southeast South Dakota (T. 95 N., R. 52 W., sec. 2). This bridge is a four-span, simple steel-girder bridge, 122 ft in length, built in 1926 and widened in 1938. The Vermillion River near Wakonda gaging station (06479000) is located about 9 mi downstream, and the Vermillion River near Vermillion streamflow-gaging station (06479010) is located about 26 mi downstream of this site (gages were discussed previously under site 6). The bridge has three square pier sets (two piers per set), 20 in. wide, located on piling. The bridge opening is classified as a vertical abutment. The site has riprap protection on the left side under the bridge. The bed material of the stream is predominantly silt and clay. Some erosion was observed in the banks on the left upstream and right downstream banks. The bank cover is 100-percent grass, weeds, and trees. The banks were eroding mainly because of the proximity of the bridge to an upstream and downstream 90-degree bend. The flood-plain cover is mainly grass and small trees. There is large potential for debris accumulation at the site because of the large number of trees upstream, large number of pier sets, and relatively small bridge opening.

The low-flow channel includes most of the bridge opening. At low to moderate flows, the stream approaches perpendicular to the upstream faces of the bridge pier sets and bridge opening. At high flows, the stream approaches at an estimated angle of 45 degrees to the bridge opening. Immediately downstream of the bridge, the channel is controlled by levees. There is a high potential for overflow of the road at this site. There are 90-degree bends a few hundred feet upstream and downstream of the site. There is a substantial contraction of the flow with the flood-plain width estimated to be 4,600 ft upstream and downstream of the bridge. Four feet of contraction scour was observed at the site. No pier or abutment scour was observed. The site has a large potential for scour because of the contracted flow and potential skewness of flow.

Site 9: Little Missouri River near Camp Crook

Site 9, the Little Missouri River bridge (32-043-278), is located on SD Highway No. 20, on the east edge of the City of Camp Crook in northwest South Dakota (T. 18 N., R. 1 E., sec. 2). This bridge is a four-span, simple steel-girder bridge, 330 ft in length, built in 1951. The bridge has three octagonal pier sets with webs, one 22 in. and two 39 in. wide, located on piling. The bridge opening is classified as a spill-through abutment. The site has riprap protection on the left side under the bridge. Also, spurs have been constructed on the left upstream bank. The bed material of the stream is predominantly sand and silt/clay. Sloughing was observed in the banks on the left upstream and downstream banks due to the bridge being located on a meander. The bank cover is 80 percent grass and small trees. The flood-plain cover is mainly grass and small trees. There is little potential for debris accumulation at the site because of the large bridge opening.

The low-flow channel is confined to the west side of the bridge opening near the westernmost pier sets. At low to high flows, the stream approaches at an estimated angle of 30 degrees to the upstream faces of the bridge pier sets and bridge opening causing sloughing of the left banks. There is large contraction of the flow with the flood-plain width estimated to be 1,200 ft upstream and downstream of the bridge. Up to 2 ft of pier scour and 5 ft of abutment scour were observed at the site. No contraction scour was observed. The site has a large potential for scour because of the large skewness of flow.

The Little Missouri River near Camp Crook streamflow-gaging station (06334500) is located at the site and has a drainage area of 1,970 mi^2. This station has been operated from 1903 through 1906 and from 1956 through 1993. The largest recorded peak discharge was 9,420 ft^3/s on March 24, 1978. The annual mean discharge at this gaging station for the period of record is 121 ft^3/s.

Sites 10 and 11: East Branch North Deer Creek near Brookings

Sites 10 and 11, the East Branch North Deer Creek bridges (06-185-074 and 06-184-074), are located on Interstate 29 north and south, respectively, 8.5 mi north of the City of Brookings in east-central South Dakota (T. 111 N., R. 49 W., sec. 7). These bridges are five-span, continuous-concrete bridges, 152 ft in length, built in 1970. No streamflow-gaging stations are located on East Branch North Deer Creek or at this site. The bridges both have four round pier sets (three piers per set), 24 in. in diameter, located on piling. The bridge openings are classified as a spill-through abutment. The sites have no riprap protection under the bridges or on the embankments. The bed material of the stream is predominantly silt and clay. The banks at the site are in good condition with a 100-percent cover of grass and weeds. The flood-plain cover is mainly grass. There is little potential for debris accumulation at the sites because of the lack of available debris and because of the relatively large bridge openings.

The low-flow channel includes the central part of the bridge openings. At low to high flows, the stream approaches at an angle to the bridge openings. However, to account for this angle, the two bridges have been offset from each other so that a line connecting the centerline of each bridge is parallel to this angle. Since the pier sets are round, they are still perpendicular to the flows. There is a large contraction of the flow with the flood-plain width estimated to be 1,000 ft upstream and downstream of the bridge. About 4 ft of contraction scour was observed at the sites. No pier or abutment scour was observed.

Sites 12 and 13: Hidewood Creek near Clear Lake

Sites 12 and 13, the Hidewood Creek bridges (20-027-207 and 20-028-207), are located on Interstate 29 north and south, respectively, 5.5 mi southeast of the intersection of I29 and Highway No. 22 near Clear Lake in northeast South Dakota (T. 114 N., R. 50 W., sec. 16). These bridges are three-span, continuous steel-girder bridges, 233 ft in length, built in 1973. No streamflow-gaging stations are located at or near these sites. The bridges both have two round pier sets (three piers per set), 33 in. in diameter, located on piling. The bridge openings are classified as a spill-through abutment. The sites have riprap protection on the 2:1 slope embankments on the left and right sides. The bed material of the stream is predominantly silt and clay. The banks at the site are in good condition with a 100-percent cover of grass and weeds. The flood-plain cover is mainly grass. There is little potential for debris accumulation at the sites because of the lack of available debris and because of the relatively large bridge openings.

The low-flow channel includes the central part of the bridge openings. At low to high flows, the stream approaches perpendicular to the upstream faces of the bridge pier sets and bridge openings. There is a substantial contraction of the flow with the flood-plain width estimated to be 2,200 ft upstream and downstream of the bridges. No pier, contraction, or abutment scour was observed at the sites.

Site 14: North Branch Dry Creek near Parkston

Site 14, the North Branch Dry Creek bridge (34-125-080), is located on SD Highway No. 44, 6.5 mi east of the City of Parkston in southeast South Dakota (T. 99 N., R. 59 W., sec. 18). This bridge is a three-span, continuous-concrete bridge, 86 ft in length, built in 1964. The bridge has two square pier sets (two piers per set), 22 in. wide, located on spread footings. The bridge opening is classified as a spill-through abutment. The site has riprap protection on the left and right slopes under the bridge. The bed material of the stream is predominantly silt and clay. Some erosion of the downstream banks was observed. The bank cover is 100 percent grass/weeds with some trees. The flood-plain cover is mainly grass. There is some potential for debris accumulation and blockage of the opening at the site because of the presence of dead trees upstream and the small bridge opening.

The low-flow channel includes the central part of the bridge opening. At low to high flows, the stream approaches perpendicular to the upstream faces of the bridge pier sets and bridge opening. There is some contraction of the flow, with the flood-plain width estimated to be 250 ft upstream and downstream of the bridge. No pier, contraction, or abutment scour was observed at the site. There is little potential for scour at the bridge due heavy riprap under the bridge.

The Dry Creek near Parkston gaging station (06478300) is located 4 mi downstream of the site and has a drainage area of 97.2 mi^2. This station has been operated from 1955 through 1980 and 1989 through 1993 as a crest-stage partial-record station. The largest recorded peak discharge was 4,210 ft^3/s on March 27, 1960.

Site 15: Snatch Creek near Springfield

Site 15, the Snatch Creek bridge (05-198-180), is located on SD Highway No. 52, 7 mi northeast of the City of Springfield in southeast South Dakota (T. 93 N., R. 59 W., sec. 2). This bridge is a two-span, simple steel-girder bridge, 72 ft in length, built in 1935. No streamflow-gaging stations are located on Snatch Creek or at this site. The bridge has one square pier set (two piers per set), 20 in. wide, located on piling. The bridge opening is classified as a vertical abutment. The site has no riprap protection under the bridge or on the embankments. The bed material of the stream is predominantly silt and clay. Some erosion of the upstream banks was observed. The bank cover is 50 to 100 percent grass and small trees. The flood-plain cover is mainly grass. There is large potential for debris accumulation and blockage of the opening at the site because of the presence of dead trees upstream and the small bridge opening.

The low-flow channel includes the central part of the bridge opening. At low to high flows, the stream approaches perpendicular to the upstream face of the bridge pier set and bridge opening. There is some contraction of the flow with the flood-plain width estimated to be 350 ft upstream and downstream of the bridge. Two feet of contraction scour was observed at the site. No pier or abutment scour was observed. Because of the characteristics of the basin, this stream would tend to be very flashy, with high flows occurring during a short time period.

Site 16: Hump Creek near McIntosh

Site 16, the Hump Creek bridge (16-329-127), is located on SD Highway No. 65, 11 mi south of the City of McIntosh in north-central South Dakota (T. 21 N., R. 22 E., sec. 24). This bridge is a five-span, continuous-concrete bridge, 174 ft in length, built in 1956. No streamflow-gaging stations are located on Hump Creek or at this site. The bridge has four square pier sets (two piers per set), 24 in. wide, located on spread footings. The bridge opening is classified as a spill-through abutment. The site has no riprap protection under the bridge or on the embankments. The bed material of the stream is predominantly sand and silt/clay. Some channel degradation was observed at the upstream and downstream banks. The soil material appears highly erodible and certainly contributes to the degradation observed. The bank cover is 90 percent grass and weeds. The flood-plain cover is mainly grass with some small trees. There is some potential for debris accumulation at the pier sets because of the presence of debris upstream of the bridge opening.

The low-flow channel includes the central part of the bridge opening. At low to high flows, the stream approaches perpendicular to the upstream faces of the bridge pier sets and bridge opening. There is some contraction of the flow, with the flood-plain width estimated to be 500 ft upstream and downstream of the bridge. Three feet of pier scour was observed at the site. No contraction or abutment scour was observed.

Site 17: Moreau River near Faith

Site 17, the Moreau River bridge (53-392-521), is located on SD Highway No. 73, 13.5 mi northwest of the City of Faith in northwest South Dakota (T. 14 N., R. 16 E., sec. 10). This bridge is a seven-span, simple steel-girder bridge, 378 ft in length, built in 1926. The bridge has three square pier sets (two piers per set) and three pointed with web pier sets, 30 and 36 in. wide, respectively, located on spread footings. The bridge opening is classified as a spill-through abutment. The site has no riprap protection under the bridge or on the embankments. The bed material of the stream is predominantly sand and silt/clay. Some bank erosion was observed in the banks on the left upstream and downstream sides due to the upstream and downstream bends. The bank cover is 80 percent grass and weeds. The flood-plain cover is mainly grass. There is some potential for debris accumulation at the pier sets because of the presence of debris upstream of the bridge opening.

The low-flow channel is confined mainly between the second and third northernmost pier sets (third span). During low to high flows, the stream approaches at an estimated angle of 30 degrees or more to the bridge opening. However, the pier sets and abutments have been skewed parallel to the flow to minimize scour. There is some contraction of the flow with the flood-plain width estimated to be 700 ft upstream and downstream of the bridge. Up to 2 ft of pier scour was observed at the site. No contraction or abutment scour was observed.

The Moreau River near Faith streamflow-gaging station (06359500) is located at the site and has a drainage area of 2,660 mi^2. This station has been operated from 1943 through 1993. The largest recorded peak discharge was 26,000 ft^3/s on April 9, 1944. The annual mean discharge at this gaging station for the period of record is 132 ft^3/s.

Site 18: South Fork Grand River near Bison

Site 18, the South Fork Grand River bridge (53-149-209), is located on SD Highway No. 75, 7.9 mi north of the intersection of Highway Nos. 75 and 20 near Bison in northwest South Dakota (T. 20 N., R. 12 E. sec. 33). This bridge is a three-span, continuous steel-girder bridge, 234 ft in length, built in 1966. The bridge has two round pier sets (24 in. in diameter) with webs, located on spread footings. The bridge opening is classified as a spill-through abutment. The site has riprap protection on the 2:1 slope embankments on the left and right sides. The bed material of the stream is predominantly silt and clay. The banks at the site are in good condition with a 100-percent cover of grass and weeds. The flood-plain cover is mainly grass. There is little potential for debris accumulation at the site because of the lack of available debris and the large bridge opening.

The low-flow channel is confined mostly around the northernmost pier set. At low to high flows, the stream approaches perpendicular to the upstream faces of the bridge pier sets and bridge opening. There is a large contraction of the flow with the flood-plain width estimated to be 1,500 ft upstream and downstream of the bridge. An estimated 1.5 to 2 ft of pier scour was observed at the pier sets and an estimated 1 ft of abutment scour. No contraction scour was observed at this site.

The South Fork Grand River near Cash streamflow-gaging station (06356500) is located at the site and has a drainage area of 1,350 mi^2. This station has been operated from 1945 through 1993. The largest recorded peak discharge was 27,000 ft^3/s on April 15, 1950. The annual mean discharge at this gaging station for the period of record is 51.3 ft^3/s.

Site 19: North Fork Grand River near Lodgepole

Site 19, the North Fork Grand River bridge (53-150-046), is located on SD Highway No. 75, 5.5 mi north of the town of Lodgepole in northwest South Dakota (T. 22 N., R. 12 E., sec. 9). This bridge is a five-span, simple and continuous steel-girder bridge, 342 ft in length, built in 1952. The bridge has four pointed pier sets with webs, 36 in. wide, located on spread footings. The bridge opening is classified as a spill-through abutment. The site has riprap protection on the left wraparound embankment. The bed material of the stream is predominantly silt and clay. The banks at the site are in good condition with a 100-percent cover of grass and weeds. The flood-plain cover is mainly grass. There is little potential for debris accumulation at the site because of the lack of available debris and the large bridge opening.

The low-flow channel is confined mostly near the two central pier sets. At low flows, the stream approaches at an estimated 15-degree angle to the upstream faces of the bridge pier sets and bridge opening. At moderate to high flows, the stream approaches almost perpendicular to the upstream faces of the bridge pier sets and bridge opening. There is some contraction of the flow, with the flood-plain width estimated to be 1,000 ft upstream and downstream of the bridge. Up to 4 ft of pier scour was observed at the pier sets. No contraction or abutment scour was observed at this site.

The North Fork Grand River near White Butte streamflow-gaging station (06355500) is located 25 mi downstream of the site and has a drainage area of 1,190 mi^2. This station has been operated from 1945 through 1993. The largest recorded peak discharge was 6,710 ft^3/s on March 28, 1978. The annual mean discharge at this gaging station for the period of record is 41.5 ft^3/s.

Site 20: French Creek near Fairburn

Site 20, the French Creek bridge (17-400-131), is located on SD Highway No. 79, 11.5 mi south of the intersection of Highway Nos. 79 and 36 near Fairburn in southwest South Dakota (T. 4 S., R. 7 E., sec. 26). This bridge is a five-span, continuous-concrete bridge, 117 ft in length, built in 1957. The bridge has four square pier sets (two piers per set), 20 in. wide, located on spread footings. The bridge opening is classified as a spill-through abutment. The site has no riprap protection under the bridge or on the embankments. The bed material of the stream is predominantly sand, silt/clay, and gravel. The banks at the site are in good condition with a 100-percent cover of grass and large trees. The flood-plain cover is mainly grass. There is potential for debris accumulation and possibly blockage of the bridge opening at the site because of the large trees upstream and the relatively small bridge opening.

The low-flow channel is confined mostly near the two central pier sets. At low to high flows, the stream approaches at an angle to the upstream faces of the bridge pier sets and bridge opening because the bridge is located on a meander of French Creek. There is some contraction of the flow, with the flood-plain width estimated to be 250 ft upstream and downstream of the bridge. Up to 3 ft of pier scour was observed at the pier sets and an estimated 2 ft of contraction scour. No abutment scour was observed at this site.

The French Creek above Fairburn streamflow-gaging station (06403300) is located 7 mi upstream of the site and has a drainage area of 105 mi^2. This station has been operated from 1982 through 1993. The largest recorded peak discharge was 329 ft^3/s on March 7, 1987. The annual mean discharge at this gaging station for the period of record is 6.5 ft^3/s.

Site 21: South Fork Grand River near Buffalo

Site 21, the South Fork Grand River bridge (32-517-215), is located on SD Highway No. 79, 6.5 mi north of the intersection of Highway Nos. 79 and 20 near Buffalo in northwest South Dakota (T. 19 N., R. 9 E., sec. 3). This bridge is a seven-span, simple steel-girder bridge, 459 ft in length, built in 1957. The bridge has six octagonal pier sets with webs, 36 in. wide. Five of the pier sets are located on spread footings and one on piling. The bridge opening is classified as a spill-through abutment. The site has some riprap protection around the piers. The site has no riprap protection on the embankments. The bed material of the stream is predominantly sand and silt/clay. Sloughing was observed in the right upstream embankment wraparound. The bank cover is 100 percent grass. The banks were eroding due to skewness of the flow through the bridge. The flood-plain cover is mainly grass. There is little potential for debris accumulation at the site because of the lack of available debris and the large bridge opening.

The low-flow channel is confined to the southern side of the bridge opening. At low to moderate flows, the stream approaches at an estimated 45 degrees to the upstream faces of the bridge pier sets and bridge opening. At high flows, the stream angle to the upstream faces of the bridge pier sets and bridge opening is estimated at 20 degrees. There is large contraction of the flow with the flood-plain width estimated to be 1,200 ft upstream and downstream of the bridge. Up to 4 ft of pier scour and up to 2 ft of abutment scour were observed at the site. No contraction scour was observed. The site has potential for scour because of the large skewness of flow.

The South Fork Grand River near Cash streamflow-gaging station (06356500) is located 22 mi downstream of the site (gage was discussed previously under site 18). The South Fork Grand River at Buffalo streamflow-gaging station (06356000) is located 35 mi upstream of the site and has a drainage area of 148 mi^2. This station has been operated from 1955 through 1993. The largest recorded peak discharge was 2,780 ft^3/s on June 14, 1963. The annual mean discharge at this gaging station for the period of record is 8.53 ft^3/s.

Site 22: Little White River near White River

Site 22, the Little White River bridge (48-250-185), is located on US Highway No. 83, 2 mi north of the City of White River in south-central South Dakota (T. 42 N., R. 29 W., sec. 23). This bridge is a five-span, simple steel-girder bridge, 314 ft in length, built in 1957. The bridge has four octagonal pier sets with webs, 36 in. wide, located on spread footings. The bridge opening is classified as a spill-through abutment. The site has no riprap protection under the bridge or on the embankments. The bed material of the stream is predominantly silt and clay. Sloughing was observed especially in the left upstream banks. The bank cover is grass and trees (25 percent upstream and 100 percent downstream). The flood-plain cover is mainly grass and trees. There is some potential for debris accumulation at the pier sets.

The low-flow channel is confined between the two northernmost pier sets (second span). At low to high flows, the stream approaches perpendicular to the upstream faces of the bridge pier sets and bridge opening. There is some contraction of the flow, with the flood-plain width estimated to be 600 ft upstream and downstream of the bridge. Up to 2 ft of pier scour and up to 2 ft of abutment scour were observed at the site. No contraction scour was observed. The site has limited potential for scour because of the relatively stable shale bottom.

The Little White River below White River streamflow-gaging station (06450500) is located at the site and has a drainage area of 1,570 mi^2 (1,310 mi^2 contributing). This station has been operated from 1949 through 1993. The largest recorded peak discharge was 13,700 ft^3/s on June 12, 1967. The annual mean discharge at this gaging station for the period of record is 128 ft^3/s.

Site 23: Horse Creek near White River

Site 23, the Horse Creek bridge (38-192-284), is located on US Highway No. 83, 12 mi north of the City of White River in south-central South Dakota (T. 3 S.,

R. 29 E., sec. 29). This bridge is a five-span, continuous-concrete bridge, 163 ft in length, built in 1956. No streamflow-gaging stations are located on Horse Creek or at this site. The bridge has four square pier sets (two piers per set), 22 in. wide, located on spread footings. The bridge opening is classified as a spill-through abutment. The site has no riprap protection under the bridge or on the embankments. The bed material of the stream is predominantly silt and clay. The banks at the site are in good condition with a 100-percent cover of grass and weeds. The flood-plain cover is mainly grass and weeds. There is some potential for debris accumulation at the pier sets.

The low-flow channel is confined in the center of the bridge opening. During low to high flows, the stream approaches at an estimated angle of 30 degrees to the bridge opening. However, the pier sets and abutments have been skewed parallel to the flow to minimize scour. There is a some contraction of the flow, with the flood-plain width estimated to be 250 ft upstream and downstream of the bridge. Up to 1.5 ft of pier scour was observed at the pier sets. No abutment or contraction scour was observed at this site.

Sites 24 and 25: False Bottom Creek near Spearfish

Sites 24 and 25, the False Bottom Creek bridges (41-126-087 and 41-126-088), are located on Interstate 90 east and west, respectively, 2.5 mi east of the City of Spearfish in western South Dakota (T. 6 N., R. 3 E., sec. 18). These bridges are three-span, continuous-concrete bridges, 106 ft in length, built in 1970. No streamflow-gaging stations are located on False Bottom Creek or at this site. The bridges each have two round pier sets (three piers per set), 24 in. in diameter, located on spread footings. The bridge openings are classified as a spill-through abutment. The site has riprap protection on the 2:1 slope embankments on the left and right sides and a natural armoring of the bed. The bed material of the stream is predominantly gravel and cobbles/boulders. The banks at the sites are in good condition with an 80-percent cover of grass. The flood-plain cover is mainly grass with some small trees. There is little potential for debris accumulation at the sites because of the lack of available debris and the stable stream bottom.

The low-flow channel includes the central part of the bridge openings. At low to high flows, the stream approaches perpendicular to the upstream faces of the bridge pier sets and bridge openings. There is a small contraction of the flow, with the flood-plain width estimated to be 170 ft upstream and downstream of the bridges. No pier, contraction, or abutment scour was observed at the sites. Because of the stable bottom and the channelization at the sites, there is little potential for scour.

Sites 26 and 27: Split Rock Creek near Brandon

Sites 26 and 27, the Split Rock Creek bridges (50-284-166 and 50-284-165), are located on Interstate 90 eastbound and westbound (about 2,000 ft east of the Brandon/Corson exit), respectively, 1 mi north of the City of Brandon in southeast South Dakota (T. 102 N., R. 48 W., sec. 26). These bridges are five-span, continuous steel-girder bridges, 330 and 337 ft in length, built in 1960. The bridges both have four octagonal pier sets (two piers per set), 33 in. wide. Three of the pier sets at each site are located on spread footings, and one of the pier sets at each site is located on piling. The bridge openings are classified as a spill-through abutment. Riprap protection is provided on the upstream left embankment of site 27. The rest of embankments at the sites have no slope protection. The bed material of the stream is predominantly silt and clay. Some bank erosion was observed in the banks on the left upstream and downstream sides. The bank cover is 100-percent grass and weeds. The flood-plain cover is mainly crops or grass with small trees. There is some potential for debris accumulation at the pier sets because of the presence of debris upstream of the bridge openings.

The low-flow channel in confined mainly in the center of the bridge openings. During low to high flows, the stream approaches at an estimated angle of 25 degrees to the bridge openings. There is some contraction of the flow with the flood-plain width estimated to be 700 ft upstream and downstream of the bridges. Up to 2.5 ft of pier scour and 1 ft of contraction scour were observed at the sites. No abutment scour was observed.

The Split Rock Creek at Corson gaging station (06482610) is located less than 1 mi upstream of the sites and has a drainage area of 464 mi^2. This station has been operated from 1951 to 1965 and 1990 through 1993 as a crest-stage partial-record gaging station and from 1965 through 1989 as a continuous-record streamflow-gaging station. The largest recorded peak discharge was 18,900 ft^3/s on May 8, 1993. The annual mean discharge at this gaging station for the period from 1965 to 1989 is 100 ft^3/s.

Site 28: White River near Presho

Site 28, the White River bridge (43-160-339), is located on US Highway No. 183, 14 mi south of the City of Presho in south-central South Dakota (T. 103 N., R. 77 W., sec. 22). This bridge is a six-span, steel-girder bridge, 433 ft in length, built in 1952. The bridge has three octagonal pier sets with webs, 39 in. wide. The other pier set had no data collected on it because of the large water depth at the pier set. Three of the pier sets are located on spread footings, and the other one is located on piling. The bridge opening is classified as a spill-through abutment. The site has no riprap protection under the bridge or on the embankments. There are, however, three spurs constructed on the upstream and downstream banks extending out into the White River. The bed material of the stream is predominantly sand with some clay. The stream carries a large amount of sediment during high flows. Degradation was observed in the banks on the left upstream side. The bank cover is 90-percent grass and weeds. The banks were eroding due to the bridge being located on a meander of the White River. The flood-plain cover is mainly grass with some small trees. There is little potential for debris accumulation at the site because of the large bridge opening.

The low-flow channel is confined to the center part of the bridge opening. At low to high flows, the stream approaches at an angle to the upstream faces of the bridge pier sets and bridge opening causing the sloughing of the left banks. There is substantial contraction of the flow, with the flood-plain width estimated to be 3,300 ft upstream and downstream of the bridge. Up to 2 ft of pier scour was observed at the site. No contraction or abutment scour was observed. The site has a large potential for scour because of the bridge being on a meander.

The White River near Oacoma streamflow-gaging station (06452000) is located 55 mi downstream from the site and has a drainage area of 10,200 mi^2. This station has been operated from 1928 through 1993. The largest recorded peak discharge was 51,900 ft^3/s on March 30, 1952. The annual mean discharge at this gaging station for the period of record is 530 ft^3/s.

Site 29: Redwater River near Belle Fourche

Site 29, the Redwater River bridge (10-105-376), is located on US Highway No. 212, 1 mi east of the intersection of Highway Nos. 212 and 85 within the City of Belle Fourche in southwest South Dakota (T. 8 N., R. 2 E., sec. 11). This bridge is a 12-span, steel-girder bridge, 384 ft in length, built in 1926. Removal and reconstruction of this bridge was initiated during 1995. The bridge has 11 pier sets (four piers per set) that are predominantly octagonal, primarily ranging from 12 to 24 in. wide, located on piling. The bridge opening is classified as a vertical abutment. The site has no riprap protection under the bridge or on the embankments. The bed material of the stream is predominantly silt and clay. The banks at the site are in good condition with a 100-percent cover of grass/weeds and trees. The flood-plain cover is mainly grass, brush, and trees. There is large potential for debris accumulation and blockage of the opening at the site because of the presence of dead trees upstream and the small bridge opening. A railroad bridge is immediately upstream of the bridge site.

The low-flow channel is confined in the center of the bridge opening. At moderate to high flows, the stream approaches at slight angle to the upstream faces of the bridge pier sets and bridge opening. There is little or no contraction of the flow with the flood-plain width estimated to be 390 ft upstream and downstream of the bridge. Little or no pier, contraction, or abutment scour was observed at this site.

The Redwater River above Belle Fourche streamflow-gaging station (06433000) is located at the site and has a drainage area of 920 mi^2. This station has been operated from 1945 through 1993. The largest recorded peak discharge was 16,400 ft^3/s on June 16, 1962. The annual mean discharge at this gaging station for the period of record is 131 ft^3/s.

Site 30: Powell Creek near Fort Pierre

Site 30, the Powell Creek bridge (59-339-327), is located on Highway No. 214, 6.5 mi southwest of the City of Fort Pierre in central South Dakota (T. 4 N., R. 30 E., sec. 15). This bridge is a single-span, steel-girder bridge, 42 ft in length, built in 1936. No streamflow-gaging stations are located on Powell Creek or at this site. The bridge has no piers. The bridge opening is classified as a vertical abutment. The site has no riprap protection under the bridge or on the embankments. The bed material of the stream is predominantly silt and clay. Erosion was observed in the banks on the left and right sides. The bank cover, composed of grass and weeds, is less than 5 percent on the upstream side and less than 25 percent on the downstream side. An upstream bend about 100 ft from the bridge site probably contributed to this bank erosion. The flood-plain cover is either bare ground or grass and weeds. There is little potential for debris accumulation at the site because of the lack of upstream debris.

The low-flow channel is confined in the center of the bridge opening. At low to high flows, the stream approaches perpendicular to the bridge opening. There is some contraction of the flow with the flood-plain width estimated to be 150 ft upstream and downstream of the bridge. Three feet of contraction scour was observed at the site. No abutment scour was observed. The site has a large potential for scour due to the highly erodible bottom.

Site 31: Willow Creek near Fort Pierre

Site 31, the Willow Creek bridge (59-374-317), is located on Highway No. 214, 3 mi southwest of the City of Fort Pierre in central South Dakota (T. 4 N., R. 31 E., sec. 8). This bridge is a three-span, steel-girder bridge, 122 ft in length, built in 1936. No streamflow-gaging stations are located on Willow Creek or at this site. The bridge has two square pier sets (two piers per set), 27 in. wide, located on piling. The bridge opening is classified as a spill-through abutment. A vertical piling wall has been constructed on the left abutment slope under the bridge. There is no other slope protection at the site. The bed material of the stream is predominantly silt and clay. Some sloughing was observed in the banks at and upstream of the bridge. The upstream bank cover is 10 percent grass and weeds, and the downstream bank cover is 100 percent trees, brush, and grass. The flood-plain cover is mainly grass/weeds with some small trees. There is some potential for debris accumulation at the pier sets.

The low-flow channel is confined to the northern part of the bridge opening. At low to high flows, the stream approaches at an angle to the upstream faces of the bridge pier sets and bridge opening, causing the erosion on the left embankments at the bridge. There is a large contraction of the flow, with the flood-plain width estimated to be 1,400 ft upstream and downstream of the bridge. Five to 10 ft of abutment scour was observed at the site. No pier or contraction scour was observed. The site has a large potential for scour because of the skewness of flow and the highly erodible bottom.

Site 32: Hidewood Creek near Estelline

Site 32, the Hidewood Creek bridge, is located on a gravel road, 4 mi north of the intersection of SD Highway No. 28 and the City of Estelline and then 0.7 mi east in northeast South Dakota. This bridge is a two-span bridge, 72 ft in length. The bridge has one pier set, which is basically a 10-in. "I" beam driven into the ground and located on piling. The bridge opening is classified as a spill-through abutment. The site has riprap protection under the bridge and on the embankments. The bed material of the stream is predominantly silt and clay. The upstream bank cover is 100 percent grass, and the downstream bank cover is cropland. The upstream flood-plain cover is grass, and the downstream flood-plain cover is crop land. There is little potential for debris accumulation at the site because of lack of upstream debris.

The low-flow channel is confined to the center of the bridge opening. At low to high flows, the stream approaches at a small angle to the upstream faces of the bridge pier sets and bridge opening. There is a large contraction of the flow, with the flood-plain width estimated to be 1,300 ft upstream and downstream of the bridge. Two feet of pier and contraction scour was observed at the site. No abutment scour was observed.

The Hidewood Creek near Estelline gaging station (06479640) is located 1.1 mi downstream of the site and has a drainage area of 164 mi^2. This station was operated as a continuous-record streamflow-gaging station from 1968 through 1985 and as a crest-stage partial-record gaging station from 1990 through 1993. The largest recorded peak discharge was 17,300 ft^3/s on June 16, 1992. The annual mean discharge at this gaging station for the period of 1968 through 1985 is 25.8 ft^3/s.

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