Field Observations and Evaluations of Streambed Scour at Bridges

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FOREWORD

This report describes the most comprehensive set of real-time field measurements of bridge scour ever assembled. It represents more than 6 years of dedicated effort by the U.S. Geological Survey researchers to collect scour data during flood events wherever they occurred in the United States. The report will be of interest to bridge engineers and hydraulic engineers involved in bridge scour evaluations and to researchers involved in developing improved bridge scour evaluation procedures. Sufficient copies will be printed to provide at least two copies to each Federal Highway Administration (FHWA) Division Office.

T. Paul Teng, P.E.
Director, Office of Infrastructure
Research and Development

Notice

Quality Assurance Statement

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The variability and complexity of site conditions make it difficult to develop methodology for predicting scour at bridges. Laboratory investigations often oversimplify or ignore many complexities common in the field. The U.S. Geological Survey, in cooperation with the Federal Highway Administration and many State highway agencies, has collected and compiled field data on scour at bridges at 79 sites located in 17 States. These data have been analyzed to isolate pier scour, contraction scour, and abutment scour. The national data base contains 493 local pier scour measurements, 18 contraction scour measurements, and 12 abutment scour measurements. The pier scour measurements were used to evaluate 26 published pier scour equations. The Froehlich Design, HEC-18, HEC-18-K4, HEC-18-K4Mu, HEC-18-K4Mo (>2 millimeter), and Mississippi equations proved to be better than the other equations for predicting pier scour for design purposes. However, comparison of the scour predicted from these equations with the observed scour clearly shows that variability in the field data is not correctly accounted for in the equations. Relations between dimensionless variables developed from laboratory experiments did not compare well with the field data. Analysis of the pier scour data indicated the importance of bed-material characteristics as a variable in the predictive equations. A new K_4 term for the HEC-18 pier-scour equation was developed based on the relative bed-material size (b/D_{50}) where b = pier width and D_{50} is the median bed material. A review of published literature found 29 references to abutment and contraction scour data; however, only a few provided complete data sets. Published comparisons of observed versus computed scour were inconclusive. A detailed comparison of computed contraction and abutment scour with field observations for two sites in Minnesota was also inconclusive. The current methodology for computing scour depth provides reasonable estimates of the maximum total scour, but the individual estimates of contraction and abutment scour did not compare well with the observed data. The accuracy of the contraction and abutment scour equations may depend on the degree of contraction, the flow distribution in and configuration of the approach, and how well the hydraulic model represents the true flow distribution.</td></tr>
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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
		NOTE: volumes greater than 1000 L shall be shown in m^3		
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
		TEMPERATURE (exact degrees)		
^oF	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	^oC
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact degrees)		
^oC	Celsius	1.8C+32	Fahrenheit	^oF
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

*SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380. (Revised March 2003)

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b is the pier width.

b_e is the effective pier width defined as $b\cos\alpha + L\sin\alpha$.

b_1 is the bottom width in the uncontracted section.

b_2 is the bottom width in the contracted section.

c_a is the pier location code in the Arkansas pier scour equation, $c_a = 0$ for main channel piers and $c_a = 1$ for piers on the banks of the main channel or on the floodplain.

D_{10} is the grain size of bed material for which 10 percent is finer.

D_{16} is the grain size of bed material for which 16 percent is finer.

D_{35} is the grain size of bed material for which 35 percent is finer.

D_{50} is the grain size of bed material for which 50 percent is finer; the median grain size.

D_{84} is the grain size of bed material for which 84 percent is finer.

D_{90} is the grain size of bed material for which 90 percent is finer.

D_{95} is the grain size of bed material for which 95 percent is finer.

D_{99} Is the grain size of bed material for which 99 percent is finer.

D_i or D_x is the grain size of bed material for which *i* or *x* percent is finer.

D_m is the mean grain size of the bed material.

DA is the drainage area.

D_{CFM} is an average of the coarse grain sizes used by Molinas; see table 3.(1)

E_b is the exponent on the ratio of bottom widths for live bed contraction scour equation.

E_n is the exponent on the ratio of roughness coefficients or live bed contraction scour equation.

E_Q is the exponent on the ratio of discharges for live bed contraction scour equation.

$f(\,)$ is an undefined function of parameters enclosed in parentheses.

F & F_o is the flow Froude number defined as $V_o/(gy_o)^{0.5}$.

F_p is the pier Froude number defined as $V_o/(gb)^{0.5}$.

G is the acceleration of gravity.

LIST OF SYMBOLS (continued)

k_i is the standard normal deviate of i.

K is a multiplying factor that varies from 1.3 to 2.3

K_d is a coefficient to correct for sediment size by Melville and Sutherland.[2]

K_i is a coefficient to correct the HEC-18 equation for sediment size by Molinas; see table 3.[1]

K_I is a coefficient to correct for flow intensity defined by Melville and Sutherland.[2]

K_s is a coefficient to correct for pier shape defined by Melville and Sutherland.[2]

K_{sc} is a coefficient for pier shape in the Simplified Chinese equation, defined by Gao et al. to be 1 for cylinders, 0.8 for round-nosed piers, and 0.66 for sharp nosed-piers.[3]

K_{S2} is a coefficient for pier shape used by Larras and is 1.0 for cylindrical piers and 1.4 for rectangular piers.[4]

K_u is 1.0 for SI units and 1.81 for customary English units in the critical velocity equation.

K_y is a coefficient to correct for flow depth defined by Melville and Sutherland.[2]

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

K_2 is a coefficient to correct for the skew of the pier to the approach flow, defined as $(cos\ \alpha + (L/b) sin\ \alpha)^{0.65}$.

K_3 is a coefficient to correct for the channel bed condition, defined as 1.1 except when medium to large dunes are present, and then it can range from 1.2 to 1.3.

K_4 is a coefficient to correct for bed material size and gradation; see table 3.

$K4Mu$ K_4 coefficient derived by Mueller.[5]

$K4Mo$ K_4 coefficient derived by Molinas.[1]

$K_{\alpha L}$ is a coefficient to correct for flow alignment defined by Melville and Sutherland (1988).[2]

L is the length of the pier.

n_1 is the Manning's n in the uncontracted section.

n_2 is the Manning's n in the contracted section.

Q_1 is the discharge in the uncontracted section.

LIST OF SYMBOLS (continued)

Q_2 is the discharge in the contracted section.

S is the slope of channel in the vicinity of the bridge.

V_o is the approach velocity for pier scour.

V_c is the critical (incipient-transport) velocity for the D_{50} size particle.

V_{cx} is the critical (incipient-transport) velocity for the D_x size particle.

V_R is a velocity intensity term used by Richardson and Davis; see table 3.[6]

V_c' is the approach velocity corresponding to critical velocity and incipient scour of the D_{50} in the accelerated flow region at the pier.

V_{cx}' is the approach velocity corresponding to critical velocity and incipient scour of the D_x in the accelerated flow region at the pier.

V_i is the approach velocity corresponding to critical velocity and incipient scour in the accelerated flow region at the pier defined by Molinas; see table 3.[1]

V_{cm} is the critical (incipient-transport) velocity for the coarse size fraction defined by Molinas; see table 3.[1]

V_{LP} is the live bed peak velocity defined by Sheppard.[7]

V_2 is the velocity in the contracted section.

y_o is the approach depth of flow for pier scour.

y_s is the depth of scour.

Y_1 is the depth in the uncontracted section.

Y_2 is the depth in the contracted section.

α is the skew of the pier to approach flow.

ϕ is a pier shape factor in Froelich's equations

σ is the coefficient of gradation.

θ is the Shield's parameter.

τ represents one or more shear stress variables.

ν is the kinematic viscosity in Shen's equation (ft^2/sec).

CHAPTER 1: INTRODUCTION

GENERAL

Bridge piers and highway embankments leading to a bridge often obstruct the flow of floodwaters, causing an increase in velocity and the development of vortices. The increased velocity and vortices often cause scour near the bridge foundations. The damage to and failure of bridges caused by scour are problems of national concern, as illustrated by the number of bridges damaged or destroyed by floods in the United States during 1985–95 (table 1).

Table 1. Number of bridges damaged and destroyed by scour, 1985–95.

Location and Year	Number of Bridges Damaged or Destroyed
Pennsylvania, West Virginia, Virginia, 1985	73
New York and New England, 1987	17
Midwestern United States, 1993	>2,500
Georgia, 1994	>1,000
Virginia, 1995	74
California, 1995	45

The U.S. Federal Highway Administration (FHWA) has issued several guidance documents addressing scour. Hydraulic Engineering Circular (HEC)–18 presents methods for predicting local and contraction scour at planned and existing bridges.[6] Although the methods presented in HEC–18 represent the state-of-the-knowledge at the time of publication, some of the potential limitations of these methods were identified.

> The current equations and methods for estimating scour at bridges are based primarily on laboratory research. Very little field data have been collected to verify the applicability and accuracy of the various design procedures for the range of soil conditions, streamflow conditions, and bridge designs encountered throughout the United States. (p. 3)[6]

The lack of and need for reliable and complete field data on scour at bridges has been a recurring conclusion of many researchers (See references 8, 9, 10, 11, and 12.) Other researchers have compiled field measurements on local pier scour.[3,13,14] These historical data sets contain valuable information, but most do not contain information on all major factors known to affect scour. Froehlich was unable to include the effect of sediment gradation in his analysis because many data sets did not include this information.[13] Johnson, in a comparison of seven published pier scour equations with field data, assumed uniform sediment size because sediment gradation information was not available for most of the data.[15]

Field measurements of contraction and abutment scour are significantly fewer than of local pier scour. In a recent review of published field data on contraction and abutment scour, 29 references were found. Of these only Norman[16] presented detailed data collected during floods and only two other references included data on abutment scour.[17,18] Seven of the 29 papers presented data on contraction scour, and another 14 discussed sites that could yield contraction scour data, but only if additional data were available.

Despite the recognized need for the collection of field data, few data were collected until the late 1980s, a deficiency that is primarily a reflection of the difficulty in collecting the necessary data. Accurate and complete field measurements of scour are difficult to obtain because of complex hydraulic conditions at bridges during floods, inability to get skilled personnel to bridge sites during floods, and problems associated with existing measuring equipment.[19,20] Cooperative research among FHWA, State highway departments, and the U.S. Geological Survey (USGS) has allowed the collection of scour data at bridges during floods. Landers and Mueller published 394 local pier scour measurements made by the USGS during the first national bridge scour study. [21] Most of these data collected by USGS contain bed material data and provide supporting channel cross sections and site-characterization data.

PURPOSE AND SCOPE

This report describes the results of the second USGS national field-data collection and analysis study on scour at bridges, funded by FHWA. The database originally developed during the first national study has been enhanced and many scour measurements added, including measurements of abutment and contraction scour.[22] Sufficient local pier scour data are now available to permit a detailed analysis of local pier scour. Scour depths computed from published pier scour equations are compared to the field measurements. Many commonly cited dimensionless variables believed to control the depth of scour are evaluated and compared with equations developed from laboratory data. The effect of the size and gradation of the bed material on the depth of scour is investigated, and a correction factor for the HEC–18 pier scour equation is proposed. Available data are insufficient to permit a detailed investigation of contraction and abutment scour; however, some basic comparisons and qualitative observations are presented on the basis of a review of the literature. The results of scour analyses for two contracted bridges are compared with real-time field data.

CHAPTER 2: COMPONENTS OF SCOUR AT BRIDGES

GENERAL

Total scour at a bridge is generally divided into three or more components. According to Richardson and Davis, bridge scour consists of the following three components:(6)

1. *Aggradation or degradation*—long-term changes in streambed elevation due to natural or human-induced causes, which can affect the reach of the river near the bridge.

2. *Contraction scour*—removal of material from the bed and banks across all or most of the channel width, resulting from the contraction of the flow area.

3. *Local scour*—removal of bed material from around piers, abutments, spurs, and embankments. Local scour is caused by the acceleration of flow and by vortices resulting from flow around an obstruction.

Documents from Canada and Australia, along with field experience, indicate the need for a fourth component of scour, termed short-term scour, which refers to natural tendencies of the stream to scour and fill during relatively short-term streamflow runoff cycles.(23,24) Neill calls this component natural scour: "Natural scour in alluvial and tidal channels (is) associated with variations in flow conditions and associated channel processes including bed material transport, bed-form migration, and channel shifting." (p. 77)(23)

Austroads defined this component as follows: "Scour due to river morphology—which would occur naturally in the stream and is a function of flow conditions and associated channel characteristics. It includes general bed movement and scour at channel contractions and bends." (p. 55)(24)

The discussion of contraction scour in Richardson and Davis implies that this natural short-term scour is included in the contraction scour component.(6) This short-term scour does not depend on the contraction, but on the sediment transport processes in the stream. Short-term scour must be accounted for in the analysis of field data, or the reported contraction scour may include short-term scour.

Data to quantify local (abutment and pier) and contraction scour must have a spatial and temporal distribution that encompasses the complex processes causing these types of scour. Scour caused by piers, abutments, and contractions cannot be measured directly, but must be interpreted from channel bathymetry data.(21,25) The spatial and temporal distribution of the data collected must include the data needed to identify the reference surfaces from which the various components of scour are measured. The components of scour cannot be isolated with any degree of certainty without adequate data to identify the reference surfaces.

AGGRADATION AND DEGRADATION

Aggradation and degradation are the result of change in a geomorphic control within the watershed, which in turn causes a long-term change in the streambed elevation. The reference surface for aggradation and degradation must be established at some discrete time in the river's history. Determining the long-term changes at a bridge site is very difficult if historical streamflow data are not available. In such situations, a paleogeomorphologist can evaluate sediment stratigraphy and vegetation to estimate the stability and magnitude of past and future geomorphic changes for a particular river. If streamflow data are available, then the development of specific gage plots, analysis of rating shifts, and analysis of historical changes in streambed elevation are the best means of estimating the stability and magnitude of geomorphic changes. Specific gage plots are developed by selecting one or more discharges to analyze using only measured stage and discharge at each discharge and then plotting a graph of stage-versus-time for each of the selected discharges. The general slope of the line will indicate the long-term change in the bed elevation, and the irregularity of the line indicates changes in roughness or short-term changes in bed elevation (figure 1). Comparing pre-flood and flood cross sections will not yield the magnitude of long-term streambed elevation change; it does yield a combination of long-term and short-term streambed elevation change, but then only if uncontracted sections (such as the approach section) are used. If the bridge section is used, the difference between pre-flood and flood cross sections will also include contraction scour.

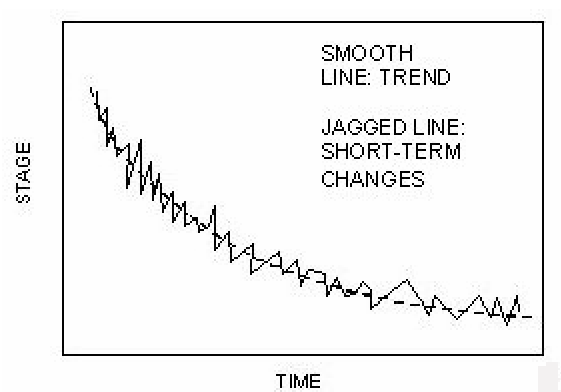


Figure 1. Illustration of a specific gage plot showing stream degradation.

SHORT-TERM SCOUR AND FILL

Short-term scour and fill is the change in bed elevation in a reach of river; it is caused by the temporary storage and transport of sediments by the river at different stages of the stream's hydrograph. Depending on the shape of the rising and falling limbs of a hydrograph, the short-term changes in bed elevation may not return to the long-term equilibrium condition at the end of a single event; returning to this equilibrium condition may take more time, perhaps several events. Some streams may never achieve a single equilibrium elevation; however, over a long period of time the specific gage plot should have a horizontal trend indicating no long-term changes (figure 2). Because prediction of short-term scour is difficult, if not impossible, and because the contraction and local scour may override the short-term scour at a structure, bridge scour design guidance has not included this in computations for depth of scour, except as the movement of bed forms.[6]

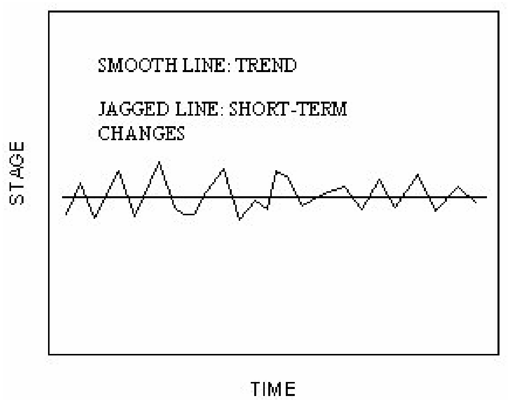


Figure 2. Example of short-term scour and fill with no long-term changes.

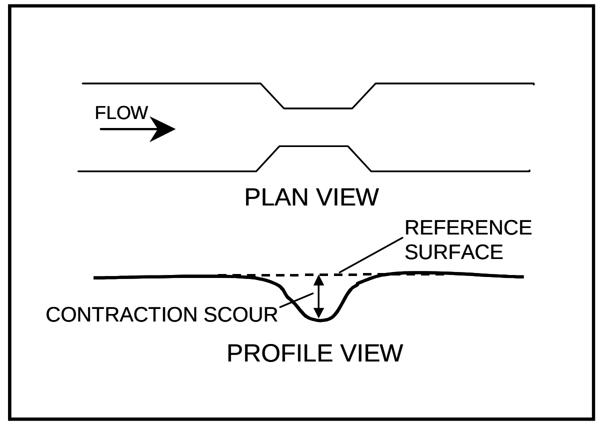


Figure 3. Illustration of reference surface for contraction scour.

If the river is not undergoing long-term changes in streambed elevation, the difference between uncontracted cross-sectional geometry measured before and during the flood provides an estimate of the short-term scour at that location; however, if long-term changes in streambed elevation are occurring on the river, this same measurement would include these long-term changes. These long-term changes cannot be easily removed from the data because long-term geomorphic changes often occur catastrophically at thresholds and not gradually at a uniform rate. A typical value of short-term scour could be estimated from the scatter of the specific gage plot around the long-term trend line (figures 1 and 2).

GENERAL SCOUR

General scour is a combination of short- and long-term scour. General scour at a specific period in time can be measured by determining the difference in bed elevation between pre-flood and flood measurements of uncontracted cross sections; however, measurements of uncontracted cross sections during floods are rarely available. Contracted sections should not be used because the scour measurements based on these sections will include contraction scour, in addition to the short- and long-term scour components.

CONTRACTION SCOUR

Contraction scour results from the contraction of the normal flow by natural contractions or manmade contractions (such as highway embankments and bridge piers); it is the difference in bed elevation between contracted and uncontracted cross sections (figure 3). Contraction scour does not necessarily occur evenly over the entire cross section; however, it usually is defined as the average difference between the contracted and uncontracted sections (figure 4). Contractions can be both lateral and vertical. A vertical contraction would occur when a bridge enters a pressure flow condition. In this report, contraction scour will refer to only lateral contractions, and the term pressure scour will be used to refer to the depth of scour caused by a vertical contraction.

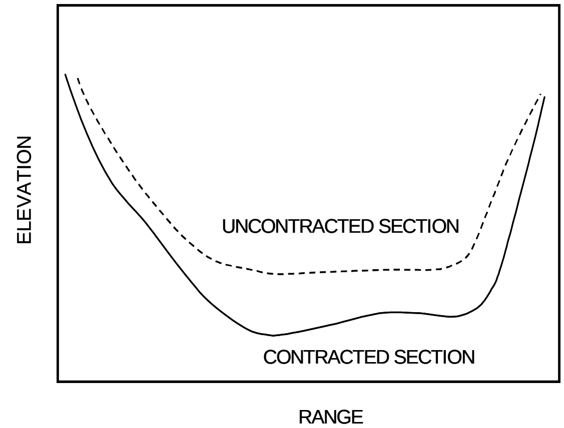


Figure 4. Illustration of nonuniformly distributed contraction scour.

Blodgett reported one of the few field investigations in which contraction scour was quantified separately from aggradation and degradation and short-term scour and fill.[26] Blodgett's contraction scour reference surface was represented by a straight line projected over the contracted section from the thalweg (minimum elevation) profile upstream and downstream from the contracted area. The contraction scour was reported as the difference between this reference surface and the thalweg of the contracted sections; Blodgett stated that using the thalweg to measure contraction scour represented a worst-case condition. Natural pool and riffle sequences must be accounted for when using the thalweg as a reference surface or the results could be misleading. Differences between thalweg elevations may not be consistent with published contraction scour equations, which are based on average changes in the contracted section required to achieve equilibrium sediment transport. Consequently, these equations cannot be used to compute the lateral distribution of contraction scour.[21]

The reference surface should characterize the mean bed elevation of an uncontracted section near the location of the contraction scour measurement. The reference surface is established by passing a surface through the average elevation of uncontracted cross sections located upstream and downstream from the contracted section.[25] For live bed conditions, the contracted and uncontracted cross sections should ideally be measured concurrently to eliminate effects from aggradation, degradation, and short-term scour. The effects of local scour on the average elevation in the contracted section are removed by excluding the locally scoured areas from the average. The cross section should be subdivided into subareas, based on the mode of transport (live bed transport or clear water conditions) and the scour in each subarea computed using the proper equation for the mode of transport.

Landers and Mueller acknowledged several potential problems with this ideal reference surface for live bed conditions:[25]

1. Identification of the bottom width over which active bed-load transport occurs is often difficult because of irregular cross section geometry.

2. Upstream and downstream cross sections may be in natural contractions or expansions because of channel bends or other factors, so they do not represent an uncontracted section at the bridge. Alternating pool and riffle sequences in some high-gradient streams can also present problems in establishing an uncontracted reference surface at the bridge.

3. Large dune bed forms can produce misleading results when dune crests or troughs predominate in one of the measured sections.

4. Slow downstream migration of large sand and gravel bars can make a measurement nonrepresentative of equilibrium conditions.

5. Measured contraction scour may not represent equilibrium scour if the scour develops over many years because of the infrequency of channel-formative flows and the resistance of the bed to scour.

6. Most flood-flow scour measurements are made only from the bridge deck along the upstream and downstream sides of the bridge because boats are usually unavailable to obtain concurrent uncontracted channel geometry.

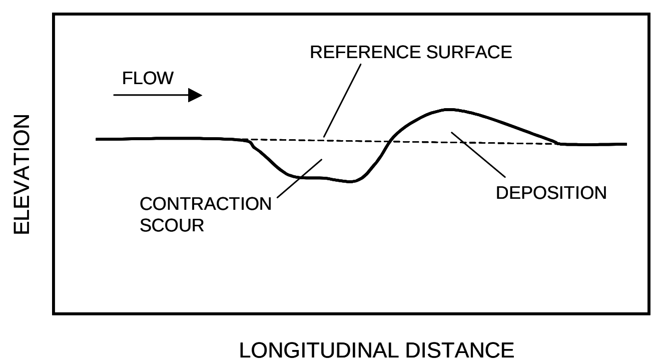


Figure 5. Illustration of a reference surface for clear water contraction scour with material deposited immediately downstream of the scour hole.

Pre-flood and (or) post-flood measurements of uncontracted sections often are used to establish a contraction scour reference surface because of the difficulty in obtaining measurements of the uncontracted sections during a flood. This reference surface is valid for clear water conditions but may not isolate contraction scour from short-term scour and degradation or aggradation for live bed conditions. Pre- and post-flood measurements may be useful for live bed contraction scour measurements if data are sufficient to support the assumption of stable approach and exit sections; however, the stability of the uncontracted sections and the accuracy of the measurement should be assessed by comparing pre- and post-flood measurements throughout the study reach or for multiple floods at a specific location.

For clear water contraction scour, collection of real-time bathymetric data are not necessary. The bed upstream of the bridge and the scour hole will maintain the same geometry after the passage of the flood because no sediment is being transported into the scour hole. Therefore, post-flood surveys can be used to measure clear water contraction scour. The post-flood survey must extend far enough downstream to end beyond the effect of the material that was scoured and deposited downstream (figure 5).

LOCAL SCOUR

Local scour near an obstruction is caused by the vortexes and flow acceleration resulting from flow striking the obstruction and moving past it. The depth of local scour is the difference between where the bed would be if the obstruction were not present and where the bed is with the obstruction in place. The concurrent ambient bed surface is the preferred reference for local scour determinations (figure 6). The concurrent ambient bed surface is determined from data collected at the time the scour hole was measured (concurrent with the scour data). This reference surface is typically taken as the average of several points measured in the vicinity of the obstruction but beyond the limits of the local scour hole.

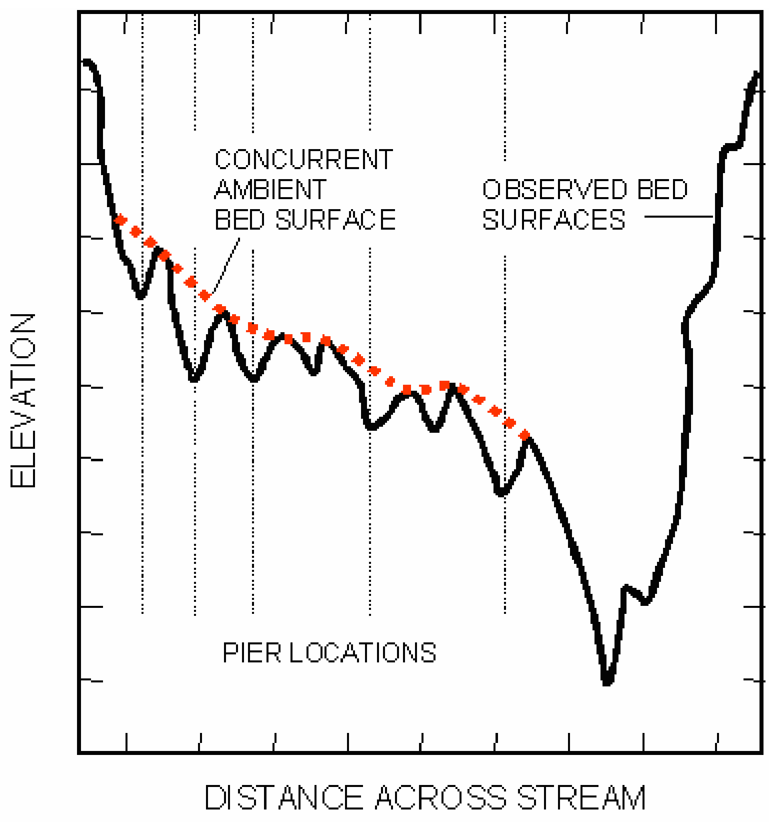


Figure 6. Illustration of reference surface sketched
on a cross section plot.

The concept and description of the concurrent ambient bed are simple; however, a representative concurrent ambient bed surface is not always apparent given the range of channel geometries and data limitations. Establishing local scour reference surfaces can be difficult and requires considerable judgment for complex cross section geometries, such as a site where the thalweg coincides with the local scour hole. Additional factors that must be evaluated in measuring local scour include remnant scour holes, debris, scour countermeasures, time-rate of scour, and dune bed forms. Determining a representative and repeatable reference surface for local scour can be difficult.[25] A good method for determining the concurrent ambient bed surface is to use a section of the river that has the same general shape as the bridge section (such as the approach, exit, or preconstruction sections), and move that section vertically until it best fits the bridge section, ignoring the scour holes. The local scour is the difference between the concurrent ambient bed surface and the lowest elevation in the scour hole. Another method is to sketch in the concurrent ambient bed surface (figure 6). Both methods are subjective and may yield different answers when applied by different people to the same cross section.

The concurrent ambient- or mean-equilibrium (for dunes) bed surface has been used as the reference surface in most flume studies of local scour. (See references 9, 27, 28, 29, and 30.) Other reference surfaces used in flume studies include the initial-condition bed surface and the water surface. Field measurements of local scour have generally used the concurrent ambient bed surface as a reference (see references 16, 31, 32, and 33); however, Inglis[34] used the water surface as the reference surface, and Jarrett and Boyle[35] used the highest observed bed elevation at the point where local scour was being measured. Other commonly used references that are not valid for isolating the local scour component include:

- The average bed elevation that includes the local scour hole(s).
- The pre- or post-flood cross sections because the piers and corresponding local scour holes may be present in these cross sections.
- The preconstruction cross section on the bridge plans because this cross section does not reflect the long-term, short-term, and contraction scour that may have occurred.

Local scour measurements based on reference surfaces other than concurrent ambient bed surface may include components of contraction scour, short-term scour, or aggradation or degradation. Total scour measurements recorded with these types of reference surfaces cannot be effectively analyzed in relation to separate local, contraction, and sediment-supply processes; therefore, the concurrent ambient bed surface should be used to determine local scour depths from the channel geometry.[25]

CHAPTER 3: DESCRIPTION OF FIELD METHODS

GENERAL

The type of data collected at a bridge is determined by the scour processes being studied and the equipment's capability to collect the desired data. The type of scour being studied (pier, abutment, or contraction) determines which variables need to be measured and the spatial distribution of these data. For local pier scour, measurement of the approach velocity, bed material properties, and cross-sectional data to define the reference surface and maximum depth of scour may be sufficient. For abutment and contraction scour, hydraulic conditions in the approach section are important. Unfortunately, the site configuration may limit the data that can be collected accurately and safely. Floodplains commonly are covered by trees and other woody vegetation, which makes measurement of the velocity and flow distribution virtually impossible. Debris and ice accumulations may prevent measuring streambed elevations in key areas. On small streams, the duration of the flood may not allow sufficient time to collect all of the desired data; all desired data are rarely collected at a site.

The USGS has defined two classes of field data: limited-detailed and detailed.(21) In general, limited-detailed data can be collected from the bridge deck, and the spatial extent of the data is restricted to the upstream and downstream edges of the bridge. Detailed data require the deployment of equipment from a boat to collect data over an area extending beyond the hydraulic effect of the bridge. Most available data are from limited-detail data collection efforts sponsored by State highway departments. The FHWA-sponsored national bridge scour studies are responsible for most of the detailed data sets that have been collected.

A brief summary of these types of data is presented here. For a more detailed discussion of limited-detail data collection see Landers and Mueller,(21) Jarrett and Boyle,(35) and Mueller and Landers.(36) A comprehensive presentation of detailed data collection and the necessary equipment can be found in Landers and Mueller,(21) Mueller and Landers,(36) and Mueller.(5)

LIMITED-DETAIL DATA

Limited-detail data primarily are collected to evaluate published equations, investigate the relations between local scour and explanatory variables, and develop envelope curves for the maximum observed scour. Limited-detail data sets should include the following data:(21)

- Water discharge.
- Water-surface elevations at the bridge.
- Cross section data along the upstream and downstream edges of the bridge.
- Cross sections, approximately one bridge width upstream and downstream of the bridge (it is desirable to measure this during the flood, but low-water approach and exit sections are usually acceptable).
- Approach flow velocity for each pier location.

- Bed material samples (it is desirable to collect these during a flood, but low-water samples are usually acceptable).
- Notes on debris accumulations, surface currents, roughness, and vegetation.
- Photographs of the bridge and stream reaches upstream and downstream.
- Water temperature.
- Bridge and pier geometry.
- Soil boring logs for the bridge crossing.

Rantz et al.[37] and Landers and Mueller[21] describe equipment and procedures for discharge and approach velocity measurements. Bed material samples can be collected using the equipment and procedures described in Landers and Mueller, Mueller and Landers, Edwards and Glysson, Ashmore et al., Yuzyk, and the International Organization for Standardization (1992). (See references 21, 36, 38, 39, 40, and 41.)

DETAILED DATA

Detailed data sets are similar to limited-detail data except the data are denser and their spatial extent broader. Ideally, detailed data sets include real-time measurements of hydraulic and channel-geometry data at several times during the flood hydrograph. Data are collected both upstream and downstream in an area that extends just beyond the hydraulic effect of the bridge. The extent of the hydraulic effect of the bridge both upstream and downstream depends on the degree of contraction, the size and configuration of the floodplains and approach embankments, and the slope of the stream. The goal is to collect data at least 10 bridge widths upstream and downstream, but this may be adjusted in the field as site conditions dictate. Detailed data sets allow distinction between local, contraction, and general scour occurring at the highway crossing, and are needed to advance the understanding of complex bridge scour processes. Detailed data sets should include the following:

- Water-discharge hydrograph.
- Water-surface elevation hydrograph.
- Water-surface slope.
- Detailed channel-geometry data at and near the bridge.
- Channel geometry in the river reach upstream and downstream of the bridge.
- Flow velocities (magnitude and direction) in the entire study reach.
- Bed material samples.
- Suspended-load and bed-load measurements (if possible).
- Notes on surface currents, channel roughness, and vegetation.
- Approximate measurements of debris piles present.

- Photographs of the bridge and stream reaches upstream and downstream.
- Water temperature.
- Bridge and pier geometry.
- Soil boring logs for the bridge crossing.

The procedures and equipment for making discharge measurements and collecting suspended-load and bed material samples are the same as those used for the limited-detail measurements. The spatial extent of the bathymetric and velocity data is much greater for detailed data sets than for limited-detail data sets; therefore, instruments must be deployed from the water surface rather than from the bridge deck. It is also highly desirable to measure the direction of flow in addition to the velocity magnitude.

CHAPTER 4: ENHANCEMENTS TO THE BRIDGE SCOUR DATA MANAGEMENT SYSTEM

The Bridge Scour Data Management System (BSDMS) was developed to support the preparation, compilation, and analysis of bridge scour data.[22] Bridge scour data are stored in the BSDMS as data sets that are defined for each bridge scour site in the database. Each data set contains more than 200 site and measurement attributes of the channel geometry, flow hydraulics, hydrology, sediment, geomorphic setting, location, and bridge specifications. The BSDMS provides interactive storage, retrieval, selection, editing, and display of bridge scour data sets; it was originally programmed in FORTRAN and was available in both DOS and Unix® versions.

Changes to the structure of the database were difficult, which restricted adapting the database to different types of data and adding new parameters as research evolved on scour at contracted bridge openings. The capabilities of the BSDMS for extracting data from the database and putting the data into tables for subsequent analysis were also limited. It was determined that the BSDMS would be more efficient and useful if it were ported to a commonly used database engine, such as Microsoft® Access, rather than to try to adapt the FORTRAN code to meet the changing requirements of the BSDMS. Microsoft Access provides a much more flexible database structure with user-friendly methods for querying and extracting data. The use of Microsoft Access will also allow future versions of the BSDMS to be served on the World Wide Web.

The enhancement of the BSDMS and the serving of the data on the Internet are an ongoing effort that has received subsequent funding from the National Cooperative Highway Research Program (NCHRP) Project 24–14. As of 2000, all data from the original BSDMS had been ported to Microsoft Access and the transferred data checked for any errors in conversion. Most variables of the original BSDMS have been retained and some new variables added to better characterize bed material and contraction and abutment scour. Some features, such as entry of hydrographs and cross sections and all graphics capabilities, currently are not supported directly by the database. Hydrographs and cross sections can be stored as individual files and referenced in the files section of the database. This simplifies the database, and the files section provides the capability of including many different types of data, such as Acoustic Doppler Current Profiler (ADCP) data, xyz topography data, digital photographs, scanned bridge plans, maps, and satellite imagery. Data are easily extracted into tables so that the user may construct ASCII data tables and import those tables into the graphics or statistical software of their choice.

Corrections and enhancements to existing data were made during the data transfer and validation process. The data from Alaska, reported in Norman, provided only the D_{90} and D_{50} grain sizes.[16] The data set in the original BSDMS included D_{90} as D_{95} and noted this in the comments; this has been changed in the new BSDMS. The D_{95}, D_{84}, and D_{16} are computed from the data provided. The D_{84} was interpolated from the D_{90} and D_{50} by use of a log-probability interpolation. The gradation parameter (σ) was computed as D_{84}/D_{50} (although this parameter could also be computed as D_{50}/D_{16} or $(D_{84}/D_{16})^{0.5}$), and the D_{95} and D_{16} were computed from the following equation:

$$D_i = D_{50}\,\sigma^{k_i} \tag{1}$$

where

D_i is the grain size of which i percent is finer;
D_{50} is the median grain size of the bed material; and
k_i is the standard normal deviate of i, defined as follows:

D_i	K_i
D_{10}	-1.282
D_{16}	-0.994
D_{35}	-0.385
D_{50}	0
D_{65}	+0.385
D_{84}	+0.994
D_{90}	+1.282
D_{95}	+1.645
$D_{99.99}$	+3.72

The limit on the number of bed material samples that could be entered for a given site in the old BSDMS did not allow full presentation of the data for sites in Ohio. The new BSDMS does not have this restriction, and all the bed material data are included from a very extensive sampling program used in the Ohio bridge scour study.[42] The database now contains 557 bed material samples collected at bridge scour study sites.

New sites and additional data at existing sites were also added to the database. Landers et al. reported 384 local pier scour measurements at 56 bridges located in 14 States.[22] This has been expanded to 493 local pier scour measurements, 18 contraction scour measurements, and 12 abutment scour measurements. The database now contains data from 79 sites located in 17 States. Some of the contraction and abutment scour data sets are not complete, but they represent the best available information and should prove useful to other researchers working on abutment and contraction scour.

CHAPTER 5: LOCAL SCOUR AT PIERS

GENERAL

Laboratory research has dominated the field of local scour at bridge piers. Such research is limited by the range of hydraulic conditions typically tested and is conducted primarily under steady-flow conditions with uniform bed material. Relations and predictive equations developed from laboratory research have not been adequately verified by use of field data. This report compares the results of predictive equations to field measurements of local pier scour. Relations developed in the laboratory are evaluated for application to conditions typical of natural streams. The effects of nonuniform bed material and other explanatory variables on field measurements of scour are investigated by use of a partial residual analysis.

A complete evaluation of all equations for the prediction of local scour around bridge piers is beyond the scope of this investigation; however, 26 commonly cited equations are compared with field measurements of scour to evaluate their potential for use as design equations. A design equation should accurately predict scour; however, predicting sediment transport and scour accurately is very difficult. If a design equation predicts too little scour, the bridge could be under-designed and the traveling public put at risk. A good design equation should be as accurate as possible, but when in error the equation should overpredict scour to ensure that the design is always safe.

Results from laboratory investigations of the effects of velocity, approach flow depth, pier width, and bed material properties on the depth of local pier scour are compared with similar plots of field data. Laboratory research has shown a relation between relative depth of scour (y_s/b) and flow intensity (V_o/V_c) for ripple- and nonripple-forming sediments and for uniform and nonuniform sediments. (See references 29, 43, 44, and 45.) Melville and Sutherland investigated the effect of relative flow depth (y_o/b) on relative depth of scour (y_s/b).[(2)] The effect of relative sediment size (b/D_{50}) on relative scour depth for relative sediment sizes ranging from 2 to 1,000 has also been studied in the laboratory.[(43,46,47)] Nonuniform sediments consistently have resulted in less relative scour than similar uniform sediments used in laboratory investigations. (See references 45, 46, 47, and 48.) The applicability of these relations and the range of the laboratory data are evaluated by plotting field data with the same combination of variables as the laboratory data.

Analysis of bridge scour field data is more complicated than analysis of laboratory data because in the field all explanatory variables have the potential to vary at the same time. In the laboratory, all explanatory variables can be held constant and a specific variable systematically changed to study its effect. These controlled laboratory investigations, however, may not adequately describe the variability and interaction of variables present in natural conditions. In the field, all variables can change and interact; the effect of individual variables cannot be easily isolated. Although variables and dimensionless parameters from field data can be compared with laboratory data, the effect of all variables is present in the field data. For example, in a comparison of the effects of velocity, the field data will also include the effects of flow depth, bed material properties, pier shape, and pier size. Most of the laboratory relations account for pier size by plotting normalized scour depth (y_s/b); however, if the pier size does not affect scour linearly, then the relation has been distorted by the selection of the parameters.

Bed material size may affect local scour through its effect on bed forms and on the energy required to transport grains from the scour hole. Initially, the energy available to erode bed material is higher at the pier than in the approach section. As erosion occurs at the pier, the scour hole forms and deepens. The energy available to transport sediment decreases as the scour hole deepens until equilibrium with upstream transport is achieved. Insufficient energy to transport a larger particle should be achieved at a shallower depth of scour than the depth needed for a smaller particle; thus, sediment size affects the depth of scour. Laboratory research is typically conducted at a constant V_o/V_c ratio: as the bed material size increases, so does the approach velocity. These laboratory conditions are not representative of most field conditions. The effect of bed material properties on the depth of scour is evaluated by comparing subsets of the field data that represent conditions similar to those modeled in the laboratory with the results of laboratory investigations. In addition, an analysis of the residuals generated by comparing the computed scour from the HEC–18 equation with field data is used to evaluate the importance and relation of bed material properties on the maximum depth of scour. Furthermore, because laboratory research has shown a significant effect of bed forms on the depth of scour in ripple-forming sediments, the difference in the depth of scour in ripple- and nonripple-forming sediments is evaluated using field data.

The depth of scour in nonuniform sediments is often less than the depth of scour in uniform sediments. In uniform sediments, the energy is sufficient to transport the material (live bed) or it is insufficient to transport the material (clear water). In nonuniform sediments, the energy of the flow may only be sufficient to transport some material, allowing the coarser material to armor the bed. Armoring can occur in both the approach section (reducing the sediment transport to the scour hole) and in the scour hole (limiting the depth of the hole). The combination of armoring that occurs depends on the energy available for transport in the approach, the energy available for transport at the pier, and the gradation of the bed material. If the armoring occurs in the approach and no armoring occurs in the scour hole, the scour can be deeper than for identical conditions in uniform sediments because the armoring of the approach has reduced the sediment supply to the scour hole. If armoring of the scour hole occurs, the depth of scour is likely to be less than that for uniform sediments regardless of the transport condition at the approach (live bed or clear water). The traditional classification of live bed and clear water conditions is insufficient to describe the conditions that may occur in nonuniform bed material.

SUMMARY OF FIELD DATA

The 493 local pier scour measurements currently available in the BSDMS were filtered to ensure that the data used for this report met basic criteria necessary to complete the objectives of this analysis (for a complete listing of the data, see appendix A). The data collection techniques typically limited the data to cross sections along the upstream and downstream edges of the bridge. For a pier aligned with the flow, maximum scour typically occurs at the nose of the pier; therefore, the data collection method previously described measures maximum scour.

Laboratory research indicates that for a pier skewed to the flow, maximum scour can occur along the sides of the pier rather than at the nose.[8] Because data were seldom collected along the sides of the piers, all measurements where the flow was not aligned with the pier were removed from the data set. Where there are measurements along the upstream and downstream edges of the bridge, only the maximum depth of scour is used. Debris accumulations on the piers have an unknown effect on local scour and often make measurement of maximum scour impossible. The effect of debris on the depth of scour (unknown, insignificant, moderate, substantial) was noted for each measurement. All measurements where the effect of debris on the depth of scour was rated "substantial" were removed from the data set. The time required for scour to reach its maximum depth in cohesive material is considerably longer than in noncohesive material.[6] Therefore, observations with scour in cohesive material also were removed from this analysis.

The hydraulic parameters measured should be the conditions that caused the measured depth of scour. It is difficult to exactly associate hydraulics with a depth of scour because of the temporal development of the scour hole. Except at a few sites, the temporal development of the scour holes reported in BSDMS is not available. It was rationalized that if the scour hole can be reasonably associated with the reported hydraulic conditions, the velocity at the pier must be competent to erode the bed material. Gao et al. published the following equation to compute the critical approach velocity that results in transport of the bed material at the pier based on the critical velocity for incipient transport of the bed material:[3]

$$V_c' = 0.645\left(\frac{D_{50}}{b}\right)^{0.053} V_c \qquad (2)$$

where

V'_c is the approach velocity corresponding to critical velocity and incipient scour in the accelerated flow region at the pier;
D_{50} is the mean grain size of the bed material;
b is the pier width; and
V_c is the critical (incipient-transport) velocity for the D_{50}-size particle.

Equation 2 was used with Neill's formulation of the critical velocity equation: [23]

$$V_c = \theta^{1/2} K_u \, 31.08 y_o^{1/6} D_{50}^{1/3} \qquad (3)$$

where

θ is the Shield's parameter;
K_u is 1.0 for SI units and 1.81 for customary English units;
y_o is the depth of flow; and
D_{50} is the median grain size.

to compute the critical approach velocity (V'_c) for transport of the D_{50} grain size at the pier. All measurements having an approach velocity (V_o) less than the critical approach velocity for transport at the pier (V'_c) were removed from the data set.

The appropriate value for the Shield's parameter, θ, has been a topic of considerable research and discussion, with no conclusive answer. Miller et al.[49] and Buffington and Montgomery[50] compiled and analyzed all available data on incipient sediment transport. Both investigations found scatter in the data caused by inconsistencies in the definition of incipient motion, the experimental method utilized, the experimental facility used, and the type of bed material used. According to these studies, the Shield's parameter may vary from 0.02 to 0.086 with a common average value for gravel of about 0.046.[50] Miller et al.[49] presented a method based on Inman[51] that relates grain size to shear velocity; this method is only valid for water at a temperature of 20 °C and for bed material with a specific gravity of 2.65. The method was presented graphically, but has been reduced to equations for the Shield's parameter by Mueller.[5]

$$\theta = 0.0019 D_{50}^{-0.384} \quad \text{for } D_{50} < 0.0009 \text{ (meter)} \tag{4}$$

$$\theta = 0.0942 D_{50}^{0.175} \quad \text{for } 0.0009 \text{ (meter)} < D_{50} < 0.020 \text{ (meter)} \tag{5}$$

$$\theta = 0.047 \quad \text{for } D_{50} > 0.020 \text{ (meter)} \tag{6}$$

The method is easily applied, provides for variation in the Shield's parameter for smaller grain sizes, and is within the range of variation defined by previous research; therefore, this method was used to evaluate the Shield's parameter needed to estimate the critical velocity for incipient sediment transport.

Of the 493 pier scour measurements in the BSDMS, 266 were selected for this analysis. The criterion that flow be aligned with the pier was satisfied by 322 measurements; 476 measurements satisfied the criterion that debris not be a substantial effect; 488 measurements were in noncohesive material; and 437 measurements satisfied the criterion that the velocity at the pier has to be competent to erode the median particle size of the bed material. Where there were measurements both at the upstream and downstream edges of the bridge, only a single measurement representing the maximum depth of scour was included; thus, 41 measurements were excluded. All criteria were satisfied by 266 measurements, which represent 106 different piers at 53 bridges located in 15 States.

A summary of the selected data and commonly used dimensionless variables is provided in table 2. The maximum and minimum values of the data and of the dimensionless variables represent a range equal to or greater than most laboratory investigations. Unlike laboratory investigations, the distribution of the data cannot be precisely controlled in the field, and the data tend to be grouped near the low end of most of the primary variables.

Table 2. Summary statistics for selected pier scour measurements.

Variable	Units	Number of Points	Minimum	25th Quartile	Median	75th Quartile	Maximum	Mean	Standard Deviation	Coefficient of Variation	Skewness
Depth of scour (y_s)	m	266	0.00	0.43	0.61	1.19	7.65	1.06	1.19	1.13	2.68
Pier width (b)	m	266	0.38	0.88	1.13	1.52	5.52	1.53	1.11	0.73	1.88
Approach velocity (V_o)	m	266	0.18	0.66	1.14	1.89	4.48	1.36	0.90	0.66	0.94
Approach depth (y_o)	m	266	0.12	2.45	4.39	6.45	20.03	4.86	3.24	0.67	1.18
D_{16}	mm	262	0.03	0.20	0.35	3.78	68.00	5.48	13.26	2.42	3.45
D_{50}	mm	266	0.15	0.48	0.74	8.00	108.00	10.61	20.74	1.95	2.75
D_{84}	mm	262	0.26	1.30	2.35	29.00	233.00	22.36	39.71	1.78	2.91
D_{95}	mm	262	0.28	2.08	7.45	44.00	350.00	34.87	58.85	1.69	2.78
Gradation coefficient (σ_g)	–	262	1.20	2.03	2.30	3.65	21.80	3.21	2.27	0.71	3.17
Drainage area (DA)	km^2	192	166	1197	3680	9402	1805222	32706	135836	4.15	11.80
Slope (S)	m/m	219	0.00010	0.00016	0.00050	0.00105	0.00500	0.00090	0.00103	1.14	1.96
y_o / b	–	266	0.12	1.87	3.05	5.31	13.84	3.91	2.81	0.72	1.21
b / D_{50}	–	266	8.47	129.54	1024.54	1828.80	14224.00	2219.03	3401.47	1.53	2.33
y_s / b	–	266	0.00	0.40	0.58	0.88	2.09	0.68	0.41	0.61	1.01
V_o / V_c	–	266	0.43	0.75	1.14	1.53	4.92	1.32	0.84	0.64	2.15
$V_o / (gy_o)^{0.5}$	–	266	0.04	0.11	0.17	0.33	0.83	0.23	0.16	0.70	1.17

EVALUATION OF PUBLISHED EQUATIONS

Discussion of Equations

Local pier scour has been a popular topic of study for many laboratory researchers. A literature review by McIntosh found that more than 35 equations had been proposed for predicting the depth of scour at a bridge pier.[52] Most local scour equations are based on research in laboratory flumes with noncohesive, uniform bed material and limited verification of results with field data.[52] In evaluating and applying scour-prediction equations, it is valuable to know the limitations of the equations, the conditions for which they were developed, how the underlying data were interpreted, and the methods used to develop the equations. Such information about each equation has been previously published in Landers and Mueller,[21] Mueller,[5] and Pritsivelis.[53] The equation, published reference, and the equation name for the 26 equations used in this report are presented in table 3.

The equations presented here were developed to predict the maximum depth of scour; there are three approaches to developing such an equation. The first is to predict the maximum depth of scour that could occur at the bridge pier under any conditions. The second is to predict, as accurately as possible, the maximum depth of scour for a given set of hydraulic and bed material conditions. These equations often are developed by multiple regression analysis and, by definition, underpredict the depth of scour for about one-half of the observations used in the equation development. The third approach is to develop a design equation. A good design equation should accurately predict the depth of scour for a given set of site and flood conditions, but when in error should always err by predicting too much scour.

Analysis of how each equation addresses pier width, approach velocity, approach depth, and bed material properties provides an indication of the effect of these variables on the depth of scour. The selected equations are formulated into two patterns. The regime equations are written in the form:

$$y_s + y_o = f\left(b, V_o, y_o, D_{50}\right) \tag{7}$$

and compute the total depth of flow including local scour. Nonregime equations are written in the form:

$$y_s = f\left(b, V_o, y_o, D_{50}\right) \tag{8}$$

and compute the depth of local scour only. The pier width is included in more than 75 percent of the equations (table 4). The regime equations have an exponent on pier width between 0.2 and 0.25. The exponent on pier width ranges from 0.6 to 0.75 in more than one-half of the nonregime equations when the pier width could be isolated. The smaller exponents on pier width for the regime equations are justified because pier width should have less effect on the total depth than on the depth of local pier scour. The exponents on approach velocity range from 0.2 to 0.68 and on approach depth from 0.135 to 0.8. This variability indicates that there is a lack of agreement among the equations on the effect of approach depth and velocity on the depth of scour. The median grain size is only included in 11 equations; it can only be isolated in 4 equations, where it has a small negative exponent.

Table 3. Summary of selected local pier scour equations.*

Name and Reference Number	**Equation**	**Equation Number**
Ahmad[54]	$y_s = K V_o^{2/3} y_o^{2/3} - y_o$	(9)
Arkansas[55]**	$y_s = 0.827 D_{50}^{-0.117} V_o^{0.684} e^{0.476(c_1)}$	(10)
Blench-Inglis I[56]**	$y_s = 1.8 b^{0.25} y_o^{0.75} - y_o$	(11)
Blench-Inglis II[56]***	$y_s = 1.53 b^{0.25} V_o^{0.5} y_o^{0.5} D_{50}^{-0.125} - y_o$	(12)
Breusers[57]	$y_s = 1.4 b$	(13)
Breusers-Hancu[53]	$y_s = b f\left(K_1 K_2 2\tanh\left(\frac{y_o}{b}\right)\right)$	(14)
Chitale[58]	$y_s = y_o\left(-5.49 F_o^2 + 6.65 F_o - 0.51\right)$	(15)
Froehlich[13]	$y_s = 0.32 \phi g^{-0.1} V_o^{0.2} y_o^{0.36} b^{0.62} D_{50}^{-0.08}$	(16)
Froehlich Design[13]	$y_s = 0.32 \phi g^{-0.1} V_o^{0.2} y_o^{0.36} b^{0.62} D_{50}^{-0.08} + b$	(17)
HEC-18[59]	$y_s = 2.0 K_1 K_2 K_3 g^{-0.215} y_o^{0.135} b^{0.65} V_o^{0.43}$	(18)
HEC-18-K4[6]	$y_s = 2.0 K_1 K_2 K_3 K_4 g^{-0.215} y_o^{0.135} b^{0.65} V_o^{0.43}$ $K_4 = \left(1 - 0.89 (1 - V_R)^2\right)^{0.5}$ $V_R = \frac{V_o - V_c'}{V_{c90} - V_c'}$	(19)
HEC-18-K4Mo[1]	$y_s = 2.0 K_1 K_2 K_3 K_4 K_i g^{-0.215} y_o^{0.135} b^{0.65} V_o^{0.43}$ $K_4 = 1.25 + 3\sqrt{\frac{D_{CFM}}{D_{50}}}\left(\frac{V_o - V_i}{V_{cm} - V_i}\right)^{0.60} ln\left(\left(\frac{V_o - V_i}{V_{cm} - V_i}\right) + 0.5\right)$ $K_i = \left(1 - \frac{V_i}{V_o}\right)^{0.45}$ $V_{cm} = 6.625 D_{CFM}^{1/3} y_o^{1/6}$ $V_i = 2.65 D_{35}^{1/3} y_o^{1/6}$ $D_{CFM} = \frac{D_{85} + 2D_{90} + 2D_{95} + D_{99}}{6}$	(20)

Table 3. Summary of selected local pier scour equations, continued.

Name and Reference Number	Equation	Equation Number
HEC-18-K4Mu[5]	$y_s = 2.0\ K_1\ K_2\ K_3\ K_4\ g^{-0.215}\ y_o^{0.135}\ b^{0.65}\ V_o^{0.43}$ $K_4 = 0.4\left(\frac{V_o - V_c'}{V_c - V_{c95}'}\right)^{0.15}$	(21)
Inglis-Poona I[34]**	$y_s = 1.7\, b^{0.22}\ V_o^{0.52}\ y_o^{0.52} - y_o$	(22)
Inglis-Poona II[34]	$y_s = 1.73\, b^{0.22}\ y_o^{0.78} - y_o$	(23)
Larras[4]**	$y_s = 1.42\ K_{s2}\ b^{0.75}$	(24)
Laursen I[60]	$y_s = 1.5\, b^{0.7} y_o^{0.3}$	(25)
Laursen II[61]	$\frac{b}{y_o} = 5.5\left(\frac{y_s}{y_o}\right)\left(\left[\left(\frac{1}{11.5}\right)\left(\frac{y_s}{y_o}\right)+1\right]^{1.70} - 1\right)$	(26)
Laursen-Callander[62]	$y_s = 1.11\, y_o^{0.5} b^{0.5}$	(27)
Melville and Sutherland[2]	$y_s = K_I\ K_d\ K_y\ K_{\alpha L}\ K_s\ b$	(28)
Mississippi[63]	$y_s = 0.9\ b_e^{0.6} y_o^{0.4}$	(29)
Molinas[1]	$y_s = 0.99\ K_1\ K_2\ K_3\ K_4\left(\frac{V_o - V_i}{V_{cm} - V_i}\right)^{0.55} b^{0.66} y_o^{0.17}$ $K_4 = 1.25 + 3\sqrt{\frac{D_{CFM}}{D_{50}}}\left(\frac{V_o - V_i}{V_{cm} - V_i}\right)^{0.60} ln\left(\left(\frac{V_o - V_i}{V_{cm} - V_i}\right) + 0.5\right)$	(30)
Shen[9]**	$y_s = 0.00073\,\nu^{-0.619}\ V_o^{0.619}\ b^{0.619}$	(31)
Shen-Maza[9]	$y_s = 11.0\ g^{-1}\ V_o^2$ for $F_p \le 0.2$ $y_s = 3.4\ g^{-0.33}\ b^{0.67}\ V_o^{0.67}$ for $F_p > 0.2$	(32)
Sheppard[64] as reported by Alkhalidi[65]	$y_s = b\, 1.5\, k\left[2.5\left(\frac{V_o}{V_c}\right) - 1\right]$ for $0.4 \le \frac{V_o}{V_c} \le 1.0$ $y_s = b\, c_2\left(\frac{V_{LP} - V_o}{V_c}\right) + c_3$ for $1.0 < \frac{V_o}{V_c} \le \frac{V_{LP}}{V_c}$ $y_s = b\ 2.4\ tanh\left(\frac{y_o}{b}\right)$ for $\frac{V_o}{V_c} > \frac{V_{LP}}{V_c}$	(33)

Table 3. Summary of selected local pier scour equations, continued.

Name and Reference Number	Equation	Equation Number
c_2	is a coefficient defined by Sheppard as $$\frac{k - 2.4\, tanh\left(1.775\left(\frac{y_o}{b}\right)^{0.618}\right)}{\left(\frac{V_{LP}}{V_c} - 1\right)}$$ where: tanh is the hyperbolic tangent function	(34)
c_3	is a coefficient defined by Sheppard as $$2.4\, tanh\left(1.775\left(\frac{y_o}{b}\right)^{0.618}\right)$$	(35)
k	is a coefficient defined by Sheppard as $$\frac{1.1\, tanh\left(1.775\left(\frac{y_o}{b}\right)^{0.618}\right)}{\left(-0.025 + 0.233\, exp\left(0.334\, log_{10}\left(\frac{b}{D_{50}}\right)\right) + \frac{3.098}{exp\left(2.411\, log_{10}\left(\frac{b}{D_{50}}\right)\right)}\right)}$$	(36)
Simplified Chinese(3)	$$y_s = 0.46\, K_{sc}\, b^{0.6}\, y_o^{0.15}\, D_m^{-0.07}\left(\frac{V_o - V_c'}{V_c - V_c'}\right)^n \quad \text{for live bed,}$$ where $$n = \left(\frac{V_c}{V_o}\right)^{9.35 - 2.23\, lg\, b} \quad \text{and } lg \text{ is } log_{10}$$ $$y_s = 0.78\, K_{sc}\, b^{0.6}\, y_o^{0.15}\, D_m^{-0.07}\left(\frac{V_o - V_c'}{V_c - V_c'}\right) \quad \text{for clear water}$$	(37)

*See List of Symbols in the front of the report for definition of symbols not defined in the table.
**Units are English units in feet.
***Units are English units in feet, except D_{50}, which is in millimeters.

Table 4. Summary of exponents for variables used in selected equations.

Equation	Pier Width	Approach Velocity	Approach Depth	D_{50}	Other Bed Material
Ahmad	–	0.667	0.667	–	–
Arkansas	–	0.684	–	-0.117	–
Blench-Inglis I*	0.25	–	0.75	–	–
Blench-Inglis II*	0.25	0.5	0.5	-0.125	–
Breusers	1.0	–	–	–	–
Breusers-Hancu	X	–	X	–	–
Chitale	–	X	X	–	–
Froehlich	0.62	0.2	0.36	-0.08	–
Froehlich Design	X	0.2	0.36	-0.08	–
HEC-18	0.65	0.43	0.135	–	–
HEC-18-K4	X	X	X	X	X
HEC-18-K4Mo	X	X	X	X	X
HEC-18-K4Mu	X	X	X	X	X
Inglis-Poona I*	0.22	0.52	X	–	–
Inglis-Poona II*	0.22	–	X	–	–
Larras	0.75	–	–	–	–
Laursen I	0.7	–	0.3	–	–
Laursen II	X	–	X	–	–
Laursen-Callander	0.5	–	0.5	–	–
Melville and Sutherland	X	X	X	X	X
Mississippi	0.6	–	0.4	–	–
Molinas	0.66	X	X	X	X
Shen	0.62	0.62	–	–	–
Shen-Maza F_p<0.2	–	2.0	–	–	–
Shen-Maza F_p>0.2	0.67	0.67	–	–	–
Sheppard	X	X	X	X	–
Simplified Chinese	X	X	X	X	–

*Regime equation that in its original form computed total depth including pier scour and approach depth (see equation 7).
X—Equation uses this variable, but the equation is complex, and this variable cannot be algebraically isolated.

Assumptions

Laboratory experiments are designed to isolate specific scour processes; thus, the resulting equations may not account for complex and dynamic field conditions. Some field conditions that effect scour are undefined in the selected equations and assumptions are required to apply the equations. The flow in the field is assumed to be steady state and uniform to allow the application of laboratory-based equations to predict scour at bridges. All equations presented here are used to estimate scour for both live bed and clear water conditions.

Many equations do not include corrections for pier shape, or they include corrections for only a few pier shapes. Pile groups are classified as round-nose or circular piers for equations that do not specify a shape correction for pile groups.[59] However, pile groups are treated as cylinders for the Simplified Chinese equation. Blench-Inglis I and II, Inglis-Poona I and II, Chitale, Ahmad, Shen, Arkansas, Breusers, and Mississippi equations do not include procedures to correct for pier shape, and corrections were not applied in this evaluation. Because the Larras equation specifies only square-nose and circular pier shapes, sharp-nose piers and pile groups are classified as circular piers in this evaluation of the Larras equation.

When it was possible, each equation was used to compute a depth pier scour for each field measurement. Several sites did not have adequate bed material data. The D_{90} grain size was interpolated from the D_{84} and D_{95} grain sizes using log-probability interpolation. The D_{99} was estimated by extrapolating the D_{84} and D_{95} values in log-probability space. If neither D_{84} nor D_{95} were reported, equations requiring D_{90} and D_{99} were not applied. A water temperature of 20 °C was assumed if no temperature was reported.

Comparison of Predictions with Field Data

This evaluation of the selected equations focuses primarily on the ability of the equations to be used as design equations for different site and flood conditions. The objective is to find an equation that accurately predicts the depth of scour for the specified conditions, but that overestimates the depth of scour when in error. The potential causes for significant inaccuracies or under-predictions are also investigated.

The ability of the equations to accurately predict the depth of scour for the variety of field conditions represented in this data set varies greatly (figure 7). Some equations (Ahmad, Breusers-Hancu, Chitale, Inglis-Poona I, Melville and Sutherland, and Shen-Maza) show trends away from the line of equality, indicating that those equations do not properly represent the processes responsible for local pier scour that occur in the field. Several equations (Arkansas, Blench-Inglis I, Blench-Inglis II, Froehlich, Shen, and Simplified Chinese) underpredict the depth of scour for a significant number of the observations and are not good candidates for design equations. The other equations have some trend along the line of equality with few underpredictions, but they display a broad scatter of data and often do not accurately predict the observed scour.

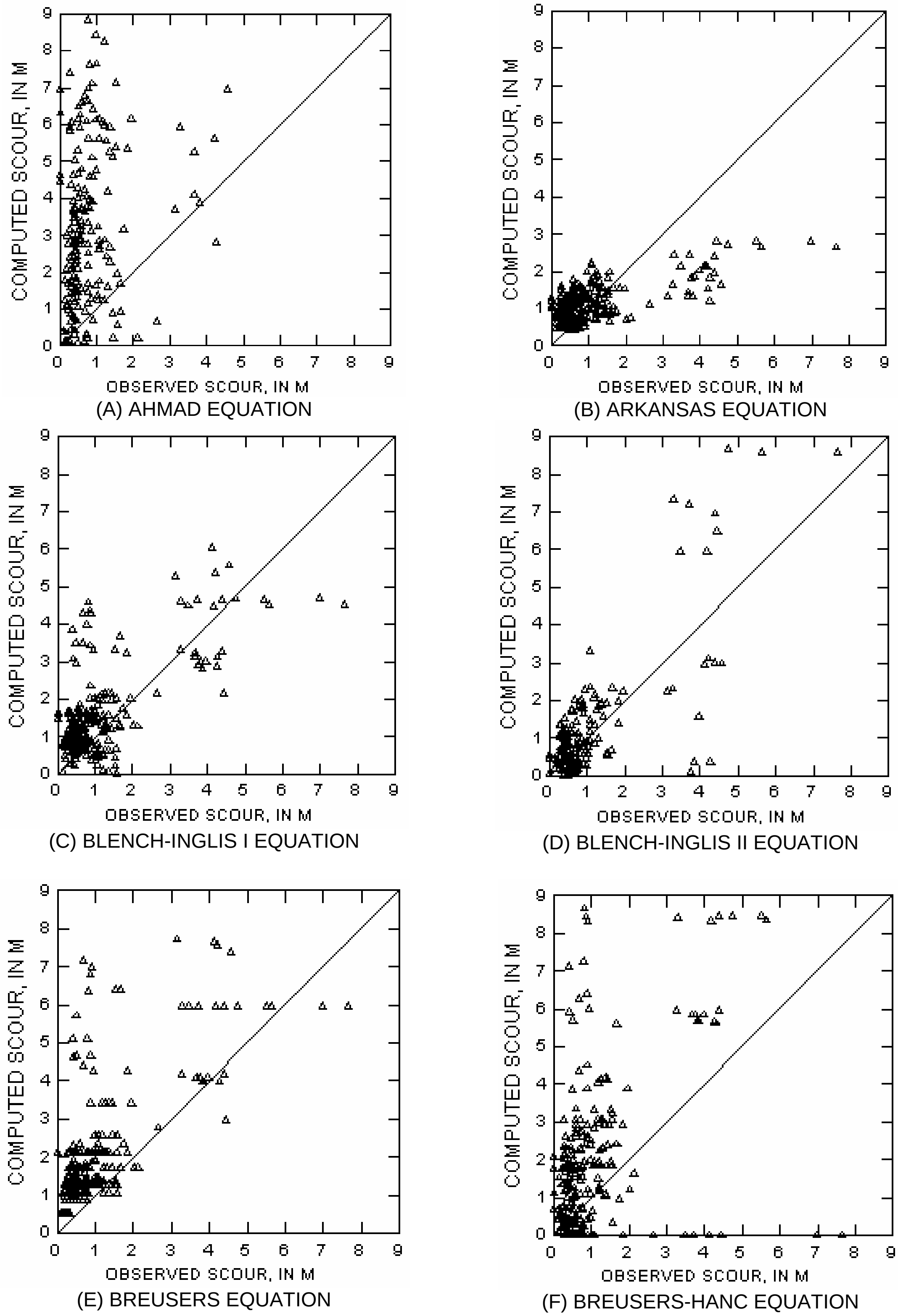


Figure 7. Scatterplots of computed versus observed scour, in meters (m), for selected pier scour equations.

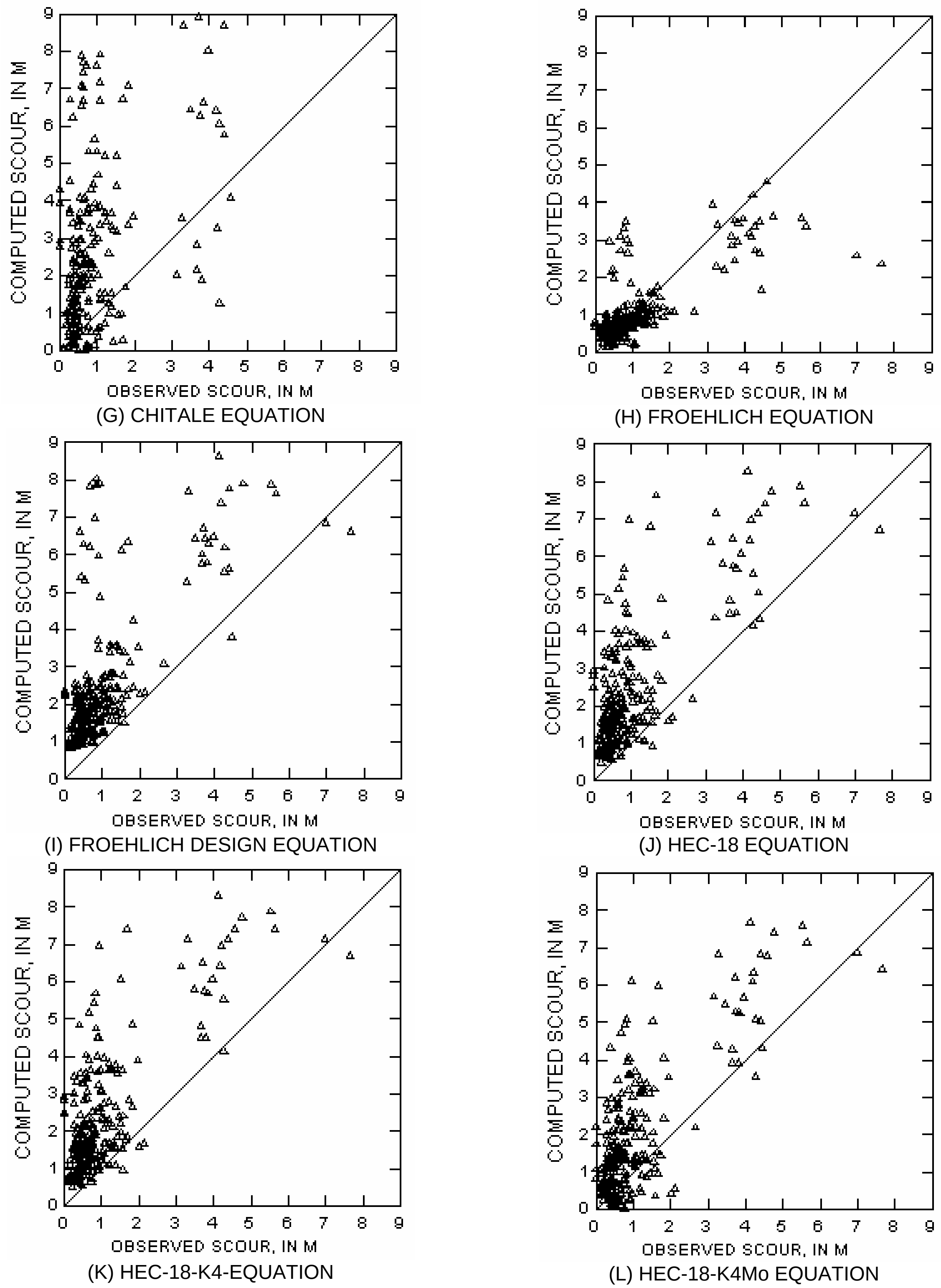


Figure 7. Scatterplots of computed versus observed scour, in meters (m), for selected pier scour equations, continued.

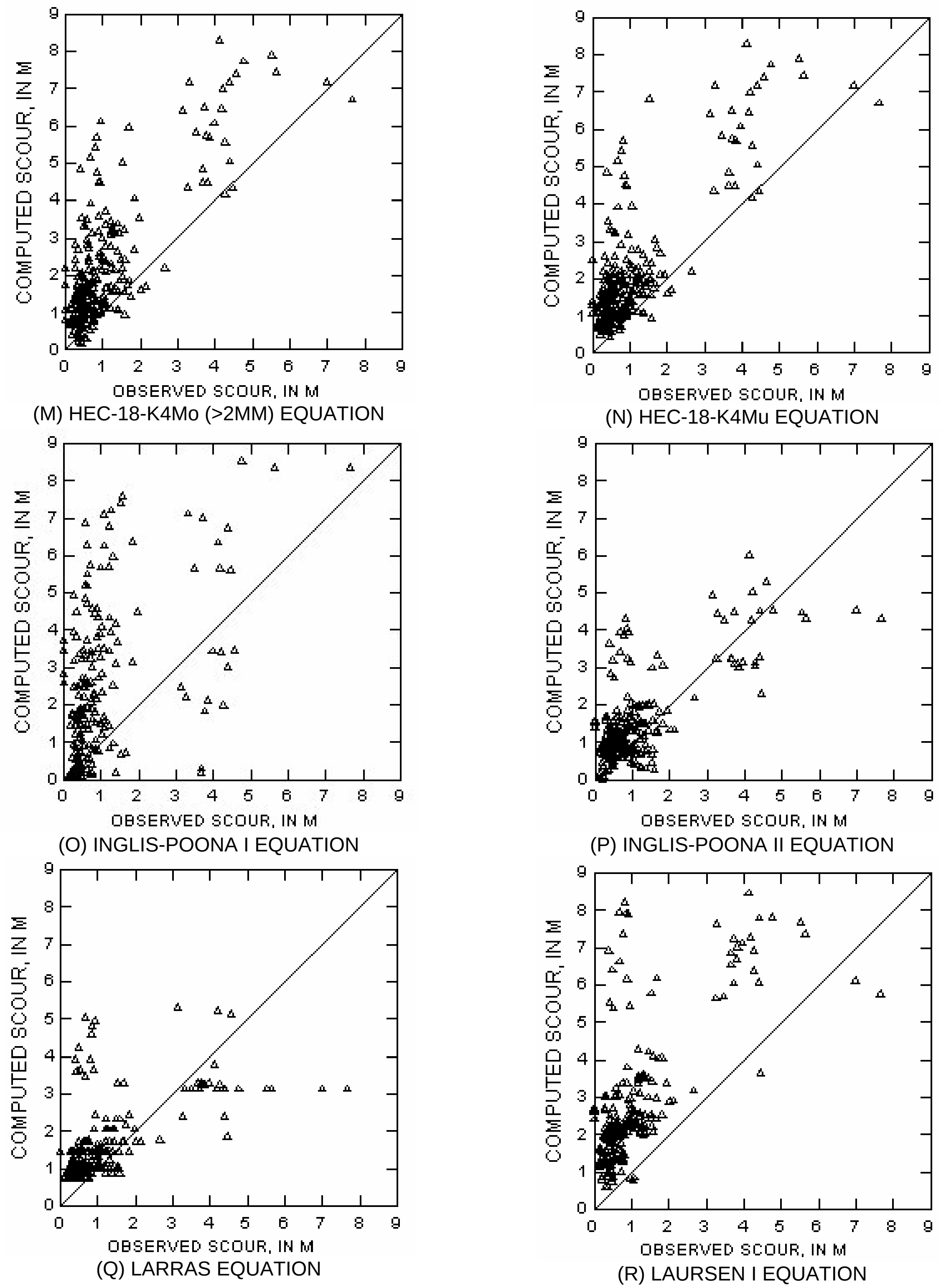


Figure 7. Satterplots of computed versus observed scour, in meters (m), for selected pier scour equations, continued.

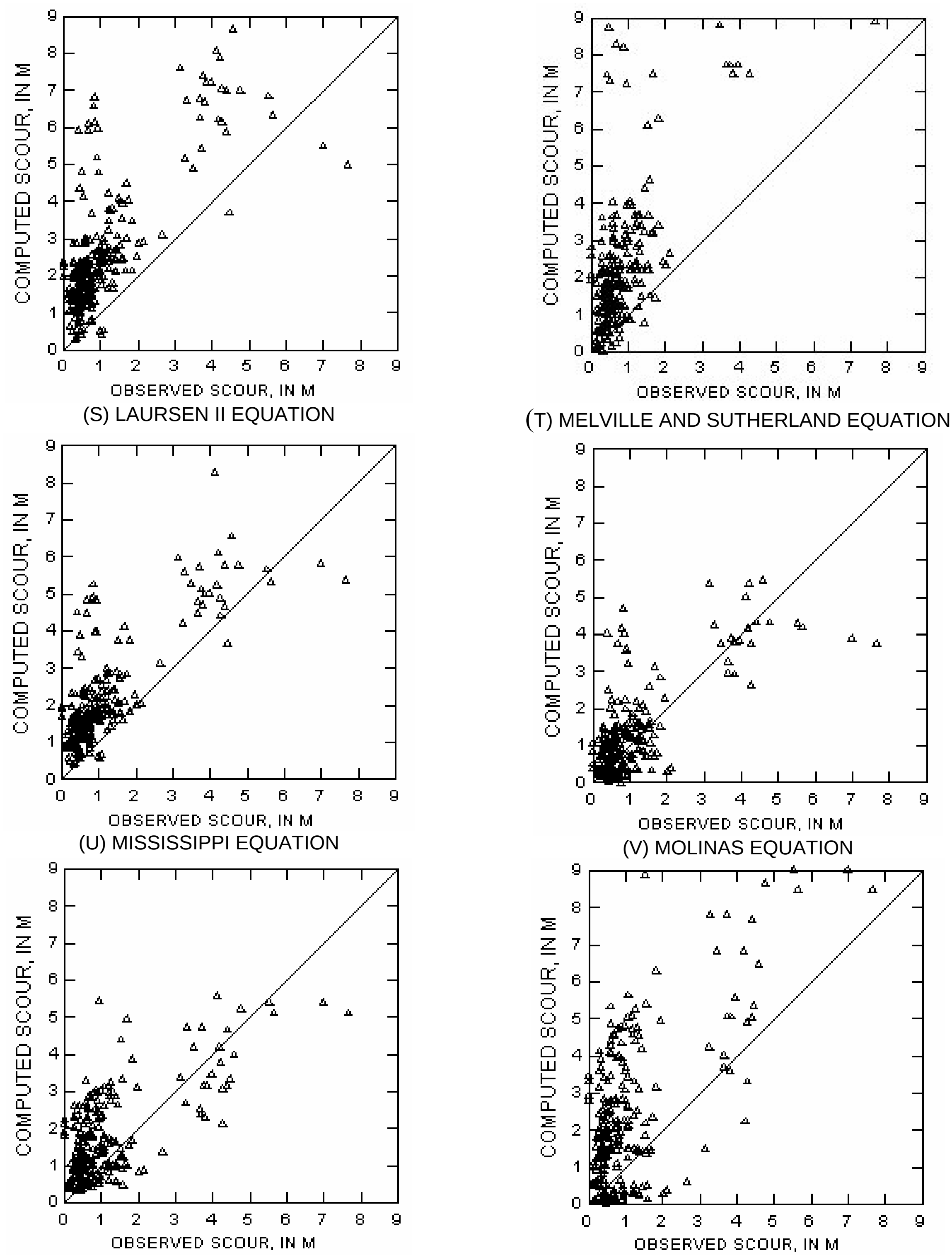
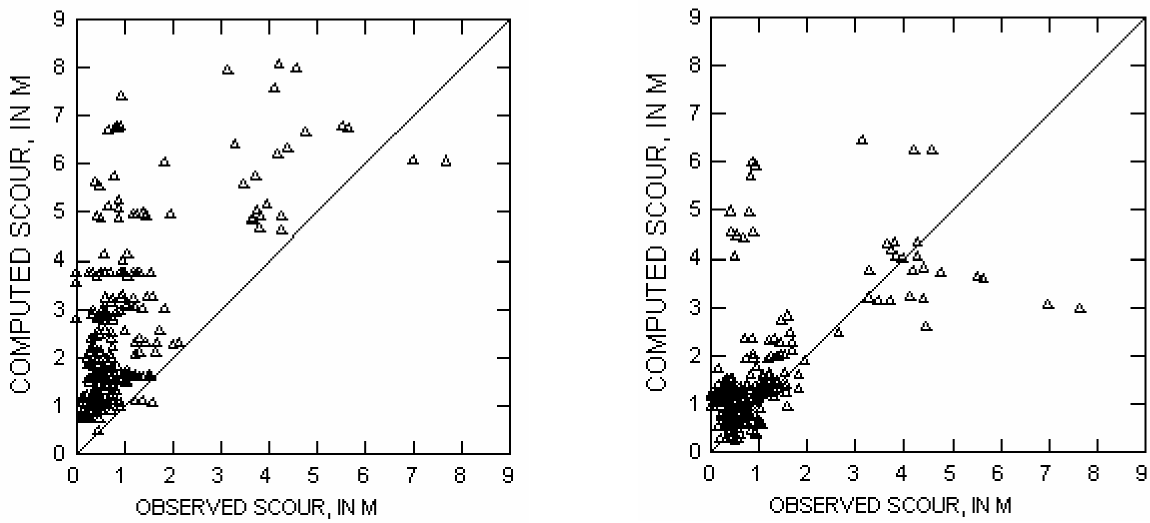


Figure 7. Scatterplots of computed versus observed scour, in meters (m), for selected pier scour equations, continued.



(Y) SHEPPARD EQUATION

(Z) SIMPLIFIED CHINESE EQUATION

Figure 7. Scatterplots of computed versus observed scour, in meters (m), for selected pier scour equations, continued.

Ranking the performance of scour-prediction equations is difficult because of the tradeoff between accuracy and underpredictions. If only accuracy is considered, the sum of squared errors can be used to evaluate the equations' performance (table 5). This statistic shows the Froehlich equation to be the most accurate; however, the Froehlich is a regression expression and underpredicted the depth of scour for 129 of 266 field observations. If the smallest number of underpredictions is used to evaluate the equations, the Froehlich Design equation is the best equation because it underestimated only four observations. The Froehlich Design equation, however, ranked 19th based on the sum of squared errors criteria. The magnitude of the underpredictions is just as important, if not more so, than the number of underpredictions; thus, the sum of squared errors for those observations that were underpredicted is another factor that should be considered. The Melville and Sutherland equation had the lowest sum of squared errors for the underpredicted observations, but this equation ranked 26th in overall sum of squared errors. The Melville and Sutherland equation did not underestimate scour by much, but grossly overestimated scour for many cases (figure 7T). The Froehlich Design, HEC-18-K4, HEC-18, HEC-18-K4Mu, and HEC-18-K4Mo (>2 mm) equations all had low sum of squared errors for the underpredicted observations. If the all the ranks are totaled, the Froehlich Design equation appears to be the top equation, followed by the HEC-18-K4Mu, HEC-18-K4, HEC-18, Mississippi, and HEC-18-K4Mo (>2 mm) equations; however, the Froehlich Design equation had the largest sum of squared errors for this group. If only the ranks based on the two sum of squared error categories are used, the HEC-18-K4Mu equation is favored and the Froehlich Design equation drops to a rank of 8.5. No single equation is conclusively better than the rest, but the top six equations generally appear to be the Froehlich Design, HEC-18-K4, HEC-18-K4Mu, HEC-18-K4Mo (>2 mm), Mississippi, and HEC-18 equations.

Table 5. Summary of the performance of the selected pier scour equations.

				Number of Underpredictions				Summation of Ranks			
				Count		SSE		All Ranks		SSE Ranks	
Equation	Number of Observations	SSE Magnitude	Rank	Number	Rank	Magnitude	Rank	Total	Rank	Total	Rank
Ahmad	266	7536.86	27	61	14	159.48	22	63	23	49	25.5
Arkansas	266	239.52	4	74	20.5	165.61	23	47.5	20	27	16
Blench-Inglis I	266	265.83	5	74	20.5	52.14	17	42.5	18	22	11
Blench-Inglis II	266	954.55	17	174	27	824.60	27	71	25	44	23
Breusers	266	670.40	13	18	9.5	7.14	9	31.5	7.5	22	11
Breusers-Hancu	266	1205.60	21	77	22	201.18	25	68	24	46	24
Chitale	266	2299.40	25	90	23	169.37	24	72	26	49	25.5
Froehlich	266	160.67	1	129	26	98.24	21	48	21	22	11
Froehlich Design	266	1067.77	19	4	1	1.51	2	22	1	21	8.5
HEC-18	266	822.38	15	13	7	2.16	4	26	4.5	19	4.5
HEC-18-K4	262	791.54	14	15	8	1.93	3	25	3	17	2
HEC-18-KMO (All)	266	495.18	11	65	16	17.01	13	40	15.5	24	13
HEC-18-KMO (> 2 mm)	266	608.79	12	21	11	2.47	6	29	6	18	3
HEC-18-K4Mu	266	448.53	9	18	9.5	2.23	5	23.5	2	14	1
Inglis-Poona I	266	1758.81	24	119	25	597.74	26	75	27	50	27
Inglis-Poona II	266	229.68	3	72	19	45.67	16	38	12	19	4.5
Larras	266	311.13	7	48	13	72.09	20	40	15.5	27	16
Laursen I	266	1277.71	23	6	2	5.20	8	33	10	31	21
Laursen II	266	930.57	16	9	3.5	10.95	12	31.5	7.5	28	18
Laursen-Callander	266	960.55	18	9	3.5	10.39	11	32.5	9	29	19.5
Melville and Sutherland	262	3092.08	26	28	12	1.45	1	39	13.5	27	14
Mississippi	266	465.05	10	12	6	7.90	10	26	4.5	20	6
Molinas	262	199.79	2	103	24	55.96	18	44	19	20	7
Shen	266	300.77	6	69	18	37.00	15	39	13.5	21	8.5
Shen-Maza	266	1133.23	20	67	17	36.90	14	51	22	34	22
Sheppard	262	1276.04	22	11	5	3.89	7	34	11	29	19.5
Simplified Chinese	254	344.46	8	62	15	56.21	19	42	17	27	16

SSE—sum of squared errors

Sheppard's equations have been revised several times as reported by Alkhalidi, Sheppard et al., and Sheppard.(65,66,67) The 2001 version of the equation as listed in table 5 was used for comparisons in their analysis because it was the version provided by Sheppard to the author at the time the analysis was conducted. That version of the Sheppard equations ranked 19th based on the two sum of squared error categories.

Since no single equation was superior to the others and none of the equations accurately predicted the scour for all conditions, it is important to assess where the equations failed. Residuals of selected equations were plotted against Froude number ($V_o/(gy_o)^{0.5}$), relative velocity (V_c/V_o), median grain size (D_{50}), pier width (b), relative bed material size (b/D_{50}), and relative depth (y_o/b) to assess where the equations may fail to properly account for the scour processes. Figure 8 shows that the Froehlich equation, which is a regression equation, has no significant patterns; it fits the data reasonably well. However, to convert the Froehlich equation from a regression equation to a design equation, Froehlich added the pier width as a factor of safety. The factor of safety increases the scatter in the data significantly. The plot of residuals versus pier width shows that factor of safety becomes too large as the pier width increases (figure 9). The HEC-18-K4 equation shows patterns of increasing overprediction as Froude number (0–0.4), median grain size, and pier width increase (figure 10). The K4, proposed by Mueller, reduces the effect of the Froude number and median grain size, but patterns are still evident in the pier width (figure 11).(5) Only pier width displays a pattern in the residuals of the Mississippi equation (figure 12). The revised HEC-18 equation, HEC-18-K4Mo, also shows patterns in the residuals with Froude number and median grain size, and the most dominant pattern is the bottom envelope on the pier width (figure 13).(1) Most underpredictions seem to occur for grain sizes less than 2 mm. Table 5 shows that two thirds of the underpredictions by HEC-18-K4Mo occur at grain sizes less than 2 mm. Thus, limiting the K_i and K_4 corrections to grain sizes greater than 2 mm improves the performance of the Molinas correction.

Although many equations have been proposed for predicting the depth of local pier scour, no equation presented here accurately predicts the depth of scour for the wide variety of conditions represented in the field data set. Six equations are identified as being better than the others when assessed for their value as design equations; however, even these display patterns in their residuals that indicate they are not properly accounting for the scour processes. Additional research and analysis are needed to develop a better local pier scour prediction equation.

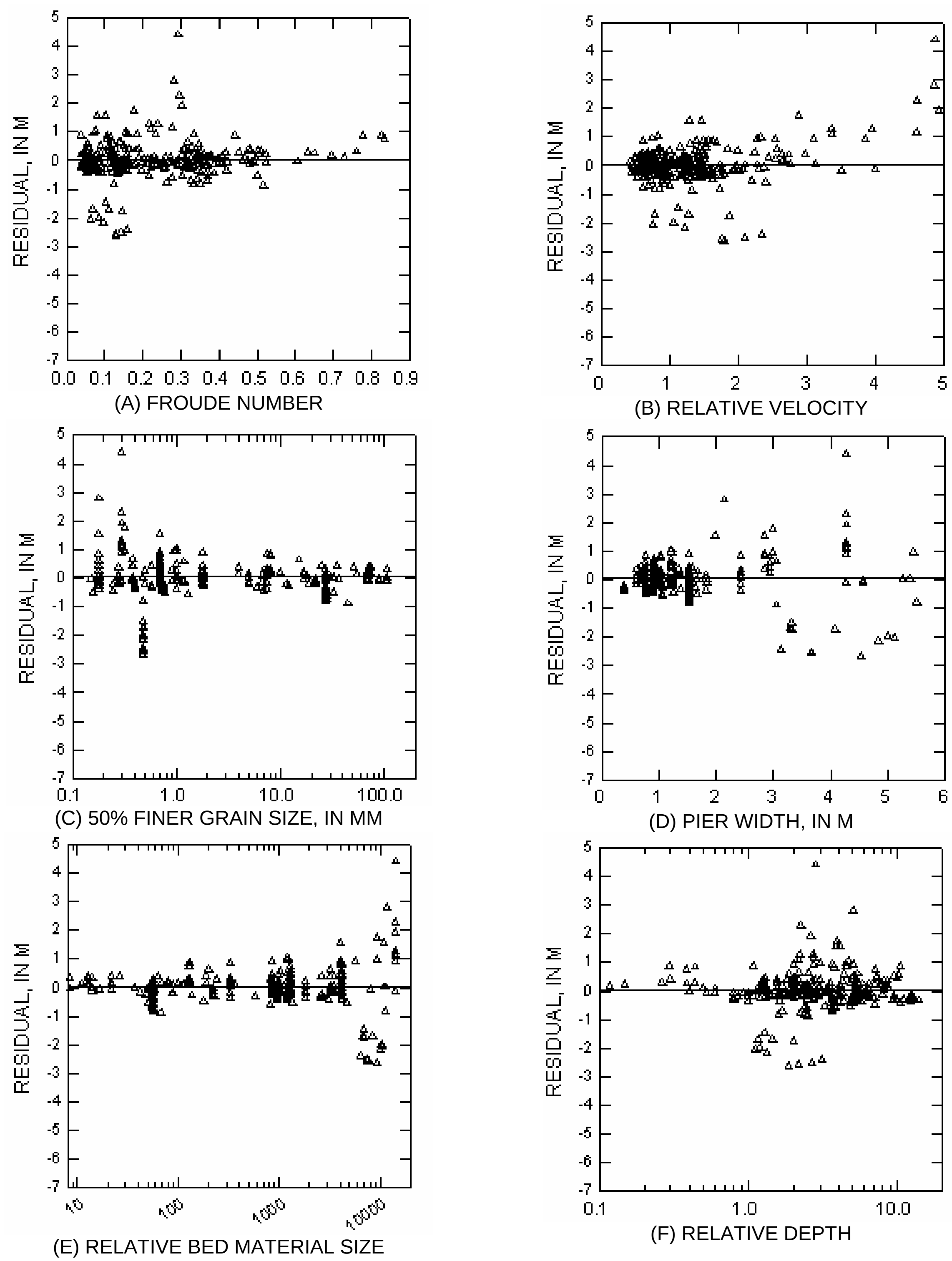


Figure 8. Evaluation of residuals for the Froehlich equation.

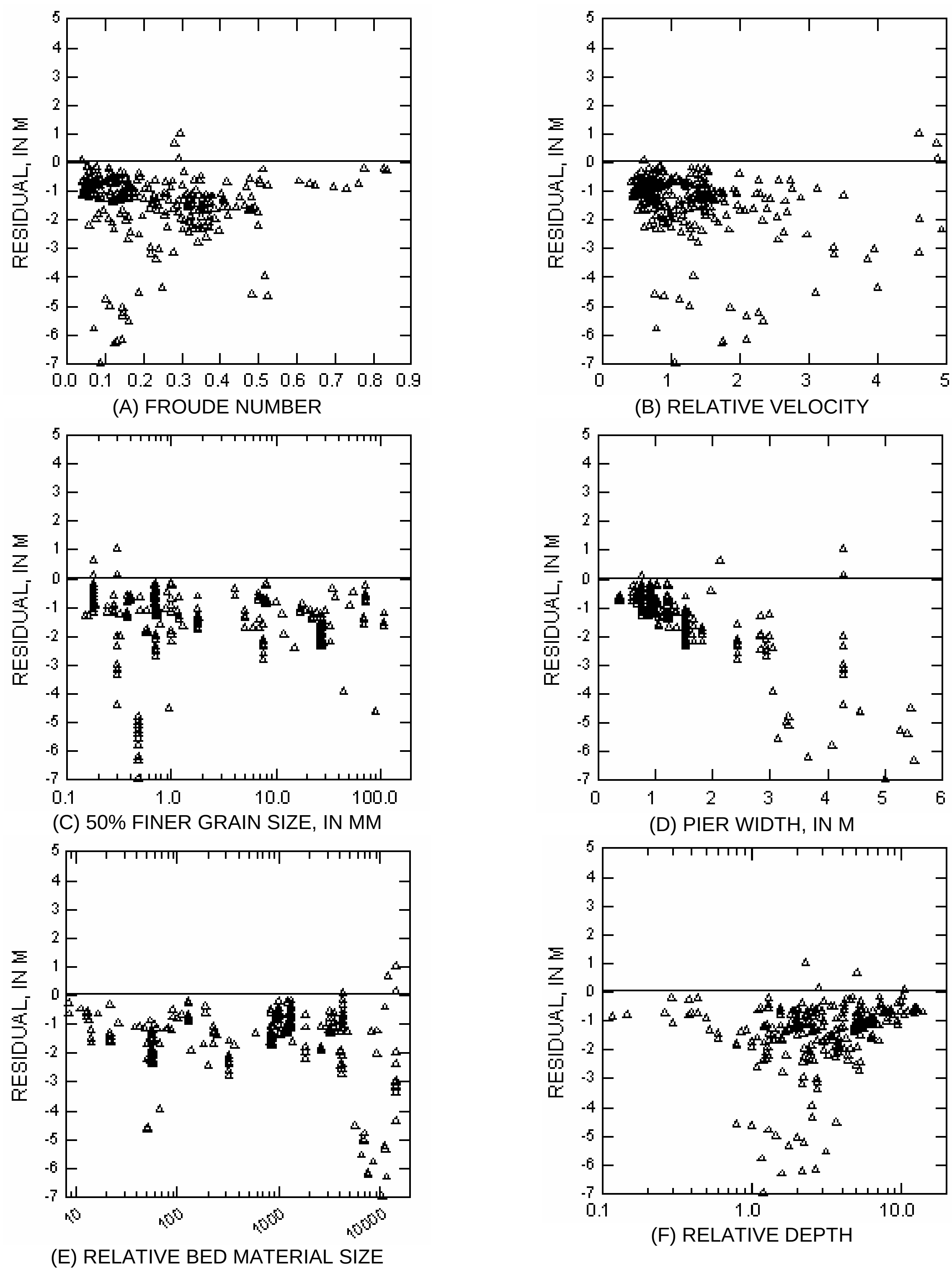


Figure 9. Evaluation of residuals for the Froehlich design equation.

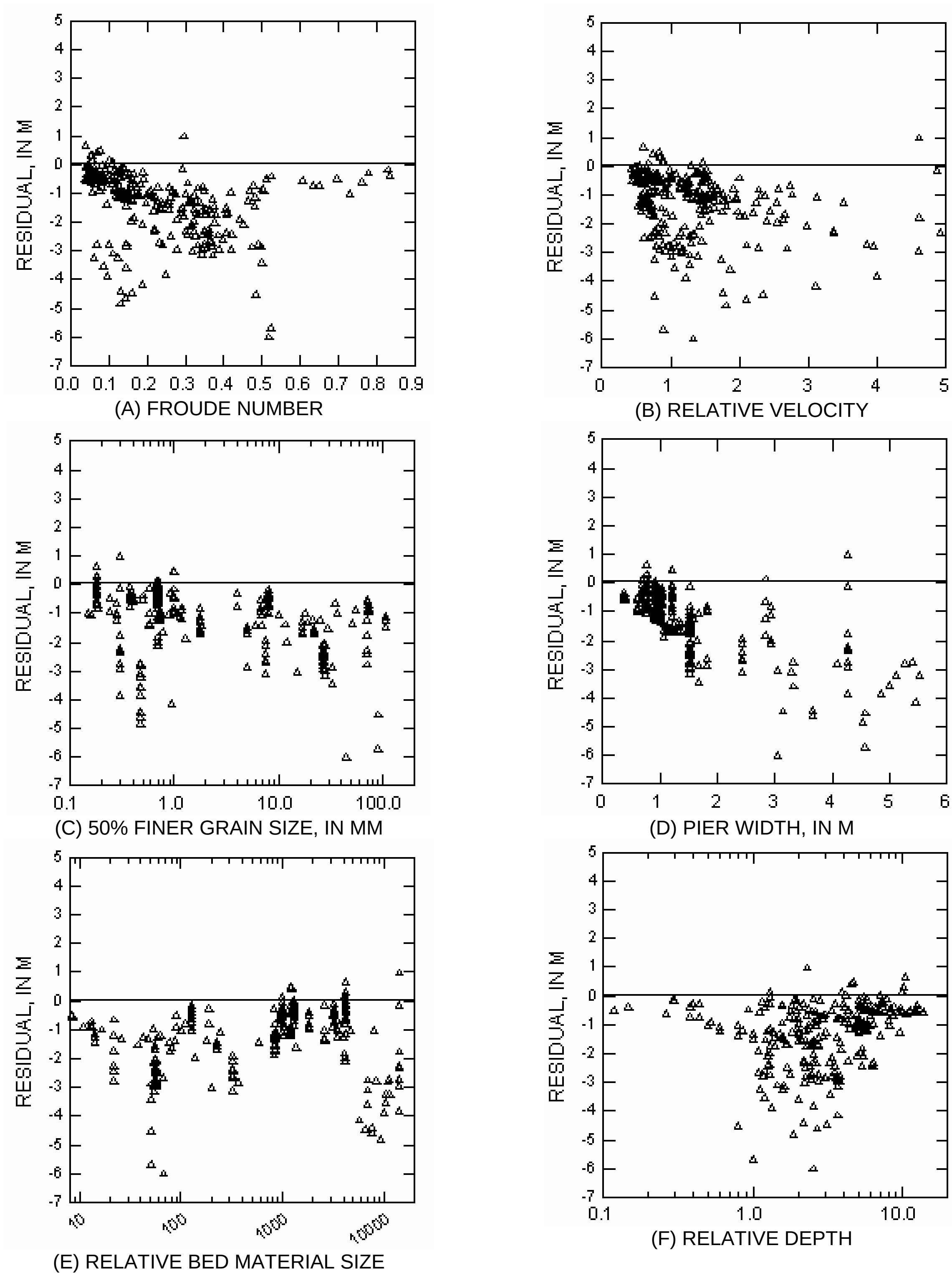


Figure 10. Evaluation of residuals for the HEC-18-K4 equation.

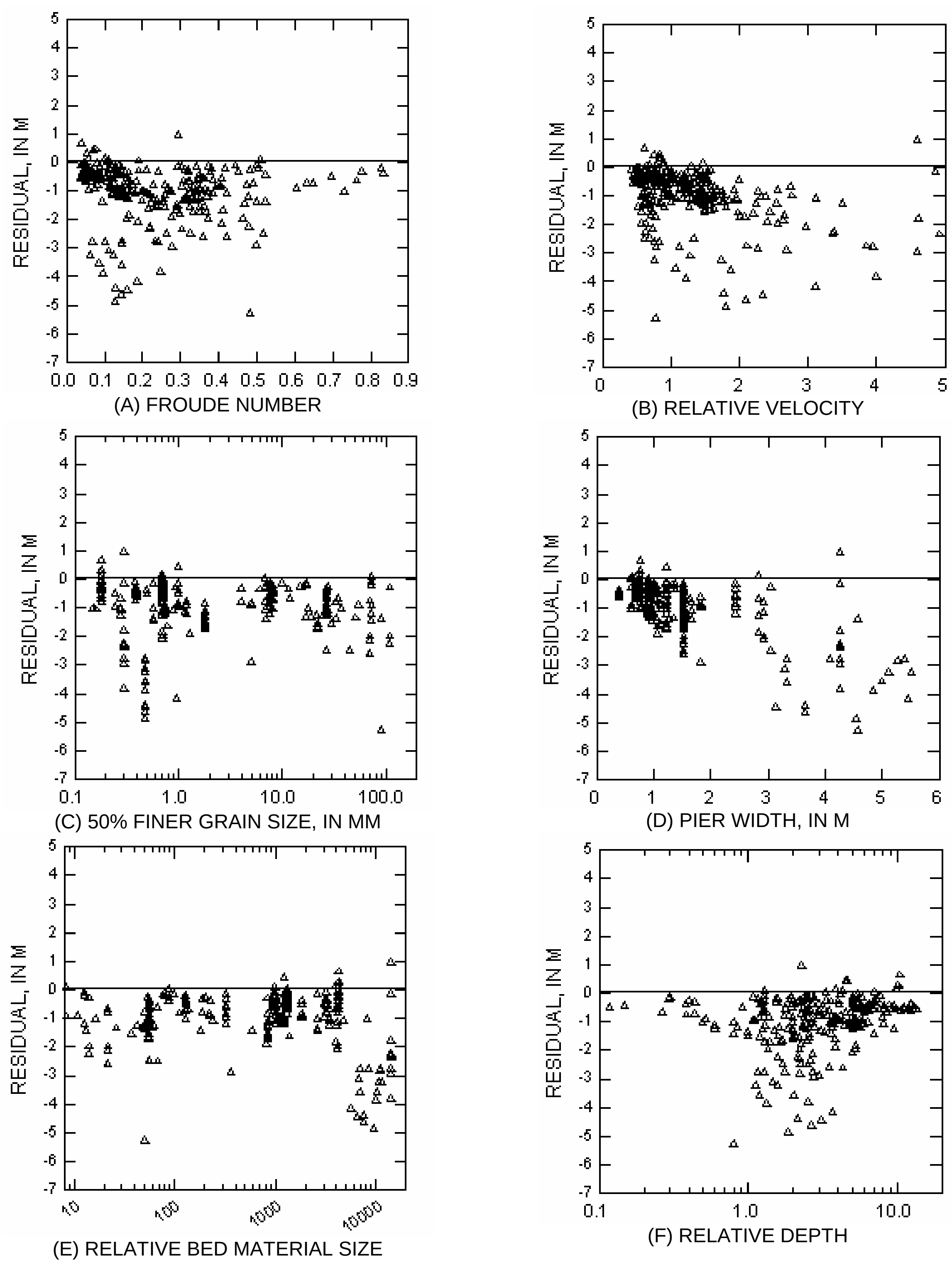


Figure 11. Evaluation of residuals for the HEC-18-K4Mu equation.

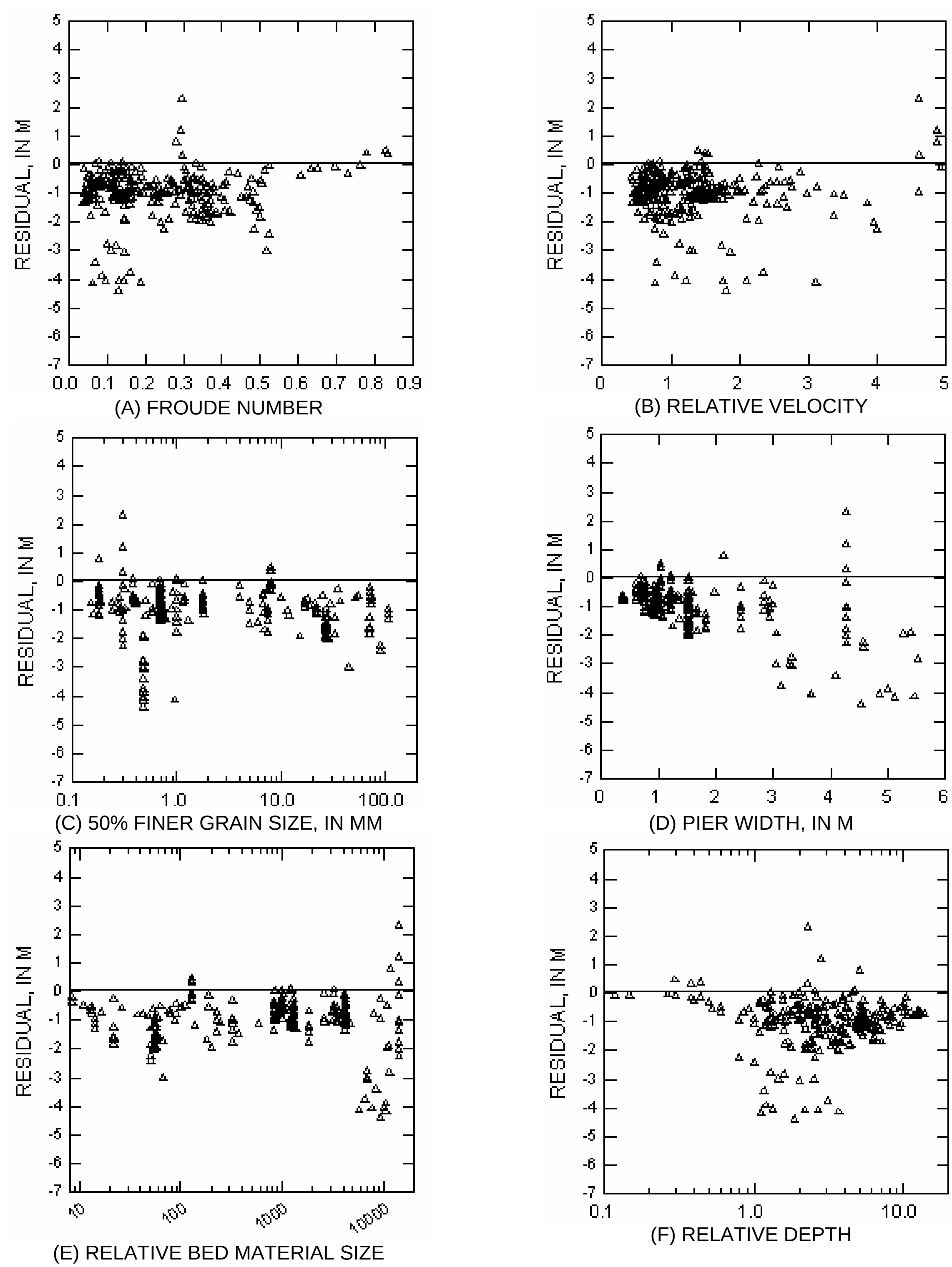


Figure 12. Evaluation of residuals for the Mississippi equation.

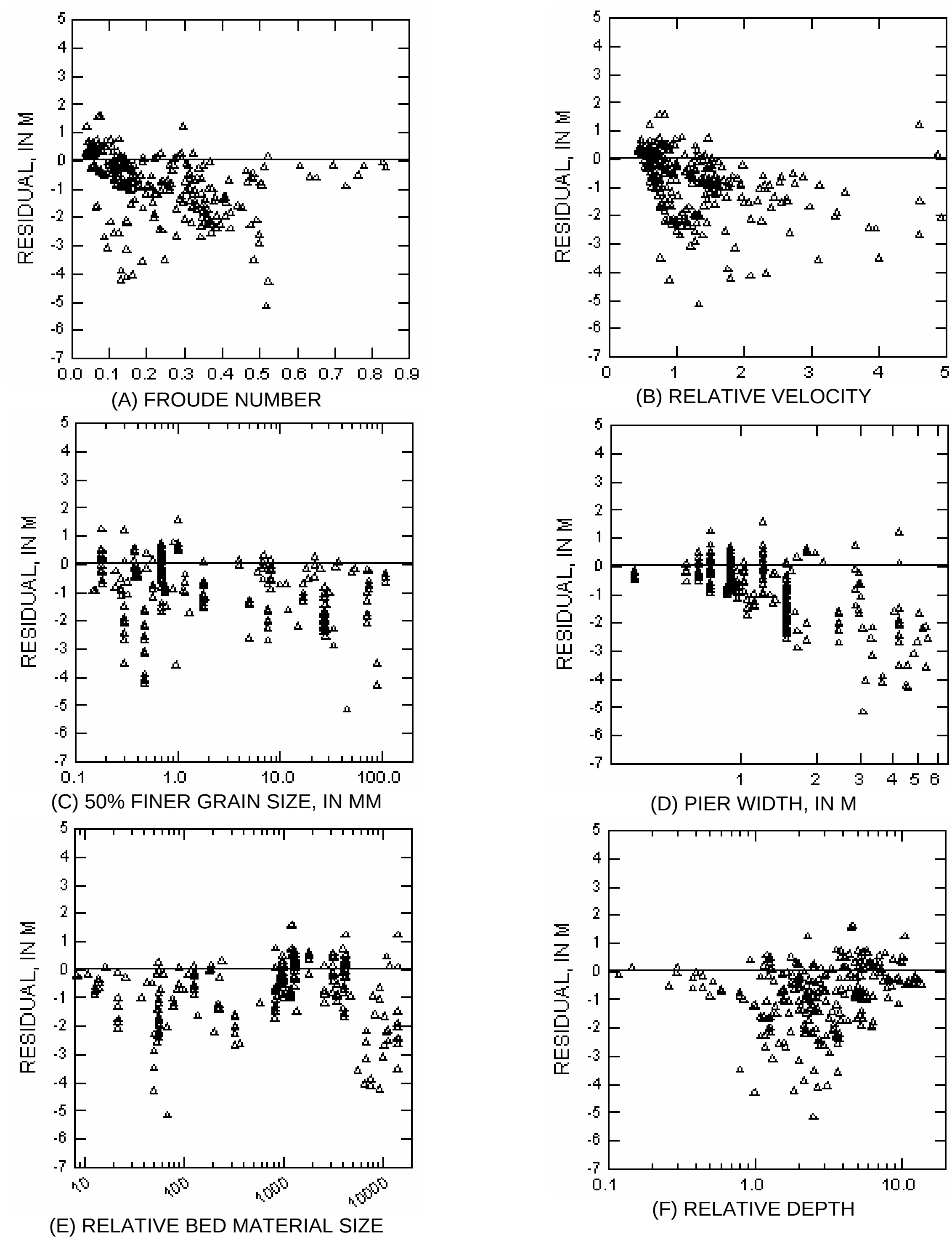


Figure 13. Evaluation of residuals for the HEC-18-K4Mo equation**.**

EVALUATION OF LABORATORY RESEARCH

General

Laboratory research has been the primary tool for defining the relations among variables affecting the depth of pier scour. The validity of these relations has not been proven in the field. Landers and Mueller evaluated many relations developed in the laboratory by use of transformed data (to obtain a more normal distribution) and smoothing techniques to assess general trends in the data.[(21)] They found only minimal agreement between the field data and laboratory-based relations. The assessment presented here investigates the relations in the field data for variable combinations commonly reported by laboratory investigations. Unlike the data set used by Landers and Mueller, all data at skewed piers were removed to prevent bias by these data, as previously discussed.[(21)] No transformations were applied unless necessary for consistency with published relations. Using the basic data without transformation results in a less uniform distribution of the data, but it provides for a more direct comparison with laboratory work.

Pier Geometry

For piers aligned with the approach flow, laboratory research indicates that streamlining the pier nose reduces the depth of scour. Many shapes have been tested in the laboratory (table 6) to determine what effect the pier shape has on the depth of scour, relative to a circular pier. The pier shapes for the field data are classified as unknown, cylinder, round-nose, sharp, or square. Landers and Mueller did not account for variables known to affect the depth of scour other than pier shape and found no correlation between pier shape and scour depth.[(21)] It is necessary to remove the effects of pier width, velocity, depth, and bed material before comparing the depth of scour among different pier shapes. Figures 14 and 15 show the effect of pier nose shape on scour. First, only the effect of pier width is accounted for by dividing the depth of scour (y_s) by the pier width (b). Figure 14 shows that the median of the relative depth of scour decreases as the pier is more streamlined; however, there is overlap in the interquartile range, and the differences are not significant. Second, the effects of pier width, velocity, depth, and bed material are removed by regressing these variables with the depth of scour and by plotting the residuals of a regression against pier shape, which is not included in the regression. The residuals from the regression analysis (figure 15) show the same trend as observed in figure 14. The pier shape does not affect the depth of scour in the field as much as in the laboratory. In the field, flow directions are variable, pier shapes vary with depth, and the effect of submerged debris is not easily accounted for; these combine to reduce the effect of pier shape on the depth of scour.

Table 6. Coefficients for the effect of pier shape relative to the scour that would be expected at a circular pier.

Shape	Length-Width Ratio	Tison[68]	Laursen and Toch[8]	Chabert and Engeldinger[69]	Garde et al. [70]	Venkatadri[71]	Neill (ed.) [23]	Dietz[72]
Circular	1	1.0	1.0	1.0	—	1.0	1.0	1.0
Lenticular	2	—	0.91	—	0.9	—	—	—
	3	—	0.76	—	0.8	—	—	—
	4	0.67	—	0.73	0.7	—	—	—
	7	0.41	—	—	—	—	0.8	—
Triangular nose 60°	—	—	—	—	—	0.75	—	0.65
Triangular nose 90°	—	—	—	—	—	1.25	—	0.76
Rectangular	1	—	—	—	—	—	—	1.22
	2	—	1.11	—	—	—	—	—
	3	—	—	—	—	—	—	1.08
	4	1.4	—	1.11	—	—	—	—
	5	—	—	—	—	—	—	—
	6	—	1.11	—	—	—	—	0.99
Rectangular chamfered	—	—	—	—	—	—	—	1.01

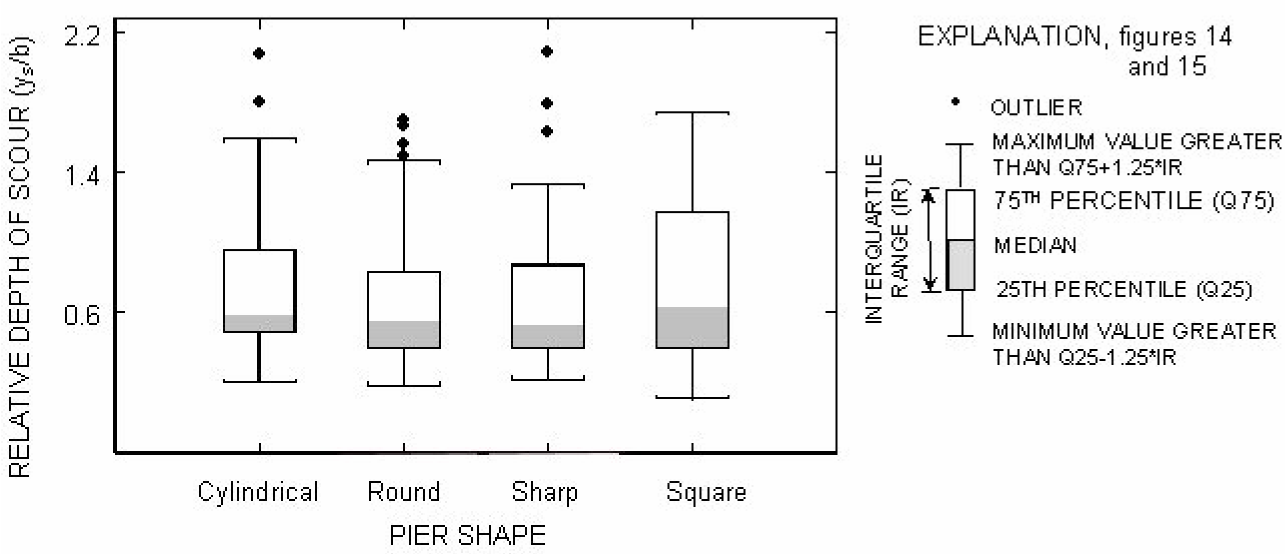


Figure 14. Box plot illustrating the effect of pier shape on relative depth of scour.

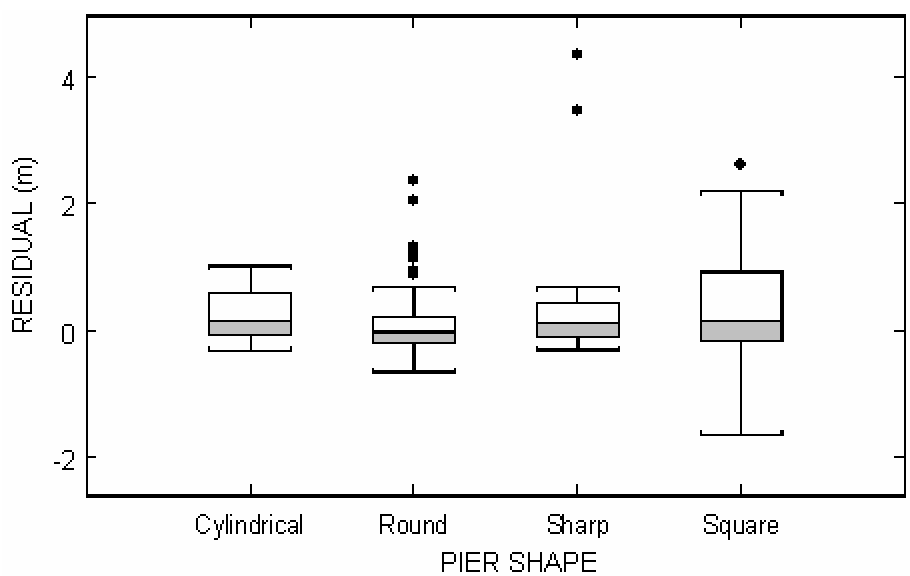


Figure 15. Box plot illustrating the effect of pier shape on the depth of scour with the effects of pier width, velocity, depth, and bed material removed by linear regression.

Relative Velocity

Through a series of laboratory experiments, Chiew found relative scour depths (y_s/b) were less for ripple-forming sediments than for nonripple-forming sediments at relative velocities (V_o/V_c) ranging from 0.6 to 2.[44] He determined that this reduction in scour depth was caused by the roughness and sediment transport associated with the formation of ripples near incipient motion. Ripple-forming sediments are those with a D_{50} less than about 0.6 mm. Figure 16 shows that the upper envelope of the field data generally fits the curves that Chiew developed.[44] A few measurements with ripple-forming sediments exceed the envelope. The maximum depth of scour observed in the field does not appear to be strongly affected by whether the sediment is ripple forming or nonripple forming. The scatter of data below the envelope curves indicates that the relation between relative depth of scour and relative velocity developed in the laboratory does not adequately explain the scour processes in the field. Nonuniformity of the bed material and variable flow depth in the field probably cause some of the scatter.

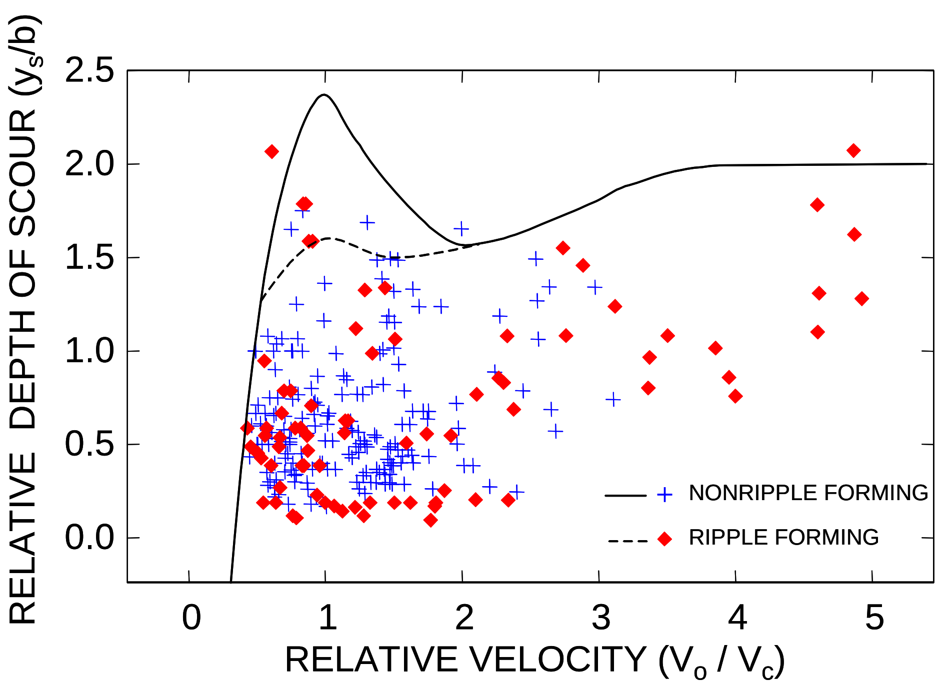


Figure 16. Comparison of field observations with the curves developed by Chiew showing the effect of sediment size andrelative velocity on relative depth of scour.[44]

Baker also did laboratory investigations of the effect of bed material properties on the relation between relative scour depths and relative velocity, using nonuniform bed material characterized by the coefficient of gradation.[45] He found that as the coefficient of gradation increased, the relative depth of scour was reduced, and the maximum scour occurred at a relative velocity greater than one. The field data categorized by the coefficient of gradation are shown in figure 17 with hand-drawn envelope curves for the four categories of gradation. The effect of gradation has no consistent pattern in the relation between normalized scour depth and relative velocity for the field observations.

Baker changed the gradation while maintaining a constant D_{50} during his experiments.(45) To simulate a constant D_{50} in the field data, Mueller used partial residuals to remove the effect of D_{50} from the field data.(5) This approach did not improve the comparison between the field data and Baker's laboratory observations.(45)

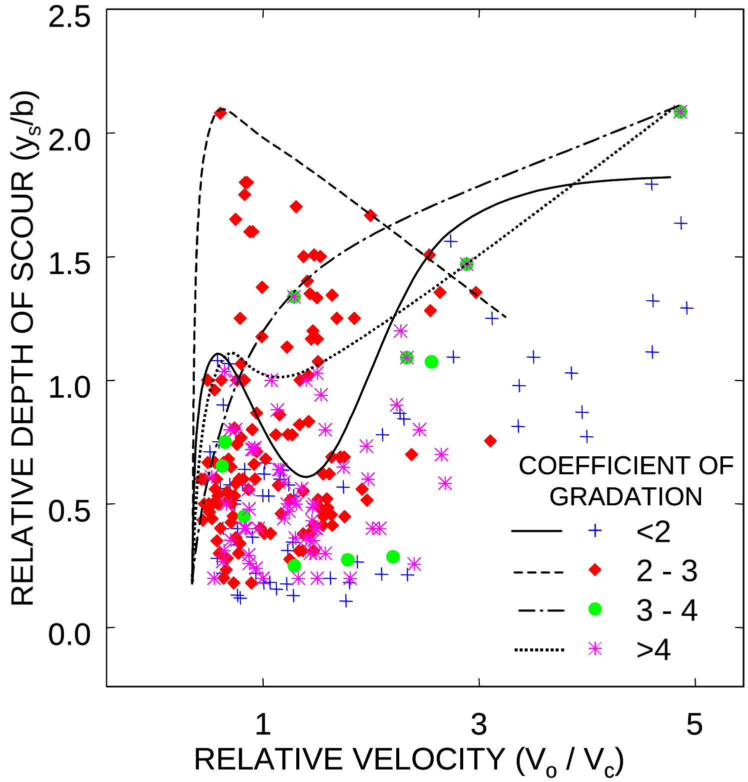


Figure 17. Effect of gradation and relative velocity on relative depth of scour for field data, with hand-drawn envelope curves for selected gradation classes.

Bed Material Parameters

The scale of laboratory experiments prevents the effect of relative sediment size (b/D_{50}) on relative scour depth from being directly compared with field conditions. The maximum relative sediment size obtained in the laboratory was about 800. In the laboratory, ripple-forming sediments had lower relative scour depths than nonripple-forming sediments for relative sediment sizes ranging from 100 to 800. The field data do not contain ripple-forming sediments with a relative sediment size less than 900 (figure 18); therefore, there is insufficient overlap between laboratory and field data to make a valid comparison. The field data show a cluster of ripple-forming sediments near a relative sediment size of 1,000 that is below the maximum scour for nonripple-forming sediments; however, the maximum relative depth of scour for ripple-forming sediments with relative sediment sizes of 4,000 exceeds the nonripple-forming sediments.

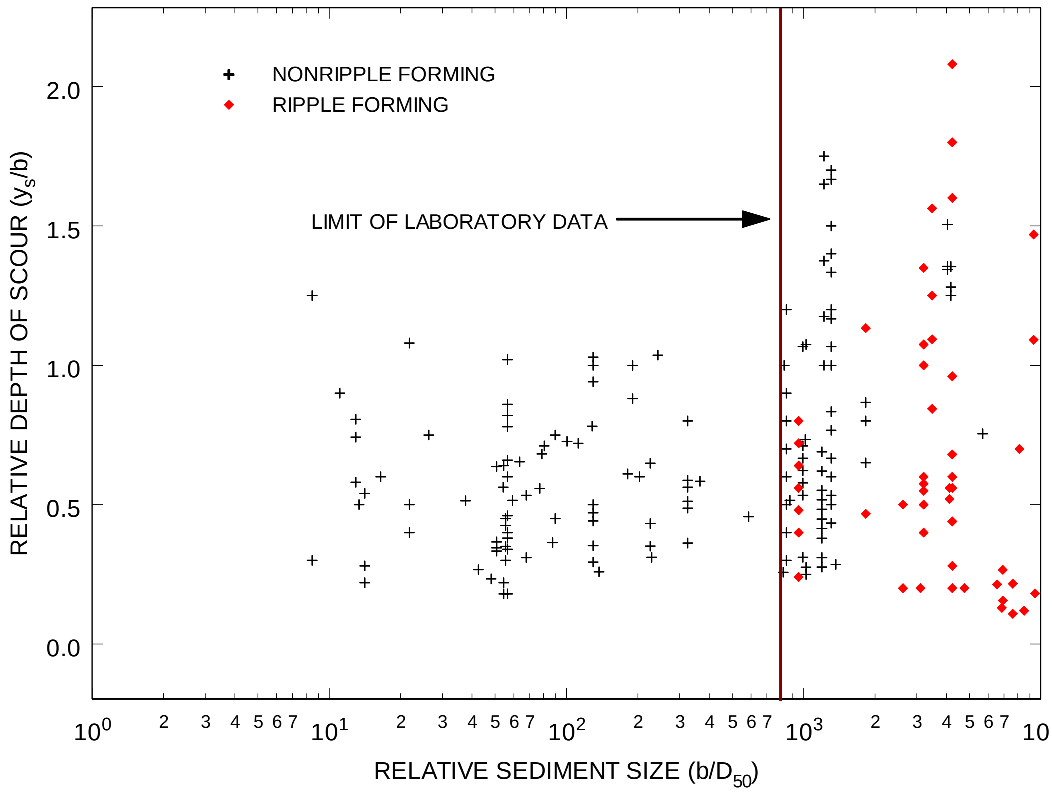


Figure 18. Effect of relative sediment size on relative depth of scour for field data.

Ettema recognized that maximum depth of scour, determined as 2.4 times the pier width, was affected by the gradation of the bed material.[47] Ettema used a series of laboratory experiments to develop a correction factor to account for the gradation of the bed material on the maximum depth of scour. Hand-drawn envelope curves in figure 19 show that the relative scour depth is greater for ripple-forming sediments than for nonripple-forming sediments when the gradation coefficient is less than about 2.5. For gradation coefficients greater than 2.5, there is a reduction in the relative depth of scour for all observations. The reduction in the relative depth of scour is larger for ripple-forming sediments than for nonripple-forming sediments. An increase in the coefficient of gradation for a constant median grain size results in an increase in the coarser size fractions of the bed material; therefore, an increase in the coarse size fractions of the bed material reduces the depth of scour, and the depth of scour is dependent on the size distribution of nonuniform bed material. The larger reduction in scour for ripple-forming sediments may be caused by armoring of the scour hole by the coarser size fractions, but the small amount of data on ripple-forming sediments for the larger gradations makes any conclusions questionable.

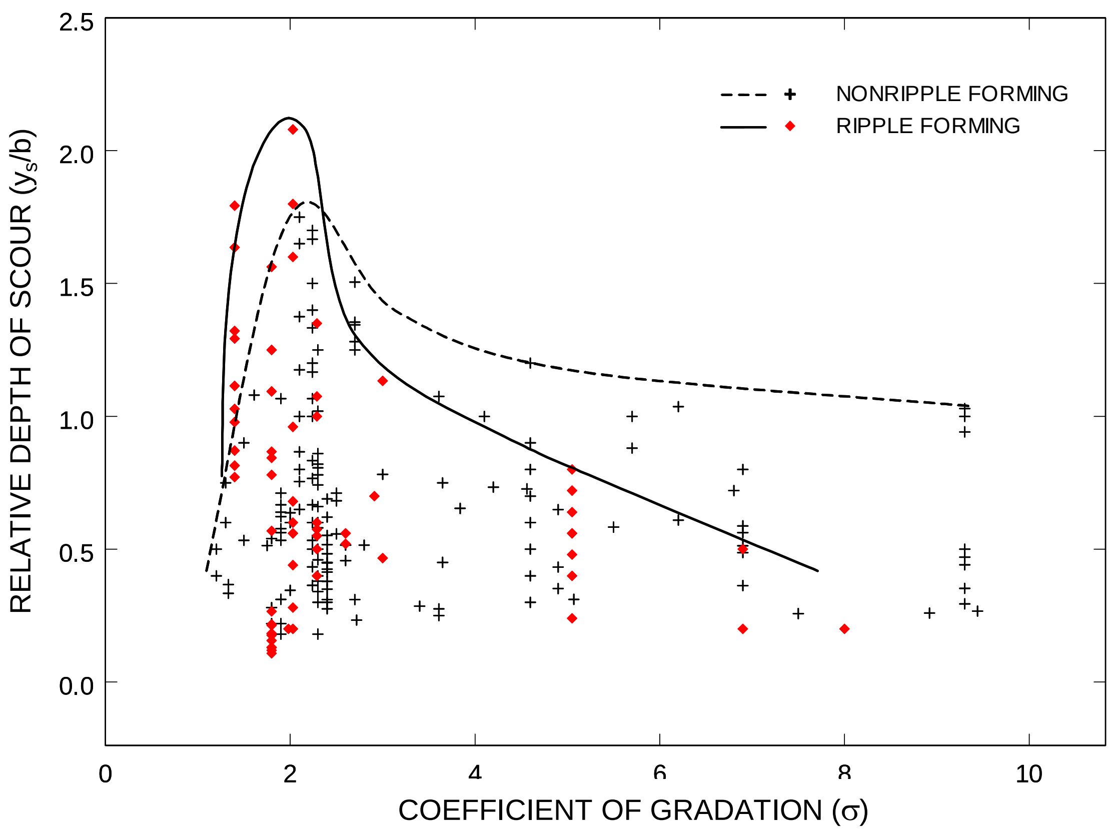


Figure 19. Effect of the coefficient of gradation on relative depth of scour for field data with hand-drawn envelope curves of ripple- and nonripple-forming sediments.

Depth of Approach Flow

Most researchers agree that for constant velocity intensity, local pier scour increases as depth of flow increases, but as the depth of flow continues to increase, the scour depth becomes almost independent of flow depth. (See references 44, 47, 69, 73, 74, 75, and 76.) Chiew[44] plotted data that he collected along with experimental data from Shen et al.,[9] Ettema,[47] and Chee[76] and concluded that the flow depth does not affect scour if the depth is greater than four times the pier width. From this research, Melville and Sutherland developed the K_y factor in their prediction equation (table 3).[2] The relation between relative flow depth and relative scour depth for the field data is shown in figure 20. Although the curve for the K_y factor envelops the data to the right, the data do not follow the trend of the curve. Most laboratory data are collected at or near incipient motion. To better compare the field data with the laboratory data, field data with sediment transport conditions near incipient motion ($0.8 < V_o/V_c < 1.2$) were selected and plotted in figure 21. Again, the field data do not follow the trend observed in the laboratory data; they indicate that the relative depth of scour tends to increase with increasing relative flow depth.

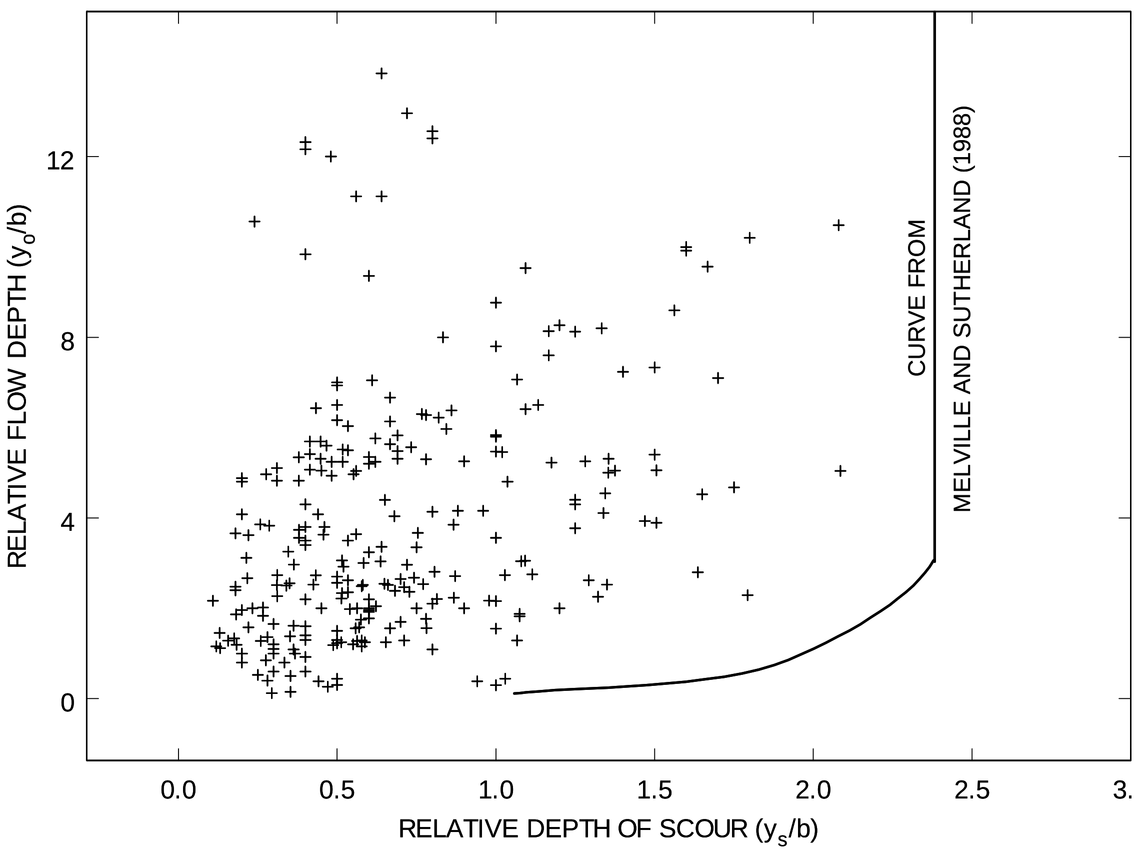


Figure 20. Effect of relative flow depth on relative depth of scour with field data compared to the relation presented by Melville and Sutherland.[2]

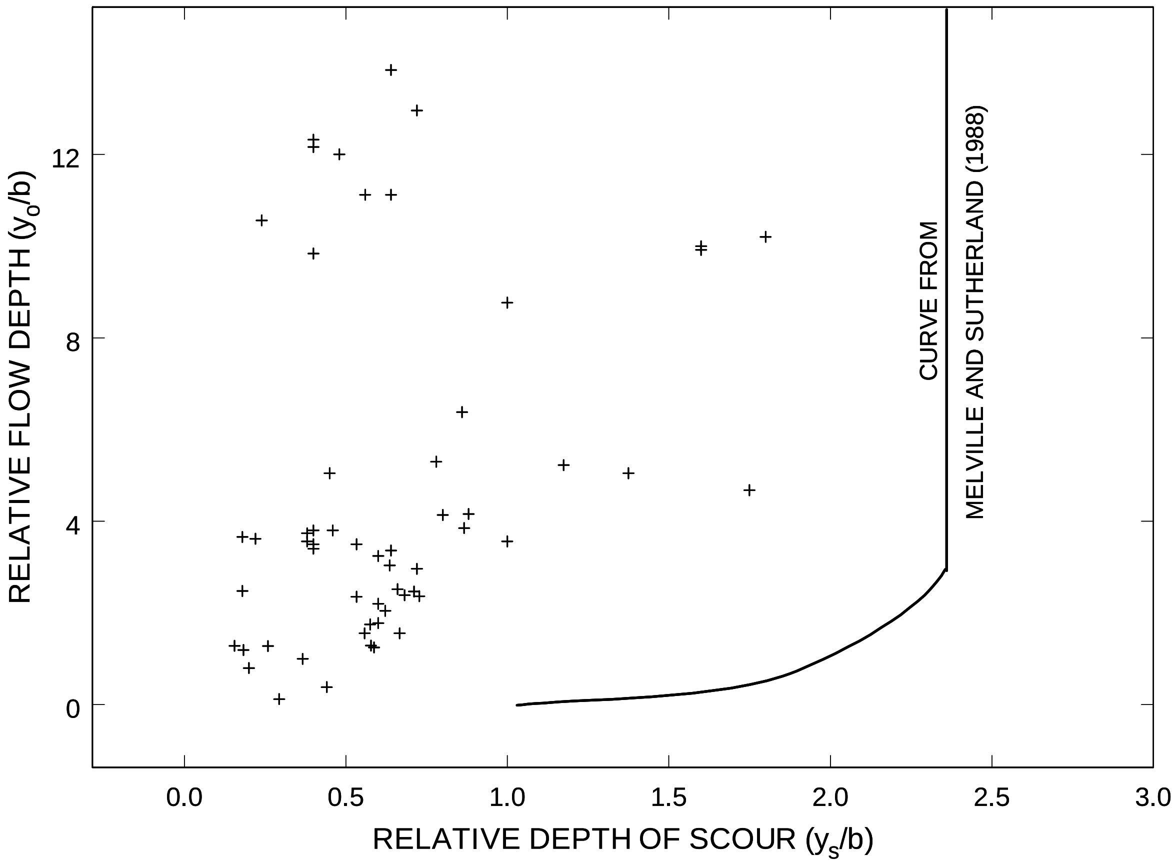


Figure 21. Effect of relative flow depth on relative depth of scour for field conditions near incipient motion ($0.8<V_o/V_c<1.2$) compared to the relation presented by Melville and Sutherland.[2]

DEVELOPMENT OF SCOUR PREDICTION METHODOLOGY

Assessment of Basic Variables

Logically, pier width, pier shape, flow depth, approach velocity, and bed material characteristics are important variables in determining the depth of scour; however, most of the design equations presented in table 3 do not contain all of these variables. The Mississippi equation, which was one of the top equations (table 5), is based on only pier width and flow depth. Therefore, it is important to evaluate the significance of each variable on the depth of scour and the potential interaction among the variables. A combination of scatter plots and multiple regression analysis will be used for this evaluation.

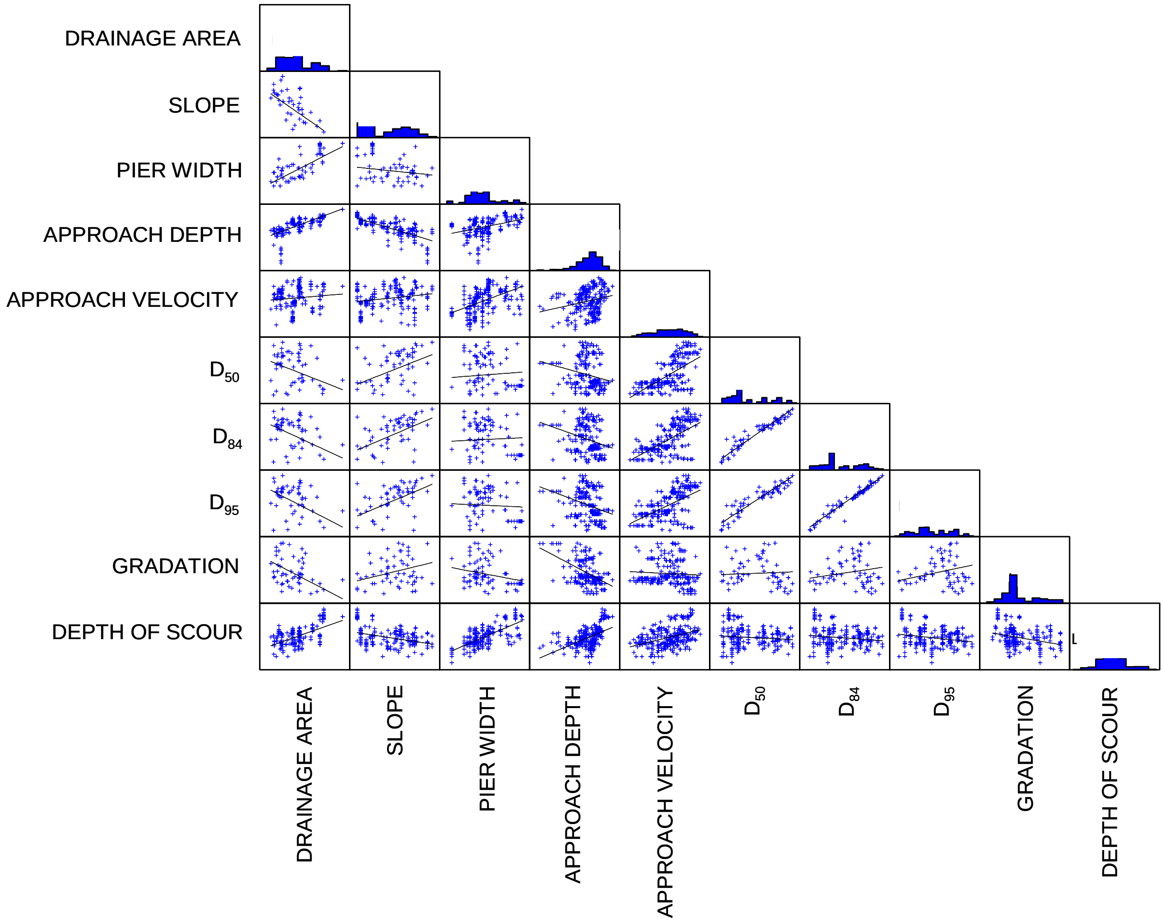


Figure 22. Scatterplot matrix and frequency distribution of basic variables and depth of scour, log-transformed.

The effect of an individual variable on the depth of scour in the field is complicated by the interactive response of the variables to the dynamic conditions. Evaluating the effect of a particular variable on the depth of scour in the laboratory is easier than in the field. In the laboratory, all variables can be held constant and one variable changed; in the field, all of the variables interact and adjust to the changing flow conditions.

Figure 22 shows a scatterplot matrix of basic variables reported in the field data with a linear least squares smooth through the data. Drainage area, slope, and pier width appear strongly correlated with scour depth. Pier width directly affects the strength of the vortex system, which erodes the material from around the base of the pier. Correspondingly, pier width shows the strongest correlation with scour depth. It is surprising that drainage area and slope have a stronger correlation with the scour depth than do approach depth or approach velocity. This strong correlation appears to be caused by the correlation of the pier width and approach depth with drainage area and slope (figure 22); thus, for these data, drainage area or slope may represent a combined effect of pier width and approach depth. There is also a positive correlation between depth of scour, approach depth, and approach velocity; however there is significant scatter in the data, indicating that these variables are less significant than pier width. The size and distribution of the bed material also affects the depth of scour, but the slope of the linear smooth is small and the scatter of the data indicates a low correlation with the depth of scour. The bed material size is well correlated with the approach velocity and slope, which is what would be expected; coarse bed streams have higher slopes and higher velocities. The bed material size classes are strongly correlated with each other, but are not linearly correlated with the gradation coefficient. The strong linear correlation between bed material sizes could cause colinearity problems in the results of multiple linear regression if different bed material size variables are included in the same equation.

Weighted multiple linear regression analysis was used to assess the importance of each variable on the depth of scour, while accounting for the interaction between variables. Bed material sizes were evaluated in separate equations because of their strong colinearity. All variables were transformed logarithmically to improve the linearity and distribution of the data. Weighted multiple linear regression computes coefficients and exponents that minimize the sum of squares of the residuals while taking into account weights assigned to each observation. If the weights for all observations are equal, approximately one-half of the data are underestimated and about one-half are overestimated (figure 23). This approach, while yielding a combination of variables that fits the middle of the data, is not appropriate for design. An envelope curve is more appropriate

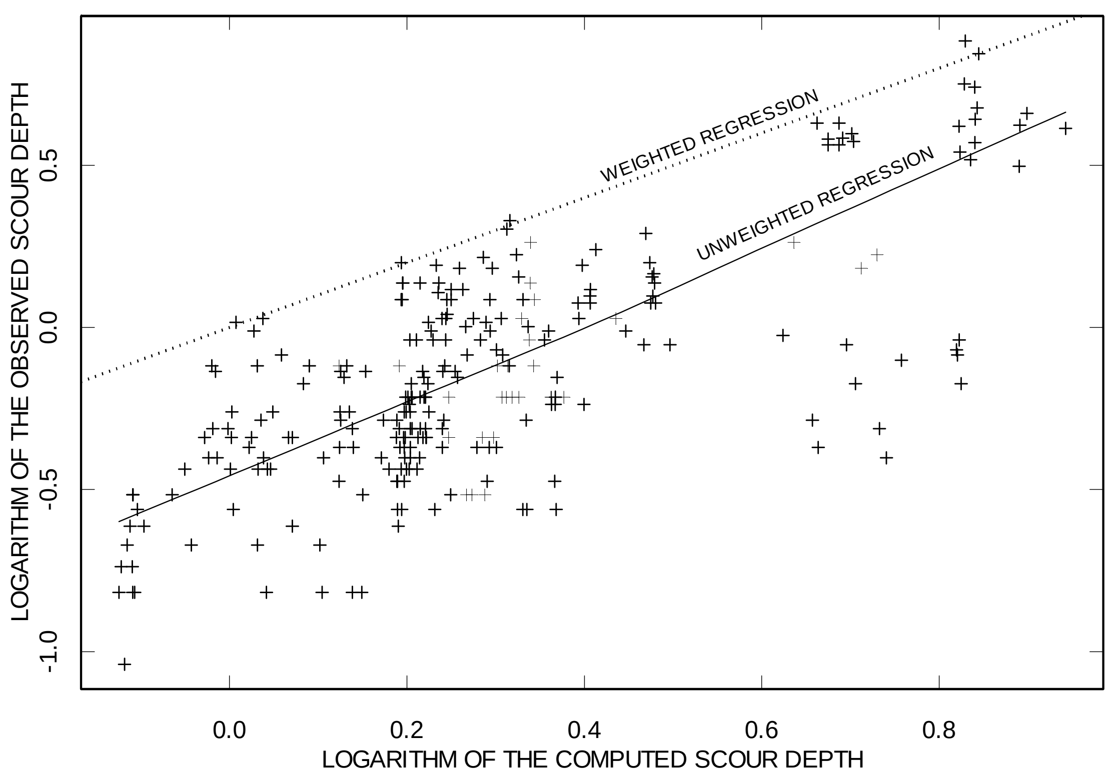


Figure 23. Example of difference between unweighted regression and weighted regression in developing a design curve.

for design. To fit an envelope curve, the regression was completed with equal weights, then the weights were adjusted so that more weight was applied to those points that were underestimated and defined the upper boundary of the data (figure 23). The weighting function (*w*), shown in equations 38 and 39, was determined by trial and error to produce a reasonable envelope curve.

$$W = \left(\left(10^{residual} \right)^2 \right)^9 \quad \text{for residuals} > 0 \tag{38}$$

$$W = \left(10^{residual} \right)^2 \quad \text{for residuals} < 0 \tag{39}$$

It was observed that small adjustments in the weighting function significantly change the sum of squared errors, the number of observations underestimated, and the statistical significance of each of the variables in the equation formulation. Therefore, the equations developed from this weighted regression approach should not be treated as the optimal envelope equation for these data, but serve as indicators as to which variables should be considered in the development of design methodology.

Regression analysis showed that the inclusion of bed material size characteristics in the equation improved the sum of squared errors (table 7). Unweighted regression indicated that only the bed material size was important and the gradation of the bed material was not statistically significant at the 0.1 level. The equation using the D_{84} sediment size resulted in the lowest sum of squared errors. It is surprising that when the bed material was removed from the equation the approach velocity was not statistically significant in the unweighted regression. It is also interesting that there is reasonable consistency in the exponents for pier width and approach depth, but the exponents on velocity vary by a factor of 10.

The weighted regression analysis produced different results than did the unweighted analysis. As with the unweighted regression, the equations containing bed material characteristics all produced lower sum of squared errors than did the equations without bed material characteristics. For the weighted analysis, all of the variables in each analysis were significant, and the equation using D_{50} produced the lowest sum of squared errors. While these equations may not be the optimal approach to predicting the depth of scour, they clearly show that bed material characteristics are important in determining the depth of scour.

Assessment of Current Methodology

A K_4 factor was added to the HEC-18 pier scour equation in the third edition of HEC-18 to account for bed material size characteristics.[(6)] FHWA derived the relation for that version of K_4 from preliminary laboratory data provided by Molinas, and it was intended as an interim adjustment factor until more detailed analyses were available (see HEC-18-K4 equation in table

3). Table 5 indicates that the sum of squared errors was only reduced from 822 to 791 by the inclusion of the K_4 term presented in the third edition of HEC-18.

Mueller developed a relationship for K_4 based on field data (see HEC-18-K4Mu in table 3).[5] Mueller used the Chinese equation for determining the approach velocity for incipient motion (equation 2) at the pier for the median grain size, but extended it to the D_{95} size fraction. The fourth edition of HEC-18 adopted Mueller's K_4 but restricted the lower limit to 0.4 and required a value of 1 if D_{50} were less than 2 mm, or D_{95} less than 20 mm. These restrictions were applied to the evaluation of this factor in table 5 (HEC-18-K4Mu). Table 5 indicates that Mueller's K_4 factor as adopted in the fourth edition of HEC-18 reduces the sum of squared errors significantly from 822 to 448. Although Mueller's 1996 K_4 factor worked quite well for the field data available for evaluation, the formation of the equation causes it to be indeterminate for some situations and behave contrary to logic in others. The equation becomes indeterminate if the velocity for incipient motion of the D_{50} grain size is smaller than the approach velocity needed to scour the D_{95} grain size at the pier. The equation behaves contrary to logic if the D_{50} grain size is held constant and only the D_{95} is varied. In this situation, K_4 increases as D_{95} increases. In the field, variables tend to change together as a system, whereas in the laboratory selected variables can be held constant and other variables can be changed arbitrarily. For the field data used by Mueller to develop the K_4 factor, an increase in D_{95} always corresponded to an increase in D_{50} (figure 22).[5] Under these conditions, the velocity intensity term proposed by Mueller provides a reasonable envelope curve, but it can produce unexpected results due to the arrangement of the variables.[5]

Table 7. Summary of weighted and unweighted regression results using basic variables.

			Exponents							
Basic Variables Used in Analysis	**Weighted**	**Coefficient**	**Pier Width**	**Approach Depth**	**Approach Velocity**	D_{50}	D_{84}	D_{95}	σ	**SSE**
b, y_o, V_o, D_{50}, σ	No	0.229	0.678	0.207	0.298	-0.114	–	–	N.S.	132.4
b, y_o, V_o, D_{50}, σ	Yes	1.19	0.765	0.166	0.058	-0.0284	–	–	-0.066	746.9
b, y_o, V_o, D_{84}, σ	No	0.247	0.651	0.166	0.319	–	-0.128	–	N.S.	121.3
b, y_o, V_o, D_{84}, σ	Yes	1.34	0.712	0.147	0.140	–	-0.038	–	-0.140	847.5
b, y_o, V_o, D_{95}, σ	No	0.270	0.619	0.176	0.300	–	–	-0.123	N.S.	127.9
b, y_o, V_o, D_{95}, σ	Yes	1.46	0.640	0.178	0.157	–	–	-0.019	-0.132	788.8
b, y_o, V_o	No	0.414	0.784	0.295	N.S.	–	–	–	–	180.8
b, y_o, V_o	Yes	1.25	0.900	0.207	0.029	–	–	–	–	1060.0

SSE—sum of squared errors
N.S.—not significant at 0.1 level
– —no value

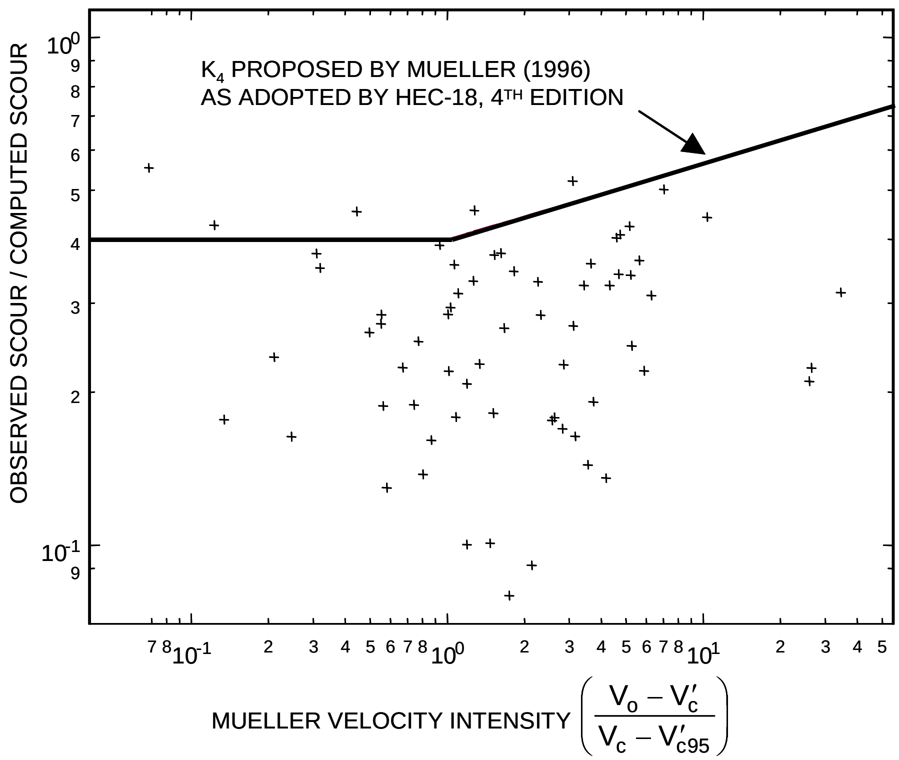


Figure 24. Relation between the ratio of the observed depth of pier scour to the depth of pier scour computed by the HEC-18 equation (idealized K4) and the K4 proposed by Mueller[5] as adopted by the fourth edition of HEC-18.[77]

Mueller's K_4, as adopted by the fourth edition of HEC-18, is compared with the expanded data set presented in this report in figure 24.[5] The equation envelops the data, with the exception of five points. Four of the points underpredicted the observed scour by less than 5 percent. Of the five points lying above the envelope curve in figure 24, four observations are from streams in Ohio. The bridge scour data sets from Ohio contained extensive bed material data, which were collected annually during low flow at most sites. These data included composite samples collected at the bridge and in the approach cross sections and local samples collected at each pier (inside the scour hole, if one were present). The bed material size reported with the scour measurement was usually the sample collected at the pier for the low flow preceding the scour measurement. All four observations for Ohio plotted below the envelope curve if the composite samples were used.

Molinas derived a new correction to the HEC-18 equation from his final laboratory data set.[1] Although several of the terms are similar to those used by Mueller and Jones,[78] Molinas redefined the equations for computing the incipient motion velocity and the approach velocity causing incipient motion at the pier (see table 3). Although this new correction provided a significant decrease in the sum of squared errors (from 822 to 495), it also significantly increased the number of observations that were underpredicted (from 13 to 65). Figure 13 showed that most of these underpredictions occurred at D_{50} less than 2 mm. If the correction developed by Molinas is only applied to D_{50} greater than 2 mm, its performance was enhanced greatly. The sum of squared errors rose to 609, but the number of observations underpredicted dropped from 65 to 21, and the sum of squared errors for the underpredictions was reduced from 17 to 2.47.

Development of New Methodology

Patterns in the performance of the HEC-18 equation clearly show the need for a K_4 term to correct the depth of scour, particularly for coarse bed materials. The HEC-18 equation showed no difference in its performance for clear water or live bed conditions (figure 25). Armoring of the scour hole could cause overpredictions by the HEC-18 equation for coarse bed material. The ability of the flow to transport the D_{95} sediment size at the pier (estimated using equation 2) was used to determine whether an armor layer would form in the scour hole and limit the depth of scour. Figure 26 shows that there is little difference in the idealized K_4 term (observed depth of scour/HEC-18 computed depth of scour) for conditions where the armoring potential is high. Mueller observed the HEC-18 equation consistently overpredicted scour in coarse bed materials.[5] Figure 27 clearly shows that for this data set, the magnitude of the overprediction increases with the median bed material size. A wide variation in the depth of scour for sand is indicated by the long whiskers in the box plot.

The K_4 term was developed by evaluating both the whole data set and only the portion with median grain sizes coarser than sand. Figure 28 shows that the depth of scour computed from the HEC-18 equation overpredicts by a larger ratio as the bed material size increases. It is interesting that there is also a negative trend in the approach velocity; this trend would indicate that the HEC-18 equation may have too high an exponent on velocity. The K_4 term should be dimensionless to maintain the dimensional homogeneity of the HEC-18 equation. Numerous combinations of variables were investigated, and the best correlation was found with the median size of the sediment relative to the pier width (b/D_{50}). The equation for the envelope curve using this variable combination is:

$$K_4 = 0.35\left(\frac{b}{D_{50}}\right)^{0.19} \tag{40}$$

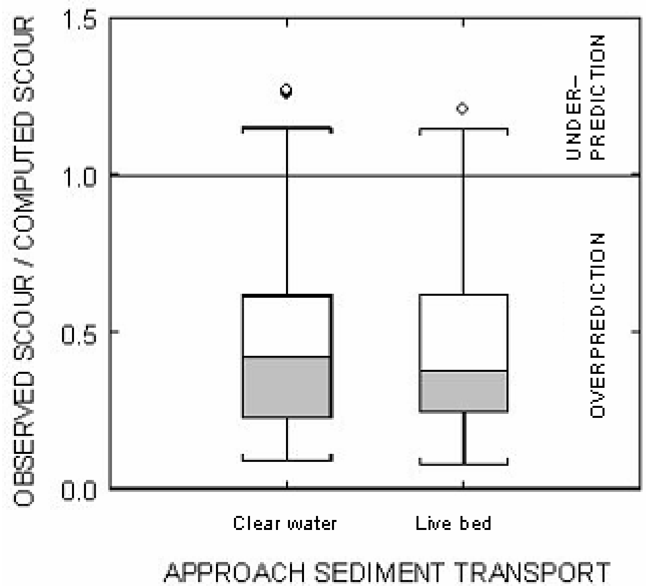


Figure 25. Box plot of the variation in the ratio of the observed depth of pier scour to the depth of pier scour computed by the HEC-18 equation (idealized K4) for clear water and live bed conditions.

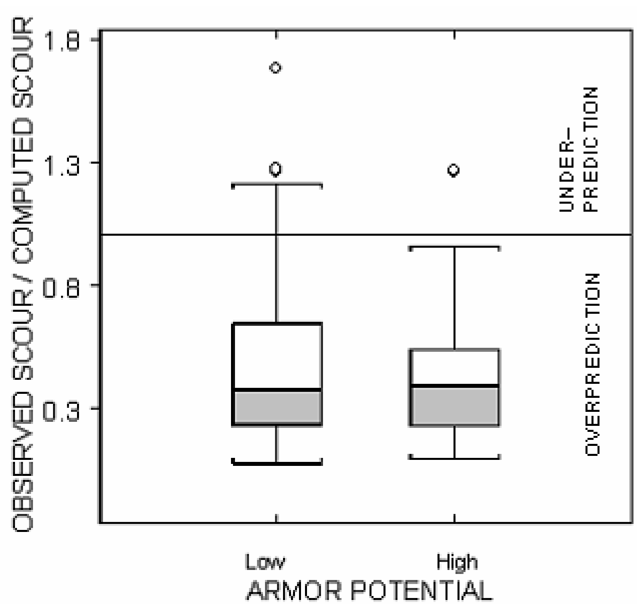


Figure 26. Box plot of the variation in the ratio of the observed depth of pier scour to the depth of pier scour computed by the HEC-18 equation (idealized K4) for low and high armor potential conditions.

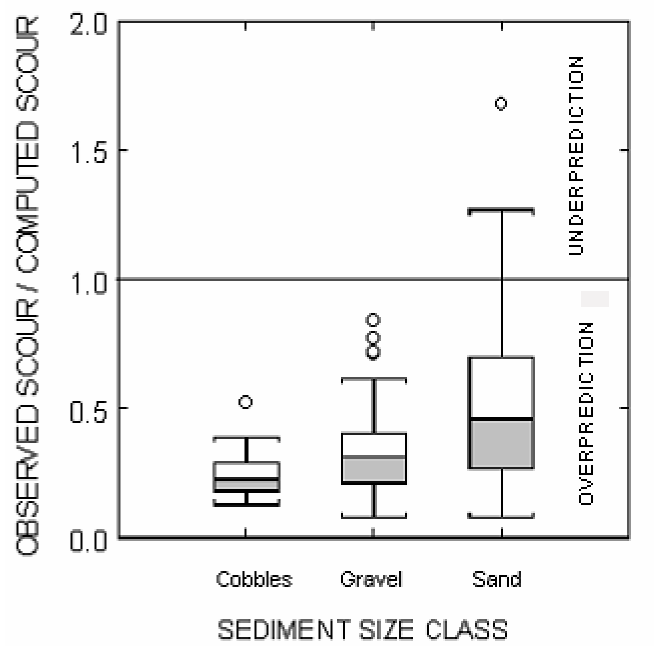


Figure 27. Box plot of the variation in the ratio of the observed depth of pier scour to the depth of pier scour computed by the HEC-18 equation (idealized K4) for sediment size classes.

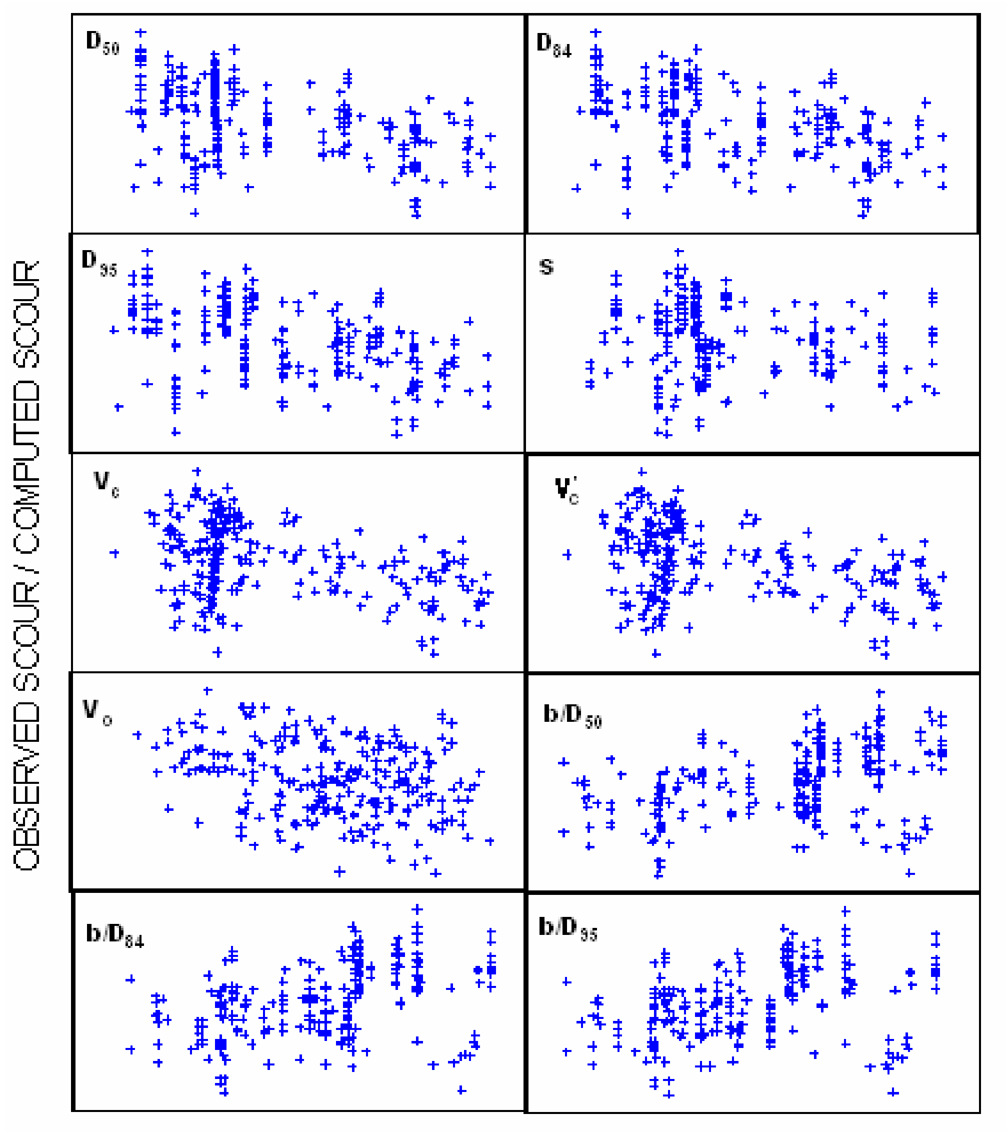


Figure 28. Relation between the ratio of the observed depth of pier scour to the depth of pier scour computed by the HEC-18 equation (idealized K4) and selected variables.

Figure 29 shows the envelope curve for K_4 developed from the b/D_{50} ratio. This curve is applicable for all grain sizes and appears to explain some of the underprediction for the HEC-18 equation for the sand sizes. If this correction is applied to all observations, the 13 observations that HEC-18 originally underpredicted (table 5) are corrected, but the sum of squared errors increases to over 2,800. The large increase in the sum of squared errors is caused by the large scatter below the curve for values of K_4 above 1. If the correction is limited to reducing the depth of scour (K_4<1), the sum of squared errors is reduced to 611, but 14 observations are underpredicted. The sum of squared errors for the 14 observations underpredicted is 2.16, the same as the HEC-18 equation before this correction (table 5). Use of only bed material size to develop a dimensionally dependent equation reduced the sum of squared errors to 520; this reduction does not seem sufficient to justify the use of a dimensionally dependent equation to compute a K_4 term.

Although the K_4 based on b/D_{50} does not perform as well as the HEC-18-K4Mu equation in table 5, the basis for this new approach is supported to an extent by the work of Sheppard, who found that b/D_{50} was an important parameter based on his laboratory research.[64]

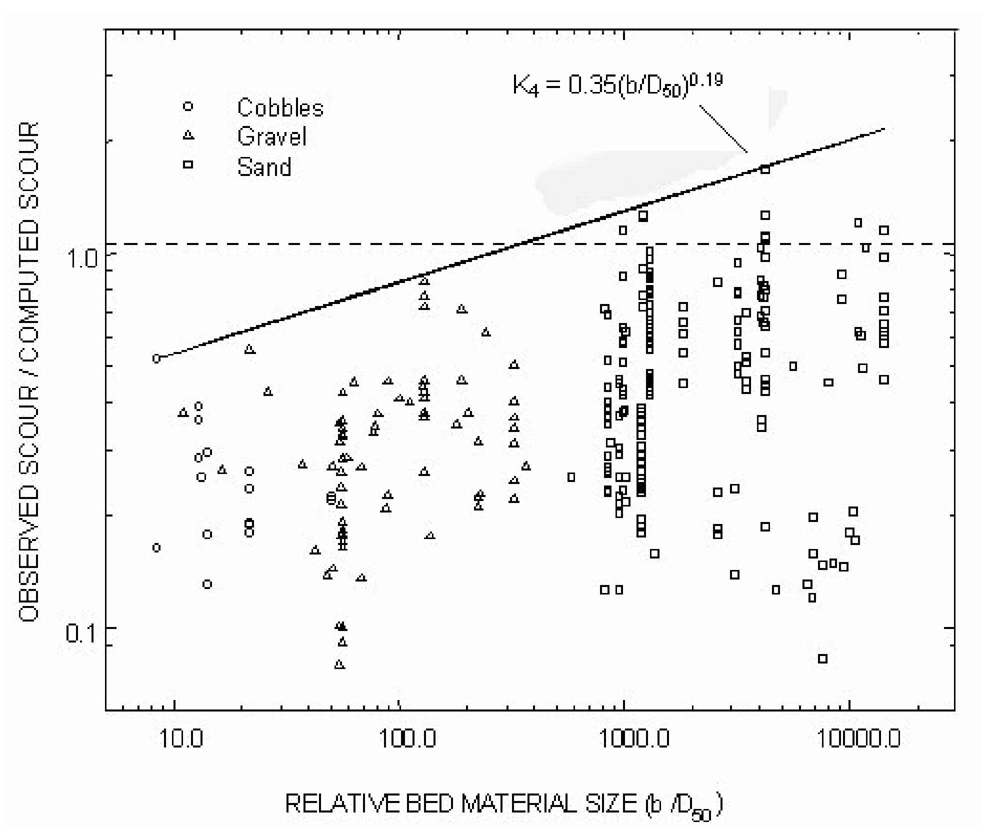


Figure 29. Relation between relative errors in computed scour using the HEC-18 equation and relative bed material size.

Importance of Sampling Bed Material

The four observations from Ohio above the envelope curve in figure 24 could be reduced or eliminated by use of the grain sizes associated with a composite sample or an average of all available composite samples. While this analysis highlights the sensitivity of Mueller's 1996 K_4 to the bed material samples, it also illustrates the importance of and uncertainty associated with determining bed material characteristics for field conditions.

The potential variability associated with characterizing the bed material can be illustrated using the Ohio bridge scour data sets. These data sets contain 419 samples of bed material, of which 149 represent composite samples for an entire cross section and the rest are point samples near the piers. Table 8 shows that individual samples can vary greatly; the average coefficient of variation is just over 1.0 for all samples, including the composite samples. If only the composite samples are considered, the variability is reduced, but the average coefficient of variation exceeds 0.7. This magnitude of variability is probably responsible for much of the scatter in the relation between depth of scour and bed material characteristics. Even if the perfect scour equation were developed, the variability of bed material characteristics used for input could result in a wide range of scour predictions, depending on the sensitivity of the equation to the bed material characteristics. Therefore, as scour equations are improved by accounting for the effect of bed material characteristics, there will be a commensurate need to ensure that sampling procedures provide representative characteristics of the bed material. If representative bed material characteristics are not obtained, the potential improvements in scour prediction will not be realized.

Table 8. Summary of variability in bed material data from sites in Ohio.

	All Samples						Composite Samples					
Site	**Number of Samples**	**Minimum**	**Maximum**	**Mean**	**SD**	**COV**	**Number of Samples**	**Minimum**	**Maximum**	**Mean**	**SD**	**COV**
41	27	0.02	24.00	7.16	7.27	1.02	8	0.13	15.50	8.11	5.71	0.70
42	17	0.13	11.50	1.91	3.21	1.68	7	0.31	1.00	0.60	0.27	0.45
43	17	0.03	54.00	13.85	17.53	1.27	7	1.40	28.00	9.17	8.71	0.95
44	17	0.70	60.00	16.89	14.38	0.85	8	0.70	17.90	10.70	5.49	0.51
45	17	0.17	13.00	5.02	4.23	0.84	7	2.80	13.00	6.52	3.26	0.50
46	3	0.16	1.60	0.73	0.76	1.04	1	0.43	0.43	0.43	–	–
47	19	0.83	51.00	19.33	15.29	0.79	8	2.35	42.00	18.89	16.48	0.87
48	18	0.01	1.09	0.26	0.30	1.15	8	0.09	0.40	0.23	0.12	0.51
59	17	0.06	7.80	1.39	2.11	1.52	7	0.16	7.80	2.64	2.86	1.08
60	23	0.70	47.90	13.20	10.20	0.77	8	1.00	47.90	14.96	15.19	1.02
61	47	0.01	59.00	18.44	14.51	0.79	7	10.20	28.00	20.64	8.11	0.39
62	17	0.54	46.00	15.28	12.78	0.84	8	10.00	46.00	22.46	12.33	0.55
63	12	0.03	32.00	8.40	9.02	1.07	6	3.50	32.00	12.68	10.78	0.85
64	28	0.01	27.00	8.74	7.96	0.91	8	5.10	20.50	11.45	5.12	0.45
65	17	0.10	2.20	0.92	0.64	0.69	7	0.12	2.00	0.88	0.64	0.73
66	28	0.10	21.00	4.63	6.17	1.33	8	0.41	15.00	5.51	6.22	1.13
67	18	0.47	47.00	15.42	11.98	0.78	8	6.40	31.50	16.00	8.83	0.55
68	6	0.48	17.50	4.75	6.44	1.35	2	4.19	17.50	10.84	9.41	0.87
69	21	0.02	20.60	10.47	7.89	0.75	7	2.20	20.00	14.24	6.57	0.46
70	17	0.03	16.00	2.12	3.88	1.84	5	0.89	3.25	1.77	0.97	0.55
71	16	0.03	8.00	2.20	2.45	1.12	7	0.90	8.00	2.35	2.51	1.07
79	17	0.11	28.00	7.73	9.28	1.20	7	0.84	18.00	7.03	6.77	0.96

SD—standard deviation
COV—coefficient of variation
– —no value

CHAPTER 6: SCOUR CAUSED BY FLOW CONTRACTION AT BRIDGES

GENERAL

Contraction and local abutment scour are a result of flow acceleration caused by encroachment of highway embankments and abutments onto the main channel and (or) floodplain. The accelerated velocity produces an increase in sediment transport from the bridge section until equilibrium conditions are reached or the bed becomes armored. Contraction scour is usually associated with flow acceleration in the contracted opening parallel to the channel and can occur throughout the bridge cross section. Abutment scour only occurs near the abutment and is associated with the localized flow curvature and acceleration near the ends of the bridge abutments as the flow separates from the channel boundary.

Contraction and abutment scour often are treated independently because they are caused by different processes as the flow accelerates through a contracted opening; however, the separation of contraction and abutment scour is often difficult in field observations of scour. Treating the components of scour as independent and additive may be appropriate for wide, shallow bridge crossings, but may not be appropriate for narrow bridge openings. HEC-18 treats contraction and abutment scour as separate and additive because the equations used to compute these scour components were developed with this assumption.

DISCUSSION OF PREDICTIVE EQUATIONS

General

Most scour equations are either theoretically derived or empirically developed from small-scale laboratory research. Controlled laboratory settings tend to oversimplify or ignore many of the complexities present in the natural setting. Laboratory research is typically conducted in straight laboratory flumes with uniform flow, noncohesive uniform bed material, and limited verification of results with field data. Although contraction and abutment scour may be interrelated, laboratory tests typically study these components independently. Before applying a scour prediction equation, it is important to recognize the limitations of the equations, the conditions for which the equations were developed, the methods used to develop them, and how the underlying data were interpreted.

Contraction Scour

Contraction scour has traditionally been classified as live bed or clear water. The live bed condition is characterized by bed material's being transported into the contracted opening from upstream of the bridge. Live bed scour is typical of scour that occurs in the main channel portion of a waterway in high-flow conditions. Clear water contraction scour occurs when the flow conveyed to the bridge crossing is not transporting bed material; thus, all material that is transported from the contracted section is sediment being scoured. Scour occurring on vegetated floodplains may be classified as clear water scour despite the potential for the shear stress in the approach section to be greater than the critical shear stress of the material comprising the floodplains.

Four approaches can be used to estimate the elevation of the scoured bed in a contracted section:

1. Regime relations for channel size and shape.
2. Empirical relations similar to at-a-station hydraulic geometry relations for width and depth.
3. Numerical sediment transport models.
4. Contraction scour equations based on particular sediment transport and uniform flow formulas.

These methods do not compute the depth of contraction scour directly, but rather the equilibrium depth of flow in the contracted section.

Regime equations attempt to quantitatively describe the shape, width, and depth of channels that are "in regime," or have reached equilibrium—a state in which they are neither aggrading nor degrading. Much of the research on the proposed regime equations was based on canals in India during the 1930s and 40s. Regime equations depend highly on the similarity of the site conditions for which they were developed. Because the equations were predominately based on conditions in India and the western United States, they usually are not applicable to areas outside these regions.

Hydraulic geometry relations are similar to regime equations and use at-a-station relations to describe the change in geometry of a single cross section for varying discharge. As with regime equations, these relations are empirical, and the limitations are dependent on the similarity between the site conditions being analyzed and those for which the equations were developed.

Numerical sediment transport models can also be used to compute the depth of scour in a contracted bridge opening. Models are available from several Federal agencies and some private companies. The primary difference among the models is whether they are two-dimensional, one-dimensional, or one-dimensional stream-tube models. The cost of applying the models and the difficulty of setting up the models to accurately replicate field conditions have limited their use for prediction of contraction scour. Numerical models are sometimes used where site conditions are complex and (or) the estimated depth of scour has a significant effect on the design and cost of a proposed bridge.

Semi-empirical contraction scour equations are based on sediment transport and uniform flow formulas. These relations are commonly used and are recommended by FHWA in HEC-18.[(6)] Because these equations are widely used, a more detailed discussion is provided. Following the traditional body of research on contraction scour, these equations are divided into the live bed and clear water classifications.

Live Bed Contraction Scour Equations

Straub was the first to develop an approach to predict contraction scour that most others would follow.[(79)] He assumed that the bed in the contracted section would scour until it reached a depth at which the local transport capacity was equal to the amount of material being supplied from upstream (sediment discharge continuity). He selected the DuBoys sediment transport equation to compute the amount of material supplied to the reach and the local transport capacity in the contraction. Straub estimated the energy dissipation rate (friction slope) in the contracted and uncontracted reaches using Manning's equation. This assumption is reasonable where flow curvature is small and pressure gradients are small compared to boundary stresses. The hydraulics in a short contraction, such as a bridge crossing, require the consideration of additional energy losses not accounted for in a roughness coefficient based on the channel composition and configuration.[80,81,82] Straub's equation, based on sediment discharge continuity, water discharge continuity, and the Manning equation, has the general form of equation 41:

$$\frac{y_2}{y_1} = \left(\frac{Q_2}{Q_1}\right)^{E_Q} \left(\frac{b_1}{b_2}\right)^{E_b} \left(\frac{n_2}{n_1}\right)^{E_n} f(\tau) \tag{41}$$

where

- y_2 is the depth in the contracted section;
- y_1 is the depth in the uncontracted section;
- Q_2 is the discharge in the contracted section;
- Q_1 is the discharge in the uncontracted section;
- E_Q is the exponent on the ratio of discharges;
- b_1 is the bottom width in the uncontracted section;
- b_2 is the bottom width in the contracted section;
- E_b is the exponent on the ratio of bottom widths;
- n_2 is the Manning's n in the contracted section;
- n_1 is the Manning's n in the uncontracted section;
- E_n is the exponent on the ratio of roughness coefficients; and
- τ represents one or more shear-stress variables.

For most applications, the shear stress-based function is assumed equal to unity.

Laursen[61] followed Straub's approach but used his own sediment transport equation.[83] Richardson and Richardson modified Laursen's live bed equation by removing the ratio of Manning's *n* in equation 41.[84] They concluded that Laursen's equation did not account correctly for the increase in transport that would occur if a plane bed existed in the contracted opening with a dune bed configuration in the approach section. For this situation, Laursen's equation would predict less scour than if the roughness coefficients were equal. The Manning's *n* ratio in Laursen's equation does, in fact, behave properly. The basic principle of estimating contraction scour is the assumption of achieving equilibrium sediment transport. With a plane bed configuration more sediment can be transported at a reduced depth than in a dune bed configuration; therefore, equilibrium sediment transport can be achieved at a shallower depth. To achieve a plane bed configuration, the streambed had to progress through the dune bed configuration in the contracted section. A deeper scour may have occurred at a lower flow with a dune configuration in the contracted section than at a higher flow with a plane bed configuration. Therefore, to predict the maximum depth of scour for design purposes, a constant Manning's *n* should be assumed in the approach and bridge sections. This yields the same result as that proposed by Richardson and Richardson.[84]

Culbertson et al. examined all of the previously published research and concluded that none of the existing equations were applicable to all channels or to abrupt contractions, such as bridges and road embankments.(85) They developed their own derivation of Straub's equation using Colby's transport relations for sand bed streams.(86) Culbertson et al. recognized the difference between a long contraction and a bridge contraction and believed that the equations that were developed based on the long-contraction assumption would provide, at best, rough estimates of contraction scour at bridges.(85) In conclusion, Culbertson et al. believed that: "Although laboratory research on alluvial channels may lead to more reliable predictions of scour and fill based on hydraulic theory and empirical equations, the scour and fill problem is inherently complicated, and evaluations based on field experience are needed." (p. 43)(85)

Table 9. Summary of live bed contraction scour equation exponents.

(See equation 41 for definitions of terms.)

Equation	$(Q_2/b_2)/(Q_1/b_1)$	b_1/b_2	Q_2/Q_1
Straub(79)	–	0.43	0.86
Straub(79)	–	0.642	–
Griffith(87)	–	0.637	–
Neill (ed.)(23)	0.67–0.85	–	–
Laursen(61)	–	0.6–0.7	0.86
Komura(88)	–	0.85	–
Komura(88)	–	0.667	–
Culbertson et al.(85)	–	0.667	–

The approach to developing live bed contraction scour equations is very similar among all researchers and differs primarily by the method of determining the sediment transport capacity. Table 9 shows a summary of the exponents of the ratios common to the equations developed by other researchers. There is good consistency in the exponents, considering that each researcher used a different sediment transport equation. In the derivation of the live bed contraction scour equations, the sediment transport equation is applied to both the contracted and uncontracted sections, and only the difference in the transport rates between these sections affects the computed depth of scour. Thus, the depth of contraction scour does not appear to be sensitive to the selection of the transport equation.

The discharge in the contracted and uncontracted sections is typically determined using a one-dimensional step-backwater computer model such as Water-Surface Profile Computations (WSPRO)(89) or Hydrologic Engineering Center-River Analysis System (HEC-RAS).(90) Stream-tube and one-dimensional models distribute the flow in a cross section based on conveyance. The flow distribution in a contracted opening is more dependent upon momentum than on roughness, making conveyance-based flow distributions inaccurate. Experience in applying backwater models indicates that distribution of the flow by conveyance may lead to overestimating the depth of contraction scour. At the uncontracted approach cross section, a conveyance-based flow distribution may place more flow in the floodplain than what actually was present, resulting in the main channel's being too low. Increased flow in the floodplain can result because: (1) the one-dimensional model assumes a constant slope for the whole cross section when, in reality, the downstream water-surface slope varies across the section; (2) the one-dimensional model does not account for flow-path lengths between main channel and the floodplains; and (3) the one-dimensional model does not account for the lateral resistance of the flow moving from the main channel to the floodplains. Conversely, the conveyance-based flow distribution may place too much flow in the main channel at the bridge because the conveyance tubes fail to represent the accelerating curvilinear flow separating from abutments and (or) road embankments. A reduction of the main channel flow in the uncontracted section coupled with an increase of main channel flow at the bridge section could lead to an overprediction of depth of contraction scour.

Clear Water Contraction Scour Equations

Clear water scour occurs where the boundary shear stress in the uncontracted section is less than or equal to the critical tractive force of the bed material, thus, preventing the supply of material into the contracted section. Laursen assumed that the maximum limit of clear water scour occurs when the boundary shear stress is equal to the critical tractive force.(91) This assumption is common among all of the proposed clear water contraction equations. The critical shear stress of a channel with a specific grain size is commonly estimated from the Shields diagram. The critical velocity for incipient motion can be computed from the Shields parameter by substituting the Manning equation for the slope term of the shear-stress equation and then using Strickler to approximate Manning's n.(92) A critical velocity equation (equation 3) can be obtained by rearranging terms and solving for velocity. By setting the velocity in the contracted section equal to the critical velocity and solving for depth, the following generic clear water contraction scour is obtained:

$$y_2 = \left(\frac{V_2^2}{\theta \left(K_u 31.08\right)^2 D_m^{2/3}} \right)^3 \tag{42}$$

where

V_2 is the average velocity in the contracted opening in meters per second; and
D_m is the mean grain size of the bed material, in meters.

On the basis of research on the effective size of bed material for riprap design and resistance to erosion presented in Richardson et al.,(93) Richardson and Richardson suggest that 1.25 D_{50} be used for D_m.(84)

The adequate determination of the critical velocity or critical shear stress and the corresponding scour is not well established for channels having cohesive bed material, bed material that varies with depth, heavily vegetated floodplains, previously developed scour holes, and armored beds. Laursen, Neill, White, Iwagaki, and Shields have all developed clear water scour relations, but all are based on assumptions that tend to oversimplify the conditions present in most field situations. (See references 94, 91, 95, 96, and 97.) The clear water scour equations were developed for flat beds, but only after a scour hole develops the hydraulics change, affecting the accuracy of the prediction equations. Proper representation of bed material is also critical to the application of clear water scour equations. Bed material samples should represent both the surface and subsurface material. Clear water contraction equations can be applied to each consecutive layer at sites where layered soils are present by modifying parameters as each layer is eroded and the next is exposed. Until recently, the effects of armoring were not accounted for in determining the depth of scour. Froehlich[98] presented a method based on the active-layer approach by Borah[99] to estimate the depth of scour required to obtain an armor layer of sufficient size and thickness to limit clear water scour.

Abutment Scour

The current knowledge of prediction of scour at abutments is derived from regime theory equations, equations used to estimate the depth of scour for spur dikes, and equations developed from small-scale physical model studies conducted in laboratory flumes. Unfortunately, none of these approaches have resulted in a satisfactory prediction equation. These approaches' inability to accurately predict scour at abutments is a result of the simplifying assumptions on which the research is based and the complexity of abutment scour in field conditions. The configuration of bridge abutments and associated embankments is complex when placed in the context of river hydraulics.

Field Conditions

The geometric configuration greatly affects the way flow is directed around the abutments. The abutment may be located in the channel, at or near the top bank, or on the floodplain. The configuration of the abutment may be vertical, have wing walls at various angles, or have a spill slope protected with riprap or some other armoring material. Although abutments with spill slopes are usually protected, the armoring can fail or be undermined by scour causing the abutment configuration to change during a flood. The embankments may not be perpendicular to the approach flow but may be angled either upstream or downstream. Drainage ditches along the toe of the embankment are common and complicate the flow patterns around the abutment.

The natural flow distribution in a river can also have a significant effect on the depth of scour at an abutment. The distribution of the approach flow blocked by the embankment is dependent on the roughness and topography of the floodplain and alignment of the main channel. The flow distribution and direction can change significantly during a flood hydrograph. Such complexity and the variability of these conditions between sites are major obstacles in developing a reliable method for predicting scour at abutments.

Discussion of Equations

Some abutment scour equations are based on data for scour at the end of spur dikes. The most notable equation of this type was published by Richardson and Huber as an equation for predicting scour when the embankment length to flow depth ratio exceeds 25.[100] This equation uses only the Froude number and depth of flow and does not account for the length of the embankment.

Most equations for predicting scour at abutments are based on small-scale physical model studies. Literature documenting the laboratory experiments reveals that the approach section of the flume usually had a constant depth with a uniform velocity distribution. The roughness of the channel was also typically uniform throughout the approach and bridge sections, and the bed material was generally composed of uniform sand. The abutments were represented by solid, nonerodible obstructions protruding from the sides of the flume with ends that varied in configuration to represent typical shapes of embankments and abutments at contracted bridge openings. These conditions are different from the conditions that occur in the field.

Several researchers have attempted to account for some conditions commonly found in the field. Dongol, Melville, and Sturm and Janjua have used models that incorporate the floodplain, some channel geometry effects, and nonuniform flow distributions.[101,102,103] Unfortunately, the amount of field data on abutment scour that can be used to evaluate the validity of the laboratory studies is limited.

The laboratory research, although not in agreement, has typically used some combination of the following variables to predict scour at abutments: (1) embankment length, (2) abutment shape, (3) depth of flow, (4) velocity, and (5) discharge. Because of the simplicity of many of the laboratory experiments, the variables in equations developed from these experiments are ambiguous when these equations are applied to field conditions. In simple flume studies, the flow depth is uniform everywhere, and there is no way to define what depth is controlling the depth of scour (the depth at the abutment, the depth in the approach upstream of the abutment, or an average depth of flow blocked by the abutment). In the field, these may all be different values; however, in the laboratory with a uniform bed they are all the same. The velocity term is also a good example of potential ambiguous variables in the field. The velocity in the contracted opening adjacent to the abutment (which would represent the accelerated curvilinear flow) would be different from the unobstructed approach velocity upstream of the abutment or the average velocity of the approach flow blocked by the length of the embankment. Flume studies often use the total length of the embankment from the flume wall to the abutment as the embankment length; however, this approach fails to account for the flow separation and recirculation zone that forms along the upstream edge of the embankment (figure 30). The effective length of the embankment on the depth of scour depends on the distribution of the approach flow and the floodplain roughness and geometry. It is important that laboratory research emulate the conditions in the field so that equations developed are representative of field conditions.

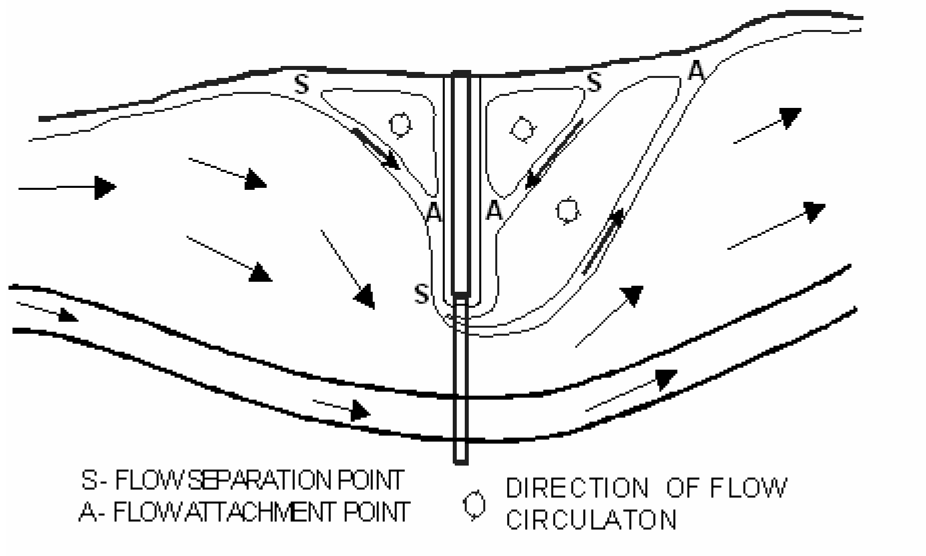


Figure 30. Illustration of flow contracted by an embankment constructed in a floodplain.

EVALUATION OF PUBLISHED FIELD OBSERVATIONS

General

Numerous papers and reports have been written on the various aspects of scour at bridges; however, complete and reliable field data are rare. Table 10 summarizes a literature survey that yielded 29 references that might have contained field data for abutment and contraction scour. A brief summary of selected references is presented.

Summary of Selected References

Research on backwater and discharge computations at single and multiple bridge openings, at bridges with openings that have been excavated, and at bridges with guide banks provided data at 60 bridges. (See references 81, 104, 105, 106, 107, and 108.) These data were collected at sites with heavily vegetated floodplains where complete real-time detailed data sets would be difficult to collect. The USGS, Mississippi District, has summarized data for two of the sites (figures 31 and 32) and made comparisons with computations based on contraction scour equations recommended in HEC-18.[109] Lithologic logs available for the site at Edinburg, MS, clearly showed the elevation of a hard clay layer (figure 31). No detailed bed material data were available for the site at Burnside, MS (figure 32); however, attempts to collect a sample of surface bed material indicate a hard surface; small amounts of clay were recovered.

Table 10. Summary of published field data for contraction and abutment scour.

Reference	Abutment	Contraction
Inglis[34]	–	G
Lane and Borland[110]	–	G
Wilson[104]	–	A
Colson and Wilson[105]	–	A
Colson and Wilson[106]	–	A
Norman[16]	–	X, C
Schnieder et al.[81]	–	A
Colson and Schnieder[107]	–	A
Blodgett[111]	–	A
Blodgett[26]	–	–
Wilson, Jr.[108]	–	A
Crumrine[112]	–	A, G
Blodgett and Harris[113]	–	X
Ahmed et al.[114]	–	A, X
Butler and Lillycrop[115]	–	G
Fischer[17]	X	–
Holnbeck et al.[18]	X	X, C
Fischer[116]	–	A
Brabets[117]	–	X, C
Fischer[118]	–	X, C
Fischer[118]	–	A
Monk[119]	–	A, G
Hagerty et al.[120]	–	A, G
Crumrine et al.[121]	–	G
Hayes[122]	–	X, C
Jackson[42]	–	X
Parker[123]	G	–
Benedict and Caldwell[124]	G	G
Mueller and Hitchcock[125]	A, G	A, G

X—data presented
A—additional analysis required
G—general observations
C—comparison with computed scour included

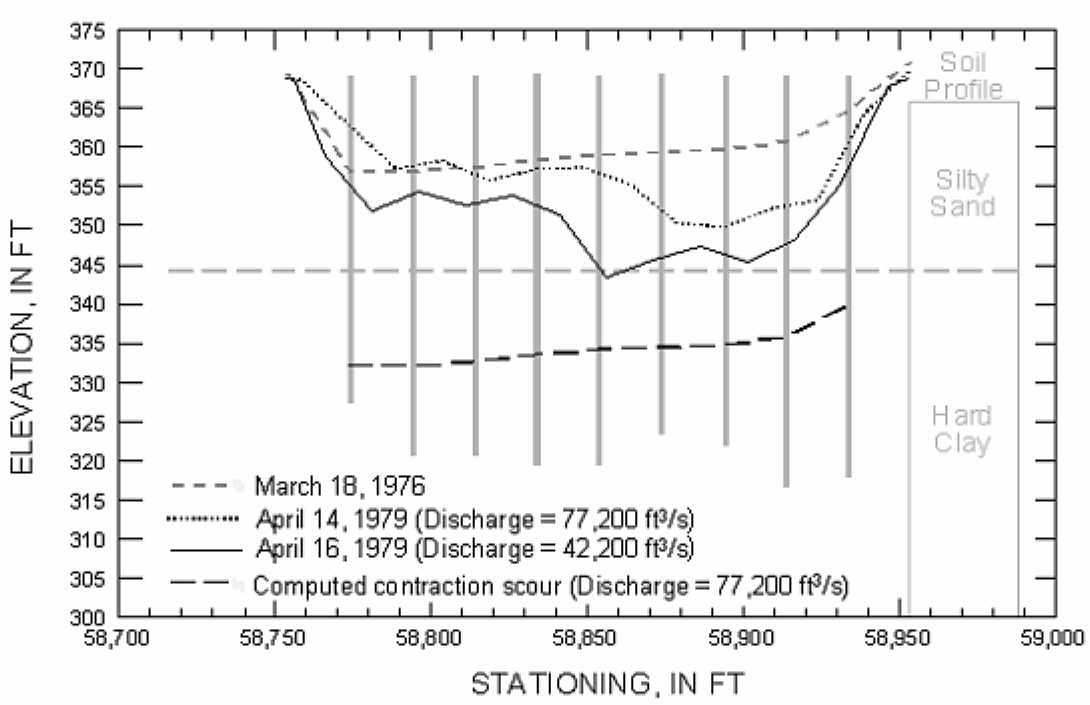


Figure 31. Comparison of measured and computed contraction scour at State Route (S.R.) 16 over the Pearl River near Edinburg, MS, at the left (east) relief bridge (1 ft = 0.305 m).

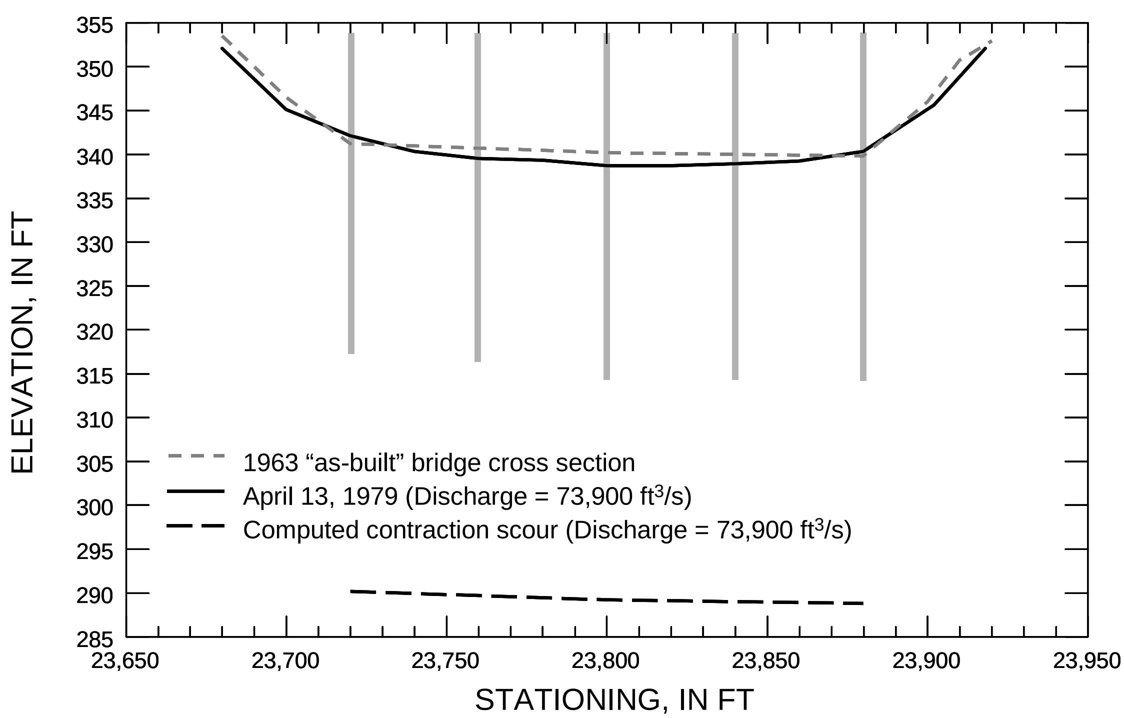


Figure 32. Comparison of measured and computed contraction scour at S.R. 15 over the Pearl River near Burnside, MS, at the left (south) relief bridge (1 ft = 0.305 m).

Norman presented detailed data and analyses of seven sites in Alaska; these are the most detailed and complete data sets available in the literature.(16) These data include concurrent measurements of uncontracted and contracted sections during floods, bed material data, velocity data, and detailed discussions of the site and data collection procedures. The local scour portions of these data are included in the national bridge scour database.(21,22) A summary table for the three sites with contraction scour measurements and a comparison with computed values of mean depth in the contracted section is presented in table 11.

Table 11. Comparison of measured mean depth to calculated mean depth at Alaskan bridges where contraction was present during flood flows (modified from Norman(16)).

Location	Section Number	Measured Depth (m)	Computed Depth of Scour (m) Straub	Laursen	Komura
Susitna River, bridge 254,	1	4.51	Uncontracted section		
8/11/71	2	4.78	4.94	4.97	5.09
	4	5.49	6.10	6.25	6.89
	5	5.61	6.16	6.34	7.04
Tazlina River, bridge 573,	1	2.80	–	–	–
9/4/71	2	3.14	2.99	2.99	3.60
Tanana River, bridge 524,	1	3.96	Uncontracted section		
8/13/71	2	4.69	5.09	5.09	2.99

Fischer presented post-flood analysis data on contraction scour occurring at S.R. 2 over the Weldon River in south-central Iowa.(17) The bridge is 68 m long and is supported by two monolithic piers and spill-through abutments. The area consists of rolling hills that surround a wide river valley with a floodplain width of 670 m at the site. Bed material at the site consists of banks of sandy, silty clay with sand and occasional boulders in the streambed underlain by glacial clay. The peak discharge of 1,930 m^3/s was determined by a combined contracted-opening and road-overflow measurement of peak discharge using the techniques outlined by Matthai and Hulsing.(80,126) The estimated peak discharge was about four times the 100-year design flood. High-water marks indicated that the water-surface fall through the bridge was 1.45 m; the depth of flow over the roadway was about 0.8 m. Bed profiles measured along the upstream and downstream edges of the bridge after the flood showed that the spill-through slope at the left abutment was scoured about 4 m to approximately the top of the glacial clay. The main channel portion of the bridge did not show appreciable scour when compared to the cross section from the construction plans; however, the bed elevation was very close to the upper limit of the glacial clay and may have refilled with sand before the post-flood measurements. The average velocity through the submerged bridge opening was estimated to be 3.0 m/s at the peak, but it was likely greater when the entire discharge passed through the bridge before overtopping. Downed trees, sand deposition on the floodplain, cobble deposition in the main channel, and eroded banks were evidence of the high velocity that passed through the bridge. None of these characteristics were observed upstream of the bridge.

Holnbeck et al. presented a case study of contraction and abutment scour at U.S. 87 over Razor Creek in Musselshell County, MT.[18] Razor Creek is an ephemeral stream with a drainage area of 44.3 km^2. The bridge, constructed in 1955 and recently replaced, had a span of 22.9 m with two timber-pile bents at 7.6-m spacing. The streambed has a sand and gravel layer ranging in thickness from 1.2 to about 4.4 m, and a grain size distribution of D_{10}, 0.065 mm; D_{50}, 2 mm; and D_{90}, 20 mm. Very dense, tan sandstone and weathered shale underlie the sand and gravel to a depth of at least 10.7 m. Peak discharges at the bridge resulting from thunderstorms in July 1986 and June 1991 were estimated by the USGS using the width-contraction, indirect measurement method.[80] These two floods are believed to be about three and four times the estimated 100-year peak discharge, respectively. Contraction scour is probably not significant at this site for floods less than the 1991 flood because the indirect measurement of the 1986 peak discharge indicated no water-surface drop through the bridge, but a water-surface drop of 2.4 m was observed for the 1991 flood. The components of total scour were computed from hydraulics estimated from a WSPRO model using the Laursen live bed contraction scour equation, the Froehlich live bed abutment scour equation, and the HEC-18 pier scour equation (table 12). The observed scour at the abutments included loss of riprap that had been placed before the 1991 flood, which may have reduced the depth of scour. The presence of sandstone in the area near the right abutment may have also limited scour depth in that area.

Table 12. Comparison of computed and measured scour at U.S. 87 on Razor Creek, MT, June 1991 (modified from Holnbeck et al.[18]).

Location	Computed Scour (m)			Observed Scour (m)
	Contraction	Local	Total	
Left abutment	0.70	2.50	3.20	0.85
Right abutment	0.70	3.66	4.36	2.23
Pile bents	0.70	1.43	2.13	0.94

Brabets presented an analysis of scour at 12 bridges located along the Copper River Highway from Flag Point to the Million Dollar Bridge in Alaska.[117] The lower Copper River is a complex and dynamic river system. The bridges analyzed are over distributary channels in the approximately 777-km^2 delta, and these channels constantly scour and fill, causing lateral channel migration. The bed material at bridges 331 and 1187 is medium gravel. Spur dikes were constructed at the left upstream abutment of bridge 331 and at both upstream abutments of bridge 1187. An approach section surveyed in May 1992 at each bridge was used to evaluate the contraction scour at this site (table 13).

Table 13. Measured and predicted mean depth of flow at bridges 331 and 1187 on the Copper River Highway, Alaska, in May 1992 (modified from Brabets[117]).

	Measured Mean Depth of Flow (m)		Predicted Mean Depth of Flow, y_2 (m)		
Bridge	y_1	y_2	**Straub**	**Laursen**	**Komura**
331	2.86	3.29	4.21	4.08	4.82
1187	2.04	2.38	2.50	2.47	2.68

Fischer presented a case study of scour caused by flooding in September 1992 at S.R. 14 over Wolf Creek in Iowa.[118] The bridge is a 30.5-m single-span steel structure supported by vertical wall concrete abutments with wingwalls. The floodplain is primarily agricultural and is approximately 400 m wide near the bridge. Dense woods line both the upstream and downstream sides of the highway near the bridge. Along the right upstream floodplain is woody riparian vegetation along the stream; pasture sits further back on the floodplain. The estimated peak discharge of 2,200 m^3/s was determined based on high-water marks and modeling of the flow using WSPRO. Fischer evaluated gravel deposits left when the road was overtopped and estimated the fall through the bridge at incipient road overtopping was 3.4 m.[118] The hydraulic analyses indicated that the flow may have been supercritical through the bridge opening at that time. The water-surface elevation continued to rise following the overtopping and completely submerged the bridge section; the fall through the bridge was reduced to 1.94 m. Information from the highway department indicates that backwater caused by channel constrictions downstream eventually reduced the fall to about 0.3 m and eventually submerged the bridge section. Using the streambed profile from the 1946 bridge plans as a reference, estimates were that 6 m of contraction scour occurred during this event. Using the methods outlined in Richardson et al., the depth of contraction scour was computed to be 9.1 m.[59] A pond about 75 m long and 50 m wide downstream of the bridge remained after the flood had receded. This large area was scoured by the high-velocity flow exiting the contracted opening.

Hayes analyzed records at gauging stations in Virginia and Maryland and identified four sites with contraction scour.[122] Routine streamflow measurements do not contain an approach-and-exit section to use as a reference surface. Hayes analyzed bed-elevation trends in the record and selected only stations where no long-term trends were present; he used the cross section geometry that existed before the bridge was constructed as the reference surface for these stable sites. Each site was visited and the appropriate cross sections surveyed and bed material samples collected. WSRPO was used to model the hydraulics at each site to allow computation of contraction scour according to the procedures recommended by Richardson and Davis.[6] Table 14 presents the data and computed depths of contraction scour. The sites on Big Pipe Creek and the Northeast Branch Anacostia River are coarse bed material streams with a D_{50} of about 20 mm. The other two sites are sand bed streams. It can be seen from the results in table 14 that the equations frequently underestimate, and no gross overestimation of scour occurred.

Table 14. Comparison of measured mean depth to calculated mean depth at bridges where contraction was present during flood flows (modified from Hayes[122]).

Site	Date	Discharge (m^3/s)	Mean Velocity (m/s)	Measured Contraction Scour (m)	Laursen Clear Water Scour (m)	Laursen Live Bed Scour (m)
S.R. 194 over Big Pipe Creek near Bruceville, MD						
	12/30/48	65.1	1.23	1.04	0.18	–
	1/5/49	51.0	1.18	1.10	0.12	–
	6/23/72	98.8	1.15	1.07	0.43	–
	3/19/75	54.9	1.18	0.73	0.00	–
	9/25/75	109.0	1.43	0.64	0.58	–
	10/23/90	75.3	1.23	0.82	0.27	–
	10/23/90	74.5	1.10	0.82	0.34	–
S.R. 410 over Northeast Branch Anacostia River at Riverdale, MD						
	4/20/40	65.7	1.06	0.27	0.00	–
	4/20/40	54.9	1.05	0.21	0.00	–
	8/9/42	83.5	1.14	0.30	0.06	–
	7/27/45	66.5	1.03	0.34	0.00	–
	7/27/45	59.5	0.99	0.30	0.00	–
	4/27/52	55.2	1.04	0.34	0.00	–
	4/13/61	58.3	1.47	0.40	–	–
	8/25/67	86.4	2.02	0.03	0.85	–
S.R. 614 over Pamunkey River near Hanover, VA						
	8/25/69	447.3	0.80	2.13	–	1.89
	6/1/71	265.9	0.66	1.40	–	1.13
	10/29/71	272.1	0.68	1.34	–	1.19
	6/25/72	512.5	0.89	2.26	–	1.98
	3/21/75	311.4	0.80	0.85	–	1.25
	4/19/83	223.1	0.60	0.76	–	0.91
	8/20/85	334.1	0.76	1.25	–	1.16
	5/31/90	419.0	0.87	0.58	–	1.65
	1/14/91	216.6	0.55	0.85	–	0.88
	4/12/93	253.7	0.59	1.07	–	1.01
	4/13/93	233.0	0.54	1.04	–	1.01
	4/19/93	250.3	0.59	1.10	–	1.07
	11/30/93	331.3	0.73	1.31	–	1.31
	3/31/94	546.4	0.91	2.10	–	1.92
S.R. 42 over Youghiogheny River at Friendsville, MD						
	2/3/50	91.7	1.38	0.18	0.00	–
	1/11/57	98.2	1.35	0.30	0.00	–
	5/8/67	142.4	1.85	0.30	0.00	–
	9/14/71	118.9	1.70	0.21	0.00	–
	2/25/77	109.3	1.55	0.24	0.00	–
	11/5/85	362.4	3.05	0.03	0.73	–
	7/13/90	212.3	2.29	0.30	0.15	–
	4/1/93	136.5	1.71	0.15	0.00	–

Jackson presented field data and associated analysis for sites in Ohio that were originally selected to minimize contraction and focus on pier scour; thus, contraction scour was only observed at three sites.[42] Data were collected from the bridge deck, so no concurrent approach-and-exit sections were measured. Jackson assessed the stability of the streambed elevation and concluded that the streambed was relatively stable and that use of the low-water approach-and-exit sections as the reference surface did not introduce significant error (less than 0.2 m). The data are presented in table 15, but no comparison with computed scour was presented.

Table 15. Contraction scour data published by Jackson.[42]

Measurement Site	Date	Measured Contraction Scour (m)	Sediment Transport Condition
U.S. 33 over Clear Creek near Rockbridge, OH	1/28/94	1.5	Clear water
S.R. 128 over Great Miami River at Hamilton, OH	7/18/92	2.3	Live bed
S.R. 22 over Todd Fork at	5/17/90	0.8	Live bed
Morrow, OH	12/18/90	0.9	Live bed

Benedict and Caldwell presented an interim summary of a study to measure and analyze the characteristics of clear water scour at bridges in the Coastal Plain and Piedmont areas of South Carolina.[124] The data are being collected during low water and reflect the maximum scour resulting from all historical floods at a given site. Field data collection techniques allow the identification and adjustment of scour depths for any infilling that may have occurred. Although no real-time hydraulic data are being collected, the investigators plan to use WSPRO to model and compare the measured scour depths with computed depths of scour obtained by applying the methods recommended in Richardson and Davis.[6]

The Coastal Plain, which comprises about 63 percent of South Carolina, is characterized by thick sand deposits, relatively flat stream slopes (0.0002 to 0.004 m/m), and heavily vegetated floodplains. The lower Coastal Plain contains many swamps that have a heavily vegetated floodplain drained by a network of shallow, poorly defined channels. The root masses of the vegetation significantly impede the transport of bed sediments, thus creating clear water scour at bridge contractions. The typical scour hole at sites with smaller bridges (bridge length of 61 m or less) was observed to develop as a single, long, and deeply cut channel extending from just upstream of the bridge to as much as 60 m downstream. Flows from around the left and right abutments overlap, and scour is concentrated near the middle of the bridge. As the bridge length increases, the observed pattern of scour changes to two separately cut holes near the abutments. The patterns described are common, but the scour patterns vary depending upon site-specific conditions. The maximum clear water scour observed at 63 sites in the Coastal Plain ranged from 0.4 to 7.2 m.

The Piedmont represents about 35 percent of South Carolina; it is characterized by clayey soils, moderately steep stream slopes (0.001 to 0.01 m/m), and incised channels on heavily vegetated floodplains. Clear water contraction and abutment scour occurs where the abutments are set back from the main channel and the embankments contract the flow in the floodplain. The typical clear water contraction scour pattern on an overbank is a shallow swale running parallel with and directly under the bridge. The typical clear water abutment scour pattern is a shallow swale oriented upstream to downstream near the abutment toe. Deviation from these patterns is infrequent where the soils are predominantly clay. The maximum clear water scour observed at 47 sites in the Piedmont range from 0 to 1.4 m. Although sandy floodplains are rare in the Piedmont, the clayey soils at some sites have a high sand content. These sites typically have wide floodplains with a pronounced contraction at the bridge. The maximum clear water scour observed at 18 Piedmont sites with sandy floodplains ranges from 0.9 to 5.5 m.

The summary of data presented by Benedict and Caldwell provides field validation of concepts commonly accepted, but previously not documented by field data.(124) The paper does not present any of the raw data, as the analysis of these data is ongoing by the USGS, South Carolina District. The results of the project will provide valuable field-based insight into clear water scour processes and will inform a substantial database for future research.

Of the 29 references, only Norman presented detailed data collected during floods.(16) Nearly all of the sites presented in the literature would require the compilation of the raw data and additional analysis to obtain complete abutment and contraction scour data sets. Only two references included data on abutment scour.(18,118) Seven papers presented data on contraction scour, and 14 discussed sites that could yield contraction scour data if additional data were available to supplement the limited data presented in the papers.

The comparison of field data with computed scour showed mixed results. Papers by Norman, Holnbeck et al., Brabets, and Fischer showed that the contraction scour equations typically overpredicted the observed scour and in a few instances, severely overpredicted the scour. (See references 16, 18, 117, and 118.) The paper by Hayes showed that the Laursen contraction scour equation underpredicted a number of measurements, and no severe overprediction was present.(122) The accuracy of the contraction scour equation may depend greatly upon the degree of contraction, the flow distribution, the configuration of the approach, and how well the hydraulic model represents the true flow distribution.

ANALYSIS OF REAL-TIME DATA

General

During record flooding in the Minnesota River Basin in April 1997, USGS, in cooperation with FHWA, deployed the USGS bridge scour data collection team to collect real-time scour (contraction and local) measurements at contracted bridge openings. An analysis of two sites that were surveyed during the April 1997 flooding is presented. Both contracted bridges span the Pomme de Terre River, where an estimated 200-year discharge was measured at the USGS Appleton gauging station (05294000) located approximately 19 km downstream of the U.S. Route 12 bridge. The compiled field data (channel and floodplain bathymetry, water discharge, water-surface elevations, roughness, and bridge geometry) were used to calibrate a step-backwater model at each site. The hydraulics and predicted depth of scour based on the calibrated model were compared with the field measurements.

U.S. Route 12 over the Pomme de Terre River

Site Description

U.S. Route 12 crosses the Pomme de Terre River about 20 km west of Danvers, MN. The single-span steel-truss structure was constructed in 1933 with a maximum span length of 26.9 m. The bridge has vertical wall abutments with wing walls; each abutment and wing wall rests on concrete footings supported on timber piling. Neither abutment was riprapped, nor were there any other scour-protection measures. A field investigation conducted in 1995 by a local engineering firm, BRW, Inc., revealed no evidence of significant scour at the abutment face.[(127)]

The upstream floodplain consists of a mixture of open agricultural land with scattered trees and brush, with a park on the upstream left bank. The area downstream of the bridge is more heavily wooded and is classified on the USGS topographic map as a wetland area. The streambed material near the bridge generally consists of fine-grained, organic, silty sand with some gravel. A sieve analysis of a surficial bed material sample indicated a median diameter of 0.15 mm. Based on the soil borings and blow counts documented in the bridge plans, the bed material appears to become harder and denser as depth increases. Because samples were not collected and analyzed, it is difficult to ascertain the makeup of the soils at depths below the surface.

During the April 1997 flood the bridge experienced both contraction and abutment scour. A large scour hole developed at the right abutment, scouring below the abutment cutoff wall and resulting in failure of the fill material behind the abutment. Slumping of the embankment slope and some deformation of the approach highway were observed. Although scour measurements showed a scour hole 2 m below the footing of the left abutment, no deformation was observed near the left abutment. These conditions resulted in closure of the bridge. Because of the age and scheduled replacement of the bridge, it was not repaired but was replaced with a new structure after the flood.

Discussion of Field Data

Data were collected during the flood (on April 5 and 9, 1997) at U.S. Route 12 over the Pomme de Terre River. A crewed boat was deployed during the initial visit on April 5, 1997. The use of the crewed boat and an ADCP allowed bathymetry and three-dimensional velocities to be measured at the bridge and in the approach and exit sections extending about 100 m upstream and 70 m downstream. Heavy vegetation and submerged obstructions in the floodplains limited data collection to the main channel. Measurements on April 9 were limited to data collected from the bridge deck. Channel bathymetry was measured along the upstream and downstream faces of the bridge and at selected locations beneath the bridge using an echo sounder deployed on a knee-board. Velocity magnitudes and water discharge were measured using a vertical axis current meter. Water-surface elevations were measured by taping down from the top-of-curb on the bridge both upstream and downstream, near the left abutment. On April 5, the water-surface elevation was 310.70 m above National Geodetic Vertical Datum (NGVD) of 1929 at the upstream edge of the bridge, and the total discharge was 141.6 m^3/s. By April 9, the water-surface had risen to an elevation of 311.5 m above NGVD of 1929, and the discharge had increased to 162.8 m^3/s.

The direction of flow through the bridge was controlled by the configuration of the upstream floodplains. The channel upstream of the bridge was straight, but the left floodplain was much wider and carried considerably more flow than the right floodplain. The flow from the left floodplain skewed the flow through the bridge opening to about 50° on average. Figure 33 is a sketch of spot elevations and the flow direction on April 9, which shows the severe skew of the flow to the bridge opening.

The appropriate reference surface was determined from an analysis of cross sections collected by BRW, Inc., on June 5, 1995 and by the USGS during the flood on April 5, 1997. On these two dates, cross sections collected approximately 90 m upstream from the bridge showed only about 0.15 m difference in the channel-bottom elevation. The flood cross section was the lower of the two. Downstream from the bridge, the cross section surveyed on June 5, 1995 (approximately 23 m downstream) had less than 0.3 m in variation when compared with the cross section surveyed on April 5, 1997 (approximately 61 m downstream), but was 0.5 m higher than the cross section surveyed on April 5, 1997 (approximately 30 m downstream). It is possible that the April 5, 1997, cross section could have been affected by the scour at the bridge section; thus, it was not considered in setting the reference surface. The WSPRO bridge section surveyed by BRW, Inc., on June 5, 1995, showed from 0.3 to 0.6 m of abutment scour in the cross section; however, the center of the channel at the bridge appeared to be representative of consistent channel slope from the upstream section to the downstream section. Because little general scour was observed at the upstream and downstream sections, the mean elevation of the unscoured portion of the WSPRO bridge section (elevation 307.9 m) was used as the contraction scour reference surface.

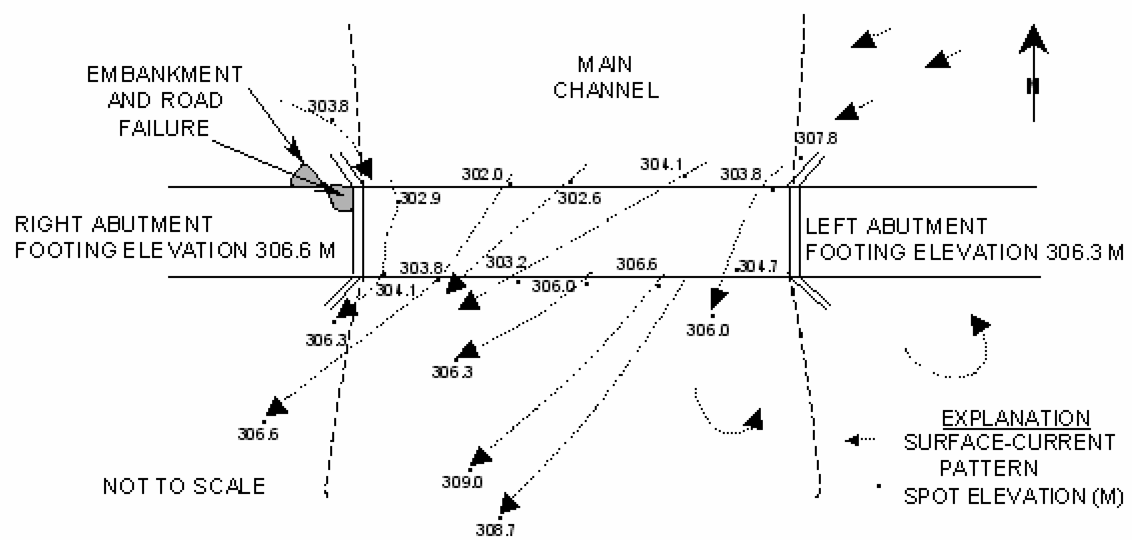


Figure 33. Illustration of U.S. Route 12 over Pomme de Terre River, Minnesota, showing spot elevations and surface current patterns on April 9, 1997.
(Elevations are in meters referenced to NGVD of 1929.)

A summary of the contraction scour data is shown in table 16. The contracted section on April 5, 1997, was measured under the bridge from data collected by an ADCP. The maximum erosion of the streambed was 2.3 m from the defined reference surface; however, when the entire streambed below the bridge was averaged the depth of contraction scour was only 0.9 m. The hydraulic data presented for April 5, 1997, were collected with the ADCP. The ADCP data had a significant amount of invalid data that were estimated in final processing. Channel banks were not clearly delineated in the approach section, creating a degree of uncertainty in the approach discharge. Overall, it is suspected that the approach discharge is ±20 percent, and the total discharge is ±10 percent. Measurements made with a sounding weight on April 9, 1997, were collected during the discharge measurement along the upstream face of the bridge, and no approach data are available. An echo sounder mounted on a knee-board was also used to make measurements on April 9, 1997. The board was floated from upstream to downstream under the bridge; the measurements reflect the depths at the upstream or downstream face of the bridge.

Table 16. Summary of contraction scour measurements at U.S. Route 12 over the Pomme de Terre River in Minnesota.

Measurement Number	Date	Location	Equipment	Scour Depth (m)	Accuracy (m)
1	4/5/97	Centerline	ADCP	0.9	0.6
2	4/9/97	Upstream	Sounding weight	3.2	0.6
3	4/9/97	Upstream	Echo sounder	3.8	0.6
4	4/9/97	Downstream	Echo sounder	1.4	0.6

	Contracted Section				Uncontracted Section			
Measurement Number	Width (m)	Depth (m)	Discharge (m^3/s)	Average Velocity (m/s)	Width (m)	Depth (m)	Discharge (m^3/s)	Average Velocity (m/s)
1	26.8	3.7	142	1.5	21.3	2.4	51.0	1.0
2	26.8	7.3	163	0.8	–	–	–	–
3	26.8	7.2	163	0.9	–	–	–	–
4	26.8	5.3	163	1.2	–	–	–	–

The reference surface used to determine the depth of abutment scour was the concurrent ambient bed; therefore, the depth of abutment scour reported is additional local scour below the depth of contraction scour (table 17). The data collected on April 5, 1997, were collected with an ADCP using a weighted average of all four beams as the measured depth. Because a weighted average was used, it is possible that the local abutment scour was not accurately measured, and no values are reported. The cross sections measured on April 9, 1997, all showed a similar pattern with abutment scour holes on each side and a sharp mound in between the scour holes but skewed toward the left bank (figure 34). It appears that the abutment scour holes may have overlapped. The highest elevation in the center of the cross section was subtracted from the reference surface to obtain the depth of contraction scour. The abutment scour was reported as the depth below the highest elevation in the center of the cross section. All velocities presented in table 17 were from the discharge measurement made along the upstream side of the bridge. Although no abutment scour was observed on April 5, 1997, the velocities at the abutments were much higher (left = 1.6 m/s, and right = 1.8 m/s).

Table 17. Summary of abutment scour data for U.S. Route 12 over the Pomme de Terre River in Minnesota.

Date	**Abutment**	**Location**	**Equipment**	**Scour Depth (m)**	**Accuracy (m)**	**Embankment Length (m)**	**Velocity at Abutment (m/s)**	**Depth at Abutment (m)**
4/9/97	Right	Upstream	Sounding weight	2.4	0.6	307	1.3	9.1
4/9/97	Right	Upstream	Echo sounder	2.1	0.6	307	1.3	9.4
4/9/97	Right	Downstream	Echo sounder	3.4	0.6	307	1.3	8.2
4/9/97	Left	Upstream	Sounding weight	0.9	0.6	121	1.2	7.6
4/9/97	Left	Upstream	Echo sounder	0.5	0.6	121	1.2	7.6
4/9/97	Left	Downstream	Echo sounder	1.8	0.6	121	1.2	6.7

Model Calibration

The HEC-RAS model, a one-dimensional step-backwater model, was calibrated to represent the field hydraulics as accurately as possible. The bathymetry from the April, 1997, flood was used to build the calibration models for the two sets of data (April 5 and 9, 1997). Because bathymetry data on April 9, were limited to the upstream and downstream edges of the bridge, the cross sections collected on April 5, were used to build the HEC-RAS model for April 9. The majority of the floodplain bathymetry utilized in building the models was taken from a full valley section found in the original bridge plans and adjusted to be consistent with topographic maps.

The water-surface elevation observed in the field rose 0.76 m between April 5 and 9. The model only showed a 0.3 m change and was unable to reproduce the observed change without unreasonable changes to the model input. This large hydraulic variation may be attributed to the U.S. Route 12 bridge reach being under a backwater condition because of some unidentified downstream condition. Large ice drifts were observed during both site visits, indicating the potential for the formation of a debris and (or) ice dam downstream of the data collection area. Analysis of the Appleton gauging station records was of little assistance because the gage was washed out on April 6, 1997, by the failure of a small upstream dam. The water-surface elevation at the upstream side of County Route 22 located about 10 km upstream changed only 0.2 m over the same period; therefore, the model was considered adjusted despite the apparent discrepancy with the water-surface elevation observed on April 9.

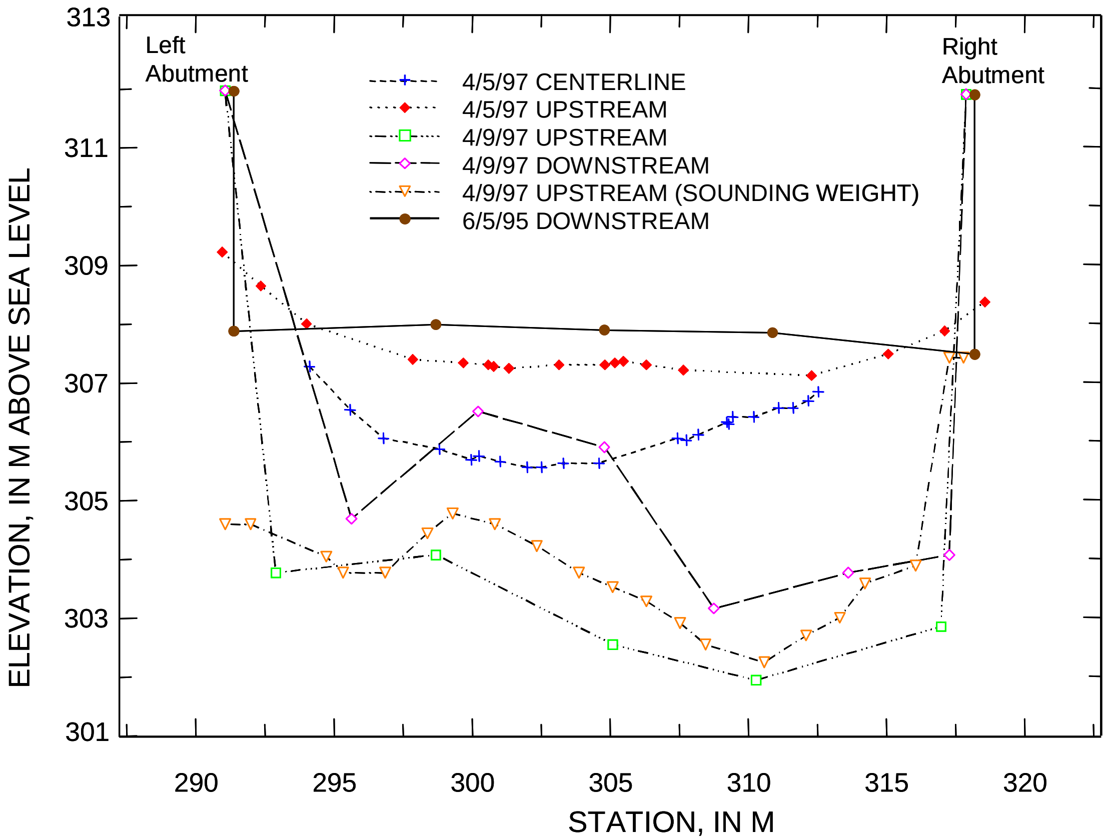


Figure 34. Measured cross sections at U.S. Route 12 over the Pomme de Terre River in Minnesota.

One of the most important factors in using one-dimensional models at contracted bridges is the ability for the model to accurately represent the velocity distribution laterally across the stream and floodplain. Figures 35 and 36 depict how the velocity distribution varied between the model and field measurements, using the geometry from April 5 and 9, 1997. The distribution shown in figure 35 reveals that the flow in the field was indeed skewed toward the right abutment. HEC-RAS did not duplicate this skewed flow pattern, but rather computed a relatively uniform flow distribution across the cross section because the model assigned flow tubes of equal conveyance through the geometrically uniform bridge section. For the scoured channel bathymetry, HEC-RAS did a better job of reproducing the observed velocity distribution (figure 36), although the model does not recognize the region of reverse flow that occurred adjacent to the left abutment. The HEC-RAS computed velocities are greater near the deeply scoured region adjacent to the right abutment because the slope and roughness are constant across the cross section, so the conveyance becomes dependent upon the depth of flow.

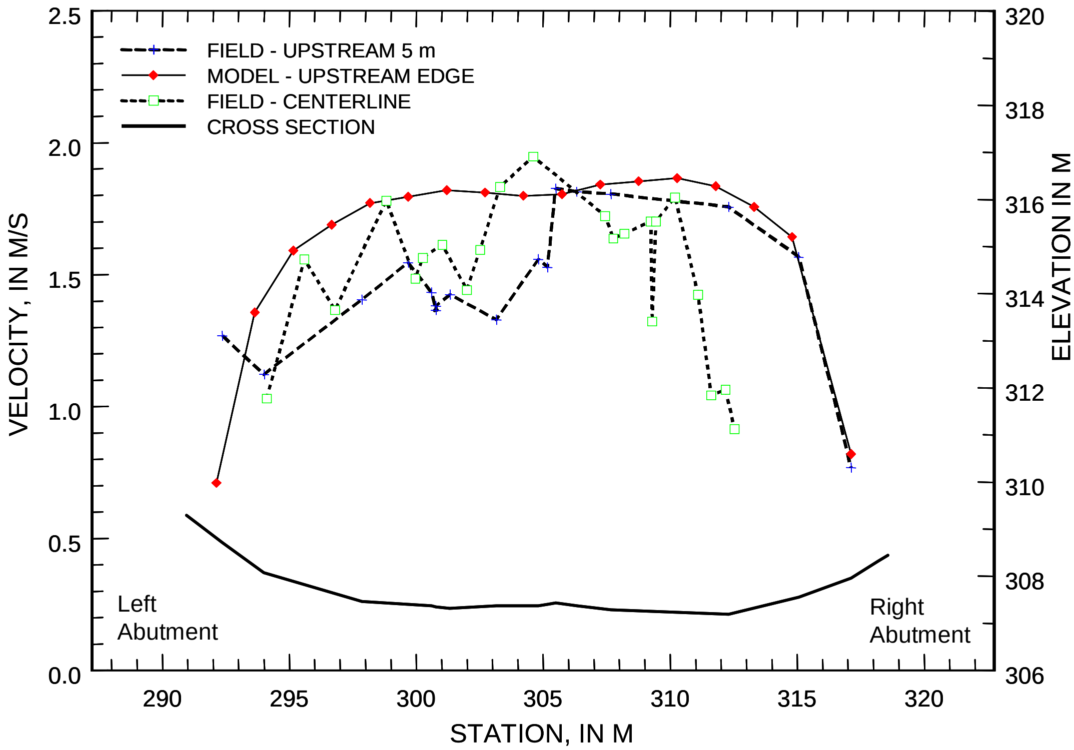


Figure 35. Comparison of observed and model velocity distributions at U.S. Route 12 over the Pomme de Terre River, Minnesota, for April 5, 1997.

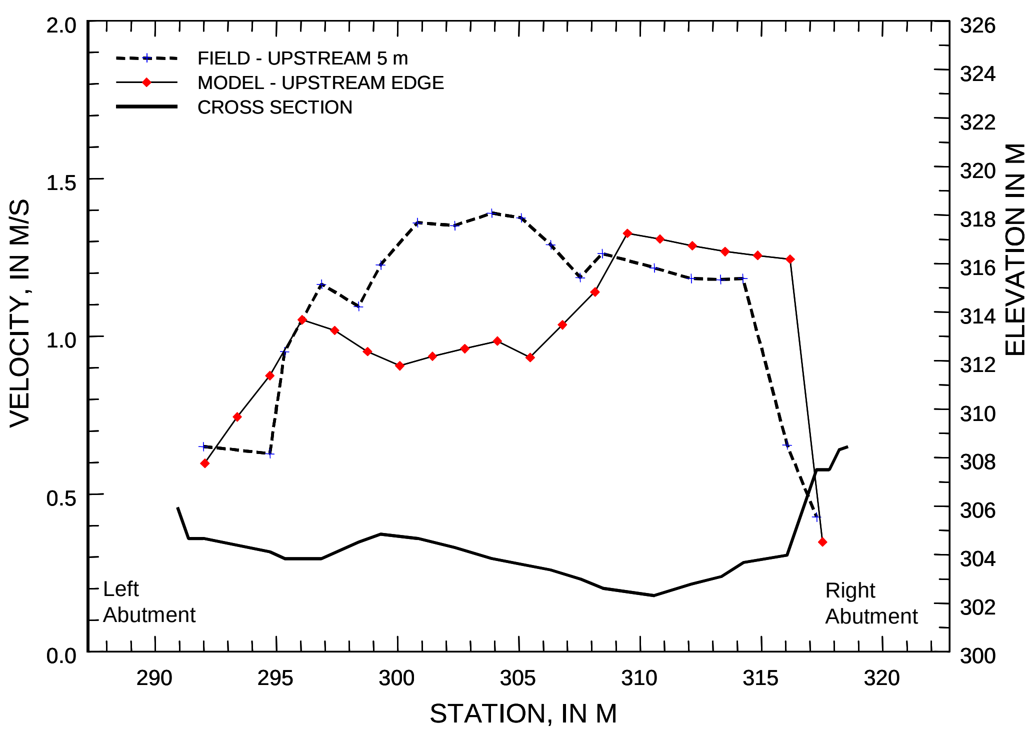


Figure 36. Comparison of observed and model velocity distributions at U.S. Route 12 over the Pomme de Terre River, Minnesota, for April 9, 1997.

Assessment of Scour Computations

The calibrated model was used to assess how accurately the scour for this flood could have been predicted. The bathymetry in the calibrated model was replaced with the original bathymetry extracted from the BRW, Inc., WSPRO model, which represented the pre-flood condition. The discharges from both April 5 and 9, were then run through the HEC-RAS model with the original bathymetry to determine the hydraulic parameters required to compute scour at the bridge. The contraction scour was computed in HEC-RAS by allowing the model to use the default equation (live bed or clear water) depending upon the hydraulic conditions. Table 18 compares the observed contraction scour to that computed by the model.

The computed depth of contraction scour was less than the observed value for all measurements. The contraction scour observed on April 9, 1997, may not be typical live bed contraction scour because depth of contraction scour could be affected by overlapping abutment scour holes. The abutment scour was computed in HEC-RAS by both the Froehlich equation and the Hydraulic in the River Environment (HIRE) equation, which are the two equations recommended in HEC-18. The HIRE equation is only applicable (but not required) if the embankment length-to-flow depth ratio at the abutment is greater than 25. In this case, the embankment to flow depth (L/a) ratio is 33. Table 19 compares the observed abutment scour to that computed by the model.

The data summarized in table 19 show the overprediction of scour that is common for abutment scour computations. Although the abutment scour equations overpredicted the local scour and the contraction scour equation underpredicted the contraction scour (table 18), when added together they predicted the total scour with reasonable accuracy and actually underpredicted the scour observed at the upstream edge of the bridge on April 9, 1997. These are somewhat surprising results that should be viewed with caution because the skew of the flow through the bridge could not be accounted for in the one-dimensional model, and the individual components were both in error. The agreement, therefore, may be somewhat coincidental.

Table 18. Comparison of observed to computed contraction scour at U.S. Route 12 over the Pomme de Terre River in Minnesota.

Date	Location (Edge of Bridge)	Equation	Depth of Scour (m) Computed	Observed
4/5/97	Upstream	Live bed	0.4	0.9
4/9/97	Upstream	Live bed	0.6	3.8
4/9/97	Downstream	Live bed	0.6	1.4

Table 19. Comparison of observed to computed abutment and total scour at U.S. Route 12 over the Pomme de Terre River in Minnesota.

				Observed		Based on Froehlich Equation		Based on HIRE Equation	
Date	**Abutment**	**Location (Edge of Bridge)**	**Equipment**	**Local Scour Depth (m)**	**Total Scour Depth (m)**	**Local Scour Depth (m)**	**Total Scour Depth (m)**	**Local Scour Depth (m)**	**Total Scour Depth (m)**
4/9/97	Right	Upstream	Sounding weight	2.4	5.6	4.6	5.2	10.8	11.4
4/9/97	Right	Upstream	Echo sounder	2.1	5.9	4.6	5.2	10.8	11.4
4/9/97	Right	Downstream	Echo sounder	3.4	4.8	4.6	5.2	10.8	11.4
4/9/97	Left	Upstream	Sounding weight	0.9	4.1	4.0	4.6	5.2	5.8
4/9/97	Left	Upstream	Echo sounder	0.5	4.3	4.0	4.6	5.2	5.8
4/9/97	Left	Downstream	Echo sounder	1.8	3.2	4.0	4.6	5.2	5.8

Swift County Route 22 over the Pomme de Terre River

Site Description

Swift County Route 22 crosses the Pomme de Terre River near Artichoke Lake, MN, and is located 10 km upstream from the U.S. Route 12 bridge. This bridge has two piers in the main channel with the abutments set at the edge of the main channel. The spill-through slopes at the abutments were protected by riprap and formed the banks of the main channel. The bridge is located in a very sinuous reach of the river with two large meanders immediately upstream and downstream of the bridge (figure 37). The floodplains are composed of farmland and forest.

During the flooding in April 1997, the USGS visited this site three times. During all three visits, the floodplain flow was concentrated in the right floodplain. This concentration of flow in the right floodplain is likely caused by the channel alignment upstream of the bridge. No defined point of reattachment along the right embankment was found during the flood. Flow was toward the main channel along the entire length of the right embankment. The flow separated from the right embankment, nearly perpendicular to the main channel flow, and joined the main flow just left of the rightmost pier (figure 38). During the visit on April 5, the flow from the right floodplain was so intense that a standing wave formed upstream of the bridge where the floodplain and main channel flow began mixing. The area from the rightmost pier to the right abutment was primarily slack and reverse flow. The depth of flow at the right abutment progressively deepened from 4.5 m on April 4, to 6 m on April 9. On April 9, a portion of the right embankment slumped, forcing Swift County officials to temporarily close the bridge until riprap was placed to protect the bridge.

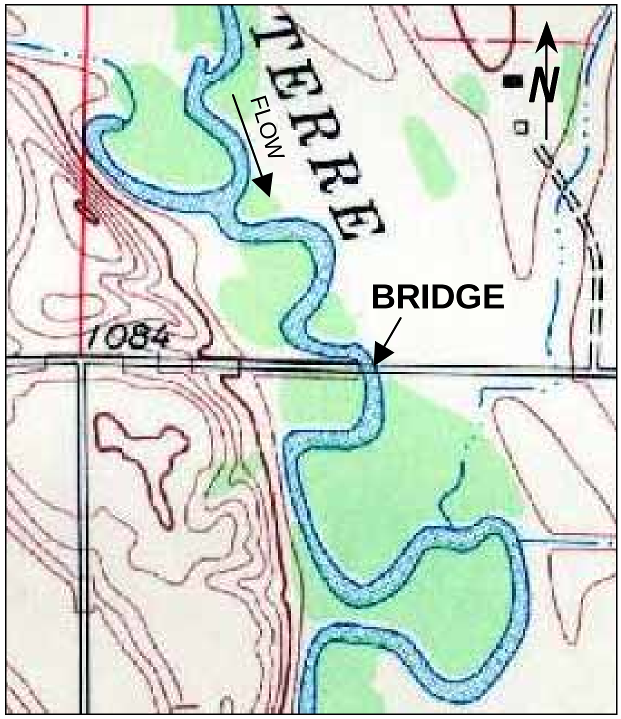


Figure 37. Plan view of Swift County Route 22 over the Pomme de Terre River, Minnesota (no scale).

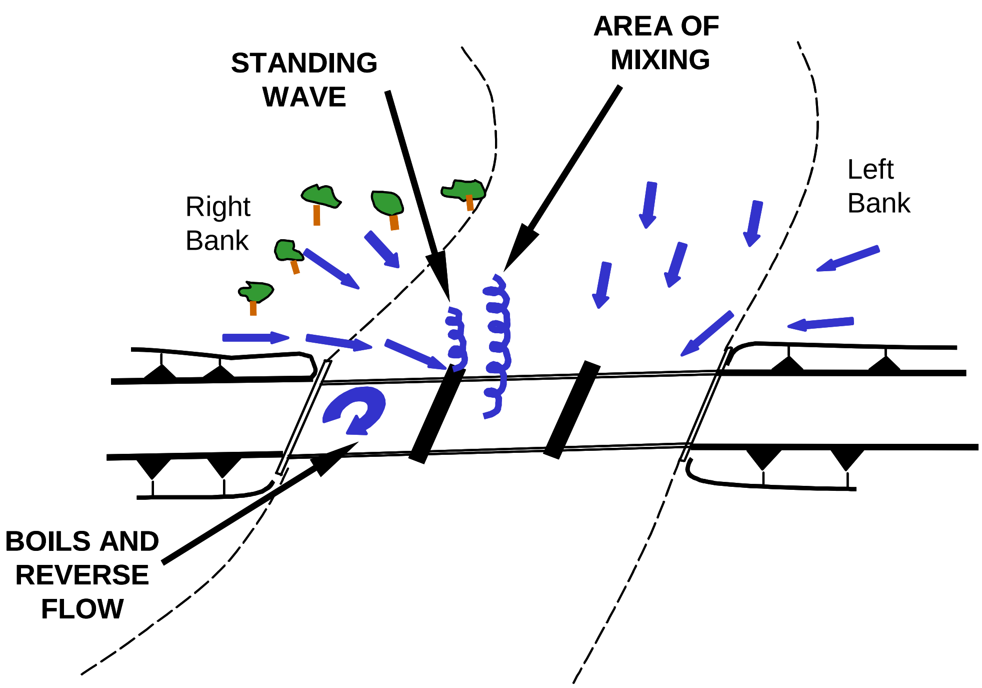


Figure 38. Sketch of flow conditions at Swift County Route 22 over the Pomme de Terre River, Minnesota (not to scale).

Discussion of Field Data

Data collection efforts were restricted to data that could be collected from the bridge deck for all three site visits during the flood (April 4, 5, and 9, 1997). All bathymetry data were collected by floating an echo sounder attached to a knee-board across the river; the sounder was controlled by a hand line from the bridge. The board was allowed to float downstream; streambed elevations were collected as far as 30 m downstream from the bridge. Data collected upstream of the bridge were restricted to the upstream edge of the bridge deck and the area around the upstream end of the right wing wall. Data could not be collected in the floodplains because of heavy vegetation. Velocity magnitudes and water discharge were measured during two of the three site visits using a vertical-axis current meter deployed along the upstream edge of the bridge. Water-surface elevations were measured at the upstream edge of the bridge from the top of the bridge deck between the left most pier and the left abutment. Table 20 summarizes the hydraulic data collected during the flood. Additional bathymetry data were collected 21 m upstream from and 30 m downstream from the bridge after the flood during a low-water site visit on July 15, 1997. Figure 39 shows the elevation and geometry changes experienced by the streambed at the bridge during the period of data collection.

The rightmost pier may have had some effect on the depth of scour at the right abutment, yet it is difficult to determine the pier's effect on the depth of local abutment scour. Limited measurements upstream of the rightmost pier showed the scour hole extended beyond the influence of the pier. The effect of the abutment is believed to be the dominant scouring factor; therefore, all scour is credited to the abutment with none reported for the pier. The observed velocity in the area at the right abutment dropped considerably as the scour-hole depth increased. The velocity at the left abutment held steady through the data collection period, as did the depth and shape of the scour hole. All abutment scour measurements were collected from the upstream edge of the bridge.

Table 20. Summary of hydraulic data collected at Swift County Route 22 over the Pomme de Terre River in Minnesota.

	Water-Surface Elevation (m, NGVD of 1929)		**Discharge**	**Velocity (m/s)**	
Date	**Upstream**	**Downstream**	**(m^3/s)**	**Average**	**Maximum**
4/4/97	317.02	316.93	–	–	–
4/5/97	317.15	317.06	132	1.3	2.5
4/9/97	317.34	–	146	1.2	1.8

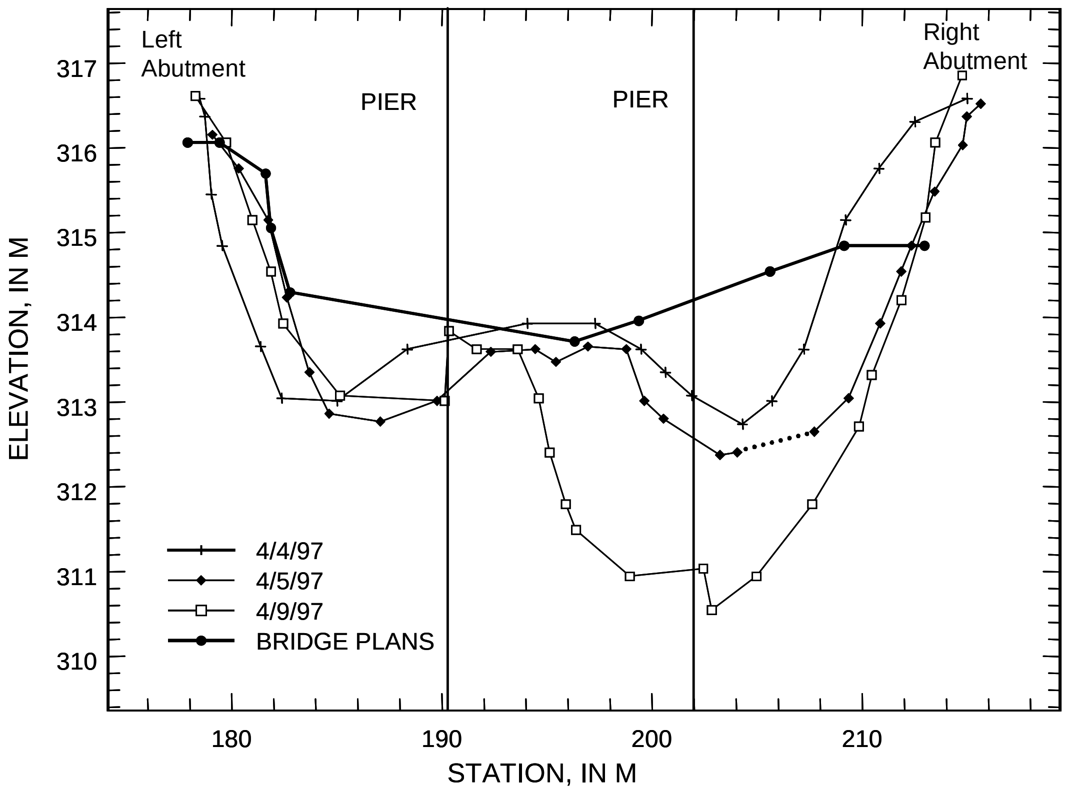


Figure 39. Cross sections collected along the upstream edge of Swift County Route 22 over the Pomme de Terre River, Minnesota.

Contraction scour is typically computed as the difference in average bed elevation between the uncontracted and contracted sections, adjusted for bed slope. Because field measurements could not be collected in the uncontracted section during the flood, a cross section collected in 1991 and included in the bridge plans was used as a reference surface. All contraction scour measurements were made along the upstream edge of the bridge. As shown in figure 39, there is less than 0.3 m difference in the bed elevation near the center of the channel (beyond the limits of the abutment scour holes) between the 1991 cross section and those collected during and after the 1997 flood. A value of zero for contraction scour is reported.

The reference surface used to determine the depth of abutment scour was the concurrent ambient bed; therefore, the depth of abutment scour reported is additional local scour below the depth of contraction scour, which for this site was negligible. A reference surface at 313.7 m above NGVD of 1929 was used to measure local abutment scour. A summary of the abutment scour data is presented in table 21.

Model Calibration

The data collected on April 5 and 9, 1997, and July 15, 1997 were utilized to build and calibrate the HEC-RAS model. Because no bathymetry data were collected during the flood in either the approach or exit sections, low-flow cross sections measured before and after the flood were used. The bathymetry data collected on July 15, 1997, along with geometry taken from the bridge plans, were the basis for the cross sections upstream and downstream of the bridge crossing. Despite the added hydraulic complexities introduced by the meander of the channel near the County Route 22 bridge, the HEC-RAS model predicted the water surface at the bridge within 0.06 m of what was measured in the field on April 5 and 9. When an ineffective flow area representing the recirculation zone between the right abutment and the rightmost pier was included, the model predicted the water-surface elevation at the bridge within 0.03 m of what was observed in the field.

The velocity distributions from the model and the field compared favorably, although the one-dimensional model could not replicate the two-dimensional features of the flow field. Figures 40 and 41 show the velocity distributions for the model, using the geometry from April 5 and 9, and field measurements collected with a vertical-axis current meter along the upstream edge of the bridge. The one-dimensional model results did not compare well with the April 5, 1997, observations (figure 40). Although the model estimated the peak velocity near the rightmost pier reasonably well, the model velocities were too high near the right bank and in the center of the main channel and too low along the left bank. Figure 41 shows that the model did a better job redistributing the flow after the scour had fully developed. The errors displayed should be expected when using a conveyance method to distribute flow that is complex and dominated by two-dimensional effects of the contraction. Since data were not available for the approach section, no comparisons could be made upstream from the bridge.

Table 21. Summary of abutment scour field data for Swift County Route 22 over the Pomme de Terre River in Minnesota.

Date	Abutment	Location	Equipment	Observed Scour Depth (m)	Accuracy (m)	Embankment Length (m)	Velocity at Abutment (m)	Depth at Abutment (m)
4/4/97	Right	Upstream	Echo sounder	1.2	0.3	157	–	4.2
4/5/97	Right	Upstream	Echo sounder	1.2	0.3	162	2.5	4.8
4/9/97	Right	Upstream	Echo sounder	3.0	0.5	166	1.0	6.4
4/4/97	Left	Upstream	Echo sounder	0.9	0.3	44	–	4.0
4/5/97	Left	Upstream	Echo sounder	0.9	0.3	47	1.5	4.4
4/9/97	Left	Upstream	Echo sounder	0.6	0.3	50	1.6	4.3

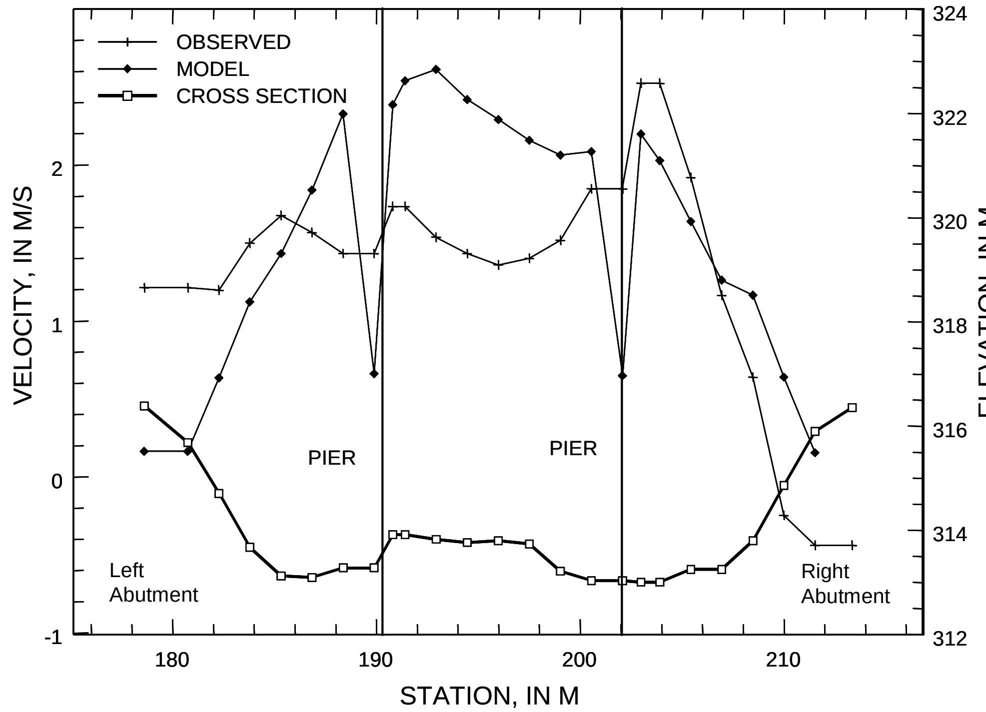


Figure 40. Comparison of observed and model velocity distributions for April 5, 1997, at Swift County Route 22 over the Pomme de Terre River, Minnesota.

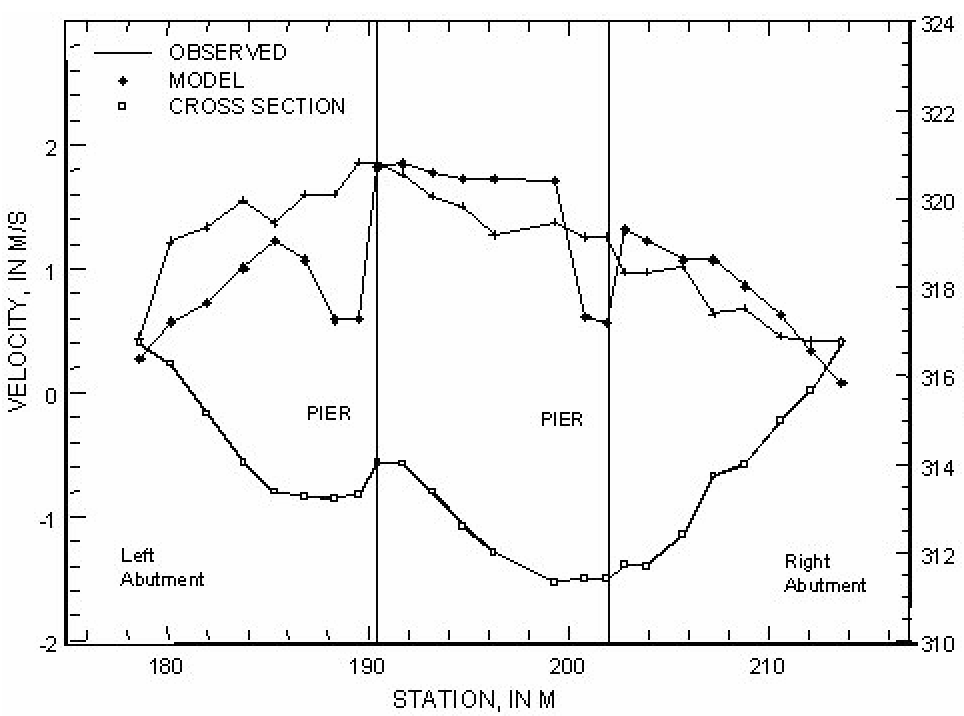


Figure 41. Comparison of observed and model velocity distributions for April 9, 1997, at Swift County Route 22 over the Pomme de Terre River, Minnesota.

Assessment of Scour Computations

The calibrated model was used to assess how accurately the scour for this flood could have been predicted. The original geometry of the bridge section was taken from the bridge plans and input into the calibrated HEC-RAS model. The approach-and-exit cross sections were modified to be consistent with the streambed elevations from the bridge plans. The ineffective flow area between the rightmost pier and the right abutment was assumed to be effective since it is unlikely that it would have been assumed ineffective without field observations. The discharges from both April 5 and 9, 1997, were then modeled with the original bathymetry to determine the hydraulic parameters needed for scour computations. The analysis did not include the data collected on April 4, 1997, because no hydraulic measurements were made during that site visit.

The contraction scour was computed in HEC-RAS by allowing the model to use the default equation (live bed or clear water) depending upon the hydraulic conditions computed by the model. The model correctly predicted little or no contraction scour for the prescribed discharges.

Abutment scour was computed in HEC-RAS by both the Froehlich and the HIRE equations. The data contained in table 22 show that the Froehlich equation did a good job predicting abutment scour, when compared to the fully developed scour holes on April 9, 1997. Because the equations predict maximum depth of scour, the Froehlich equation correctly overpredicted the depth of scour, when compared to the scour holes at the right abutment measured on April 5, 1997, which had not fully developed. The HIRE equation overpredicted scour for all situations.

Table 22. Comparison of observed to computed abutment scour at Swift County Route 22 over the Pomme de Terre River in Minnesota.

				Local Scour Depth		
Date	**Abutment**	**Location**	**Equipment**	**Observed (m)**	**Froehlich Equation (m)**	**HIRE Equation (m)**
4/5/97	Right	Upstream	Echo sounder	1.2	2.9	3.8
4/5/97	Left	Upstream	Echo sounder	0.8	0.7	2.8
4/9/97	Right	Upstream	Echo sounder	3.0	3.3	4.1
4/9/97	Left	Upstream	Echo sounder	0.6	0.9	3.1

CHAPTER 7: SUMMARY AND CONCLUSIONS

The analysis and prediction of scour at bridges is complex. Scale models and design methodology must account for the variability of site conditions and the potential interaction of the various components of scour. Scour at bridges has traditionally been classified into the categories of degradation, contraction scour, and local scour (abutment and pier). These categories do not explicitly account for the natural scour and deposition that occurs in a river during a flood or series of floods. Data collected at bridges during floods must be carefully analyzed to determine the magnitude of local and contraction scour. If appropriate reference surfaces are not selected, the scour reported may reflect scour caused by other processes.

Researchers and design engineers have agreed that field data on bridge scour are needed to validate laboratory experiments and to ensure the reliability of design methodology. The USGS, in cooperation with FHWA and many State highway agencies, has collected and compiled data on scour at bridges. The national database has been expanded and now contains 493 local pier scour measurements, 18 contraction scour measurements, and 12 abutment scour measurements from 79 sites located in 17 States.

Various researchers have proposed many pier scour equations, but none have accurately and conservatively predicted the scour observed in the field. Most equations are based on scaled laboratory experiments that did not account for the complexity of the field conditions: Relations among dimensionless variables developed from these laboratory experiments did not compare well with the field data. The Froehlich Design, HEC-18, HEC-18-K4, HEC-18-K4Mu, HEC-18-K4Mo (>2 mm), and Mississippi equations proved to be better than other equations for predicting pier scour for design purposes. The comparison of the scour depths predicted from these equations with scour depths measured in the field clearly showed that processes are reflected in the field data that are not correctly accounted for in these equations. Analysis of the pier scour data indicated the importance of bed material characteristics as an explanatory variable in the predictive equations. A new K_4 term for the HEC-18 equation was developed based on the relative bed material size (b/D_{50}).

Although caused by the same geometric contraction of the floodplain, contraction and abutment scour have traditionally been studied and treated separately. Contraction scour equations are based on theoretically developed equations that do not account for the complexity of the flow conditions in the field. Likewise, most abutment scour equations are developed from laboratory experiments that oversimplify or ignore many complexities common at highway bridges. Field data are needed to evaluate published approaches to computing contraction and abutment scour. A review of the literature found 29 references with mention of contraction and (or) abutment scour data, but only one presented detailed data collected during floods. Only two references included data on abutment scour. Published comparisons of field data with computed scour showed mixed results. Four papers showed the contraction scour equation typically overpredicted the observed scour and in a few instances severely overpredicted the scour; however, one paper showed the Laursen contraction scour equation underpredicted a number of measurements, and no severe overprediction was observed. The accuracy of the contraction and abutment scour equations recommended in HEC-18 may depend greatly upon the degree of contraction, the flow distribution, the configuration of the approach, and how well the hydraulic model represents the true flow distribution.

Comparison of computed abutment and contraction scour depths with depths measured in the field for U.S. Route 12 and Swift County Route 22 over the Pomme de Terre River in Minnesota provides insight to the capabilities and limitations of using one-dimensional models and the available abutment and contraction scour equations to predict scour at contracted bridge openings. The application of the methods outlined in HEC-18 to these sites showed a variability of results similar to the comparisons published in the literature. HEC-RAS and the equations recommended in HEC-18 provided reasonable predictions for maximum total scour at the two bridges; however, the magnitudes of the individual components (abutment and contraction) of scour did not compare well with the field data. Although field data in the approach sections were inadequate to provide a comprehensive evaluation of the ability of a one-dimensional model to represent a complex two-dimensional flow field, the comparisons that could be made showed the one-dimensional model computed flow distributions comparable to the field data for the fully developed scour hole conditions, but were less accurate for initial conditions and in areas of highly curvilinear flow.

Overall, the methodology for computing scour at bridges published in HEC-18 provides estimates that are generally conservative, in that the depth of scour is usually overpredicted. The complexity and variability of conditions at bridges make the development of predictive methodology difficult. The equations oversimplify most conditions, but modification of the methodology to account for site complexity and variability is not a simple task. New methodologies must balance the desire to fully explain complex processes with the need to provide procedures that are time and cost effective to apply. Additional field data and model studies are needed to continue to improve scour prediction methodology.

APPENDIX A: PIER SCOUR FIELD DATA

Table 23. List of sites.

Site	Stream	State	Route Class	Route Number/Name	Drainage Area (km^2)	Slope in Vicinity (m/m)
1	Susitna River	AK	State	3	29785	0.00040
2	Knik River	AK	Unknown	Old Glenn Hwy.	3108	0.00069
3	Knik River	AK	State	1	–	0.00100
4	Tazlina River	AK	State	4	6915	0.00210
5	Tanana River	AK	State	2	34965	0.00060
6	Tanana River	AK	State	3	66304	0.00015
7	Snow River	AK	State	9	389	–
8	Red River	AR	U.S.	71	124398	–
9	Red River	AR	Interstate	30	135550	–
10	Red River	AR	U.S.	82	136428	–
11	South Platte River	CO	State	37	24859	0.00093
12	South Platte River	CO	County	87	31391	0.00132
13	Arkansas River	CO	County	613	–	0.00050
14	Rio Grande River	CO	U.S.	285	4118	0.00075
15	Leipsic River	DE	State	9	–	–
16	Assawoman Bay	DE	State	54	–	–
17	South Altamaha River	GA	Interstate	95	36260	–
18	Eel River	IN	State	59	2279	0.00035
19	Wabash River	IN	State	163	30355	0.00014
20	White River	IN	State	157	11375	0.00020
21	Red River	LA	State	3032	157213	0.00010
22	Red River	LA	State	3032	157213	0.00010
23	Youghiogheny	MD	State	42	764	0.00500
24	Big Pipe Creek	MD	State	194	264	0.00157
25	Choptank River	MD	State	287	–	–
26	Pearl River	MS	State	25	8107	0.00019
27	Pearl River	MS	State	25	8107	0.00019
28	Homochitto River	MS	U.S.	84	469	0.00093
29	Pearl River	MS	U.S.	98	14815	0.00019
30	Pearl River	MS	U.S.	98	14815	0.00019
31	Clarks Fork Yellowstone River	MT	U.S.	310	4685	0.00700
32	Gallatin River	MT	U.S.	191	2137	0.00630
33	Yellowstone River	MT	U.S.	89	7366	0.00220
34	Badger Creek	MT	U.S.	89	619	0.00390
35	Otselic River	NY	State	23	396	0.00040
36	Chemung River	NY	State	427	6491	0.00075
37	Schoharie Creek	NY	State	30	1383	0.00200
38	Susquehanna River	NY	County	314	5781	0.00057
39	Genesee River	NY	County	Bailey Road	2549	0.00090
40	Delaware River	NY	City	6	7951	0.00114
41	Great Miami River	OH	State	128	9402	0.00049

Table 23. List of sites, continued.

Site	Stream	State	Route Class	Route Number/Name	Drainage Area (km^2)	Slope in Vicinity (m/m)
42	Hocking River	OH	State	278	1492	0.00038
43	Honey Creek	OH	State	67	386	0.00140
44	Little Miami River	OH	State	350	1748	0.00084
45	Ottawa River	OH	County	122	337	0.00144
46	Scioto River	OH	State	4	1368	0.00008
47	Todd Fork	OH	State	22	679	0.00179
48	Killbuck Creek	OH	County	621	1197	0.00023
49	Pamunkey River	VA	State	614	2800	0.00012
50	Nottoway River	VA	State	653	3680	0.00016
51	Bush River	VA	U.S.	460	166	0.00110
52	Dan River	VA	U.S.	501	7071	0.00025
53	Tye River	VA	State	56	240	0.00290
54	Little Nottoway River	VA	State	603	–	0.00200
55	Reed Creek	VA	State	649	–	0.00010
56	North Fork Holston River	VA	State	633	–	0.00100
57	Mississippi River	IL	State	51/150	1835274	0.00030
58	Mississippi River	MN	State	3	95312	0.00022
59	Clear Creek	OH	U.S.	33	238	0.00190
60	Grand River	OH	State	84	1774	0.00109
61	Great Miami River	OH	State	41	2401	0.00023
62	Mad River	OH	U.S.	36	420	0.00136
63	Massies Creek	OH	U.S.	68	219	0.00357
64	Maumee River	OH	U.S.	127	5895	0.00022
65	Salt Creek	OH	U.S.	50	741	0.00082
66	Scioto River	OH	State	159	9969	0.00035
67	Sugar Creek	OH	U.S.	250	805	0.00087
68	Tuscarawas River	OH	City	Walnut Road	1329	0.00008
69	Tascarwas River	OH	County	14	6216	0.00047
70	Wakatomika Creek	OH	State	16	363	0.00062
71	Walnut Creek	OH	County	17	559	0.00068
72	Minnesota River	MN	County	14	10619	–
73	Pomme De Terre River	MN	County	22	2165	0.00060
74	Brazos River	TX	County	2004	113810	0.00030
75	Mississippi River	MO	Interstate	255	–	0.00010
76	Mississippi River	MO	State	799	1805230	–
77	Middle Fork Crow River	MN	State	4	–	0.00100
78	Pomme De Terre River	MN	U.S.	12	2189	0.00050
79	Agulaize River	OH	State	198	518	0.00060

Table 24. Pier scour observations.

Measurement	Site	State	Date	Time	Pier	Upstream/ Downstream	Pier Type	Pier Shape	Pier Width (m)	Pier Length (m)	Skew (deg)	Velocity (m/s)	Depth (m)	Debris Effects	Bed Material	D16 (mm)	D50 (mm)	D84 (mm)	D95 (mm)	Gradation	Scour Depth (m)	Accuracy (m)
1	2	AK	7/11/1965	15:30	5	Upstream	Single	Sharp	1.8	8.8	0	3.7	5.5	Unknown	Noncohesive	0.92	5.00	27.00	83.00	5.5	1.1	0.2
2	3	AK	6/24/1966	–	1	Upstream	Single	Round	1.5	11.2	0	1.5	2.1	Unknown	Noncohesive	0.39	1.80	8.00	22.00	4.6	0.6	0.2
3	3	AK	6/28/1966	–	1	Upstream	Single	Round	1.5	11.2	0	0.9	0.9	Unknown	Noncohesive	0.39	1.80	8.00	22.00	4.6	0.5	0.2
4	3	AK	6/24/1966	–	2	Upstream	Single	Round	1.5	11.2	0	1.6	2.0	Unknown	Noncohesive	0.39	1.80	8.00	22.00	4.6	0.6	0.2
5	3	AK	6/28/1966	–	2	Upstream	Single	Round	1.5	11.2	0	1.0	0.9	Unknown	Noncohesive	0.39	1.80	8.00	22.00	4.6	0.6	0.2
6	3	AK	6/17/1966	–	3	Upstream	Single	Round	1.5	11.2	0	0.5	1.2	Unknown	Noncohesive	0.08	0.58	4.00	14.00	6.9	0.3	0.2
7	3	AK	6/24/1966	–	3	Upstream	Single	Round	1.5	11.2	0	1.6	3.0	Unknown	Noncohesive	0.39	1.80	8.00	22.00	4.6	0.9	0.2
8	3	AK	6/28/1966	–	3	Upstream	Single	Round	1.5	11.2	0	1.1	1.8	Unknown	Noncohesive	0.39	1.80	8.00	22.00	4.6	0.5	0.2
9	3	AK	6/17/1966	–	4	Upstream	Single	Round	1.5	11.2	0	0.8	1.5	Unknown	Noncohesive	0.08	0.58	4.00	14.00	6.9	0.3	0.2
10	3	AK	6/24/1966	–	4	Upstream	Single	Round	1.5	11.2	0	2.0	3.2	Unknown	Noncohesive	0.39	1.80	8.00	22.00	4.6	1.2	0.2
11	3	AK	6/28/1966	–	4	Upstream	Single	Round	1.5	11.2	0	1.2	2.4	Unknown	Noncohesive	0.39	1.80	8.00	22.00	4.6	0.6	0.2
12	3	AK	6/17/1966	–	5	Upstream	Single	Round	1.5	11.2	0	0.9	1.2	Unknown	Noncohesive	0.08	0.58	4.00	14.00	6.9	0.3	0.2
13	3	AK	6/24/1966	–	5	Upstream	Single	Round	1.5	11.2	0	1.8	3.0	Unknown	Noncohesive	0.39	1.80	8.00	22.00	4.6	1.4	0.2
14	3	AK	6/28/1966	–	5	Upstream	Single	Round	1.5	11.2	0	1.1	2.3	Unknown	Noncohesive	0.39	1.80	8.00	22.00	4.6	0.8	0.2
15	3	AK	6/17/1966	–	6	Upstream	Single	Round	1.5	11.2	0	0.3	0.5	Unknown	Noncohesive	0.08	0.58	4.00	14.00	6.9	0.8	0.2
16	3	AK	6/24/1966	–	6	Upstream	Single	Round	1.5	11.2	0	2.1	2.6	Unknown	Noncohesive	0.39	1.80	8.00	22.00	4.6	1.1	0.2
17	3	AK	6/28/1966	–	6	Upstream	Single	Round	1.5	11.2	0	1.1	1.5	Unknown	Noncohesive	0.39	1.80	8.00	22.00	4.6	0.5	0.2
18	3	AK	6/17/1966	–	7	Upstream	Single	Round	1.5	11.2	0	0.2	0.6	Unknown	Noncohesive	0.08	0.58	4.00	14.00	6.9	1.2	0.2
19	3	AK	6/24/1966	–	7	Upstream	Single	Round	1.5	11.2	0	1.8	3.0	Unknown	Noncohesive	0.39	1.80	8.00	22.00	4.6	1.8	0.2
20	3	AK	6/28/1966	–	7	Upstream	Single	Round	1.5	11.2	0	1.0	2.0	Unknown	Noncohesive	0.39	1.80	8.00	22.00	4.6	0.8	0.2
21	7	AK	9/23/1970	–	5	Unknown	Single	Round	1.0	0.0	0	1.6	1.5	Insignificant	Noncohesive	3.00	7.60	18.00	31.00	3.0	0.8	0.2
22	1	AK	7/2/1971	–	1	Upstream	Single	Sharp	1.5	6.1	0	2.0	5.8	Insignificant	Noncohesive	58.00	70.00	85.00	96.00	1.2	0.8	0.2
23	1	AK	8/11/1971	–	1	Upstream	Single	Sharp	1.5	6.1	0	3.0	5.3	Insignificant	Noncohesive	58.00	70.00	85.00	96.00	1.2	0.6	0.2
24	1	AK	7/2/1971	–	2	Upstream	Single	Sharp	1.5	6.1	0	2.6	4.1	Insignificant	Noncohesive	58.00	70.00	85.00	96.00	1.2	0.8	0.2
25	1	AK	8/11/1971	–	2	Unknown	Single	Sharp	1.5	6.1	0	2.9	6.6	Insignificant	Noncohesive	58.00	70.00	85.00	96.00	1.2	0.6	0.3
26	1	AK	7/2/1971	–	3	Upstream	Single	Sharp	1.5	6.1	0	2.1	3.4	Insignificant	Noncohesive	58.00	70.00	85.00	96.00	1.2	0.6	0.2
27	1	AK	8/11/1971	–	3	Unknown	Single	Sharp	1.5	6.1	0	3.5	5.2	Insignificant	Noncohesive	58.00	70.00	85.00	96.00	1.2	0.6	0.3
28	1	AK	7/2/1971	–	4	Downstream	Single	Sharp	1.5	6.1	0	1.5	4.1	Substantial	Noncohesive	58.00	70.00	85.00	96.00	1.2	1.5	0.2
29	1	AK	8/11/1971	–	4	Downstream	Single	Sharp	1.5	6.1	0	2.9	5.3	Substantial	Noncohesive	58.00	70.00	85.00	96.00	1.2	1.5	0.3
30	5	AK	7/16/1971	–	1	Downstream	Single	Round	1.5	9.4	37	2.2	3.7	Moderate	Noncohesive	5.00	14.00	42.00	86.00	3.0	1.8	0.2
31	5	AK	7/16/1971	–	2	Downstream	Single	Round	1.5	9.4	37	2.2	3.7	Moderate	Noncohesive	5.00	14.00	42.00	86.00	3.0	2.1	0.2
32	5	AK	7/16/1971	–	3	Downstream	Single	Round	1.5	9.4	37	2.1	4.6	Moderate	Noncohesive	5.00	14.00	42.00	86.00	3.0	1.8	0.2
33	5	AK	7/16/1971	–	4	Downstream	Single	Round	1.5	13.5	37	1.7	4.3	Moderate	Noncohesive	5.00	14.00	42.00	86.00	3.0	2.4	0.2
34	6	AK	8/17/1967	–	1	Downstream	Group	Sharp	3.0	14.6	0	2.6	6.7	Moderate	Noncohesive	12.00	15.00	19.00	23.00	1.3	1.8	0.2
35	4	AK	9/2/1971	–	1	Upstream	Group	Round	4.6	0.0	0	2.9	3.7	Insignificant	Noncohesive	68.00	90.00	120.00	144.00	1.3	1.5	0.2

Table 24. Pier scour observations, continued.

Mea-surement	Site	State	Date	Time	Pier	Upstream/ Downstream	Pier Type	Pier Shape	Pier Width (m)	Pier Length (m)	Skew (deg)	Velocity (m/s)	Depth (m)	Debris Effects	Bed Material	D16 (mm)	D50 (mm)	D84 (mm)	D95 (mm)	Grad-ation	Scour Depth (m)	Ac-curacy (m)
36	4	AK	9/4/1971	–	1	Upstream	Group	Round	4.6	0.0	0	3.5	4.6	Insignificant	Noncohesive	68.00	90.00	120.00	144.00	1.3	1.7	0.2
37	8	AR	5/9/1990	15:00	8	Downstream	Group	Round	2.1	9.4	11	2.7	12.3	Unknown	Unknown	–	0.12	–	–	–	2.3	0.2
38	8	AR	5/9/1990	15:00	9	Downstream	Group	Round	2.1	9.4	8	3.9	13.0	Unknown	Unknown	–	0.12	–	–	–	3.4	0.2
39	9	AR	5/12/1990	14:00	4	Upstream	Unknown	Sharp	2.1	0.0	0	2.9	10.8	Unknown	Unknown	–	0.18	–	–	–	4.4	0.2
40	9	AR	5/12/1990	14:00	5	Upstream	Unknown	Sharp	2.0	0.0	0	0.7	8.1	Unknown	Unknown	–	0.18	–	–	–	2.7	0.2
41	10	AR	5/14/1990	14:00	7	Downstream	Group	Round	3.0	10.9	0	1.9	11.7	Unknown	Unknown	–	0.32	–	–	–	4.4	0.2
42	10	AR	5/14/1990	14:00	8	Downstream	Group	Square	3.0	10.9	14	2.3	13.5	Unknown	Unknown	–	0.32	–	–	–	1.8	0.2
43	10	AR	5/14/1990	14:00	10	Downstream	Group	Round	3.0	10.9	0	1.5	9.1	Unknown	Unknown	–	0.32	–	–	–	3.3	0.2
44	13	CO	5/23/1984	9:00	1	Upstream	Single	Round	1.2	6.4	12	1.0	2.1	Unknown	Noncohesive	0.40	1.19	5.15	10.70	3.6	0.6	0.2
45	13	CO	5/23/1984	10:30	1	Downstream	Single	Round	1.2	6.4	12	1.0	2.1	Unknown	Noncohesive	0.40	1.19	5.15	10.70	3.6	0.4	0.2
46	13	CO	6/5/1984	14:00	1	Upstream	Single	Round	1.2	6.4	0	1.6	2.3	Unknown	Noncohesive	0.40	1.19	5.15	10.70	3.6	1.3	0.3
47	13	CO	6/5/1984	15:00	1	Downstream	Single	Round	1.2	6.4	0	1.6	2.3	Unknown	Noncohesive	0.40	1.19	5.15	10.70	3.6	0.5	0.2
48	13	CO	9/27/1984	10:00	1	Upstream	Single	Round	1.2	6.4	19	0.8	0.7	Unknown	Noncohesive	0.34	0.64	1.19	2.87	1.9	0.6	0.2
49	13	CO	9/27/1984	11:00	1	Downstream	Single	Round	1.2	6.4	19	0.8	0.7	Unknown	Noncohesive	0.34	0.64	1.19	2.87	1.9	0.4	0.2
50	13	CO	5/23/1984	9:00	3	Upstream	Single	Round	1.2	6.4	0	0.7	0.6	Unknown	Noncohesive	0.40	1.19	5.15	10.70	3.6	0.3	0.2
51	13	CO	5/23/1984	10:30	3	Downstream	Single	Round	1.2	6.4	0	0.7	0.6	Unknown	Noncohesive	0.40	1.19	5.15	10.70	3.6	0.3	0.2
52	13	CO	6/5/1984	14:00	3	Upstream	Single	Round	1.2	6.4	0	1.0	1.0	Unknown	Noncohesive	0.40	1.19	5.15	10.70	3.6	0.3	0.2
53	13	CO	6/5/1984	15:00	3	Downstream	Single	Round	1.2	6.4	0	1.0	1.0	Unknown	Noncohesive	0.40	1.19	5.15	10.70	3.6	0.3	0.2
54	14	CO	5/22/1984	12:00	1	Upstream	Single	Sharp	0.9	27.4	26	1.6	1.2	Unknown	Noncohesive	8.81	29.80	68.90	80.80	3.0	0.5	0.2
55	14	CO	5/22/1984	14:00	1	Downstream	Single	Sharp	0.9	27.4	26	1.6	1.2	Unknown	Noncohesive	8.81	29.80	68.90	80.80	3.0	0.5	0.2
56	14	CO	9/25/1984	8:00	1	Upstream	Single	Sharp	0.9	27.4	36	1.1	0.3	Unknown	Noncohesive	8.81	29.80	68.90	80.80	3.0	0.3	0.2
57	11	CO	5/21/1984	10:00	3	Upstream	Single	Sharp	0.5	7.1	20	1.4	1.8	Unknown	Noncohesive	0.43	1.10	5.12	10.40	3.5	0.3	0.2
58	11	CO	5/21/1984	13:00	3	Downstream	Single	Sharp	0.5	7.1	20	1.4	1.8	Unknown	Noncohesive	0.43	1.10	5.12	10.40	3.5	0.7	0.2
59	11	CO	5/21/1984	10:00	4	Upstream	Single	Sharp	0.5	7.1	20	1.8	1.6	Unknown	Noncohesive	0.43	1.10	5.12	10.40	3.5	0.3	0.2
60	11	CO	5/21/1984	13:00	4	Downstream	Single	Sharp	0.5	7.1	20	1.8	1.6	Unknown	Noncohesive	0.43	1.10	5.12	10.40	3.5	0.8	0.2
61	11	CO	6/26/1984	14:00	4	Upstream	Single	Sharp	0.5	7.1	43	1.0	1.4	Unknown	Noncohesive	0.43	1.10	5.12	10.40	3.5	0.5	0.2
62	11	CO	6/26/1984	16:00	4	Downstream	Single	Sharp	0.5	7.1	43	1.0	1.4	Unknown	Noncohesive	0.43	1.10	5.12	10.40	3.5	0.7	0.2
63	11	CO	10/3/1984	8:00	4	Upstream	Single	Sharp	0.5	7.1	0	0.6	0.6	Substantial	Noncohesive	0.43	1.80	7.86	14.80	4.3	0.5	0.2
64	11	CO	10/3/1984	10:00	4	Downstream	Single	Sharp	0.5	7.1	0	0.6	0.6	Substantial	Noncohesive	0.43	1.80	7.86	14.80	4.3	0.7	0.2
65	11	CO	5/21/1984	13:00	5	Downstream	Single	Sharp	0.5	7.1	20	1.4	1.3	Unknown	Noncohesive	0.43	1.10	5.12	10.40	3.5	0.9	0.2
66	11	CO	6/26/1984	16:00	5	Downstream	Single	Sharp	0.5	7.1	43	1.0	1.0	Unknown	Noncohesive	0.43	1.10	5.12	10.40	3.5	0.5	0.2
67	11	CO	10/3/1984	10:00	5	Downstream	Single	Sharp	0.5	7.1	43	0.8	0.7	Unknown	Noncohesive	0.43	1.80	7.86	14.80	4.3	0.4	0.2
68	12	CO	5/18/1984	11:00	1	Upstream	Single	Square	0.3	7.3	26	0.8	1.0	Unknown	Noncohesive	0.40	0.94	3.51	6.98	3.0	0.6	0.2
69	12	CO	5/18/1984	13:30	1	Downstream	Single	Square	0.3	7.3	26	0.8	1.0	Unknown	Noncohesive	0.40	0.94	3.51	6.98	3.0	0.6	0.2
70	12	CO	6/25/1984	14:00	1	Upstream	Single	Square	0.3	7.3	15	0.7	0.6	Unknown	Noncohesive	0.40	0.94	3.51	6.98	3.0	0.2	0.2

Table 24. Pier scour observations, continued.

Measurement	Site	State	Date	Time	Pier	Upstream/ Downstream	Pier Type	Pier Shape	Pier Width (m)	Pier Length (m)	Skew (deg)	Velocity (m/s)	Depth (m)	Debris Effects	Bed Material	D16 (mm)	D50 (mm)	D84 (mm)	D95 (mm)	Gradation	Scour Depth (m)	Accuracy (m)
71	12	CO	6/25/1984	16:30	1	Downstream	Single	Square	0.3	7.3	15	0.7	0.6	Unknown	Noncohesive	0.40	0.94	3.51	6.98	3.0	0.4	0.2
72	12	CO	10/2/1984	14:00	1	Upstream	Single	Square	0.3	7.3	15	0.7	0.2	Unknown	Noncohesive	0.30	0.67	2.10	9.30	2.6	0.5	0.2
73	12	CO	10/2/1984	16:00	1	Downstream	Single	Square	0.3	7.3	15	0.7	0.2	Unknown	Noncohesive	0.30	0.67	2.10	9.30	2.6	0.0	0.2
74	12	CO	5/18/1984	11:00	2	Upstream	Single	Square	0.3	7.3	26	1.1	1.0	Substantial	Noncohesive	0.40	0.94	3.51	6.98	3.0	0.2	0.2
75	12	CO	5/18/1984	13:00	2	Downstream	Single	Square	0.3	7.3	26	1.1	1.0	Substantial	Noncohesive	0.40	0.94	3.51	6.98	3.0	0.5	0.2
76	12	CO	5/18/1984	11:00	3	Upstream	Single	Square	0.3	7.3	14	1.2	1.0	Unknown	Noncohesive	0.40	0.94	3.51	6.98	3.0	0.3	0.2
77	12	CO	5/18/1984	13:30	3	Downstream	Single	Square	0.3	7.3	14	1.2	1.0	Unknown	Noncohesive	0.40	0.94	3.51	6.98	3.0	0.3	0.2
78	12	CO	6/25/1984	14:00	3	Upstream	Single	Square	0.3	7.3	20	0.8	0.3	Unknown	Noncohesive	0.40	0.94	3.51	6.98	3.0	0.3	0.2
79	12	CO	6/25/1984	16:30	3	Downstream	Single	Square	0.3	7.3	20	0.8	0.3	Unknown	Noncohesive	0.40	0.94	3.51	6.98	3.0	0.0	0.2
80	12	CO	5/18/1984	11:00	4	Upstream	Single	Square	0.3	7.3	23	1.2	1.3	Substantial	Noncohesive	0.40	0.94	3.51	6.98	3.0	0.2	0.2
81	12	CO	5/18/1984	13:30	4	Downstream	Single	Square	0.3	7.3	23	1.2	1.3	Substantial	Noncohesive	0.40	0.94	3.51	6.98	3.0	0.3	0.2
82	12	CO	6/25/1984	14:00	4	Upstream	Single	Square	0.3	7.3	16	1.0	0.3	Unknown	Noncohesive	0.40	0.94	3.51	6.98	3.0	0.4	0.2
83	12	CO	6/25/1984	16:30	4	Downstream	Single	Square	0.3	7.3	16	1.0	0.3	Unknown	Noncohesive	0.40	0.94	3.51	6.98	3.0	0.2	0.2
84	12	CO	10/2/1984	14:00	4	Upstream	Single	Square	0.3	7.3	16	0.7	1.0	Unknown	Noncohesive	0.30	0.67	2.10	9.30	2.6	0.4	0.2
85	12	CO	5/18/1984	11:00	5	Upstream	Single	Square	0.3	7.3	16	1.2	1.9	Unknown	Noncohesive	0.40	0.94	3.51	6.98	3.0	0.6	0.2
86	12	CO	5/18/1984	13:30	5	Downstream	Single	Square	0.3	7.3	16	1.2	1.9	Unknown	Noncohesive	0.40	0.94	3.51	6.98	3.0	0.5	0.2
87	12	CO	6/25/1984	14:00	5	Upstream	Single	Square	0.3	7.3	11	1.1	0.4	Unknown	Noncohesive	0.40	0.94	3.51	6.98	3.0	0.5	0.2
88	12	CO	6/25/1984	16:30	5	Downstream	Single	Square	0.3	7.3	11	1.1	0.4	Unknown	Noncohesive	0.40	0.94	3.51	6.98	3.0	0.4	0.2
89	12	CO	10/2/1984	14:00	5	Upstream	Single	Square	0.3	7.3	11	1.1	0.9	Unknown	Noncohesive	0.40	0.94	3.51	6.98	2.6	0.5	0.2
90	12	CO	10/2/1984	16:00	5	Downstream	Single	Square	0.3	7.3	11	1.1	0.9	Unknown	Noncohesive	0.40	0.94	3.51	6.98	2.6	0.5	0.2
91	12	CO	5/18/1984	11:00	6	Upstream	Single	Square	0.3	7.3	16	1.2	2.7	Unknown	Noncohesive	0.40	0.94	3.51	6.98	3.0	0.7	0.2
92	12	CO	5/18/1984	13:30	6	Downstream	Single	Square	0.3	7.3	16	1.2	2.7	Unknown	Noncohesive	0.40	0.94	3.51	6.98	3.0	0.8	0.2
93	12	CO	6/25/1984	14:00	6	Upstream	Single	Square	0.3	7.3	8	1.1	0.5	Unknown	Noncohesive	0.40	0.94	3.51	6.98	3.0	0.5	0.2
94	12	CO	6/25/1984	16:30	6	Downstream	Single	Square	0.3	7.3	8	1.1	0.5	Unknown	Noncohesive	0.40	0.94	3.51	6.98	3.0	0.4	0.2
95	12	CO	10/2/1984	14:00	6	Upstream	Single	Square	0.3	7.3	8	1.1	0.6	Unknown	Noncohesive	0.30	0.67	2.10	9.30	2.6	0.6	0.2
96	12	CO	10/2/1984	16:00	6	Downstream	Single	Square	0.3	7.3	8	1.1	0.6	Unknown	Noncohesive	0.30	0.67	2.10	9.30	2.6	0.6	0.2
97	12	CO	5/18/1984	11:00	7	Upstream	Single	Square	0.3	7.3	14	1.6	2.8	Substantial	Noncohesive	0.40	0.94	3.51	6.98	3.0	0.4	0.2
98	12	CO	5/18/1984	13:30	7	Downstream	Single	Square	0.3	7.3	14	1.6	2.8	Substantial	Noncohesive	0.40	0.94	3.51	6.98	3.0	0.4	0.2
99	12	CO	6/25/1984	14:00	7	Upstream	Single	Square	0.3	7.3	13	1.1	0.7	Unknown	Noncohesive	0.40	0.94	3.51	6.98	3.0	0.5	0.2
100	12	CO	6/25/1984	16:30	7	Downstream	Single	Square	0.3	7.3	13	1.1	0.7	Unknown	Noncohesive	0.40	0.94	3.51	6.98	3.0	0.4	0.2
101	12	CO	10/2/1984	14:00	7	Upstream	Single	Square	0.3	7.3	13	0.7	0.7	Unknown	Noncohesive	0.30	0.67	2.10	9.30	2.6	0.6	0.2
102	12	CO	10/2/1984	16:00	7	Downstream	Single	Square	0.3	7.3	13	0.7	0.7	Unknown	Noncohesive	0.30	0.67	2.10	9.30	2.6	0.9	0.2
103	12	CO	5/18/1984	11:00	8	Upstream	Single	Square	0.3	7.3	11	1.3	2.9	Substantial	Noncohesive	0.40	0.94	3.51	6.98	3.0	0.2	0.2
104	12	CO	5/18/1984	13:30	8	Downstream	Single	Square	0.3	7.3	11	1.3	2.9	Substantial	Noncohesive	0.40	0.94	3.51	6.98	3.0	0.6	0.2
105	12	CO	10/2/1984	14:00	8	Upstream	Single	Square	0.3	7.3	11	0.7	0.8	Insignificant	Noncohesive	0.30	0.67	2.10	9.30	2.6	0.2	0.2

Table 24. Pier scour observations, continued.

Mea-surement	Site	State	Date	Time	Pier	Upstream/ Downstream	Pier Type	Pier Shape	Pier Width (m)	Pier Length (m)	Skew (deg)	Velocity (m/s)	Depth (m)	Debris Effects	Bed Material	D16 (mm)	D50 (mm)	D84 (mm)	D95 (mm)	Grad-ation	Scour Depth (m)	Ac-curacy (m)
106	12	CO	10/2/1984	16:00	8	Downstream	Single	Square	0.3	7.3	11	0.7	0.8	Unknown	Noncohesive	0.30	0.67	2.10	9.30	2.6	0.3	0.2
107	16	DE	9/24/1991	14:00	C	Upstream	Group	Cylindrical	0.8	13.1	0	0.3	3.2	Unknown	Unknown	0.09	0.18	0.37	0.65	2.0	0.7	0.3
108	16	DE	6/8/1992	9:00	C	Upstream	Group	Cylindrical	0.8	13.1	0	0.3	3.1	Unknown	Unknown	0.09	0.18	0.37	0.65	2.0	0.5	0.3
109	16	DE	6/8/1992	10:30	C	Upstream	Group	Cylindrical	0.8	13.1	0	0.3	3.1	Unknown	Unknown	0.09	0.18	0.37	0.65	2.0	0.3	0.3
110	16	DE	9/24/1991	14:00	E	Upstream	Group	Cylindrical	0.8	13.1	0	0.3	8.0	Unknown	Unknown	0.09	0.18	0.37	0.65	2.0	1.6	0.3
111	16	DE	6/8/1992	9:00	E	Upstream	Group	Cylindrical	0.8	13.1	0	0.5	7.8	Unknown	Unknown	0.09	0.18	0.37	0.65	2.0	1.4	0.3
112	16	DE	6/8/1992	10:30	E	Upstream	Group	Cylindrical	0.8	13.1	0	0.5	7.8	Unknown	Unknown	0.09	0.18	0.37	0.65	2.0	1.4	0.3
113	16	DE	9/24/1991	14:00	F	Upstream	Group	Cylindrical	0.8	13.1	0	0.4	7.1	Unknown	Unknown	0.06	0.18	0.37	0.65	2.0	0.5	0.3
114	16	DE	6/8/1992	9:00	F	Upstream	Group	Cylindrical	0.8	13.1	0	0.5	7.6	Unknown	Unknown	0.06	0.18	0.37	0.65	2.0	1.2	0.3
115	16	DE	6/8/1992	10:30	F	Upstream	Group	Cylindrical	0.8	13.1	0	0.5	7.6	Unknown	Unknown	0.06	0.18	0.37	0.65	2.0	1.2	0.3
116	16	DE	9/24/1991	14:00	G	Upstream	Group	Cylindrical	0.8	13.1	0	0.3	3.8	Unknown	Unknown	0.06	0.18	0.37	0.65	2.0	0.4	0.3
117	16	DE	6/8/1992	10:30	G	Upstream	Group	Cylindrical	0.8	13.1	0	0.3	3.7	Unknown	Unknown	0.06	0.18	0.37	0.65	2.0	0.2	0.3
118	16	DE	6/8/1992	9:00	H	Upstream	Group	Cylindrical	0.8	13.1	0	0.2	1.5	Unknown	Unknown	0.06	0.18	0.37	0.65	2.0	0.5	0.3
119	16	DE	6/8/1992	10:30	H	Upstream	Group	Cylindrical	0.8	13.1	0	0.2	1.5	Unknown	Unknown	0.06	0.18	0.37	0.65	2.0	0.5	0.3
120	16	DE	6/8/1992	9:00	I	Upstream	Group	Cylindrical	0.8	13.1	0	0.2	0.3	Unknown	Unknown	0.06	0.18	0.37	0.65	2.0	0.2	0.3
121	15	DE	11/28/1988	16:00	C2	Upstream	Group	Square	0.4	8.2	0	0.6	4.7	Unknown	Unknown	0.06	0.40	1.40	8.00	5.1	0.2	0.3
122	15	DE	11/15/1990	13:30	C2	Upstream	Group	Square	0.4	8.2	0	0.7	5.3	Unknown	Unknown	0.06	0.40	1.40	8.00	5.1	0.2	0.3
123	15	DE	12/2/1990	13:00	C2	Upstream	Group	Square	0.4	8.2	0	0.4	4.8	Unknown	Unknown	0.06	0.40	1.40	8.00	5.1	0.3	0.3
124	15	DE	12/2/1990	14:00	C2	Upstream	Group	Square	0.4	8.2	0	0.7	4.2	Unknown	Unknown	0.06	0.40	1.40	8.00	5.1	0.2	0.3
125	15	DE	12/4/1990	15:00	C2	Upstream	Group	Square	0.4	8.2	0	0.4	4.7	Unknown	Unknown	0.06	0.40	1.40	8.00	5.1	0.3	0.3
126	15	DE	10/7/1991	13:00	C2	Upstream	Group	Square	0.4	8.2	0	0.5	4.6	Unknown	Unknown	0.06	0.40	1.40	8.00	5.1	0.2	0.3
127	15	DE	6/10/1992	9:30	C2	Upstream	Group	Square	0.4	8.2	0	0.5	4.6	Unknown	Unknown	0.06	0.40	1.40	8.00	5.1	0.2	0.3
128	15	DE	6/10/1992	10:30	C2	Upstream	Group	Square	0.4	8.2	0	0.5	4.2	Unknown	Unknown	0.06	0.40	1.40	8.00	5.1	0.2	0.3
129	15	DE	6/10/1992	11:30	C2	Upstream	Group	Square	0.4	8.2	0	0.5	3.7	Unknown	Unknown	0.06	0.40	1.40	8.00	5.1	0.2	0.3
130	15	DE	11/15/1990	13:30	C3	Upstream	Group	Square	0.4	8.2	0	0.5	4.0	Unknown	Unknown	0.06	0.40	1.40	8.00	5.1	0.1	0.3
131	15	DE	6/10/1992	10:30	C3	Upstream	Group	Square	0.4	8.2	0	0.5	4.9	Unknown	Unknown	0.06	0.40	1.40	8.00	5.1	0.3	0.3
132	17	GA	2/12/1990	14:45	11	Upstream	Group	Square	1.2	9.9	0	0.6	5.7	Insignificant	Noncohesive	0.55	1.00	2.45	4.10	2.1	2.1	0.1
133	17	GA	2/12/1990	17:05	11	Upstream	Group	Square	1.2	9.9	0	0.5	5.5	Insignificant	Noncohesive	0.55	1.00	2.45	4.10	2.1	2.0	0.1
134	17	GA	2/12/1990	13:00	12	Upstream	Group	Square	1.2	10.8	0	0.5	7.1	Insignificant	Noncohesive	0.55	1.00	2.45	4.10	2.1	1.2	0.1
135	17	GA	2/12/1990	14:45	12	Upstream	Group	Square	1.2	10.8	0	0.7	6.4	Insignificant	Noncohesive	0.55	1.00	2.45	4.10	2.1	1.4	0.1
136	17	GA	2/12/1990	17:05	12	Upstream	Group	Square	1.2	10.8	0	0.7	6.2	Insignificant	Noncohesive	0.55	1.00	2.45	4.10	2.1	1.7	0.1
137	17	GA	2/12/1990	13:00	13	Upstream	Group	Square	1.8	10.8	0	0.5	8.0	Insignificant	Noncohesive	0.55	1.00	2.45	4.10	2.1	1.2	0.1
138	17	GA	2/12/1990	14:45	13	Upstream	Group	Square	1.8	10.8	0	0.7	7.6	Insignificant	Noncohesive	0.55	1.00	2.45	4.10	2.1	1.5	0.1
139	17	GA	2/12/1990	17:05	13	Upstream	Group	Square	1.8	10.8	0	0.7	7.0	Insignificant	Noncohesive	0.55	1.00	2.45	4.10	2.1	1.6	0.1
140	57	IL	8/3/1993	–	11	Upstream	Single	Square	4.0	11.6	11	2.4	22.5	Insignificant	Noncohesive	0.30	0.60	1.30	4.20	2.1	7.1	0.6

Table 24. Pier scour observations, continued.

Measurement	Site	State	Date	Time	Pier	Upstream/ Downstream	Pier Type	Pier Shape	Pier Width (m)	Pier Length (m)	Skew (deg)	Velocity (m/s)	Depth (m)	Debris Effects	Bed Material	D16 (mm)	D50 (mm)	D84 (mm)	D95 (mm)	Gradation	Scour Depth (m)	Accuracy (m)
141	57	IL	8/12/1993	–	11	Upstream	Single	Square	4.1	11.6	4	2.0	22.4	Insignificant	Noncohesive	0.30	0.60	1.30	4.20	2.1	6.2	0.6
142	57	IL	9/13/1993	–	11	Upstream	Single	Square	4.7	11.6	4	1.8	16.7	Insignificant	Noncohesive	0.30	0.60	1.30	4.20	2.1	6.5	0.6
143	18	IN	11/13/1992	12:30	1	Downstream	Single	Round	0.9	10.2	0	0.3	5.1	Unknown	Noncohesive	0.25	0.50	2.25	8.00	3.0	0.4	0.2
144	18	IN	11/13/1992	10:30	2	Upstream	Single	Round	0.9	10.2	0	0.8	5.9	Unknown	Noncohesive	0.25	0.50	2.25	8.00	3.0	1.0	0.2
145	19	IN	1/3/1991	11:00	8	Upstream	Single	Round	0.9	10.5	5	1.6	9.4	Unknown	Noncohesive	0.24	0.34	0.34	0.55	1.4	0.7	0.2
146	19	IN	1/3/1991	11:45	8	Downstream	Single	Round	0.9	10.5	5	1.6	9.4	Unknown	Noncohesive	0.24	0.34	0.34	0.55	1.4	0.5	0.2
147	20	IN	1/3/1991	12:30	2	Upstream	Single	Round	0.9	14.5	5	1.1	3.4	Unknown	Noncohesive	0.24	0.90	4.20	7.00	4.2	0.4	0.2
148	20	IN	1/4/1991	12:00	2	Upstream	Single	Round	0.9	14.5	5	1.0	3.2	Unknown	Noncohesive	0.24	0.90	4.20	7.00	4.2	0.4	0.2
149	20	IN	1/4/1991	13:00	2	Downstream	Single	Round	0.9	14.5	5	1.0	3.2	Unknown	Noncohesive	0.24	0.90	4.20	7.00	4.2	0.2	0.2
150	20	IN	1/3/1991	12:30	3	Upstream	Single	Round	0.9	13.1	0	1.3	5.1	Unknown	Noncohesive	0.24	0.90	4.20	7.00	4.2	0.7	0.2
151	20	IN	1/3/1991	12:30	4	Upstream	Single	Round	0.6	12.8	10	1.6	6.6	Unknown	Noncohesive	0.24	0.90	4.20	7.00	4.2	1.1	0.2
152	20	IN	1/4/1991	12:00	4	Upstream	Single	Round	0.6	12.8	10	1.9	6.2	Unknown	Noncohesive	0.24	0.90	4.20	7.00	4.2	1.2	0.2
153	21	LA	5/19/1990	14:20	4	Upstream	Single	Sharp	4.3	16.5	0	2.6	11.6	Insignificant	Noncohesive	0.20	0.30	0.40	0.50	1.4	3.7	0.6
154	21	LA	5/19/1990	15:30	4	Downstream	Single	Sharp	4.3	16.5	0	2.6	12.3	Insignificant	Noncohesive	0.20	0.30	0.40	0.50	1.4	0.9	0.3
155	21	LA	5/22/1990	13:35	4	Upstream	Single	Sharp	4.3	16.5	0	2.1	9.4	Insignificant	Noncohesive	0.20	0.30	0.40	0.50	1.4	3.5	0.3
156	21	LA	5/22/1990	14:45	4	Downstream	Single	Sharp	4.3	16.5	0	2.1	9.4	Insignificant	Noncohesive	0.20	0.30	0.40	0.50	1.4	1.1	0.3
157	21	LA	5/19/1990	14:20	5	Upstream	Single	Sharp	4.3	16.5	0	3.2	11.9	Insignificant	Noncohesive	0.20	0.30	0.40	0.50	1.4	7.0	0.6
158	21	LA	5/19/1990	15:30	5	Downstream	Single	Sharp	4.3	16.5	0	3.2	12.6	Insignificant	Noncohesive	0.20	0.30	0.40	0.50	1.4	5.2	0.6
159	21	LA	5/22/1990	13:35	5	Upstream	Single	Sharp	4.3	16.5	0	2.9	9.8	Insignificant	Noncohesive	0.20	0.30	0.40	0.50	1.4	7.7	0.3
160	21	LA	5/22/1990	14:45	5	Downstream	Single	Sharp	4.3	16.5	0	2.9	9.8	Insignificant	Noncohesive	0.20	0.30	0.40	0.50	1.4	5.6	0.3
161	22	LA	5/17/1990	13:30	4	Upstream	Single	Round	4.3	12.2	0	2.5	11.7	Insignificant	Noncohesive	0.20	0.30	0.40	0.50	1.4	4.4	0.3
162	22	LA	5/17/1990	15:10	4	Downstream	Single	Round	4.3	12.2	0	2.5	11.7	Insignificant	Noncohesive	0.20	0.30	0.40	0.50	1.4	3.9	0.6
163	22	LA	5/19/1990	10:25	4	Upstream	Single	Round	4.3	12.2	0	2.6	10.8	Insignificant	Noncohesive	0.20	0.30	0.40	0.50	1.4	3.3	0.6
164	22	LA	5/19/1990	12:55	4	Downstream	Single	Round	4.3	12.2	0	2.6	11.3	Insignificant	Noncohesive	0.20	0.30	0.40	0.50	1.4	2.1	0.3
165	22	LA	5/22/1990	8:55	4	Downstream	Single	Round	4.3	12.2	0	2.1	9.3	Insignificant	Noncohesive	0.20	0.30	0.40	0.50	1.4	4.2	0.3
166	22	LA	5/22/1990	10:30	4	Upstream	Single	Round	4.3	12.2	0	2.1	9.3	Insignificant	Noncohesive	0.20	0.30	0.40	0.50	1.4	3.8	0.3
167	22	LA	5/17/1990	13:30	5	Upstream	Single	Round	4.3	12.2	0	3.0	12.0	Insignificant	Noncohesive	0.20	0.30	0.40	0.50	1.4	4.5	0.3
168	22	LA	5/17/1990	15:10	5	Downstream	Single	Round	4.3	12.2	0	3.0	11.7	Insignificant	Noncohesive	0.20	0.30	0.40	0.50	1.4	4.8	0.6
169	22	LA	5/19/1990	10:25	5	Upstream	Single	Round	4.3	12.2	0	3.2	11.2	Insignificant	Noncohesive	0.20	0.30	0.40	0.50	1.4	5.5	0.3
170	22	LA	5/19/1990	12:55	5	Downstream	Single	Round	4.3	12.2	0	3.2	11.7	Insignificant	Noncohesive	0.20	0.30	0.40	0.50	1.4	5.2	0.3
171	22	LA	5/22/1990	8:55	5	Downstream	Single	Round	4.3	12.2	0	2.9	9.6	Insignificant	Noncohesive	0.20	0.30	0.40	0.50	1.4	5.6	0.6
172	22	LA	5/22/1990	10:30	5	Upstream	Single	Round	4.3	12.2	0	2.9	9.7	Insignificant	Noncohesive	0.20	0.30	0.40	0.50	1.4	3.7	0.3
173	24	MD	6/23/1972	17:30	1	Upstream	Single	Round	1.2	9.8	0	0.8	3.5	Unknown	Unknown	13.00	22.00	76.00	160.00	2.4	0.4	0.3
174	24	MD	9/25/1975	19:00	1	Upstream	Single	Round	1.2	9.8	0	1.3	3.1	Unknown	Unknown	13.00	22.00	76.00	160.00	2.4	0.4	0.3
175	24	MD	6/23/1972	17:30	2	Upstream	Single	Round	1.2	9.8	0	1.1	2.4	Unknown	Unknown	13.00	22.00	76.00	160.00	2.4	0.7	0.3

Table 24. Pier scour observations, continued.

Measurement	Site	State	Date	Time	Pier	Upstream/ Downstream	Pier Type	Pier Shape	Pier Width (m)	Pier Length (m)	Skew (deg)	Velocity (m/s)	Depth (m)	Debris Effects	Bed Material	D16 (mm)	D50 (mm)	D84 (mm)	D95 (mm)	Gradation	Scour Depth (m)	Accuracy (m)
176	24	MD	9/25/1975	19:00	2	Upstream	Single	Round	1.2	9.8	0	1.6	2.4	Unknown	Unknown	13.00	22.00	76.00	160.00	2.4	0.5	0.3
177	24	MD	5/29/1990	20:30	2	Upstream	Single	Round	1.2	9.8	0	1.0	1.9	Unknown	Unknown	13.00	22.00	76.00	160.00	2.4	0.3	0.3
178	24	MD	10/23/1990	15:30	2	Upstream	Single	Round	1.2	9.8	0	1.6	2.0	Unknown	Unknown	13.00	22.00	76.00	160.00	2.4	0.4	0.3
179	24	MD	10/23/1990	20:00	2	Upstream	Single	Round	1.2	9.8	0	1.6	3.1	Unknown	Unknown	13.00	22.00	76.00	160.00	2.4	0.5	0.3
180	25	MD	2/23/1989	11:00	CNTR	Upstream	Single	Square	1.2	10.7	0	0.1	1.1	Unknown	Unknown	0.18	0.38	0.94	2.60	2.3	0.7	0.3
181	25	MD	3/25/1989	14:00	CNTR	Upstream	Single	Square	1.2	10.7	0	0.4	2.2	Unknown	Unknown	0.18	0.38	0.94	2.60	2.3	0.7	0.3
182	25	MD	7/28/1991	10:30	CNTR	Upstream	Single	Square	1.2	10.7	0	0.2	1.5	Unknown	Unknown	0.18	0.38	0.94	2.60	2.3	0.6	0.3
183	25	MD	2/23/1989	11:00	LEFT	Upstream	Single	Square	1.2	10.7	0	0.7	1.9	Unknown	Noncohesive	0.18	0.38	0.94	2.60	2.3	1.2	0.3
184	25	MD	3/25/1989	14:00	LEFT	Upstream	Single	Square	1.2	10.7	0	0.8	3.1	Unknown	Noncohesive	0.18	0.38	0.94	2.60	2.3	1.6	0.3
185	25	MD	7/28/1991	10:30	LEFT	Upstream	Single	Square	1.2	10.7	0	0.8	2.2	Unknown	Unknown	0.18	0.38	0.94	2.60	2.3	1.3	0.3
186	25	MD	2/23/1989	11:00	RT	Upstream	Single	Square	1.2	10.7	0	0.3	1.1	Unknown	Unknown	0.18	0.38	0.94	2.60	2.3	0.5	0.3
187	25	MD	2/23/1989	14:00	RT	Upstream	Single	Square	1.2	10.7	0	0.6	2.1	Unknown	Unknown	0.18	0.38	0.94	2.60	2.3	0.7	0.3
188	25	MD	7/28/1991	10:30	RT	Upstream	Single	Square	1.2	10.7	0	0.3	1.5	Unknown	Unknown	0.18	0.38	0.94	2.60	2.3	0.7	0.3
189	23	MD	7/13/1990	13:00	LEFT	Upstream	Single	Sharp	1.5	12.7	0	2.3	2.4	Unknown	Unknown	68.00	108.00	233.00	350.00	1.8	0.3	0.3
190	23	MD	4/1/1993	9:30	LEFT	Upstream	Single	Sharp	1.5	12.7	0	2.1	2.1	Unknown	Unknown	68.00	108.00	233.00	350.00	1.8	0.4	0.3
191	23	MD	7/13/1990	13:00	RT	Upstream	Single	Sharp	1.5	12.7	0	2.6	3.0	Unknown	Unknown	68.00	108.00	233.00	350.00	1.8	0.8	0.3
192	23	MD	4/1/1993	9:30	RT	Upstream	Single	Sharp	1.5	12.7	0	1.9	2.4	Unknown	Unknown	68.00	108.00	233.00	350.00	1.8	0.5	0.3
193	58	MN	3/28/1969	14:30	9	Upstream	Single	Square	5.0	9.8	0	0.7	5.9	Insignificant	Noncohesive	0.20	0.48	0.67	1.10	21.8	0.9	0.0
194	58	MN	4/3/1969	11:57	9	Upstream	Single	Square	4.8	9.8	0	0.8	6.5	Insignificant	Noncohesive	0.20	0.48	0.67	1.10	1.8	0.9	0.1
195	58	MN	4/9/1969	11:52	9	Upstream	Single	Square	4.5	9.8	0	1.2	8.5	Insignificant	Noncohesive	0.20	0.48	0.67	1.10	1.8	0.8	0.1
196	58	MN	4/15/1969	12:40	9	Upstream	Single	Square	5.3	9.8	0	1.6	11.8	Insignificant	Noncohesive	0.20	0.48	0.67	1.10	1.8	4.6	0.2
197	58	MN	4/23/1969	11:37	9	Upstream	Single	Square	5.4	9.8	0	1.4	9.5	Insignificant	Noncohesive	0.20	0.48	0.67	1.10	1.8	4.2	0.2
198	58	MN	4/28/1969	12:15	9	Upstream	Single	Square	5.5	9.8	0	1.1	8.7	Insignificant	Noncohesive	0.20	0.48	0.67	1.10	1.8	3.1	0.1
199	58	MN	6/4/1969	13:55	9	Upstream	Single	Square	5.1	9.8	0	0.5	5.7	Unknown	Noncohesive	0.20	0.48	0.67	1.10	1.8	0.7	0.0
200	58	MN	3/28/1969	14:30	10	Upstream	Single	Square	3.3	9.8	0	0.7	4.3	Insignificant	Noncohesive	0.20	0.48	6.70	1.10	1.8	0.5	0.0
201	58	MN	4/3/1969	11:57	10	Upstream	Single	Square	3.3	9.8	0	0.8	4.8	Insignificant	Noncohesive	0.20	0.48	6.70	1.10	1.8	0.4	0.0
202	58	MN	4/9/1969	11:52	10	Upstream	Single	Square	3.3	9.8	0	1.2	6.7	Insignificant	Noncohesive	0.20	0.48	6.70	1.10	1.8	0.9	0.0
203	58	MN	4/15/1969	12:40	10	Upstream	Single	Square	3.1	9.8	0	1.6	9.8	Insignificant	Noncohesive	0.20	0.48	0.67	1.10	1.8	0.7	0.0
204	58	MN	4/23/1969	11:37	10	Upstream	Single	Square	3.7	9.8	0	1.4	9.8	Insignificant	Noncohesive	0.20	0.48	0.67	1.10	1.8	0.8	0.0
205	58	MN	4/28/1969	12:15	10	Upstream	Single	Square	3.7	9.8	0	1.1	7.9	Insignificant	Noncohesive	0.20	0.48	0.67	1.10	1.8	0.4	0.0
206	58	MN	6/4/1969	13:35	10	Upstream	Single	Square	4.1	9.8	0	0.5	4.7	Insignificant	Noncohesive	0.20	0.48	0.67	1.10	1.8	0.5	0.0
207	75	MO	7/14/1993	–	8	Upstream	Single	Square	2.8	9.0	0	0.0	12.6	Insignificant	Noncohesive	0.28	0.70	2.10	4.60	2.7	3.7	0.9
208	75	MO	7/17/1993	–	8	Upstream	Single	Square	2.8	9.0	0	1.9	14.3	Insignificant	Noncohesive	0.28	0.70	2.10	4.60	2.7	4.3	0.6
209	75	MO	7/19/1993	–	8	Upstream	Single	Square	2.8	9.0	0	2.0	15.1	Insignificant	Unknown	0.28	0.70	2.10	4.60	2.7	3.8	0.6
210	75	MO	8/17/1993	–	8	Upstream	Single	Square	2.8	9.0	0	1.2	12.9	Insignificant	Noncohesive	0.28	0.70	2.10	4.60	2.7	3.8	0.6

Table 24. Pier scour observations, continued.

Measurement	Site	State	Date	Time	Pier	Upstream/ Downstream	Pier Type	Pier Shape	Pier Width (m)	Pier Length (m)	Skew (deg)	Velocity (m/s)	Depth (m)	Debris Effects	Bed Material	D16 (mm)	D50 (mm)	D84 (mm)	D95 (mm)	Gradation	Scour Depth (m)	Accuracy (m)
211	75	MO	9/16/1993	–	8	Upstream	Single	Square	2.8	9.0	0	1.1	11.0	Insignificant	Noncohesive	0.28	0.70	2.10	4.60	2.7	4.3	0.6
212	75	MO	7/14/1993	–	9	Upstream	Single	Square	2.9	9.0	0	0.0	13.3	Insignificant	Noncohesive	0.28	0.70	2.10	4.60	2.7	4.0	0.9
213	75	MO	7/17/1993	–	9	Upstream	Single	Square	2.9	9.0	0	2.3	14.6	Insignificant	Noncohesive	0.28	0.70	2.10	4.60	2.7	4.0	0.6
214	75	MO	7/19/1993	–	9	Upstream	Single	Square	2.9	9.0	0	2.0	15.4	Insignificant	Noncohesive	0.28	0.70	2.10	4.60	2.7	3.7	0.6
215	75	MO	8/17/1993	–	9	Upstream	Single	Square	2.9	9.0	0	1.4	12.9	Insignificant	Noncohesive	0.28	0.70	2.10	4.60	2.7	3.7	0.6
216	75	MO	9/16/1993	–	9	Upstream	Single	Square	2.9	9.0	0	1.2	11.0	Insignificant	Noncohesive	0.28	0.70	2.10	4.60	2.7	3.7	0.6
217	76	MO	7/15/1993	–	10	Upstream	Single	Sharp	5.5	20.7	0	2.6	20.0	Insignificant	Noncohesive	0.55	0.96	2.50	4.03	2.1	4.1	0.6
218	28	MS	1/25/1990	9:00	3	Upstream	Single	Cylindrical	2.4	2.4	0	1.9	3.0	Insignificant	Noncohesive	0.49	7.51	23.20	27.00	6.9	1.2	0.3
219	28	MS	8/27/1992	10:10	3	Downstream	Single	Cylindrical	2.4	2.4	0	1.9	2.6	Substantial	Noncohesive	0.49	7.51	23.20	27.00	6.9	1.0	0.3
220	28	MS	12/21/1972	6:00	4	Upstream	Single	Cylindrical	2.4	2.4	0	2.1	3.9	Unknown	Noncohesive	0.49	7.51	23.20	27.00	6.9	0.9	0.2
221	28	MS	4/25/1973	5:30	4	Upstream	Single	Cylindrical	2.4	2.4	0	1.9	2.7	Unknown	Noncohesive	0.49	7.51	23.20	27.00	6.9	0.9	0.2
222	28	MS	1/25/1990	9:00	4	Upstream	Single	Cylindrical	2.4	2.4	0	2.1	2.9	Insignificant	Noncohesive	0.49	7.51	23.20	27.00	6.9	1.2	0.3
223	28	MS	8/27/1992	11:10	4	Upstream	Single	Cylindrical	2.4	2.4	0	2.3	2.7	Insignificant	Noncohesive	0.49	7.51	23.20	27.00	6.9	2.0	0.3
224	28	MS	1/25/1990	9:00	5	Upstream	Single	Cylindrical	2.4	2.4	0	1.7	3.0	Insignificant	Noncohesive	0.49	7.51	23.20	27.00	6.9	1.4	0.3
225	28	MS	8/27/1992	10:55	5	Upstream	Single	Cylindrical	2.4	2.4	0	2.0	3.1	Insignificant	Noncohesive	0.49	7.51	23.20	27.00	6.9	1.4	0.3
226	26	MS	2/25/1991	14:30	12L	Upstream	Group	Square	0.4	8.0	28	0.8	2.9	Insignificant	Cohesive	–	–	–	–	–	0.6	0.2
227	26	MS	5/1/1991	10:00	12L	Downstream	Group	Square	0.4	8.0	23	0.8	5.1	Insignificant	Cohesive	–	–	–	–	–	0.8	0.2
228	26	MS	2/25/1991	14:30	14L	Upstream	Group	Square	0.4	8.0	18	1.0	7.3	Insignificant	Cohesive	–	–	–	–	–	0.0	0.2
229	26	MS	5/1/1991	10:00	14L	Upstream	Group	Square	0.4	8.0	16	1.3	6.4	Insignificant	Cohesive	–	–	–	–	–	0.0	0.2
230	26	MS	2/25/1991	14:30	15L	Upstream	Group	Square	0.4	8.0	16	1.2	8.9	Insignificant	Noncohesive	0.36	0.54	1.20	2.90	1.8	0.4	0.2
231	26	MS	5/1/1991	10:00	15L	Downstream	Group	Square	0.4	8.0	14	1.3	8.8	Insignificant	Noncohesive	0.36	0.54	1.20	2.90	1.8	0.9	0.2
232	26	MS	2/25/1991	14:30	16L	Upstream	Group	Square	0.8	8.0	16	1.2	8.8	Insignificant	Noncohesive	0.36	0.54	1.20	2.90	1.8	0.4	0.2
233	26	MS	5/1/1991	10:00	16L	Upstream	Group	Square	0.8	8.0	8	1.4	8.2	Moderate	Noncohesive	0.36	0.54	1.20	2.90	1.8	0.4	0.2
234	26	MS	1/31/1990	15:00	17L	Upstream	Group	Cylindrical	1.8	6.2	11	0.9	5.3	Insignificant	Noncohesive	0.26	0.39	0.90	1.30	1.9	0.6	0.2
235	26	MS	2/25/1991	14:30	17L	Upstream	Group	Cylindrical	1.6	6.2	16	1.0	6.7	Insignificant	Noncohesive	0.26	0.39	0.90	1.30	1.9	1.1	0.2
236	26	MS	5/1/1991	10:00	17L	Upstream	Group	Cylindrical	1.4	6.2	11	1.1	6.5	Insignificant	Noncohesive	0.26	0.39	0.90	1.30	1.9	1.2	0.2
237	26	MS	1/31/1990	15:00	18L	Upstream	Group	Cylindrical	1.8	6.2	11	0.4	5.3	Moderate	Noncohesive	0.26	0.39	0.90	1.30	1.9	0.5	0.2
238	26	MS	2/25/1991	14:30	18L	Upstream	Group	Cylindrical	1.6	6.2	16	0.6	6.4	Moderate	Noncohesive	0.26	0.39	0.90	1.30	1.9	0.6	0.2
239	26	MS	5/1/1991	10:00	18L	Downstream	Group	Cylindrical	1.5	6.2	14	0.7	6.8	Insignificant	Noncohesive	0.26	0.39	0.90	1.30	1.9	0.8	0.2
240	27	MS	5/1/1991	11:20	15R	Upstream	Group	Square	0.4	8.0	16	1.6	9.3	Insignificant	Noncohesive	0.36	0.54	1.20	2.90	1.8	0.4	0.2
241	27	MS	2/25/1991	15:20	16R	Upstream	Group	Square	0.8	8.0	16	1.2	8.4	Insignificant	Noncohesive	0.36	0.54	1.20	2.90	1.8	0.9	0.2
242	27	MS	5/1/1991	11:20	16R	Downstream	Group	Square	0.8	8.0	11	1.4	8.1	Insignificant	Noncohesive	0.36	0.54	1.20	2.90	1.8	0.6	0.2
243	27	MS	2/25/1991	15:20	17R	Upstream	Group	Cylindrical	1.7	6.2	16	0.9	7.1	Insignificant	Noncohesive	0.26	0.39	0.90	1.30	1.9	0.5	0.2
244	27	MS	5/1/1991	11:20	17R	Upstream	Group	Cylindrical	1.5	6.2	8	1.0	6.6	Insignificant	Noncohesive	0.26	0.39	0.90	1.30	1.9	1.2	0.2
245	27	MS	2/25/1991	15:20	18R	Upstream	Group	Cylindrical	1.6	6.2	20	0.7	6.2	Insignificant	Noncohesive	0.26	0.39	0.90	1.30	1.9	1.7	0.2

Table 24. Pier scour observations, continued.

Mea-surement	Site	State	Date	Time	Pier	Upstream/ Downstream	Pier Type	Pier Shape	Pier Width (m)	Pier Length (m)	Skew (deg)	Velocity (m/s)	Depth (m)	Debris Effects	Bed Material	D16 (mm)	D50 (mm)	D84 (mm)	D95 (mm)	Grad-ation	Scour Depth (m)	Ac-curacy (m)
246	27	MS	5/1/1991	11:20	18R	Upstream	Group	Cylindrical	1.6	6.2	14	0.6	7.0	Insignificant	Noncohesive	0.26	0.39	0.90	1.30	1.9	1.1	0.2
247	29	MS	1/30/1990	14:15	4	Downstream	Single	Square	1.6	8.2	14	2.1	6.8	Insignificant	Noncohesive	0.39	6.90	15.00	20.00	6.2	1.5	0.2
248	29	MS	5/10/1991	14:45	4	Upstream	Single	Square	1.6	8.2	8	2.1	7.5	Insignificant	Noncohesive	0.39	6.90	15.00	20.00	6.2	0.7	0.2
249	29	MS	1/30/1990	14:15	5	Upstream	Single	Square	1.9	8.2	8	2.0	8.6	Insignificant	Noncohesive	0.39	6.90	15.00	20.00	6.2	1.6	0.2
250	29	MS	5/10/1991	14:45	5	Downstream	Single	Square	1.8	8.2	11	2.0	8.8	Insignificant	Noncohesive	0.39	6.90	15.00	20.00	6.2	1.2	0.2
251	29	MS	1/30/1990	14:15	6	Upstream	Single	Square	1.7	8.1	0	1.1	8.0	Insignificant	Noncohesive	0.39	6.90	15.00	20.00	6.2	1.7	0.3
252	29	MS	5/10/1991	14:45	6	Downstream	Single	Square	1.7	8.1	11	1.6	9.2	Insignificant	Noncohesive	0.39	6.90	15.00	20.00	6.2	2.3	0.3
253	29	MS	1/30/1990	14:15	7	Upstream	Group	Square	1.2	7.0	0	0.6	7.0	Insignificant	Noncohesive	0.39	6.90	15.00	20.00	6.2	1.2	0.2
254	29	MS	5/10/1991	14:45	7	Downstream	Group	Square	1.2	7.0	0	0.9	8.8	Insignificant	Noncohesive	0.39	6.90	15.00	20.00	6.2	0.8	0.3
255	30	MS	1/27/1990	14:55	4	Upstream	Group	Cylindrical	1.6	6.4	8	1.6	8.2	Unknown	Noncohesive	0.39	6.90	15.00	20.00	6.2	0.6	0.2
256	30	MS	1/30/1990	15:00	4	Upstream	Group	Cylindrical	1.7	6.4	8	1.4	7.6	Unknown	Noncohesive	0.39	6.90	15.00	20.00	6.2	1.0	0.2
257	30	MS	5/10/1991	10:45	4	Upstream	Group	Cylindrical	1.7	6.4	11	1.6	8.9	Unknown	Noncohesive	0.39	6.90	15.00	20.00	6.2	0.4	0.2
258	30	MS	1/27/1990	12:30	5	Upstream	Group	Cylindrical	1.7	6.4	22	2.3	8.7	Unknown	Noncohesive	0.39	6.90	15.00	20.00	6.2	2.3	0.2
259	30	MS	1/30/1990	15:00	5	Upstream	Group	Cylindrical	2.0	6.4	8	1.7	8.7	Unknown	Noncohesive	0.39	6.90	15.00	20.00	6.2	1.0	0.2
260	30	MS	2/5/1990	17:35	5	Upstream	Group	Cylindrical	1.8	6.4	16	1.3	7.8	Unknown	Noncohesive	0.39	6.90	15.00	20.00	6.2	0.6	0.2
261	30	MS	5/10/1991	10:45	5	Downstream	Group	Cylindrical	1.7	6.4	11	2.0	8.8	Unknown	Noncohesive	0.39	6.90	15.00	20.00	6.2	1.4	0.2
262	30	MS	1/27/1990	14:55	6	Upstream	Group	Cylindrical	1.7	6.4	16	1.7	9.2	Insignificant	Noncohesive	0.39	6.90	15.00	20.00	6.2	1.5	0.2
263	30	MS	1/30/1990	15:00	6	Downstream	Group	Cylindrical	1.7	6.4	14	2.1	8.4	Unknown	Noncohesive	0.39	6.90	15.00	20.00	6.2	2.0	0.2
264	30	MS	2/5/1990	17:35	6	Downstream	Group	Cylindrical	1.8	6.4	18	1.3	7.7	Unknown	Noncohesive	0.39	6.90	15.00	20.00	6.2	2.0	0.2
265	30	MS	5/10/1991	10:45	6	Upstream	Group	Cylindrical	1.6	6.4	14	2.2	8.3	Unknown	Noncohesive	0.39	6.90	15.00	20.00	6.2	3.0	0.2
266	34	MT	5/21/1991	15:30	P1	Upstream	Single	Sharp	1.0	11.0	0	1.6	0.5	Moderate	Noncohesive	0.35	8.00	30.00	48.00	9.3	0.4	0.2
267	34	MT	6/4/1991	12:40	P1	Upstream	Single	Sharp	1.0	11.0	0	1.3	0.5	Moderate	Noncohesive	0.35	8.00	30.00	48.00	9.3	0.5	0.2
268	34	MT	6/21/1991	14:50	P1	Upstream	Single	Sharp	1.0	11.0	0	1.3	0.4	Moderate	Noncohesive	0.35	8.00	30.00	48.00	9.3	0.5	0.2
269	34	MT	5/21/1991	15:30	P3	Upstream	Single	Sharp	1.0	11.0	0	1.6	0.4	Moderate	Noncohesive	0.35	8.00	30.00	48.00	9.3	1.0	0.1
270	34	MT	6/4/1991	12:40	P3	Upstream	Single	Sharp	1.0	11.0	0	1.4	0.3	Moderate	Noncohesive	0.35	8.00	30.00	48.00	9.3	1.0	0.1
271	34	MT	6/21/1991	14:50	P3	Upstream	Single	Sharp	1.0	11.0	0	1.6	0.5	Moderate	Noncohesive	0.35	8.00	30.00	48.00	9.3	1.1	0.1
272	34	MT	5/21/1991	15:30	P4	Upstream	Single	Sharp	1.0	11.0	0	1.2	0.3	Moderate	Noncohesive	0.35	8.00	30.00	48.00	9.3	0.5	0.1
273	34	MT	6/4/1991	12:40	P4	Upstream	Single	Sharp	1.0	11.0	0	0.8	0.1	Moderate	Noncohesive	0.35	8.00	30.00	48.00	9.3	0.3	0.1
274	34	MT	6/21/1991	14:50	P4	Upstream	Single	Sharp	1.0	11.0	0	0.6	0.2	Moderate	Noncohesive	0.35	8.00	30.00	48.00	9.3	0.4	0.1
275	31	MT	6/6/1991	11:15	1	Upstream	Single	Sharp	1.3	15.2	5	2.5	2.6	Insignificant	Noncohesive	17.00	39.00	90.00	140.00	2.3	1.1	0.1
276	31	MT	6/10/1991	12:30	1	Upstream	Single	Sharp	1.3	15.2	5	2.1	2.0	Insignificant	Noncohesive	17.00	39.00	90.00	140.00	2.3	0.8	0.1
277	31	MT	6/13/1991	12:30	1	Upstream	Single	Sharp	1.3	15.2	5	2.1	2.3	Insignificant	Noncohesive	17.00	39.00	90.00	140.00	2.3	0.8	0.1
278	31	MT	6/18/1991	12:45	1	Upstream	Single	Sharp	1.3	15.2	5	1.5	1.4	Insignificant	Noncohesive	17.00	39.00	90.00	140.00	2.3	1.0	0.1
279	31	MT	6/6/1991	11:15	2	Upstream	Single	Sharp	1.3	15.2	5	2.0	2.2	Insignificant	Noncohesive	17.00	39.00	90.00	140.00	2.3	0.8	0.1
280	31	MT	6/10/1991	12:30	2	Upstream	Single	Sharp	1.3	15.2	5	1.8	1.7	Insignificant	Noncohesive	17.00	39.00	90.00	140.00	2.3	0.9	0.1

Table 24. Pier scour observations, continued.

Measurement	Site	State	Date	Time	Pier	Upstream/ Downstream	Pier Type	Pier Shape	Pier Width (m)	Pier Length (m)	Skew (deg)	Velocity (m/s)	Depth (m)	Debris Effects	Bed Material	D16 (mm)	D50 (mm)	D84 (mm)	D95 (mm)	Gradation	Scour Depth (m)	Accuracy (m)
281	31	MT	6/13/1991	12:30	2	Upstream	Single	Sharp	1.3	15.2	5	2.1	1.9	Insignificant	Noncohesive	17.00	39.00	90.00	140.00	2.3	0.9	0.1
282	31	MT	6/18/1991	12:45	2	Upstream	Single	Sharp	1.3	15.2	5	1.1	1.2	Insignificant	Noncohesive	17.00	39.00	90.00	140.00	2.3	0.9	0.1
283	32	MT	6/6/1991	13:35	1	Upstream	Single	Sharp	1.0	12.0	3	2.6	1.5	Insignificant	Noncohesive	38.00	95.00	230.00	330.00	2.5	0.2	0.1
284	32	MT	6/18/1992	14:45	1	Upstream	Single	Sharp	1.0	12.0	3	1.6	1.0	Insignificant	Noncohesive	38.00	95.00	230.00	330.00	2.5	0.4	0.1
285	32	MT	6/23/1993	–	1	Upstream	Single	Sharp	1.0	12.0	3	1.9	1.0	Insignificant	Noncohesive	38.00	95.00	230.00	330.00	2.5	0.6	0.1
286	32	MT	6/6/1991	13:35	2	Upstream	Single	Sharp	1.0	12.0	3	3.2	1.7	Insignificant	Noncohesive	38.00	95.00	230.00	330.00	2.5	1.7	0.2
287	32	MT	6/18/1992	14:45	2	Upstream	Single	Sharp	1.0	12.0	3	2.1	1.1	Insignificant	Noncohesive	38.00	95.00	230.00	330.00	2.5	1.4	0.2
288	32	MT	6/23/1993	0:00	2	Upstream	Single	Sharp	1.0	12.0	3	2.1	1.2	Insignificant	Noncohesive	38.00	95.00	230.00	330.00	2.5	1.4	0.2
289	33	MT	5/21/1993	16:10	1	Upstream	Single	Sharp	0.9	10.4	0	2.4	2.7	Unknown	Noncohesive	28.00	73.00	150.00	190.00	2.3	0.8	0.2
290	33	MT	5/27/1993	10:00	1	Upstream	Single	Sharp	0.9	10.4	0	2.5	2.5	Unknown	Noncohesive	28.00	73.00	150.00	190.00	2.3	0.7	0.2
291	33	MT	6/30/1993	10:30	1	Upstream	Single	Sharp	0.9	10.4	0	1.5	2.0	Unknown	Noncohesive	28.00	73.00	150.00	190.00	2.3	0.6	0.2
292	33	MT	5/21/1993	16:10	2	Upstream	Single	Sharp	1.0	10.4	0	2.3	2.5	Unknown	Noncohesive	28.00	73.00	150.00	190.00	2.3	0.5	0.1
293	33	MT	5/27/1993	10:00	2	Upstream	Single	Sharp	0.9	10.4	0	2.4	2.4	Unknown	Noncohesive	28.00	73.00	150.00	190.00	2.3	0.5	0.1
294	33	MT	6/30/1993	10:30	2	Upstream	Single	Sharp	1.0	10.4	0	1.5	1.9	Unknown	Noncohesive	28.00	73.00	150.00	190.00	2.3	0.3	0.1
295	33	MT	5/21/1993	16:10	3	Upstream	Single	Sharp	0.9	10.4	0	1.0	2.3	Unknown	Noncohesive	28.00	73.00	150.00	190.00	2.3	0.1	0.1
296	33	MT	5/27/1993	10:00	3	Upstream	Single	Sharp	0.9	10.4	0	1.1	2.1	Unknown	Noncohesive	28.00	73.00	150.00	190.00	2.3	0.1	0.1
297	33	MT	6/30/1993	10:30	3	Upstream	Single	Sharp	0.9	10.4	0	1.1	1.8	Unknown	Noncohesive	28.00	73.00	150.00	190.00	2.3	0.1	0.1
298	36	NY	10/23/1970	–	1	Upstream	Single	Round	1.5	14.6	0	2.0	3.8	Unknown	Noncohesive	11.00	27.00	58.00	106.00	2.3	0.5	0.2
299	36	NY	6/24/1972	–	1	Upstream	Single	Round	1.5	14.6	0	4.1	8.3	Unknown	Noncohesive	11.00	27.00	58.00	106.00	2.3	1.6	0.2
300	36	NY	10/23/1970	–	2	Upstream	Single	Round	1.5	14.6	0	1.8	3.7	Unknown	Noncohesive	11.00	27.00	58.00	106.00	2.3	0.3	0.2
301	36	NY	6/24/1972	–	2	Upstream	Single	Round	1.5	14.6	0	3.9	9.5	Unknown	Noncohesive	11.00	27.00	58.00	106.00	2.3	1.2	0.2
302	36	NY	3/25/1980	–	2	Upstream	Single	Round	1.5	14.6	0	2.4	4.9	Unknown	Noncohesive	11.00	27.00	58.00	106.00	2.3	0.9	0.2
303	36	NY	10/23/1970	–	3	Upstream	Single	Round	1.5	14.6	0	1.5	3.4	Unknown	Noncohesive	11.00	27.00	58.00	106.00	2.3	0.0	0.2
304	36	NY	6/24/1972	–	3	Upstream	Single	Round	1.5	14.6	0	3.7	9.6	Unknown	Noncohesive	11.00	27.00	58.00	106.00	2.3	1.2	0.2
305	36	NY	9/28/1975	–	3	Upstream	Single	Round	1.5	14.6	0	3.2	5.8	Unknown	Noncohesive	11.00	27.00	58.00	106.00	2.3	0.7	0.2
306	36	NY	3/25/1980	–	3	Upstream	Single	Round	1.5	14.6	0	2.3	3.8	Unknown	Noncohesive	11.00	27.00	58.00	106.00	2.3	1.0	0.2
307	36	NY	10/23/1970	–	4	Upstream	Single	Round	1.5	14.6	0	1.0	2.4	Unknown	Noncohesive	11.00	27.00	58.00	106.00	2.3	0.0	0.2
308	36	NY	6/24/1972	–	4	Upstream	Single	Round	1.5	14.6	0	3.4	9.7	Unknown	Noncohesive	11.00	27.00	58.00	106.00	2.3	1.3	0.3
309	36	NY	9/28/1975	–	4	Upstream	Single	Round	1.5	14.6	0	2.9	5.7	Unknown	Noncohesive	11.00	27.00	58.00	106.00	2.3	0.6	0.2
310	36	NY	3/25/1980	–	4	Upstream	Single	Round	1.5	14.6	0	2.1	3.7	Unknown	Noncohesive	11.00	27.00	58.00	106.00	2.3	0.0	0.2
311	36	NY	10/23/1970	–	5	Upstream	Single	Round	1.5	14.6	0	0.8	2.0	Unknown	Noncohesive	11.00	27.00	58.00	106.00	2.3	0.0	0.2
312	36	NY	6/24/1972	–	5	Upstream	Single	Round	1.5	14.6	0	3.2	8.1	Unknown	Noncohesive	11.00	27.00	58.00	106.00	2.3	1.2	0.2
313	36	NY	9/28/1975	–	5	Upstream	Single	Round	1.5	14.6	0	2.7	5.4	Unknown	Noncohesive	11.00	27.00	58.00	106.00	2.3	0.6	0.2
314	36	NY	3/25/1980	–	5	Upstream	Single	Round	1.5	14.6	0	2.0	3.6	Unknown	Noncohesive	11.00	27.00	58.00	106.00	2.3	0.0	0.2
315	36	NY	10/23/1970	–	6	Upstream	Single	Round	1.5	14.6	0	0.5	1.7	Unknown	Noncohesive	11.00	27.00	58.00	106.00	2.3	0.0	0.2

Table 24. Pier scour observations, continued.

Mea-surement	Site	State	Date	Time	Pier	Upstream/ Downstream	Pier Type	Pier Shape	Pier Width (m)	Pier Length (m)	Skew (deg)	Velocity (m/s)	Depth (m)	Debris Effects	Bed Material	D16 (mm)	D50 (mm)	D84 (mm)	D95 (mm)	Grad-ation	Scour Depth (m)	Ac-curacy (m)
316	36	NY	6/24/1972	–	6	Upstream	Single	Round	1.5	14.6	0	2.7	5.8	Unknown	Noncohesive	11.00	27.00	58.00	106.00	2.3	0.6	0.2
317	36	NY	9/28/1975	–	6	Upstream	Single	Round	1.5	14.6	0	2.3	3.8	Unknown	Noncohesive	11.00	27.00	58.00	106.00	2.3	0.3	0.2
318	36	NY	3/25/1980	–	6	Upstream	Single	Round	1.5	14.6	0	1.6	2.6	Unknown	Noncohesive	11.00	27.00	58.00	106.00	2.3	0.0	0.2
319	40	NY	9/16/1992	–	1	Upstream	Single	Round	3.0	17.7	0	4.5	7.7	Unknown	Noncohesive	14.00	45.00	103.00	80.00	2.7	0.9	0.2
320	39	NY	10/21/1998	–	3	Upstream	Single	Sharp	1.8	6.7	0	2.9	6.4	Unknown	Noncohesive	18.00	27.00	42.00	80.00	1.5	1.0	0.2
321	35	NY	10/24/1990	–	1	Upstream	Single	Round	0.9	12.2	30	2.1	3.1	Insignificant	Noncohesive	18.00	32.00	55.00	80.00	1.8	1.6	0.1
322	37	NY	11/17/1989	–	1	Upstream	Single	Round	1.7	13.1	0	3.7	5.5	Insignificant	Noncohesive	15.00	33.00	60.00	103.00	2.0	0.6	0.1
323	37	NY	11/17/1989	–	2	Upstream	Single	Round	1.7	13.1	0	3.4	5.1	Insignificant	Noncohesive	15.00	33.00	60.00	103.00	2.0	1.1	0.1
324	38	NY	8/27/1991	–	2	Upstream	Single	Round	1.5	12.2	0	2.5	5.5	Unknown	Noncohesive	14.00	28.00	53.00	80.00	1.9	0.3	0.1
325	38	NY	8/27/1991	–	3	Upstream	Single	Round	1.5	12.2	0	2.7	5.6	Unknown	Noncohesive	14.00	28.00	53.00	80.00	1.9	0.3	0.1
326	38	NY	8/27/1991	–	4	Upstream	Single	Round	1.5	12.2	0	2.2	5.1	Moderate	Noncohesive	14.00	28.00	53.00	80.00	1.9	1.0	0.1
327	79	OH	7/13/1992	14:30	1	Upstream	Single	Round	0.8	9.4	40	0.5	3.2	Insignificant	Noncohesive	0.01	0.12	2.40	5.80	20.3	0.5	0.2
328	79	OH	7/17/1992	16:10	1	Upstream	Single	Round	0.8	9.4	50	0.3	2.2	Insignificant	Noncohesive	0.01	0.12	2.40	5.80	20.3	0.2	0.2
329	59	OH	1/28/1994	10:00	1	Upstream	Single	Round	1.2	9.4	0	1.1	3.2	Unknown	Noncohesive	0.03	0.15	0.28	0.35	2.9	0.9	0.2
330	60	OH	12/31/1992	11:05	1	Upstream	Single	Round	1.6	10.9	0	1.5	2.1	Insignificant	Noncohesive	0.39	12.00	31.00	44.00	8.9	0.4	0.2
331	60	OH	12/31/1992	11:05	3	Upstream	Single	Round	1.6	10.9	85	0.8	1.8	Insignificant	Noncohesive	0.09	9.00	33.50	71.00	19.3	0.3	0.2
332	41	OH	5/16/1990	10:00	2	Upstream	Single	Round	1.1	24.9	0	1.4	3.9	Insignificant	Noncohesive	0.75	1.82	5.00	8.80	2.6	0.5	0.2
333	41	OH	7/18/1992	12:45	2	Upstream	Single	Round	1.1	24.9	8	1.5	3.7	Insignificant	Noncohesive	0.75	1.82	5.00	8.80	2.6	0.4	0.2
334	41	OH	1/29/1994	9:10	2	Upstream	Single	Round	1.1	24.9	0	1.8	4.1	Insignificant	Noncohesive	0.25	1.30	14.00	18.00	7.5	0.3	0.2
335	41	OH	5/16/1990	10:00	3	Upstream	Single	Round	1.1	24.9	0	1.4	4.1	Insignificant	Noncohesive	0.14	0.78	1.60	1.80	3.4	0.3	0.2
336	61	OH	7/17/1992	11:40	1	Upstream	Single	Sharp	0.9	17.4	60	0.2	2.2	Insignificant	Noncohesive	–	0.01	0.08	0.33	–	0.2	0.2
337	61	OH	11/13/1992	12:10	1	Upstream	Single	Sharp	0.9	17.4	60	0.2	1.8	Insignificant	Noncohesive	–	0.06	0.18	5.40	–	0.2	0.2
338	61	OH	1/29/1994	10:55	1	Upstream	Single	Sharp	0.9	17.4	66	0.2	1.6	Unknown	Noncohesive	0.57	8.20	32.00	40.00	7.5	0.5	0.2
339	61	OH	7/17/1992	11:40	2	Upstream	Single	Sharp	0.9	17.4	66	0.2	2.3	Insignificant	Noncohesive	–	0.01	0.70	0.96	–	0.4	0.2
340	61	OH	11/13/1992	12:10	2	Upstream	Single	Sharp	0.9	17.4	66	0.2	1.9	Insignificant	Noncohesive	0.05	0.25	9.50	25.00	14.1	0.5	0.2
341	61	OH	1/29/1994	10:55	2	Upstream	Single	Sharp	0.9	17.4	66	0.3	1.7	Unknown	Noncohesive	3.50	18.00	33.00	43.00	3.1	0.5	0.2
342	61	OH	7/17/1992	11:40	3	Upstream	Single	Sharp	0.9	17.4	63	0.5	2.8	Insignificant	Noncohesive	2.20	36.30	64.00	72.00	5.4	1.0	0.2
343	61	OH	11/13/1992	12:10	3	Upstream	Single	Sharp	0.9	17.4	63	0.5	2.4	Insignificant	Noncohesive	1.65	22.50	39.50	47.00	4.9	0.8	0.2
344	61	OH	1/29/1994	10:55	3	Upstream	Single	Sharp	0.9	17.4	63	0.6	2.4	Unknown	Noncohesive	4.00	21.00	65.00	71.00	4.0	1.0	0.2
345	61	OH	7/17/1992	11:40	4	Upstream	Single	Sharp	0.9	17.4	62	0.8	3.0	Insignificant	Noncohesive	5.00	12.70	60.00	69.00	3.5	0.7	0.2
346	61	OH	11/13/1992	12:10	4	Upstream	Single	Sharp	0.9	17.4	64	0.6	2.7	Insignificant	Noncohesive	1.40	14.00	36.00	41.00	5.1	1.2	0.2
347	61	OH	1/29/1994	10:55	4	Upstream	Single	Sharp	0.9	17.4	60	0.7	2.4	Unknown	Noncohesive	6.50	27.00	43.00	48.00	2.6	1.5	0.2
348	61	OH	7/17/1992	11:40	5	Upstream	Single	Sharp	0.9	17.4	65	0.8	2.9	Insignificant	Noncohesive	1.60	4.10	10.30	38.00	6.4	0.6	0.2
349	61	OH	11/13/1992	12:10	5	Upstream	Single	Sharp	0.9	17.4	65	0.6	2.6	Insignificant	Noncohesive	2.70	29.00	58.00	70.00	4.6	0.9	0.2
350	61	OH	1/29/1994	10:55	5	Upstream	Single	Sharp	0.9	17.4	66	0.6	2.2	Unknown	Noncohesive	4.00	32.00	53.00	63.00	3.6	0.8	0.2

Table 24. Pier scour observations, continued.

Measurement	Site	State	Date	Time	Pier	Upstream/ Downstream	Pier Type	Pier Shape	Pier Width (m)	Pier Length (m)	Skew (deg)	Velocity (m/s)	Depth (m)	Debris Effects	Bed Material	D16 (mm)	D50 (mm)	D84 (mm)	D95 (mm)	Gradation	Scour Depth (m)	Accuracy (m)
351	61	OH	7/17/1992	11:40	6	Upstream	Single	Sharp	0.9	17.4	60	0.8	2.9	Insignificant	Noncohesive	1.00	35.50	62.00	70.00	7.9	0.8	0.2
352	61	OH	11/13/1992	12:10	6	Upstream	Single	Sharp	0.9	17.4	57	0.9	2.6	Insignificant	Noncohesive	9.50	31.00	56.00	79.00	2.4	1.0	0.2
353	61	OH	1/29/1994	10:55	6	Upstream	Single	Sharp	0.9	17.4	60	0.7	2.3	Unknown	Noncohesive	9.00	34.00	55.00	65.00	2.5	1.0	0.2
354	61	OH	7/17/1992	11:40	7	Upstream	Single	Sharp	0.9	17.4	60	0.8	3.0	Insignificant	Noncohesive	0.98	4.10	58.00	70.00	7.7	0.9	0.2
355	61	OH	11/13/1992	12:10	7	Upstream	Single	Sharp	0.9	17.4	60	0.7	2.5	Insignificant	Noncohesive	7.80	56.00	80.00	90.00	3.2	1.1	0.2
356	61	OH	1/29/1994	10:55	7	Upstream	Single	Sharp	0.9	17.4	60	0.7	2.3	Unknown	Noncohesive	4.20	24.00	45.00	62.00	3.3	1.1	0.2
357	42	OH	12/31/1990	13:00	1	Upstream	Single	Round	0.8	9.1	22	1.5	5.3	Insignificant	Noncohesive	0.86	2.85	4.70	6.00	2.3	0.8	0.2
358	42	OH	7/14/1992	10:50	1	Upstream	Single	Round	0.8	9.1	32	0.9	3.6	Insignificant	Noncohesive	0.42	11.50	55.00	80.00	11.4	0.8	0.2
359	42	OH	7/17/1992	12:40	1	Upstream	Single	Round	0.8	9.1	20	1.2	3.5	Insignificant	Noncohesive	0.42	11.50	55.00	80.00	11.4	0.7	0.2
360	42	OH	1/29/1994	13:50	1	Upstream	Single	Round	0.8	9.1	14	1.8	5.2	Unknown	Noncohesive	0.68	2.50	8.00	14.00	3.4	0.7	0.2
361	42	OH	12/31/1990	13:00	2	Upstream	Single	Round	0.8	9.1	16	0.7	4.2	Insignificant	Noncohesive	0.03	0.17	0.25	0.45	2.8	0.3	0.2
362	42	OH	7/14/1992	10:50	2	Upstream	Single	Round	0.8	9.1	12	0.6	2.3	Insignificant	Noncohesive	0.05	0.15	0.30	0.38	6.3	0.3	0.2
363	42	OH	7/17/1992	12:40	2	Upstream	Single	Round	0.8	9.1	8	1.2	2.0	Insignificant	Noncohesive	0.05	0.15	0.30	0.38	6.3	0.3	0.2
364	42	OH	1/29/1994	13:50	2	Upstream	Single	Round	0.8	9.1	0	0.8	3.7	Unknown	Noncohesive	0.07	0.16	0.26	0.38	2.0	0.2	0.2
365	43	OH	2/2/1990	15:30	2	Upstream	Single	Round	1.0	10.9	0	1.2	2.0	Insignificant	Noncohesive	9.30	18.00	34.00	44.00	1.9	0.5	0.2
366	43	OH	7/15/1992	14:25	2	Upstream	Single	Round	0.9	10.9	12	1.2	2.1	Insignificant	Noncohesive	12.00	54.00	69.00	85.00	2.4	0.4	0.2
367	43	OH	7/17/1992	10:40	2	Upstream	Single	Round	0.9	10.9	0	1.2	2.1	Insignificant	Noncohesive	12.00	54.00	69.00	85.00	2.4	0.5	0.2
368	48	OH	5/17/1990	12:15	1	Upstream	Single	Round	0.8	9.6	0	0.7	2.2	Insignificant	Noncohesive	0.08	0.19	0.54	1.10	2.6	0.4	0.2
369	48	OH	7/23/1990	13:05	1	Upstream	Single	Round	0.8	9.6	0	0.9	2.8	Insignificant	Noncohesive	0.08	0.19	0.54	1.10	2.6	0.4	0.2
370	44	OH	5/16/1990	14:30	1	Upstream	Single	Round	0.8	7.4	0	1.4	2.6	Insignificant	Noncohesive	14.70	35.00	38.00	39.00	1.6	0.2	0.2
371	44	OH	12/19/1990	10:10	1	Upstream	Single	Round	0.8	7.4	0	1.5	2.3	Insignificant	Noncohesive	14.70	35.00	38.00	39.00	1.6	0.8	0.3
372	44	OH	12/19/1990	10:10	2	Upstream	Single	Round	0.8	7.4	0	1.1	1.7	Insignificant	Noncohesive	12.00	60.00	71.00	75.00	2.4	0.2	0.2
373	62	OH	1/5/1993	12:10	1	Upstream	Single	Round	1.6	11.3	0	1.0	0.9	Substantial	Noncohesive	11.50	19.00	32.00	42.00	1.7	1.2	0.2
374	62	OH	7/2/1993	10:30	1	Upstream	Single	Round	1.6	11.3	0	1.2	1.6	Substantial	Noncohesive	3.80	25.00	56.00	69.00	3.8	1.4	0.2
375	62	OH	1/28/1994	11:25	1	Upstream	Single	Round	1.6	11.3	0	1.4	2.0	Unknown	Noncohesive	3.80	25.00	56.00	69.00	3.8	1.0	0.2
376	63	OH	1/28/1994	10:50	1	Upstream	Group	Square	0.3	14.4	14	0.3	1.8	Unknown	Noncohesive	0.12	0.46	1.20	1.80	3.2	0.2	0.2
377	64	OH	7/14/1992	10:45	2	Upstream	Single	Sharp	1.2	16.4	32	0.8	1.7	Insignificant	Noncohesive	4.10	17.30	44.00	53.00	3.3	0.4	0.2
378	64	OH	7/18/1992	10:15	2	Upstream	Single	Sharp	1.2	16.4	37	1.1	3.8	Insignificant	Noncohesive	4.10	17.30	44.00	53.00	3.3	0.5	0.2
379	64	OH	1/6/1993	13:10	2	Upstream	Single	Sharp	1.2	16.4	15	1.5	6.0	Insignificant	Noncohesive	4.10	12.00	44.00	53.00	2.6	0.2	0.2
380	64	OH	7/14/1992	10:45	3	Upstream	Single	Sharp	1.2	16.4	36	0.9	1.5	Insignificant	Noncohesive	0.34	1.30	3.00	6.60	3.0	0.3	0.2
381	64	OH	7/18/1992	10:15	3	Upstream	Single	Sharp	1.2	16.4	32	0.8	3.6	Substantial	Noncohesive	0.34	1.30	3.00	6.60	3.0	0.4	0.2
382	64	OH	1/6/1993	13:10	3	Upstream	Single	Sharp	1.2	16.4	10	0.9	5.8	Moderate	Noncohesive	0.18	2.85	8.80	14.00	7.0	0.2	0.2
383	45	OH	8/22/1990	10:40	1	Upstream	Single	Round	0.8	11.4	0	0.8	1.6	Unknown	Noncohesive	0.90	4.00	26.00	45.00	5.7	0.8	0.2
384	45	OH	12/30/1990	14:30	1	Upstream	Single	Round	0.8	11.4	0	1.3	3.2	Unknown	Noncohesive	0.90	4.00	26.00	45.00	5.7	0.7	0.2
385	45	OH	7/17/1992	14:15	1	Upstream	Single	Round	0.8	11.4	0	1.2	2.3	Moderate	Noncohesive	0.66	6.80	30.50	44.00	6.8	0.5	0.2

Table 24. Pier scour observations, continued.

Measurement	Site	State	Date	Time	Pier	Upstream/ Downstream	Pier Type	Pier Shape	Pier Width (m)	Pier Length (m)	Skew (deg)	Velocity (m/s)	Depth (m)	Debris Effects	Bed Material	D16 (mm)	D50 (mm)	D84 (mm)	D95 (mm)	Gradation	Scour Depth (m)	Accuracy (m)
386	45	OH	8/22/1990	10:40	2	Upstream	Single	Round	0.8	11.4	0	0.2	1.5	Insignificant	Noncohesive	0.08	0.25	5.00	7.90	8.0	0.2	0.2
387	45	OH	12/30/1990	14:30	2	Upstream	Single	Round	0.8	11.4	0	0.7	3.1	Insignificant	Noncohesive	0.08	0.25	5.00	7.90	8.0	0.2	0.2
388	65	OH	1/28/1994	10:50	1	Upstream	Single	Round	1.3	8.3	18	1.5	4.5	Insignificant	Noncohesive	0.10	0.30	0.75	1.40	2.7	0.6	0.2
389	65	OH	1/28/1994	10:50	2	Upstream	Single	Round	1.3	8.3	18	1.7	6.0	Unknown	Noncohesive	0.46	2.20	8.00	15.00	4.2	0.9	0.2
390	46	OH	5/14/1990	16:05	1	Upstream	Single	Sharp	1.1	10.2	8	0.4	1.8	Moderate	Noncohesive	0.03	0.17	0.36	0.70	3.5	0.2	0.2
391	46	OH	5/18/1990	10:50	1	Upstream	Single	Sharp	1.1	10.2	8	0.5	2.5	Insignificant	Noncohesive	0.03	0.17	0.36	0.70	3.5	0.1	0.2
392	46	OH	12/31/1990	11:55	1	Upstream	Single	Sharp	1.1	10.2	8	0.8	4.2	Insignificant	Noncohesive	0.03	0.17	0.36	0.70	3.5	0.2	0.2
393	66	OH	1/29/1994	11:50	28	Upstream	Single	Round	1.4	18.3	0	0.9	3.7	Insignificant	Noncohesive	0.70	6.00	18.00	22.00	5.1	0.4	0.2
394	66	OH	1/29/1994	11:50	29	Upstream	Single	Round	1.4	18.3	8	1.5	5.9	Unknown	Noncohesive	0.18	0.59	1.30	1.70	2.7	1.3	0.2
395	66	OH	1/29/1994	11:50	30	Upstream	Single	Round	1.4	18.3	10	1.3	5.7	Unknown	Noncohesive	0.04	2.41	16.00	19.00	19.8	1.9	0.2
396	67	OH	1/14/1993	11:15	1	Upstream	Single	Round	0.9	18.8	0	1.3	1.7	Insignificant	Noncohesive	0.74	21.50	66.00	82.00	9.4	0.2	0.2
397	67	OH	12/6/1993	8:20	1	Upstream	Single	Round	0.9	18.8	0	1.3	1.8	Unknown	Noncohesive	6.10	19.00	45.00	63.00	2.7	0.2	0.2
398	69	OH	1/15/1993	10:45	2	Upstream	Single	Round	1.0	8.9	0	1.2	2.3	Insignificant	Noncohesive	0.46	1.15	3.10	8.50	2.6	0.5	0.2
399	69	OH	12/6/1993	10:00	2	Upstream	Single	Round	1.0	8.9	0	1.5	2.4	Insignificant	Noncohesive	1.30	10.00	27.00	40.00	4.6	0.7	0.2
400	69	OH	1/15/1993	10:45	3	Upstream	Single	Round	1.0	8.9	0	1.3	3.0	Moderate	Unknown	4.40	11.50	22.00	33.00	2.2	0.4	0.2
401	69	OH	12/6/1993	10:00	3	Upstream	Single	Round	1.0	8.9	0	1.4	3.1	Unknown	Noncohesive	3.70	17.00	29.00	38.00	2.8	0.5	0.2
402	47	OH	5/16/1990	11:45	1	Upstream	Single	Round	1.1	11.9	0	1.6	1.6	Insignificant	Noncohesive	0.67	5.00	16.00	22.00	4.9	0.4	0.2
403	47	OH	5/17/1990	9:05	1	Upstream	Single	Round	1.1	11.9	0	2.1	2.9	Insignificant	Noncohesive	0.67	5.00	16.00	22.00	4.9	0.7	0.2
404	47	OH	12/18/1990	14:30	1	Upstream	Single	Round	1.1	11.9	0	1.8	3.1	Insignificant	Noncohesive	0.67	5.00	16.00	22.00	4.9	0.5	0.2
405	47	OH	1/28/1994	12:50	1	Upstream	Single	Round	1.1	11.9	0	1.8	2.5	Insignificant	Noncohesive	13.00	30.00	40.00	46.00	1.8	0.6	0.2
406	47	OH	5/16/1990	11:45	2	Upstream	Single	Round	1.3	11.9	0	1.7	2.0	Insignificant	Noncohesive	4.55	17.00	29.00	40.00	2.5	0.7	0.2
407	47	OH	5/17/1990	9:05	2	Upstream	Single	Round	1.3	11.9	0	2.1	3.2	Insignificant	Noncohesive	4.55	17.00	29.00	40.00	2.5	0.9	0.2
408	47	OH	12/18/1990	14:30	2	Upstream	Single	Round	1.4	11.9	0	2.0	3.4	Insignificant	Noncohesive	4.55	17.00	29.00	40.00	2.5	1.0	0.2
409	47	OH	1/28/1994	12:50	2	Upstream	Single	Round	1.3	11.9	0	1.7	2.7	Insignificant	Unknown	37.00	51.00	63.00	70.00	1.3	1.0	0.2
410	71	OH	7/13/1992	12:30	1	Upstream	Single	Round	0.6	10.2	0	0.9	2.0	Insignificant	Noncohesive	1.20	6.80	16.00	23.00	3.7	0.5	0.2
411	71	OH	7/17/1992	15:00	1	Upstream	Single	Round	0.6	10.2	0	1.2	3.1	Moderate	Noncohesive	1.20	6.80	16.00	23.00	3.7	0.3	0.2
412	74	TX	10/23/1994	–	6	Downstream	Group	Square	22.9	14.0	0	2.4	12.5	Substantial	Cohesive	–	–	–	–	–	7.5	0.9
413	51	VA	5/29/1990	12:00	2	Upstream	Single	Round	0.8	13.1	0	0.6	2.7	Unknown	Unknown	0.29	0.92	4.80	10.50	4.1	0.8	0.3
414	52	VA	10/24/1990	11:00	1	Upstream	Single	Round	1.0	25.3	0	1.6	6.2	Unknown	Noncohesive	0.14	0.28	0.46	0.77	1.8	1.1	0.3
415	52	VA	10/25/1990	11:00	1	Upstream	Single	Round	1.0	25.3	0	1.9	7.9	Unknown	Noncohesive	0.14	0.28	0.46	0.77	1.8	1.2	0.3
416	52	VA	4/22/1992	12:00	1	Upstream	Single	Round	1.0	25.3	0	1.3	5.8	Unknown	Noncohesive	0.14	0.28	0.46	0.77	1.8	0.8	0.3
417	52	VA	10/25/1990	11:00	2	Upstream	Single	Round	1.0	25.3	0	2.2	9.3	Unknown	Noncohesive	0.14	0.28	0.46	0.77	1.8	1.1	0.3
418	52	VA	4/22/1992	12:00	2	Upstream	Single	Round	1.0	25.3	0	1.7	8.4	Unknown	Noncohesive	0.14	0.28	0.46	0.77	1.8	1.5	0.3
419	54	VA	5/2/1989	13:00	1	Upstream	Single	Round	0.7	8.5	0	0.7	1.7	Unknown	Unknown	0.35	0.69	1.30	1.90	1.9	0.4	0.3
420	54	VA	8/24/1990	10:30	1	Upstream	Single	Round	0.7	8.5	0	0.7	1.8	Unknown	Unknown	0.35	0.69	1.30	1.90	1.9	0.4	0.3

Table 24. Pier scour observations, continued.

Mea-surement	Site	State	Date	Time	Pier	Upstream/ Downstream	Pier Type	Pier Shape	Pier Width (m)	Pier Length (m)	Skew (deg)	Velocity (m/s)	Depth (m)	Debris Effects	Bed Material	D16 (mm)	D50 (mm)	D84 (mm)	D95 (mm)	Grad-ation	Scour Depth (m)	Ac-curacy (m)
421	54	VA	8/24/1990	14:30	1	Upstream	Single	Round	0.7	8.5	0	0.6	1.6	Unknown	Unknown	0.35	0.69	1.30	1.90	1.9	0.2	0.3
422	54	VA	3/29/1991	12:30	1	Upstream	Single	Round	0.7	8.5	0	0.4	0.9	Unknown	Unknown	0.35	0.69	1.30	1.90	1.9	0.4	0.3
423	54	VA	3/29/1991	15:30	1	Upstream	Single	Round	0.7	8.5	0	0.5	1.4	Unknown	Unknown	0.35	0.69	1.30	1.90	1.9	0.4	0.3
424	54	VA	3/29/1991	17:30	1	Upstream	Single	Round	0.7	8.5	0	0.5	1.6	Unknown	Unknown	0.35	0.69	1.30	1.90	1.9	0.4	0.3
425	54	VA	5/2/1989	13:00	2	Upstream	Single	Round	0.7	8.5	0	0.2	1.0	Unknown	Unknown	0.35	0.69	1.30	1.90	1.9	0.5	0.3
426	54	VA	8/24/1990	10:30	2	Upstream	Single	Round	0.7	8.5	0	0.3	0.9	Unknown	Unknown	0.35	0.69	1.30	1.90	1.9	0.7	0.3
427	54	VA	5/2/1989	13:00	3	Upstream	Single	Round	0.7	8.5	0	0.2	0.9	Unknown	Unknown	0.35	0.69	1.30	1.90	1.9	0.5	0.3
428	54	VA	8/24/1990	10:30	3	Upstream	Single	Round	0.7	8.5	0	0.3	0.8	Unknown	Unknown	0.35	0.69	1.30	1.90	1.9	0.4	0.3
429	54	VA	8/24/1990	14:30	3	Upstream	Single	Round	0.7	8.5	0	0.2	0.6	Unknown	Unknown	0.35	0.69	1.30	1.90	1.9	0.4	0.3
430	54	VA	3/29/1991	17:30	3	Upstream	Single	Round	0.7	8.5	0	0.5	1.1	Unknown	Unknown	0.35	0.69	1.30	1.90	1.9	0.5	0.3
431	56	VA	5/29/1990	11:00	1	Upstream	Single	Round	0.6	9.0	0	1.2	2.2	Unknown	Noncohesive	18.00	37.00	75.00	180.00	2.0	0.1	0.3
432	56	VA	5/29/1990	11:00	2	Upstream	Single	Round	0.6	9.0	0	1.4	2.4	Unknown	Unknown	18.00	37.00	75.00	180.00	2.0	0.1	0.3
433	56	VA	3/30/1991	10:00	2	Upstream	Single	Round	0.6	9.0	0	1.4	2.6	Unknown	Unknown	18.00	37.00	75.00	180.00	2.0	0.2	0.3
434	56	VA	2/26/1992	10:00	2	Upstream	Single	Round	0.6	9.0	0	1.4	3.0	Unknown	Unknown	18.00	37.00	75.00	180.00	2.0	0.3	0.3
435	56	VA	3/4/1993	13:30	2	Upstream	Single	Round	0.6	9.0	0	1.5	3.0	Unknown	Unknown	18.00	37.00	75.00	180.00	2.0	0.2	0.3
436	56	VA	3/4/1993	17:30	2	Upstream	Single	Round	0.6	9.0	0	1.6	3.3	Unknown	Unknown	18.00	37.00	75.00	180.00	2.0	0.4	0.3
437	50	VA	1/16/1991	10:30	CNTR	Upstream	Single	Round	0.9	9.8	0	0.9	4.7	Unknown	Unknown	0.35	0.74	2.00	4.00	2.4	0.3	0.3
438	50	VA	1/18/1991	10:30	CNTR	Upstream	Single	Round	0.9	9.8	0	0.8	4.9	Unknown	Unknown	0.35	0.74	2.00	4.00	2.4	0.5	0.3
439	50	VA	4/4/1991	14:30	CNTR	Upstream	Single	Round	0.9	9.8	0	1.0	5.0	Unknown	Unknown	0.35	0.74	2.00	4.00	2.4	0.4	0.3
440	50	VA	4/5/1991	12:00	CNTR	Upstream	Single	Round	0.9	9.8	0	1.0	4.8	Unknown	Unknown	0.35	0.74	2.00	4.00	2.4	0.4	0.3
441	50	VA	3/12/1992	11:30	CNTR	Upstream	Single	Round	0.9	9.8	0	0.9	4.5	Unknown	Unknown	0.35	0.74	2.00	4.00	2.4	0.4	0.3
442	50	VA	3/11/1993	9:30	CNTR	Upstream	Single	Round	0.9	9.8	0	1.1	5.0	Unknown	Unknown	0.35	0.74	2.00	4.00	2.4	0.4	0.3
443	50	VA	3/17/1993	9:30	CNTR	Upstream	Single	Round	0.9	9.8	0	1.0	5.1	Unknown	Unknown	0.35	0.74	2.00	4.00	2.4	0.5	0.3
444	50	VA	3/18/1993	12:00	CNTR	Upstream	Single	Round	0.9	9.8	0	1.1	4.8	Unknown	Unknown	0.35	0.74	2.00	4.00	2.4	0.6	0.3
445	50	VA	3/19/1993	11:00	CNTR	Upstream	Single	Round	0.9	9.8	0	1.1	5.2	Unknown	Unknown	0.35	0.74	2.00	4.00	2.4	0.6	0.3
446	50	VA	4/1/1993	10:00	CNTR	Upstream	Single	Round	0.9	9.8	0	1.1	4.7	Unknown	Unknown	0.35	0.74	2.00	4.00	2.4	0.6	0.3
447	50	VA	1/16/1991	10:30	LEFT	Upstream	Single	Round	0.9	9.8	0	0.9	4.3	Unknown	Unknown	0.35	0.74	2.00	4.00	2.4	0.3	0.3
448	50	VA	1/18/1991	10:30	LEFT	Upstream	Single	Round	0.9	9.8	0	0.8	4.4	Unknown	Unknown	0.35	0.74	2.00	4.00	2.4	0.2	0.3
449	50	VA	4/4/1991	14:30	LEFT	Upstream	Single	Round	0.9	9.8	0	0.9	4.5	Unknown	Unknown	0.35	0.74	2.00	4.00	2.4	0.3	0.3
450	50	VA	4/5/1991	12:00	LEFT	Upstream	Single	Round	0.9	9.8	0	0.9	4.3	Unknown	Unknown	0.35	0.74	2.00	4.00	2.4	0.3	0.3
451	50	VA	3/12/1992	11:30	LEFT	Upstream	Single	Round	0.9	9.8	0	0.9	4.4	Unknown	Unknown	0.35	0.74	2.00	4.00	2.4	0.5	0.3
452	50	VA	3/11/1993	9:30	LEFT	Upstream	Single	Round	0.9	9.8	0	1.0	4.6	Unknown	Unknown	0.35	0.74	2.00	4.00	2.4	0.4	0.3
453	50	VA	3/17/1993	9:30	LEFT	Upstream	Single	Round	0.9	9.8	0	1.0	4.4	Unknown	Unknown	0.35	0.74	2.00	4.00	2.4	0.4	0.3
454	50	VA	3/18/1993	12:00	LEFT	Upstream	Single	Round	0.9	9.8	0	1.0	4.6	Unknown	Unknown	0.35	0.74	2.00	4.00	2.4	0.5	0.3
455	50	VA	3/19/1993	11:00	LEFT	Upstream	Single	Round	0.9	9.8	0	1.0	4.6	Unknown	Unknown	0.35	0.74	2.00	4.00	2.4	0.5	0.3

Table 24. Pier scour observations, continued.

Measurement	Site	State	Date	Time	Pier	Upstream/ Downstream	Pier Type	Pier Shape	Pier Width (m)	Pier Length (m)	Skew (deg)	Velocity (m/s)	Depth (m)	Debris Effects	Bed Material	D16 (mm)	D50 (mm)	D84 (mm)	D95 (mm)	Gradation	Scour Depth (m)	Accuracy (m)
456	50	VA	4/1/1993	10:00	LEFT	Upstream	Single	Round	0.9	9.8	0	1.0	4.7	Unknown	Unknown	0.35	0.74	2.00	4.00	2.4	0.4	0.3
457	49	VA	5/31/1990	13:30	CNTR	Upstream	Single	Round	0.9	10.7	0	1.4	8.7	Unknown	Unknown	0.32	0.70	1.60	2.80	2.2	1.5	0.3
458	49	VA	1/14/1991	12:00	CNTR	Upstream	Single	Round	0.9	10.7	0	1.0	7.3	Unknown	Unknown	0.32	0.70	1.60	2.80	2.2	0.8	0.3
459	49	VA	12/13/1992	11:30	CNTR	Upstream	Single	Round	0.9	10.7	0	1.0	4.9	Unknown	Unknown	0.32	0.70	1.60	2.80	2.2	1.4	0.3
460	49	VA	12/14/1992	10:00	CNTR	Upstream	Single	Round	0.9	10.7	0	1.0	6.9	Unknown	Unknown	0.32	0.70	1.60	2.80	2.2	1.1	0.3
461	49	VA	12/15/1992	11:00	CNTR	Upstream	Single	Round	0.9	10.7	0	0.9	6.7	Unknown	Unknown	0.32	0.70	1.60	2.80	2.2	1.4	0.3
462	49	VA	1/11/1993	12:00	CNTR	Upstream	Single	Round	0.9	10.7	0	0.9	6.5	Unknown	Unknown	0.32	0.70	1.60	2.80	2.2	1.6	0.3
463	49	VA	1/12/1993	10:30	CNTR	Upstream	Single	Round	0.9	10.7	0	0.9	6.6	Unknown	Unknown	0.32	0.70	1.60	2.80	2.2	1.3	0.3
464	49	VA	4/12/1993	12:00	CNTR	Upstream	Single	Round	0.9	10.7	0	1.0	7.5	Unknown	Unknown	0.32	0.70	1.60	2.80	2.2	1.2	0.3
465	49	VA	4/13/1993	9:30	CNTR	Upstream	Single	Round	0.9	10.7	0	1.0	7.4	Unknown	Unknown	0.32	0.70	1.60	2.80	2.2	1.1	0.3
466	49	VA	4/19/1993	10:00	CNTR	Upstream	Single	Round	0.9	10.7	0	1.0	7.6	Unknown	Unknown	0.32	0.70	1.60	2.80	2.2	1.1	0.3
467	49	VA	5/31/1990	13:30	LEFT	Upstream	Single	Round	0.9	10.7	0	0.5	6.5	Unknown	Unknown	0.32	0.70	1.60	2.80	2.2	1.0	0.3
468	49	VA	12/13/1992	11:30	LEFT	Upstream	Single	Round	0.9	10.7	0	0.4	5.0	Unknown	Unknown	0.32	0.70	1.60	2.80	2.2	0.5	0.3
469	49	VA	12/14/1992	10:00	LEFT	Upstream	Single	Round	0.9	10.7	0	0.4	5.3	Unknown	Unknown	0.32	0.70	1.60	2.80	2.2	0.9	0.3
470	49	VA	12/15/1992	11:00	LEFT	Upstream	Single	Round	0.9	10.7	0	0.3	5.2	Unknown	Unknown	0.32	0.70	1.60	2.80	2.2	0.6	0.3
471	49	VA	1/11/1993	12:00	LEFT	Upstream	Single	Round	0.9	10.7	0	0.3	4.8	Unknown	Unknown	0.32	0.70	1.60	2.80	2.2	0.5	0.3
472	49	VA	1/12/1993	10:30	LEFT	Upstream	Single	Round	0.9	10.7	0	0.3	5.0	Unknown	Unknown	0.32	0.70	1.60	2.80	2.2	0.9	0.3
473	49	VA	4/12/1993	12:00	LEFT	Upstream	Single	Round	0.9	10.7	0	0.5	5.5	Unknown	Unknown	0.32	0.70	1.60	2.80	2.2	0.5	0.3
474	49	VA	4/13/1993	9:30	LEFT	Upstream	Single	Round	0.9	10.7	0	0.4	5.6	Unknown	Unknown	0.32	0.70	1.60	2.80	2.2	0.6	0.3
475	49	VA	4/19/1993	10:00	LEFT	Upstream	Single	Round	0.9	10.7	0	0.5	5.8	Unknown	Unknown	0.32	0.70	1.60	2.80	2.2	0.7	0.3
476	49	VA	5/31/1990	13:30	RT	Upstream	Single	Round	0.9	10.7	0	0.6	8.0	Unknown	Unknown	0.32	0.70	1.60	2.80	2.2	0.9	0.3
477	49	VA	12/13/1992	11:30	RT	Upstream	Single	Round	0.9	10.7	0	0.4	5.9	Unknown	Unknown	0.32	0.70	1.60	2.80	2.2	0.5	0.3
478	49	VA	12/14/1992	10:00	RT	Upstream	Single	Round	0.9	10.7	0	0.4	6.1	Unknown	Unknown	0.32	0.70	1.60	2.80	2.2	0.6	0.3
479	49	VA	12/15/1992	11:00	RT	Upstream	Single	Round	0.9	10.7	0	0.2	5.7	Unknown	Unknown	0.32	0.70	1.60	2.80	2.2	0.4	0.3
480	49	VA	1/11/1993	12:00	RT	Upstream	Single	Round	0.9	10.7	0	0.3	5.6	Unknown	Unknown	0.32	0.70	1.60	2.80	2.2	0.5	0.3
481	49	VA	1/12/1993	10:30	RT	Upstream	Single	Round	0.9	10.7	0	0.3	5.9	Unknown	Unknown	0.32	0.70	1.60	2.80	2.2	0.4	0.3
482	49	VA	4/12/1993	12:00	RT	Upstream	Single	Round	0.9	10.7	0	0.4	6.3	Unknown	Unknown	0.32	0.70	1.60	2.80	2.2	0.5	0.3
483	49	VA	4/13/1993	9:30	RT	Upstream	Single	Round	0.9	10.7	0	0.3	6.4	Unknown	Unknown	0.32	0.70	1.60	2.80	2.2	0.5	0.3
484	49	VA	4/19/1993	10:00	RT	Upstream	Single	Round	0.9	10.7	0	0.3	7.1	Unknown	Unknown	0.32	0.70	1.60	2.80	2.2	0.9	0.3
485	55	VA	3/29/1991	14:30	2	Upstream	Single	Round	0.6	9.1	0	1.1	0.8	Unknown	Noncohesive	38.00	55.00	84.00	130.00	1.5	0.5	0.3
486	55	VA	6/5/1992	10:30	2	Upstream	Single	Round	0.6	9.1	0	1.7	3.2	Unknown	Noncohesive	38.00	55.00	84.00	130.00	1.5	0.6	0.3
487	55	VA	3/24/1993	9:30	2	Upstream	Single	Round	0.6	9.1	0	2.0	3.2	Unknown	Noncohesive	38.00	55.00	84.00	130.00	1.5	0.5	0.3
488	53	VA	5/3/1989	12:00	2	Upstream	Single	Round	0.6	12.5	0	0.6	0.5	Unknown	Noncohesive	33.00	72.00	170.00	250.00	2.3	0.2	0.3
489	53	VA	5/7/1989	9:00	2	Upstream	Single	Round	0.6	12.5	0	1.6	0.7	Unknown	Noncohesive	33.00	72.00	170.00	250.00	2.3	0.2	0.3
490	53	VA	4/22/1992	11:00	2	Upstream	Single	Round	0.6	12.5	0	1.6	1.7	Unknown	Noncohesive	33.00	72.00	170.00	250.00	2.3	0.5	0.3

Table 24. Pier scour observations, continued.

Mea-surement	Site	State	Date	Time	Pier	Upstream/ Downstream	Pier Type	Pier Shape	Pier Width (m)	Pier Length (m)	Skew (deg)	Velocity (m/s)	Depth (m)	Debris Effects	Bed Material	D16 (mm)	D50 (mm)	D84 (mm)	D95 (mm)	Grad-ation	Scour Depth (m)	Ac-curacy (m)
491	53	VA	5/3/1989	12:00	3	Upstream	Single	Round	0.6	12.5	0	1.2	1.2	Unknown	Noncohesive	33.00	72.00	170.00	250.00	2.3	0.3	0.3
492	53	VA	5/7/1989	9:00	3	Upstream	Single	Round	0.6	12.5	0	1.6	1.5	Unknown	Noncohesive	33.00	72.00	170.00	250.00	2.3	0.4	0.3
493	53	VA	4/22/1992	11:00	3	Upstream	Single	Round	0.6	12.5	0	2.6	2.6	Unknown	Noncohesive	33.00	72.00	170.00	250.00	2.3	0.8	0.3

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