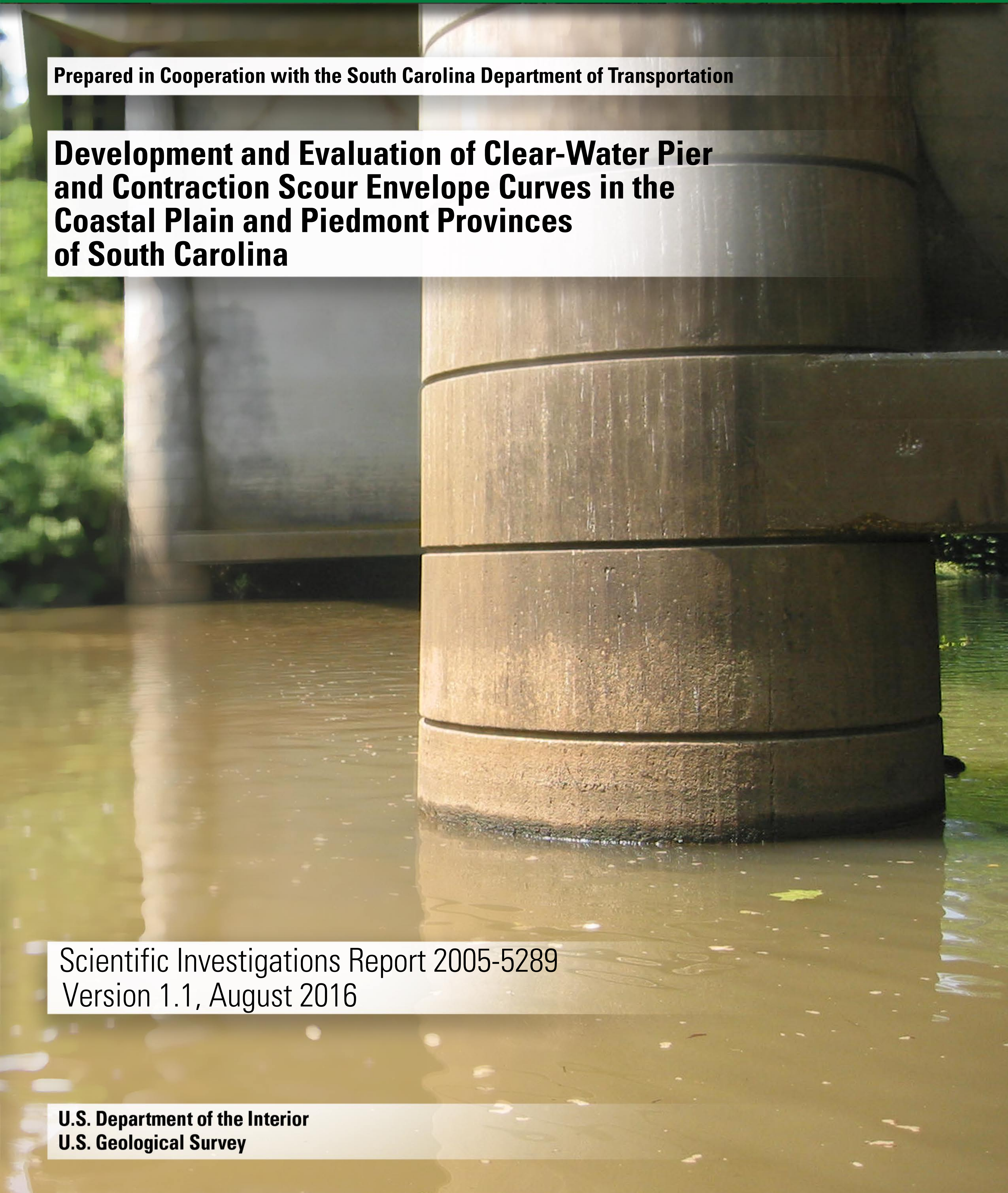
Prepared in Cooperation with the South Carolina Department of Transportation

Development and Evaluation of Clear-Water Pier and Contraction Scour Envelope Curves in the Coastal Plain and Piedmont Provinces of South Carolina



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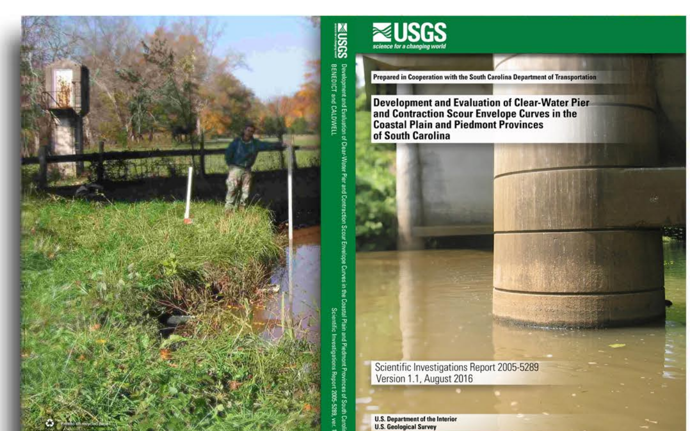

Cover Photos:
Clear-water pier scour at structure 304004900400 at S.C. Route 49, crossing the Enoree River in Laurens County, South Carolina.

(U.S. Geological Survey, South Carolina Water Science Center, November 13, 2002)

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By Stephen T. Benedict and Andral W. Caldwell

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Contents

Figures

Tables

CONVERSION FACTORS, TEMPERATURE, DATUMS, AND ABBREVIATIONS and ACRONYMS

Multiply	By	To obtain
	Length	
inch (in.)	25.4	millimeter (mm)
foot (ft)	0.3048	meter (m)
mile (mi)	1.609	kilometer (km)
	Area	
square mile (mi^2)	2.590	square kilometer (km^2)
	Volume	
cubic foot (ft^3)	0.02832	cubic meter (m^3)
	Flow rate	
foot per foot (ft/ft)	0.02832	meter per meter (m/m)
square foot per second (ft^2/s)	0.02832	square meter per second (m^2/s)
cubic foot per second (ft^3/s)	0.02832	cubic meter per second (m^3/s)
foot per second (ft/s)	0.02832	meter per second (m/s)

Equations for temperature conversion between degrees Celsius (°C) and degrees Fahrenheit (°F):

$$°C = (°F - 32) / 1.8$$
$$°F = (1.8 \times °C) + 32$$

Horizontal coordinate information (latitude and longitude) is referenced to the North American Datum of 1983 (NAD 83). Vertical coordinate information is referenced to the North American Vertical Datum of 1988 (NAVD 88).

Abbreviations and Acronyms:

CSU	Colorado State University
DVD	digital video disc
FHWA	Federal Highway Administration
HEC-18	Hydraulic Engineering Circular 18
NBSD	National Bridge Scour Database
PVC	polyvinyl chloride
SCDOT	South Carolina Department of Transportation
SCPCSD	South Carolina Clear-Water Pier- and Contraction-Scour Database
USGS	U.S. Geological Survey
WSPRO	water-surface profile model
mm	millimeter
<	less than
≤	less than or equal to
>	greater than
≥	greater than or equal to

Development and Evaluation of Clear-Water Pier and Contraction Scour Envelope Curves in the Coastal Plain and Piedmont Provinces of South Carolina

By Stephen T. Benedict and Andral W. Caldwell

Abstract

The U.S. Geological Survey in cooperation with the South Carolina Department of Transportation collected clear-water pier- and contraction-scour data at 116 bridges in the Coastal Plain and Piedmont Physiographic Provinces of South Carolina. Pier-scour depths collected in both provinces ranged from 0 to 8.0 feet. Contraction-scour depths collected in the Coastal Plain ranged from 0 to 3.9 feet. Using hydraulic data estimated with a one-dimensional flow model, predicted clear-water scour depths were computed with scour equations from the Federal Highway Administration Hydraulic Engineering Circular 18 and compared with measured scour. This comparison indicated that predicted clear-water scour depths, in general, exceeded measured scour depths and at times were excessive. Predicted clear-water contraction scour, however, was underpredicted approximately 30 percent of the time by as much as 7.1 feet.

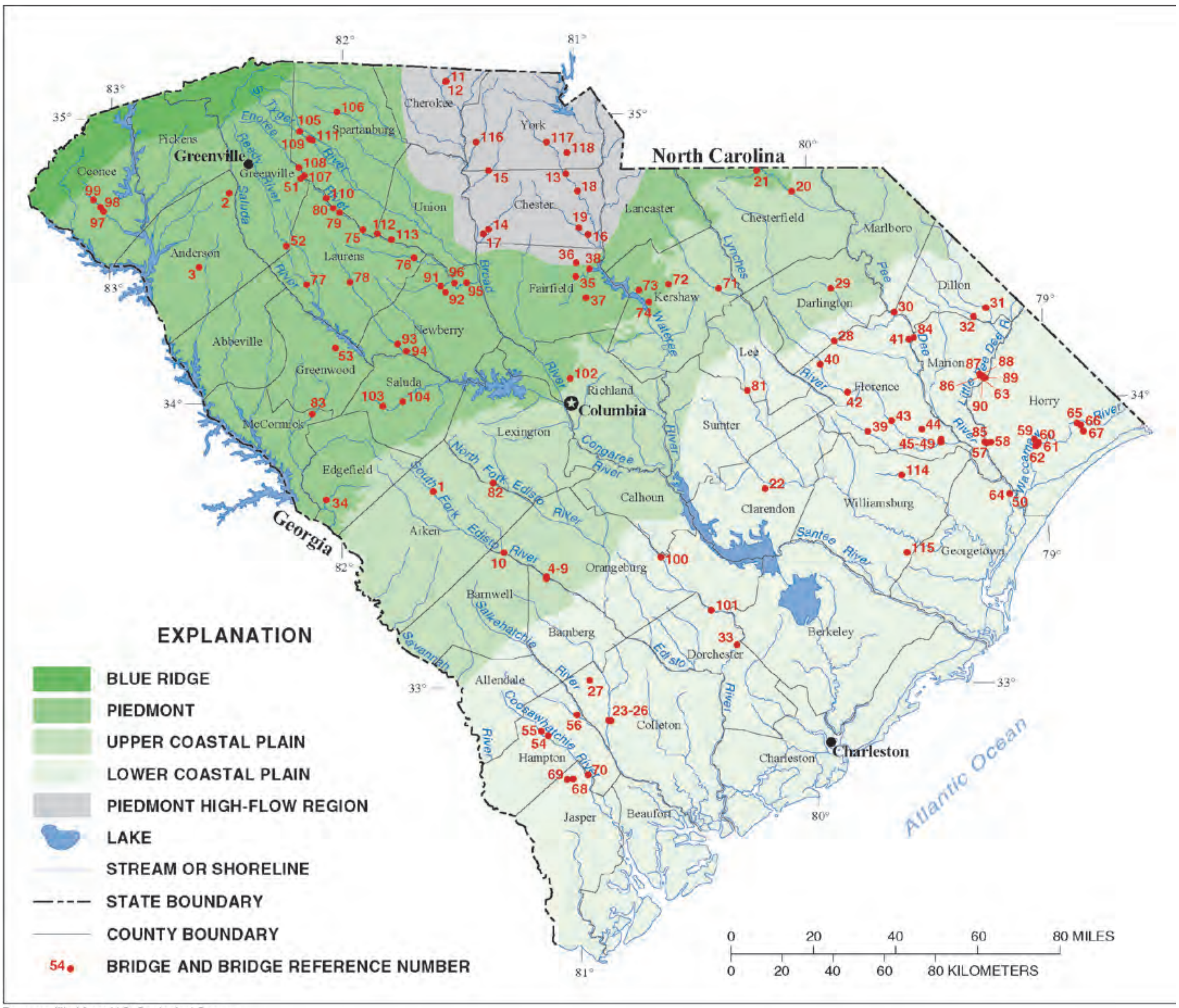
The investigation focused on clear-water pier scour, comparing trends in the laboratory and field data. This comparison indicated that the range of dimensionless variables (relative depth, flow intensity, relative grain size) used in laboratory investigations of pier scour, were similar to the range for field data in South Carolina, further indicating that laboratory relations may have some applicability to field conditions in South Carolina. Variables determined to be important in developing pier scour in laboratory studies were investigated to understand their influence on the South Carolina field data, and many of these variables appeared to be insignificant under field conditions in South Carolina. The strongest explanatory variables were pier width and approach velocity. Envelope curves developed from the field data are useful tools for evaluating reasonable ranges of clear-water pier and contraction scour in South Carolina. A modified version of the Hydraulic Engineering Circular 18 pier-scour equation also was developed as a tool for evaluating clear-water pier scour. The envelope curves and modified equation offer an improvement over the current methods for predicting clear-water scour in South Carolina.

Data from this study were compiled into a database that includes photographs, measured scour depths, predicted scour depths, limited basin characteristics, limited soil data, and modeled hydraulic data. The South Carolina database can be used to compare studied sites with unstudied sites to evaluate the potential for scour at the unstudied sites. In addition, the database can be used to evaluate the performance of various methods for predicting clear-water pier and contraction scour.

Introduction

In 1996, the U.S. Geological Survey (USGS) in cooperation with the South Carolina Department of Transportation (SCDOT) initiated a study to investigate clear-water abutment and contraction scour in the Coastal Plain and Piedmont Physiographic Provinces of South Carolina (Benedict, 2003). (These regions in South Carolina will hereafter in the report be referred to as the Coastal Plain and Piedmont.) Clear-water abutment scour was investigated in both provinces. The investigation of clear-water contraction scour was limited to the Piedmont. The study included the collection of field data at 144 bridges, limited comparisons of predicted and measured scour depths, and development of envelope curves as supplementary tools for evaluating clear-water abutment and contraction scour in South Carolina. Comparisons of predicted and measured scour indicated that the laboratory-derived equations performed poorly. Overpredictions often were excessive, as large as 26.3 feet (ft), and underpredictions as large as 6.7 ft occasionally occurred. The field data envelope curves developed in the 1996 study (Benedict, 2003) represent an upper bound of historic scour, providing some indicator of scour depths that may occur at sites with similar characteristics. Although the envelope curves have limitations in their application, they provide a useful supplementary tool to evaluate anticipated ranges of clear-water abutment and contraction-scour depths in South Carolina.

Based on the findings of the 1996 investigation (Benedict, 2003), it became clear that a similar approach could be used to develop envelope curves for other components of bridge scour, including clear-water pier scour, live-bed pier scour, live-bed contraction scour, and clear-water contraction scour outside of the Piedmont. These envelope curves, in conjunction with the previously developed envelope curves (Benedict, 2003), could provide supplementary tools for the full evaluation of scour at bridges in South Carolina. Realizing the benefits that can be derived from such tools, the USGS in cooperation with the SCDOT initiated a study in October 2002 to investigate clear-water pier scour in the Coastal Plain and Piedmont and clear-water contraction scour in the Coastal Plain. (Live-bed scour is not addressed in this investigation.) Because clear-water pier and contraction scour infrequently occurs in the main channel of South Carolina streams, this investigation focused on clear-water pier and contraction scour on the overbanks (also called the floodplain) of Coastal Plain and Piedmont bridges (fig. 1). The general objectives of this study were to (1) collect field observations of clear-water pier and contraction scour, (2) use the data to evaluate the methods in Hydraulic Engineering Circular 18 (HEC-18; Richardson and Davis, 2001) for predicting scour, and (3) if possible, develop envelope curves to help evaluate these components of scour in South Carolina. The scope of the investigation was limited to clear-water pier and contraction scour in the sandy soils of the Coastal Plain and clear-water pier scour in the cohesive overbank soils of Piedmont streams. These regions (fig. 1) are characterized by thick floodplain vegetation that promotes conditions for clear-water scour. The contrast of soil types



Base modified from U.S. Geological Survey digital data, 1:2,000,000 scale, 1972

Figure 1. Locations of physiographic provinces and bridge-scour study sites in South Carolina. (Refer to appendix 2 at back of report to identify bridge with corresponding number.)

between the regions provides valuable insights into the scour processes for South Carolina streams.

Because field data for bridge scour are limited, scour trends observed in the South Carolina data can help other States understand anticipated scour trends within their own boundaries. The scour trends in South Carolina likely will be most applicable to States with similar regional characteristics to South Carolina. However, states with differing regional characteristics can gain some insights regarding anticipated scour trends within their boundaries and, if desired, can use the approach in this investigation to develop regional bridge-scour envelope curves for their own States.

Purpose and Scope

The purpose of this report is to describe (1) techniques used to collect clear-water pier- and contraction-scour data at 116 bridges in the Coastal Plain and Piedmont of South Carolina, (2) a limited comparison of predicted clear-water pier- and contraction-scour depths to measured scour depths, (3) selected relations in the field data, and (4) envelope curves that can be used to estimate ranges of anticipated clear-water pier and contraction scour at bridges in the Coastal Plain and Piedmont of South Carolina. In addition, a compilation of the data developed for each bridge is available at https://pubs.er.usgs.gov/publication/sir20055289. This compilation includes photographs, measured scour depths, predicted scour depths, limited basin characteris-tics, limited soil data, and modeled hydraulic data, which can be viewed using Microsoft Access.

Previous Investigations

The USGS in cooperation with the SCDOT investigated scour in South Carolina in three previous studies. In the first investigation of level-1 bridge scour (1990–92), limited structural, hydraulic, geomorphic, and vegetative data were collected at 3,506 bridges and culverts in South Carolina, and observed- and potential-scour indexes were developed for each site (Hurley, 1996). These indexes, along with other variables, were used by the SCDOT to select sites in need of additional bridge-scour investigation.

In the second cooperative investigation of level-2 bridge scour (1992–95), detailed bridge-scour studies of 293 bridges in South Carolina were conducted using methods presented in HEC-18 (Richardson and others, 1991, 1993). Predicted scour depths determined in these studies were compared to bridge foundations to provide an indicator of the vulnerability of the bridges to failure. This information was used by the SCDOT to assist in determining if additional studies and(or) remedial actions were required to protect bridges from the threat of scour.

The level-1 and level-2 bridge-scour studies gave a qualitative overview of scour, which helped form general concepts of the type, magnitude, and frequency of scour throughout the State. In addition, the level-2 bridge-scour studies provided evidence of the apparent discrepancy between the predicted and measured scour. This information was helpful in developing the approach for the third cooperative investigation, which was of clear-water abutment and contraction scour (1996–99; Benedict, 2003). In the third investigation, field data were collected at 144 bridges, limited comparisons were made of predicted and measured scour depths, and field-data envelope curves were developed for evaluating clear-water abutment and contraction scour in South Carolina. The assumptions and techniques used in the third investigation were, in large measure, the guiding principles for the current investigation of clear-water pier and contraction scour.

Description of Study Area

South Carolina has an area of about 31,100 square miles (mi^2) and is divided into three physiographic provinces—the Blue Ridge, Piedmont, and Coastal Plain. The Coastal Plain is divided into upper and lower regions (fig. 1). The study area includes most of South Carolina but generally excludes the Blue Ridge and the tidally influenced area of the lower Coastal Plain.

The Piedmont covers approximately 35 percent of South Carolina and lies between the Blue Ridge and Coastal Plain (fig. 1). Land-surface elevations range from about 400 ft near the Fall Line (Coastal Plain boundary) to approximately 1,000 ft at the Blue Ridge boundary. The general topography includes rolling hills, elongated ridges, and moderately deep to shallow valleys. The drainage patterns are well developed with well-defined channels and densely vegetated floodplains. Stream slopes in the Piedmont range from approximately 0.00015 to 0.0100 foot per foot (ft/ft).

The geology of the Piedmont consists of fractured crystalline rock overlain by moderately to poorly permeable silty-clay loams. Alluvial deposits along the valley floors consist of clay, silt, and sand, and form varying degrees of cohesive soils (Guimaraes and Bohman, 1992). The cohesive soils typically found in Piedmont floodplains provide some resistance to scour that can reduce scour depths in this region. The thick floodplain vegetation significantly impedes sediment transport, promoting clear-water scour conditions in the floodplain.

In this study, 53 bridges in the Piedmont were surveyed for clear-water pier scour. Limited data indicate that peak flows are higher in the northeastern region of the Piedmont than in the western region (Guimaraes and Bohman, 1992; Feaster and Tasker, 2002). This area is designated as the Piedmont high-flow region (fig. 1), and 15 of the 53 Piedmont sites are located in this region. (Two sites are located just outside of the high-flow region, but because the basins are within the region, these two sites were considered as being within the Piedmont high-flow region.) Stream slopes and drainage areas for the 53 sites range from 0.00015 to 0.00290 ft/ft (fig. 2) and 10.7 to 1,620 mi^2 (fig. 3), respectively.

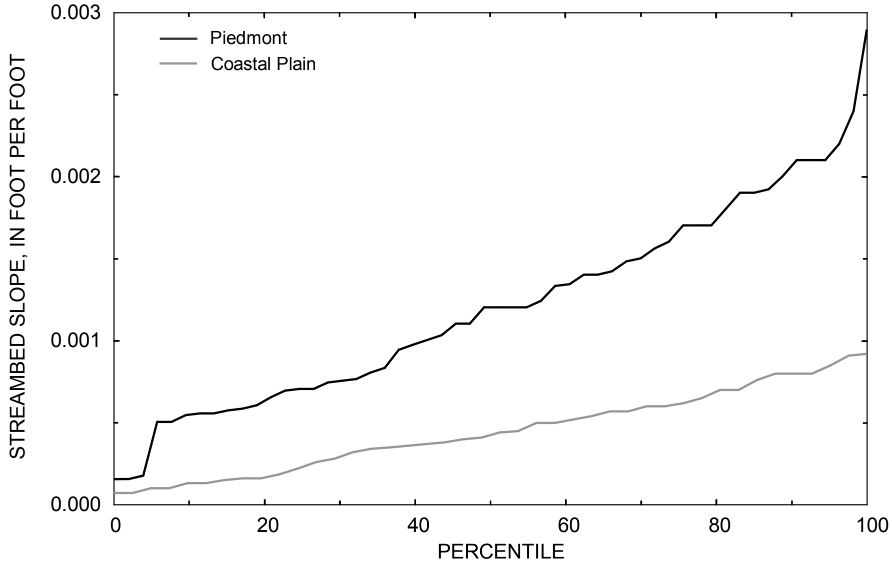


Figure 2. Distribution of streambed slopes for selected bridges in the Coastal Plain and Piedmont Physiographic Provinces of South Carolina.

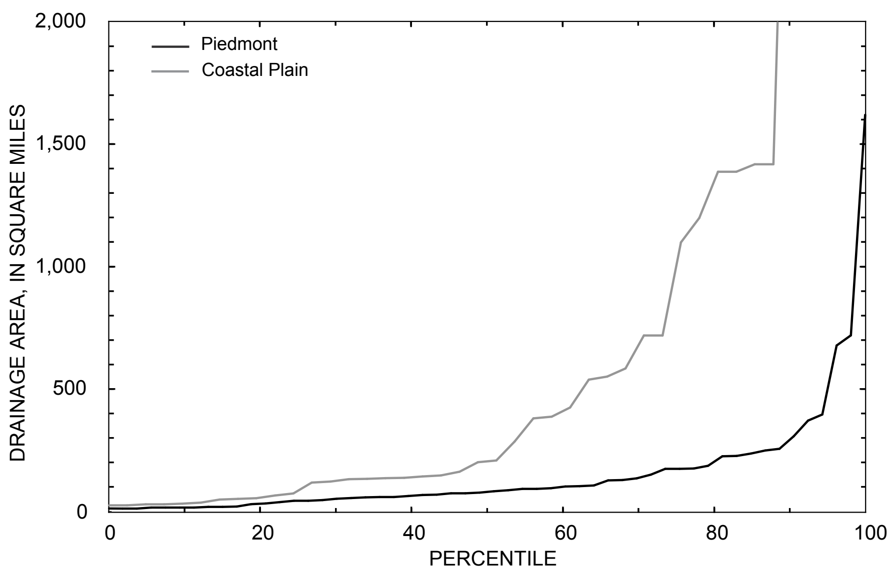


Figure 3. Distribution of drainage areas for selected bridges in the Coastal Plain and Piedmont Physiographic Provinces of South Carolina. (Note: Vertical scale has been truncated for graph clarity at small drainage areas.)

The upper Coastal Plain is bounded by the Piedmont and lower Coastal Plain, and covers approximately 20 percent of the State (fig. 1). The general topography in the upper Coastal Plain consists of rounded hills with gradual slopes, and land-surface elevations that range from less than 200 ft to more than 700 ft. The geology consists primarily of sedimentary rocks composed of layers of sand, silt, clay, and gravel underlain by igneous rocks (Zalants, 1990). A shallow surface layer of permeable sandy soils is common. Low-flow channels bounded by densely vegetated floodplains characterize upper Coastal Plain streams. Stream slopes are moderate, ranging from approximately 0.0005 to 0.0040 ft/ft (Guimaraes and Bohman, 1992). In this study, 12 bridges in the upper Coastal Plain were surveyed for clear-water contraction and pier scour.

The lower Coastal Plain covers about 43 percent of the State (fig. 1). The topographic relief in the lower Coastal Plain is less pronounced than that of the upper Coastal Plain, and land-surface elevations range from 0 ft at the coast to nearly 200 ft at the boundary with the upper Coastal Plain. The geology of the lower Coastal Plain consists of loosely consolidated sedimentary rocks of sand, silt, clay, and gravel overlain by permeable sandy soils (Zalants, 1991). Stream slopes range from approximately 0.0001 to 0.0040 ft/ft, and streamflow patterns are tidally influenced near the coast (Guimaraes and Bohman, 1992).

Although large rivers flow within well-defined channels in the lower Coastal Plain, the region is noted for its numerous swamps, which have wide, densely vegetated floodplains that are drained by a network of shallow, poorly defined channels. Because of the thick vegetation, the shallow channels have large root masses at or just below the ground surface. These root masses significantly impede the transport of streambed sediments, thereby promoting clear-water scour conditions at bridge contractions. In this study, 51 bridges in the lower Coastal Plain were surveyed for clear-water contraction and pier scour; 43 of these sites are in swamps. Stream slopes and drainage areas for the 63 sites in the upper and lower Coastal Plain range from 0.00007 to 0.00092 ft/ft (fig. 2) and 26.3 to 13,000 mi^2 (fig. 3), respectively.

Approach

Laboratory investigations of bridge scour have frequently used envelope curves to display the trends of scour and to develop tools for evaluating the potential for scour (Breusers and others, 1977; Dongol, 1993; Melville and Coleman, 2000). With the current use of micro computers to model complex physical phenomenon, the use of envelope curves for evaluating bridge scour seems too simplistic and somewhat archaic. However, the use of simple envelope curves, in large measure, stems from the limited understanding of the complex mechanisms that create scour. The following quotes from selected researchers highlight this fact. Describing the findings of an extensive literature review of pier scour, Breusers and others (1977) state that it is

> *. . . clear, as in many other fields of sediment transport, up to now no entirely satisfactory theoretical and experimental results have been obtained, because the processes involved of water and sediment movement are too complicated and experimental data are incomplete and sometimes conflicting.*

Melville and Coleman (2000) in their extensive summary of the state-of-the-knowledge and practice of bridge scour note

> *The theoretical basis for the structural design of bridges is well established. In contrast, the mechanism of flow and erosion in mobile-boundary channels has not been well defined and it is not possible to estimate with confidence the river boundary changes that may occur at a bridge subject to a given flood. This is not only due to the extreme complexity of the problem, but also to the fact that river characteristics, bridge constriction geometry, and soil and water interaction are different for each bridge as well as for each flood.*

The limited understanding of the “extreme complexity” associated with bridge scour has necessitated the use of envelope curves for defining scour trends in laboratory investigations and is a practice that likely will be associated with this discipline for years to come. Although envelope curves of laboratory data cannot provide a refined estimate of bridge scour, they are useful tools in helping the practitioner understand the upper-bound trends of scour for various conditions. There are, however, known problems associated with small-scale laboratory investigations of bridge scour, including over simplification of site conditions within the laboratory and scaling issues, both of which may lead to unreasonable estimates of scour when scaled to the field (Ettema and others, 1998).

One approach to minimizing these problems is to use field data, rather than laboratory data, to define bridge scour envelopes. Field envelope curves can avoid the problems associated with small-scale laboratory investigations and provide the practitioner with a better understanding of scour trends within the field setting. This is the approach that the current investigation takes to develop scour-evaluating tools for clear-water pier scour and contraction scour in South Carolina. Numerous field observations of clear-water pier and contraction scour data were collected in the Coastal Plain and Piedmont of South Carolina, and dominant explanatory variables were utilized to develop envelope curves to define the upper bound of scour. These envelope curves can be used to help evaluate the potential for bridge scour in these regions of South Carolina.

Data Collection

When using field envelope curves to evaluate scour potential, it is important to understand the limitations of the data used to develop the envelope curve. The following sections describe assumptions regarding the data, criteria for site selection, and techniques for collecting and interpreting the field data.

Data Assumptions

In the previous investigation (Benedict, 2003), various assumptions were made about the collected scour data. These primarily included the assumption of clear-water scour conditions and the assumption that the collected field data encompassed scour resulting from large flows. Many of the bridges used in this investigation (92 of 116 sites) are the same bridges used in the previous investigation of contraction and abutment scour (Benedict, 2003). Therefore, assumptions in the previous investigation and justification of those assumptions should be applicable to this investigation. An overview of the assumptions follows. Additional details can be found in Benedict (2003).

Clear-Water Scour Conditions

As in the previous investigation (Benedict, 2003), data collection in the current study focused on clear-water bridge scour in contrast to live-bed scour. Live-bed scour occurs at a bridge when the approaching flow velocity exceeds the critical velocity for eroding sediments of a given size, thus transporting sediments along the streambed and into the area of scour. Because sediments are being transported into the area of scour, scour holes partially or totally refill with sediments as flood flows recede, making it difficult to measure scour depths during low-flow and post-flood conditions. In contrast, under clear-water scour conditions, approaching flow velocities do not exceed the critical velocity, and sediments are not transported into the area of scour. Therefore, scour holes developed under clear-water scour conditions are not refilled, and an unobscured record of the maximum scour depth is preserved at the bridge. This record can be readily measured during low-flow and post-flood investigations, and the measured scour represents the maximum clear-water scour that has occurred during the life of the bridge. This assumes that the scoured region at a selected bridge has not been disturbed by bridge repair or maintenance. (A questionnaire was sent to SCDOT maintenance engineers to determine if the regions of scour could have been disturbed by past repairs or maintenance. Based on questionnaire responses, it was concluded that most bridges had not been disturbed.) Because of the relative ease of measuring clear-water scour in contrast to live-bed scour, the focus of this study was on the collection of clear-water scour data. The term “clear-water scour” can be misleading, because it implies that flows must literally be “clear” of any sediments to be classified as clear-water scour conditions. However, under clear-water scour conditions, it is common to have clays and silts suspended in flood flows even though flow velocities are insufficient to move sediments along the streambed. Therefore, it is important to keep in mind the difference between the transport of suspended sediments, which can exist under clear-water scour conditions, and the transport of sediments along the streambed, which distinguishes live-bed scour from clear-water scour. Because clear-water pier and contraction scour infrequently occur in the main channel of South Carolina streams, this investigation focused on clear-water pier and contraction scour on the overbanks (also called the floodplain) of Coastal Plain and Piedmont bridges.

The assumption that bridge-scour data collected in this study is clear-water scour can be justified in several ways. Clear-water scour occurs when the approaching flow velocities are insufficient to move streambed sediments into the area of scour or when site conditions, such as streambed armoring or vegetation, impede the transport of sediments along the bed. For the floodplains of South Carolina streams, both of these conditions frequently occur. The floodplains of South Carolina typically are covered by thick vegetation that impedes the transport of sediments along the streambed, promoting clear-water scour conditions. Under these conditions, even if the velocity of the floodplain flow exceeds the critical velocity of the bed material, sediment transport along the streambed is negligible. (Clear-water scour conditions created by vegetated floodplains is a phenomenon acknowledged by other investigators (Laursen, 1963; Richardson and Davis, 2001)). In addition to the impediment of sediment transport by vegetation, floodplain velocities in South Carolina often are sufficiently low to create clear-water scour conditions. This can be illustrated using hydraulic data generated from a one-dimensional flow model for the 116 bridges in this investigation. For the 63 bridges in the Coastal Plain, the average unconstricted velocity in the approach floodplain for the 100-year flow ranged from approximately 0.1 to 1.1 feet per second (ft/s) with a mean value of 0.4 ft/s. The percentile plot for the ratio of the approaching floodplain velocity to the critical velocity of the median grain size for the Coastal Plain bridges indicates that 96 percent of the bridges, theoretically, should have clear-water scour conditions in the floodplain (fig. 4). (Critical velocity was estimated with the equation presented in HEC-18 (Richardson and Davis, 2001)).

For the 53 bridges in the Piedmont, the average unconstricted velocity in the approach floodplain for the

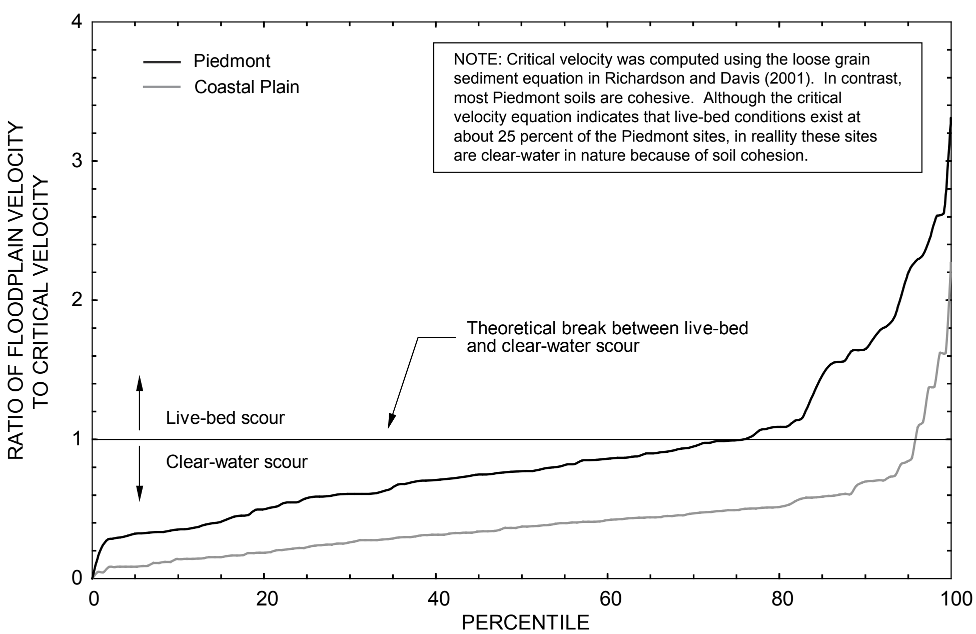


Figure 4. Distribution of the ratio of the average floodplain velocity to the critical velocity of the median grain size for selected bridges in the Coastal Plain and Piedmont Physiographic Provinces of South Carolina.

100-year flow ranged from approximately 0.1 to 4.0 ft/s with a mean value of 1.0 ft/s. The percentile plot for the ratio of the approaching floodplain velocity to the critical velocity of the median grain size for the Piedmont bridges indicates that 75 percent of the bridges, theoretically, should have clear-water scour conditions in the floodplain (fig. 4). The critical velocity equation from HEC-18 (Richardson and Davis, 2001) used to develop this plot was derived for noncohesive sediments and will not accurately represent the critical velocities for the cohesive soils commonly found in the Piedmont. Although the graph in figure 4 indicates that live-bed conditions exist at about 25 percent of the Piedmont bridges, in reality these sites are clear-water in nature because of soil cohesion.

A final justification for clear-water scour conditions can be based on the limited amount of measured infill in the overbank contraction-scour holes. Percentile plots for measured infill for clear-water contraction-scour holes in the Piedmont and Coastal Plain of South Carolina are shown in figure 5. For the 75 previous observations of clear-water contraction scour on the overbanks of Piedmont streams (Benedict, 2003), the measured infill ranged from 0 to 0.7 ft with a median of 0 ft and a mean of 0.05 ft. For the 64 observations of clear-water contraction scour on the overbanks of the Coastal Plain streams, the measured infill ranged from 0 to 1.1 ft with a median of 0 ft and a mean of 0.2 ft. The median and mean values indicate that infill for overbank contraction scour in the Piedmont and Coastal Plain typically was small and often did not exist. Because clear-water scour theoretically does not have any infill, this trend indicates that the overbank areas of bridges in South Carolina are clear-water in nature and that infill, generally, is minimal. (Interestingly, the Piedmont region that has larger flow velocities and, thus, larger potential for sediment transport has less frequent infill than the Coastal Plain. This can be attributed to the clayey soils of the Piedmont region that greatly impede sediment transport along the streambed.)

Because measured scour in the current and previous (Benedict, 2003) investigations is classified as clear-water scour and theoretically should have no infill, the question must be asked as to why infill was observed at some sites. There are two probable causes that could account for the observed infill, including (1) limited sediment transport into the scoured area and (2) human error in discerning and measuring infill. In the preceding paragraphs, it was shown that the clear-water scour classification of the study sites in the previous (Benedict, 2003) and current investigations was justifiable based on the heavily vegetated floodplains and the small flow velocities. Although approach streamflow velocities (beyond the influence of the bridge) may be sufficiently small to create clear-water scour conditions, the flow contracts as it nears the bridge and the increased velocity of the approaching flow in close proximity to the bridge may be sufficient to transport small amounts of sediment bed loads into the bridge. This, in some measure, could account for small amounts of infill. Regardless of the minor amounts of infill occasionally found at some sites, it is still appropriate to classify these sites as clear-water in nature.

Error associated with measuring infill also may account for observed infill at some sites. Estimating sediment infill in scour holes is a subjective and often difficult task in the field setting. For shallow scour holes that are not submerged, visual inspection of the scoured area can be made to evaluate sediment infill. In the case of submerged scour holes, probing with a rod and retrieving small sediment cores must be done to evaluate the infill. At some sites, the amount of infill can be determined readily. However, at other sites the distinction between infill and unscoured material is unclear, making the measurement difficult. In such cases, field investigators typically make a conservative estimate of the infill (larger than what may have actually occurred) to assure that the scour depth overestimates rather than underestimates the maximum scour at the site. Therefore, infill estimates at such sites may overestimate the true infill, and it is probable that the estimated infill depths at certain sites in the current and previous investigations are too high.

The data in figures 4 and 5, in conjunction with the thick floodplain vegetation and the cohesive Piedmont soils, indicate that clear-water scour

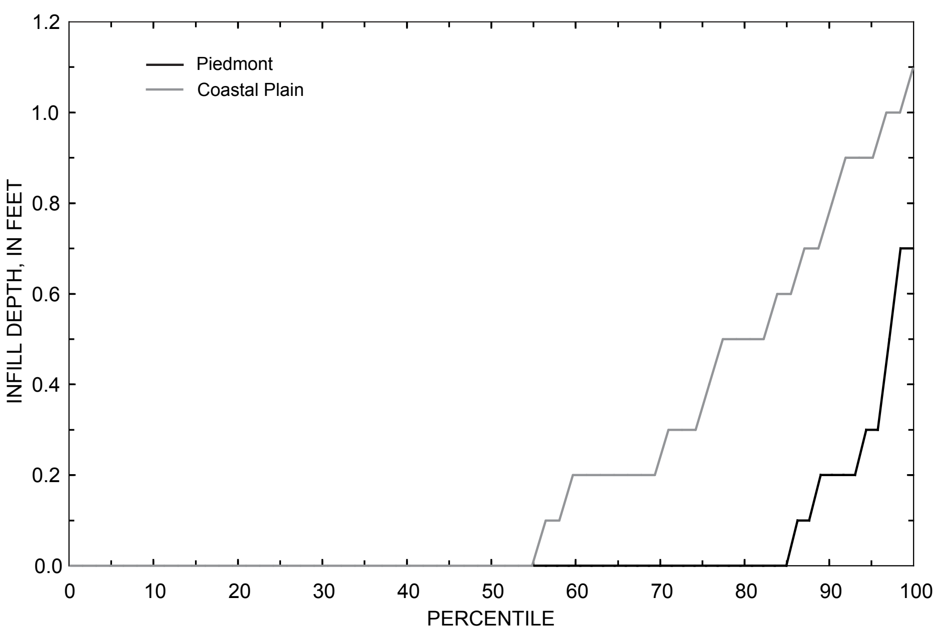


Figure 5. Distribution of sediment infill depth measured at clear-water contraction scour holes for selected bridges in the Coastal Plain and Piedmont Physiographic Provinces of South Carolina.

conditions prevail in the floodplains of the 116 bridges used in the current investigation. Therefore, it is reasonable to assume that pier- and contraction-scour data collected in the floodplains of these sites will represent scour resulting from clear-water scour conditions.

Assumption of Large Floods

As demonstrated in the previous investigation (Benedict, 2003), when sufficient scour data are collected at a large number of bridges, the data can be used to develop envelope curves for evaluating ranges of anticipated scour depths for given site conditions. For example, if collected data for clear-water pier-scour depths range from 0.0 to 2.0 ft for 1-ft-wide piles in the clayey soils of the Piedmont, it would be reasonable to assume that an upper limit for scour depth under such conditions would be approximately 2.0 ft. When using measured scour data in such a manner, it must be assumed that the collected field data represent scour resulting from floods, such as those approaching the 100-year flood-flow magnitude. If the collected field data represent scour that has resulted only from minor floods, then the data cannot be used to evaluate scour resulting from large floods. However, if the measured data represent scour resulting from large floods, it is reasonable to use such data to evaluate the scour potential at other bridges with similar site characteristics.

The assumption that pier- and contraction-scour data collected in this investigation represent scour resulting from large flows is critical. The previous investigation (Benedict, 2003) justified this assumption by demonstrating from risk analysis, streamgage records, and historic flood records that approximately 80 percent of 144 bridges likely had flows equal to or exceeding 70 percent of the 100-year flow. Because the current investigation uses 92 of the bridges from the previous investigation, it is reasonable to assume that the justification for the assumption of large flows is applicable. A summary of these assumptions and how they apply to the bridges in the current investigation will be presented. For more details on these assumptions refer to Benedict (2003).

Benedict (2003) defines a large flow as any flow equal to or exceeding 70 percent of the 100-year flow. Although this definition is arbitrary, it was chosen, in part, because 70 percent of the 100-year rural flow, as determined by the South Carolina flood-frequency regression equations (Guimaraes and Bohman, 1992), is approximately equal to the 25-year rural flow. If 70 percent of the 100-year rural flow is assumed equal to the 25-year rural flow, then statistical analysis, with respect to exceedance probability (also called risk analysis), can be made. Using risk analysis, Benedict (2003) demonstrated that bridges 30 years or older have a high probability (71 percent) of having flows equal to or exceeding the 25-year flow. In the current investigation, 75 of 116 bridges were 30 years or older in 2002 (fig. 6), indicating that large flows likely had occurred at these bridges. In addition, 18 of the 41 bridges under 30 years of age are known to have had flows exceeding the 25-year recurrence interval (9 of which equaled or exceeded the 100-year flow). The risk analysis, in conjunction with known maximum historic flows, indicates that large flows likely have occurred at approximately 80 percent of the bridges in this investigation, giving support to the assumption that a significant portion of the scour data collected in the investigation will represent scour resulting from large flows.

The assumption of large flows also can be substantiated with streamgage data. A review of all streamgage records in South Carolina for the period 1976–2002 identified 50 streamgages, of which 6 are indirect flow-measurement sites (table 1), having flows equal to or exceeding approximately 70 percent of the 100-year flow. (The period of record 1976–2002 was selected because clear-water scour data collection began in late 2002 and approximately 85 percent of the bridges were in place in 1976, indicating that major floods occurring during 1976–2002 may have influenced those bridges.) Included in the 50 streamgages are 35 gages influenced by the major floods of October 1990, October 1992, August 1995, and September 1999. These floods are described in Benedict (2003), and the locations of the streamgages affected by these floods are shown in figure 7. The ratio of peak flow to the 100-year flow for the 50 streamgages ranges from 0.66 to 3.3 with a median ratio of 0.9 (table 1). This indicates that a significant number of large floods have occurred throughout the State since 1976. Figure 7 also shows bridge sites in the current investigation that were in place in 1976 (99 of 116 bridges) as well as the 50 streamgages having flows equal to or exceeding approximately 70 percent of the 100-year flow. The significant aerial overlap indicates the high probability that large floods occurred at many if not most of the bridges in the current study that are 26 years old or older. Because of the close proximity of these bridges to the 50 streamgages, it was possible to estimate the maximum historic

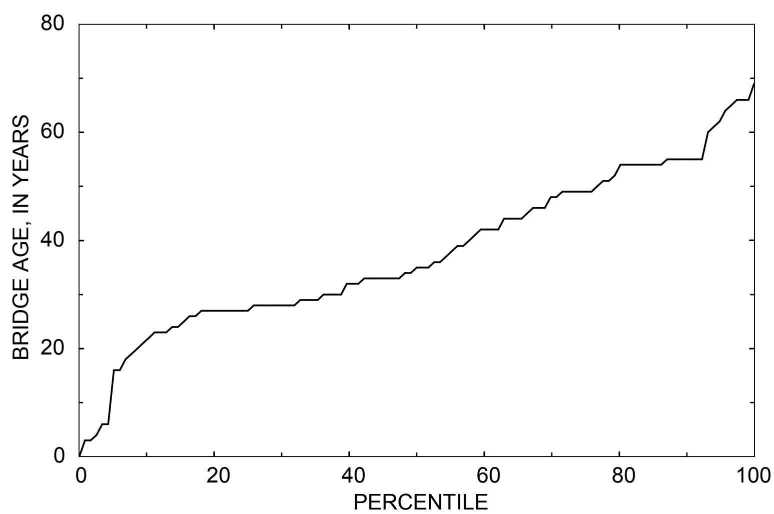


Figure 6. Distribution of bridge age at selected bridges in the Coastal Plain and Piedmont Physiographic Provinces of South Carolina.

Table 1. Selected U.S. Geological Survey streamgaging stations in South Carolina with maximum peak flows equal to or exceeding 70 percent of the 100-year flow during the period 1976 to 2002.

[USGS, U.S. Geological Survey; mi², square mile; ft³/s, cubic foot per second; SC, South Carolina Route; SR, Secondary Road]

USGS gaging station number (fig. 7)	Station name	Calendar year for peak flow	Drainage area (mi^2)	Maximum peak flow (ft^3/s)	Estimated 100-year flow (ft^3/s)	Ratio of maximum peak flow to 100-year flow
02110500	Waccamaw River near Longs	1999	1,110	28,200	23,600[a]	1.19
02110704	Waccamaw River at Conway	1999	1,420[b]	24,800	24,800[c]	1.00
02131000	Pee Dee River at Pee Dee	1999	8,830	103,000	155,000[d]	0.68
02135000	Little Pee Dee River at Galivants Ferry	1999	2,790	27,600	32,100[e]	0.86
02158500	South Tyger River near Reidville	1995	106	9,650	7,980[a]	1.21
02159000	South Tyger River near Woodruff	1995	174	16,500	11,300[a]	1.46
02160326	Enoree River at Pelham	1995	84.2	11,300	8,070[f]	1.40
02160390	Enoree River near Woodruff	1995	249	50,400	15,900[f]	3.17
02160500	Enoree River near Enoree	1995	307	43,800	21,500[a]	2.04
02160700	Enoree River at Whitmire	1995	444	31,200	24,300[a]	1.28
02164000	Reedy River near Greenville	1995	48.6	5,400	6,300[g]	0.86
02164110	Reedy River above Fork Shoals	1995	104	8,200	9,210[f]	0.89
02165000	Reedy River near Ware Shoals	1995	236	9,980	13,500[a]	0.74
02175500	Salkehatchie River near Miley	1992	341	4,360	4,840[e]	0.90
02176500	Coosawhatchie River near Hampton	1992	203	8,820	8,250[a]	1.07
02130900	Black Creek near Mcbee	1990	108	4,500	2,590[a]	1.74
02130910	Black Creek near Hartsville	1990	173	4,450	3,160[d]	1.41
02131309	Fork Creek at Jefferson	1990	24.3	8,690	4,020[a]	2.16
02131472	Hanging Rock Creek near Kershaw	1990	23.9	3,760	3,820[a]	0.98
02135000	Little Pee Dee River at Galivants Ferry	1983	2,790	24,400	32,100[e]	0.76
02135300	Scape Ore Swamp near Bishopville	1990	96	4,500	2,930[a]	1.54
02147000	Catawba River near Catawba	1976	3,530	73,600	105,000[d]	0.70
02147500	Rocky Creek at Great Falls	1989	194	16,300	20,100[a]	0.81
02153500	Broad River near Gaffney	1976	1,490	84,900	94,800[e]	0.90
02154500	North Pacolet River at Fingerville	1995	116	8,160	11,000[a]	0.74
02155500	Pacolet River near Fingerville	1995	212	13,700	18,300[d]	0.75
02156000	Pacolet River near Clifton	1976	320	27,700	30,200[d]	0.92
02156500	Broad River near Carlisle	1976	2,790	123,000	137,000[e]	0.90
02157500	Middle Tyger River at Lyman	1977	68.3	3,990	5,170[a]	0.77
02160000	Fairforest Creek near Union	1976	183	11,700	11,700[a]	1.00
02160105	Tyger River near Delta	1976	759	37,500	35,600[a]	1.05
02161500	Broad River at Richtex	1976	4,850	146,000	210,000[e]	0.70
02162350	Middle Saluda River near Cleveland	1986	21	5,190	5,940[a]	0.87
02163500	Saluda River near Ware Shoals	1995	580	20,900	25,500[a]	0.82
02166970	Ninety-Six Creek near Ninety-Six	1990	17.4	1,410	1,930[a]	0.73
02172500	South Fork Edisto River near Montmorenci	1983	198	3,210	4,520[a]	0.71
02174000	Edisto River near Branchville	1993	1,720	14,000	18,500[a]	0.76
02174250	Cow Castle Creek near Bowman	1979	23.4	2,340	2,410[a]	0.97
02175000	Edisto River near Givhans	1993	2,730	22,700	28,500[e]	0.80
02185200	Little River near Walhalla	1976	72	10,100	14,200[a]	0.71

Table 1. Selected U.S. Geological Survey streamgaging stations in South Carolina with maximum peak flows equal to or exceeding 70 percent of the 100-year flow during the period 1976 to 2002.—Continued

[USGS, U.S. Geological Survey; mi^2, square mile; ft^3/s, cubic foot per second; SC, South Carolina Route; SR, Secondary Road]

USGS gaging station number (fig. 7)	Station name	Calendar year for peak flow	Drainage area (mi^2)	Maximum peak flow (ft^3/s)	Estimated 100-year flow (ft^3/s)	Ratio of maximum peak flow to 100-year flow
02186000	Twelve Mile Creek near Liberty	1998	106	6,800	8,730[a]	0.78
02187900	Broadway Creek near Anderson	1995	26.4	2,720	3,580[a]	0.76
02192500	Little River near Mt. Carmel	1995	217	14,800	15,900[a]	0.93
02196000	Stevens Creek near Modoc	1990	545	27,800	32,900[a]	0.84
Indirect	King Creek at SC 3 in Allendale County	1992	17.2	1,560	1,880[h]	0.83
Indirect	Coosawhatchie River at SR 87 in Jasper County	1992	382	14,100	13,300[h]	1.06
Indirect	Whippy Swamp at SR 13 in Hampton County	1992	134	10,100	6,860[h]	1.47
Indirect	Gaul Branch at SR 107 in Allendale County	1992	8.5	2,240	1,200[h]	1.86
Indirect	Gaul Creek at SC 3 in Allendale County	1992	17.9	4,320	1,930[h]	2.24
Indirect	Coosawhatchie River at SR 21 in Allendale County	1992	48.1	11,900	3,590[h]	3.31

[a]Flood-frequency estimate was based on weighted flood-frequency values as published in Feaster and Tasker (2002).

[b]This site is tidally influenced and has a drainage area that is difficult to define. The drainage area should be considered as an approximation.

[c]This site is tidally influenced making it difficult to estimate flood frequencies. During the September 1999 flood, the USGS gaging station 02110500, Waccamaw River near Longs, which is upstream from this gaging station, had flows slightly exceeding the 100-year flow. Therefore, for purposes of this study, the 1999 maximum historic flow at this gaging station was assumed equal to the magnitude of the 100-year flow. This flow estimate is not recommended for applications beyond the scope of this investigation.

[d]This is a regulated stream making it difficult to determine flood frequencies. For purposes of this study, a log-Pearson Type III analysis of data at this gaging station was made to obtain an approximation of the 100-year flow. This flow estimate is not recommended for applications beyond the scope of this investigation.

[e]Flood-frequency estimate was based on a log-Pearson Type III analysis of the gaging station data as published in Feaster and Tasker (2002).

[f]At the time flood frequencies were estimated at this site, the length of record at the gaging station was less than 10 years. Therefore, the flood-frequency estimate was based on methods presented in Feaster and Tasker (2002) for ungaged sites.

[g]The basin for this gage is urbanized but basin characteristics are outside the limits of the South Carolina flood-frequency urban equations (Bohman, 1992). For purposes of this study, a log-Pearson Type III analysis of data at this gaging station was made to obtain an approximation of the 100-year flow. This flow estimate is not recommended for applications beyond the scope of this investigation.

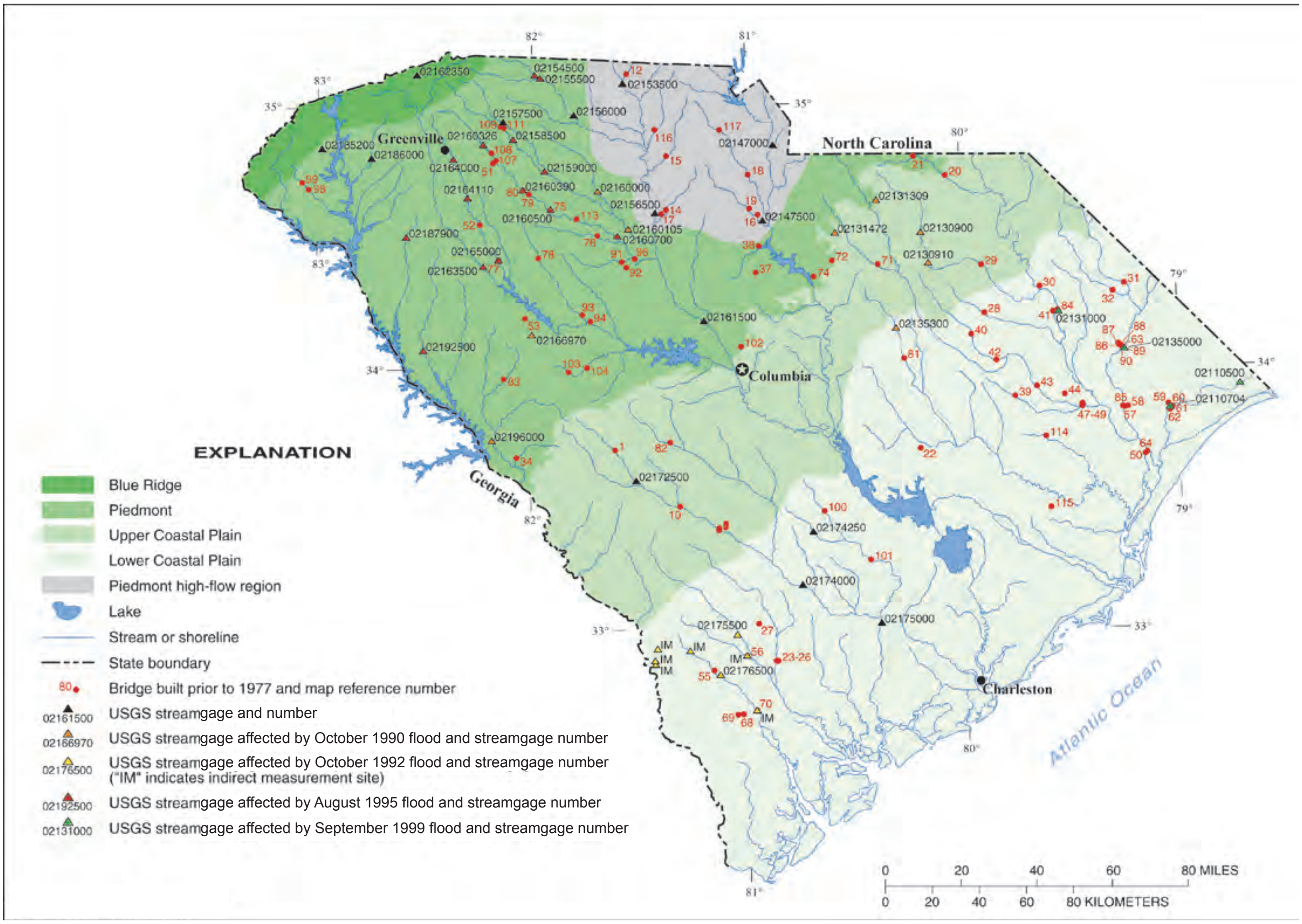
[h]Flood-frequency estimate was based on methods presented in Feaster and Tasker (2002) for ungaged sites.

flows at 34 bridge crossings that included 51 individual bridges (table 2). Twenty-seven of the 51 bridges are associated with multiple-bridge openings and 24 are single-bridge openings. Methods used to estimate the maximum historic flows included (1) collecting streamgage data at the bridge, (2) shifting streamgage data to the ungaged bridge site using methods presented in Feaster and Tasker (2002), (3) interpolating streamgage data by drainage area when a bridge was located between two streamgages, and (4) making indirect computations of peak flows at the bridge from previous documentation of historic floods. Table 2 lists these sites along with the historic peak flows and the method used to estimate the flow. The ratio of historic peak flow to the 100-year flow for the 34 bridge crossings ranges from 0.62 to 4.65 with a median ratio of 1.0. This range indicates that floods close to or exceeding the 100-year flow have occurred at these sites and the scour measurements at these bridges will represent scour resulting from large floods.

Although not all bridges in the current study are known to have been associated with at least one large flood, based on risk analysis, streamgage records, and peak-flow estimates at selected bridge crossings, evidence is strong that floods equal to or exceeding 70 percent of the 100-year flow have occurred at many if not most of the sites in this investigation. This supports the assumption that the scour data collected in this study represent scour resulting from large floods; therefore, the data likely will provide a good indicator for anticipated ranges of scour related to high-flow conditions at bridges in South Carolina.

Site Selection

Field data for this study were collected at 93 multiple- and single-bridge crossings encompassing 116 individual bridges in South Carolina, including 53 bridges in the Piedmont and 63 bridges in the Coastal Plain. There were 10



Base modified from U.S. Geological Survey
digital data, 1:2,000,000 scale, 1972

Figure 7. Locations of selected bridges in the South Carolina Coastal Plain and Piedmont Physiographic Provinces built prior to 1977 and streamgaging stations recording at least one flow equal to or exceeding 70 percent of the 100-year flow during 1976–2002. (Refer to appendix 2 at back of report to identify bridge with corresponding number.)

multiple-bridge crossings in the investigation, all located in the Coastal Plain and encompassing 33 individual bridges. Additionally, there were 83 single-bridge crossings. Dual bridges, which are parallel bridges in close proximity to each other spanning the same river, were considered to be one bridge rather than separate bridges. To minimize costs, many of the bridges used in the previous investigation (92 bridges; Benedict, 2003) also were used in the current study. Bridges in the previous investigation were selected from several sources, including (1) a list provided by the SCDOT of bridges with known scour problems, (2) bridges previously studied by the USGS in the level-2 bridge-scour study, (3) bridges influenced by known maximum historic floods, and (4) Piedmont bridges with wide, flat floodplains indicating a high potential for scour. A description of the selection sources and selection process is found in Benedict (2003). Fifty-two of the bridges in the previous investigation were not used in the current study because site conditions precluded the ability to measure pier scour. In particular, bridges 240 ft or less in length often developed a large single scour hole that encompassed the bridge opening. The large scour hole typically obscured the pier-scour holes making these sites inappropriate for the current investigation. To obtain more sites for data collection, level-2 studies from previous investigations were reviewed and an additional 24 bridges were selected. Particular attention was given to selecting longer bridges (greater than 240 ft in length) that would allow the development of overbank contraction scour and pier scour distinct from abutment-scour holes. Older bridges with higher probabilities of having withstood large floods also were given priority in the selection of additional bridges.

Techniques for the Collection and Interpretation of Field Data

Basic field data collected at each clear-water scour site included (1) measurements of scour depths; (2) collection of bed-material samples, if not collected in the previous investigation; and (3) site description by photographs, sketches, and

Table 2. Estimate of maximum historic flows at selected bridge crossings in South Carolina.

[SCDOT, South Carolina Department of Transportation; mi², square mile; ft³/s, cubic foot per second; Method for estimating peak flow: 1, gage at site; 2, shift of gage data by drainage area ratio; 3, interpolation between gages; 4, indirect measurement; USGS, U.S. Geological Survey; US, United States Route; SC, South Carolina Route; I, Interstate Highway; SR, Secondary Road]

County	Road	Stream	SCDOT structure number	Drainage area at bridge (mi²)	Estimate of maximum historic flow at bridge (ft³/s)	Calendar year for maximum historic flow at bridge	Estimate of 100-year flow at bridge (ft³/s)	Ratio of maximum historic flow to 100-year flow at bridge	Method for estimating maximum historic flow at bridge	USGS gaging station number at or near bridge	Peak flow at USGS gaging station concurrent with life of bridge (ft³/s)	Drainage area at USGS gaging station (mi²)
Bamberg	U.S. 321	South Edisto River Swamp[a]	052032100300	720	7,350	1964	9,220[b]	0.80	1	02173000	7,350	720
		South Edisto River Swamp[a]	052032100400									
		South Edisto River[a]	052032100500									
Barnwell	SC 39	South Edisto River	064003900200	540	6,140[c]	1964	7,750[d]	0.79	1	02173000	7,350	720
Chester	SC 97	Turkey Creek	124009700100	81.5	9,510	1990	11,700[e]	0.81	1	021563931	9,510	81.5
Chester	SC 97	Rocky Creek	124009700800	126	23,900	1967	15,800[d]	1.51	2	02143500	31,300	194
Chester	SC 901	Rocky Creek	124090100200	103	21,000	1967	13,600[d]	1.54	2	02143500	31,300	194
Darlington	I–20	Jeffries Creek	161002020400	30.0	2,370	1994	1,800[f]	1.32	2	02131110	3,220	46.6
Dillon	I–95	Great Pee Dee River[a]	171009530100	8,230	98,500	1979	145,000[g]	0.68	2	02131000	103,000	8,830
Dillon	SC 41	Little Pee Dee River	174004100200	586	7,250	1995	10,100[d]	0.72	2	02132500	6,760	524
Florence/ Marion	U.S. 76	Great Pee Dee River[a]	212007621100	8,830	103,000	1979	155,000[h]	0.66	1	02131000	103,000	8,830
		Great Pee Dee River[a]	342007620100									
Florence	U.S. 378	Lynches River	212037800900	1,100	14,500	1990	22,600[f]	0.64	2	02132000	13,900	1,030
Florence	SC 41	Lynches River[a]	214004100200	1,390	16,800	1990	27,000[f]	0.62	2	02132000	13,900	1,030
		Lynches River Swamp[a]	214004100300									
		Lynches River Swamp[a]	214004100400									
Georgetown/ Horry	U.S. 701	Yauhannah Lake[a]	222070100400	12,990[i]	131,000	1979	199,000[g]	0.66	2	02131000	103,000	8,830
		Great Pee Dee River overflow[a]	262070100100									
Greenville	SR 68	Reedy River	237006800100	173	9,530	1995	11,900[d]	0.80	3	02165000	9,980	236
Hampton	U.S. 601	Coosawhatchie River	243060100300	203	8,820	1992	8,250[b]	1.07	1	02176500	8,820	203
Hampton	SC 363	Coosawhatchie River	254036300100	124	6,470	1992	6,430[d]	1.01	2	02176500	8,820	203
Hampton	SR 13	Whippy Swamp[a]	257001300500	134	10,100	1992	6,860[j]	1.47	4	Indirect measurement	10,100	134

Table 2. Estimate of maximum historic flows at selected bridge crossings in South Carolina.—Continued

[SCDOT, South Carolina Department of Transportation; mi², square mile; ft³/s, cubic foot per second; Method for estimating peak flow: 1, gage at site; 2, shift of gage data by drainage area ratio; 3, interpolation between gages; 4, indirect measurement; USGS, U.S. Geological Survey; US, United States Route; SC, South Carolina Route; I, Interstate Highway; SR, Secondary Road]

County	Road	Stream	SCDOT structure number	Drainage area at bridge (mi²)	Estimate of maximum historic flow at bridge (ft³/s)	Calendar year for maximum historic flow at bridge	Estimate of 100-year flow at bridge (ft³/s)	Ratio of maximum historic flow to 100-year flow at bridge	Method for estimating maximum historic flow at bridge	USGS gaging station number at or near bridge	Peak flow at USGS gaging station concurrent with life of bridge (ft³/s)	Drainage area at USGS gaging station (mi²)
Horry/Marion	U.S. 378	Little Pee Dee River[a]	262037800100	3,040	30,100	1964	36,600[f]	0.82	2	02135000	27,600	2,790
		Little Pee Dee River Swamp[a]	262037800200									
		Little Pee Dee River Swamp[a]	342037800800									
Horry	U.S. 501	Waccamaw River[a]	262050103100	1,420[i]	24,000	1999	24,000[k]	1.00	1	02110704	24,000	1,420
		Waccamaw River[a]	262050103200									
		Waccamaw River[a]	262050103300									
Horry	U.S. 501	Waccamaw River	262050104300	1,420[i]	24,000	1999	24,000[k]	1.00	1	02110704	24,000	1,420
Horry/Marion	U.S. 501	Little Pee Dee River[a]	262050110100	2,790	27,600	1964	33,500[l]	0.82	1	02135000	27,600	2,790
		Little Pee Dee River[a]	342050110700									
		Little Pee Dee River[a]	342050110800									
		Little Pee Dee River[a]	342050110900									
		Little Pee Dee River[a]	342050111000									
		Little Pee Dee River[a]	342050111100									
Horry	SC 22	Waccamaw River[a]	264002220200	1,200	27,000	1999	27,000[k]	1.00	3	02110500	28,200	1,110
		Waccamaw River[a]	264002220300									
		Waccamaw River[a]	264002220400									
Jasper	SR 87	Coosawhatchie River	277008700100	382	14,100	1992	13,300[j]	1.06	4	Indirect measurement	14,100	382
Laurens	SC 49	Enoree River	304004900400	307	43,800	1995	21,500[b]	2.04	1	02160500	43,800	307
Laurens	SR 36	Reedy River	307003600200	236	9,870	1995	13,500[b]	0.73	1	02165000	9,980	236
Laurens	SR 112	Enoree River	307011200100	256	50,000	1995	18,200[d]	2.75	3	02160390	50,400	249
Laurens	SR 263	Enoree River	307026300100	249	50,400	1995	17,700[e]	2.85	1	02160390	50,400	249

Table 2. Estimate of maximum historic flows at selected bridge crossings in South Carolina.—Continued

[SCDOT, South Carolina Department of Transportation; mi^2, square mile; ft^3/s, cubic foot per second; Method for estimating peak flow: 1, gage at site; 2, shift of gage data by drainage area ratio; 3, interpolation between gages; 4, indirect measurement; USGS, U.S. Geological Survey; US, United States Route; SC, South Carolina Route; I, Interstate Highway; SR, Secondary Road]

County	Road	Stream	SCDOT structure number	Drainage area at bridge (mi^2)	Estimate of maximum historic flow at bridge (ft^3/s)	Calendar year for maximum historic flow at bridge	Estimate of 100-year flow at bridge (ft^3/s)	Ratio of maximum historic flow to 100-year flow at bridge	Method for estimating maximum historic flow at bridge	USGS gaging station number at or near bridge	Peak flow at USGS gaging station concurrent with life of bridge (ft^3/s)	Drainage area at USGS gaging station (mi^2)
Lee	U.S. 401	Scape Ore Swamp	312040100100	133	5,520	1990	3,940[d]	1.40	2	02135300	4,500	96.0
Newberry	SR 45	Enoree River	367004500100	719	22,400	1995	30,900[j]	0.72	3	02160700	31,200	444
Newberry	SR 81	Enoree River	367008100200	677	23,400	1995	29,800[j]	0.79	3	02160700	31,200	444
Spartanburg	U.S. 29	South Tyger River	422002900100	76.0	7,840	1995	7,090[d]	1.11	2	02158500	9,650	106
Spartanburg	SR 62	South Tyger River	427006200500	91.8	13,900	1995	7,620[d]	1.82	4	Indirect measurement	13,900	91.8
Spartanburg	SR 118	Enoree River	427011800001	186	64,200	1995	13,800[d]	4.65	3	02160390	50,400	249
Spartanburg	SR 242	South Tyger River	427024200200	94.4	8,400	1995	7,690[d]	1.09	3	02158500	9,650	106
Union	SR 22	Enoree River	447002200100	395	35,700	1995	22,900[d]	1.56	3	02160700	31,200	444

[a]This bridge is part of a multiple-bridge crossing. Peak-flow information was listed for the first bridge but applies to all bridges at the crossing.

[b]Flood-frequency estimate was based on weighted flood-frequency values as published in Feaster and Tasker (2002).

[c]A flow of 5,510 ft^3/s was inadvertently used in this study to represent the maximum historic flow for the 1990 flood at the S.C. Route 39 crossing of the South Edisto River. This is a 10-percent reduction in the estimated maximum historic flow of 6,140 ft^3/s and should not produce significant errors in the WSPRO (Shearman, 1990) model and predicted scour computations at this site.

[d]Flood-frequency estimate was based on methods presented in Feaster and Tasker (2002) for ungaged sites near a gaging station on the same stream.

[e]At the time flood frequencies were estimated at this site, the length of record at the gaging station was less than 10 years. Therefore, the flood-frequency estimate was based on methods presented in Feaster and Tasker (2002) for ungaged sites.

[f]Flood-frequency estimate was based on methods presented in Feaster and Tasker (2002) for ungaged sites near a gaging station on the same stream. These methods were modified to account for the drainage area being in more than one physiographic province.

[g]This is a regulated stream making it difficult to determine flood frequencies. For purposes of this study, flood-frequency estimates from a log-Pearson Type III analysis of data at gaging station 02131000, Pee Dee River at Pee Dee, were shifted to the ungaged site assuming all of the drainage area was in the lower Coastal Plain. This flow estimate is not recommended for applications beyond the scope of this investigation.

[h]This is a regulated stream making it difficult to determine flood frequencies. For purposes of this study, a log-Pearson Type III analysis of data at this gaging station was made to obtain an approximation of the 100-year flow. This flow estimate is not recommended for applications beyond the scope of this investigation.

[i]This site is tidally influenced and has a drainage area that is difficult to define. The drainage area should be considered as an approximation.

[j]Flood-frequency estimate was based on methods presented in Feaster and Tasker (2002) for ungaged sites.

[k]This site is tidally influenced making it difficult to estimate flood frequencies. During the September 1999 flood, the gaging station 02110500, Waccamaw River near Longs, which is upstream from this gaging station, had flows slightly exceeding the 100-year flow. Therefore, for purposes of this study, the 1999 maximum historic flow at this site was assumed equal to the magnitude of the 100-year flow. This flow estimate is not recommended for applications beyond the scope of this investigation.

[l]Flood-frequency estimate was based on methods presented in Feaster and Tasker (2002) for gaged sites. These methods were modified to account for the drainage area being in more than one physiographic province.

written records. Because the field conditions varied between the Coastal Plain and Piedmont, the approach for collecting field data in these regions was slightly different.

Data Collection in the Coastal Plain

In the Coastal Plain swamps, standing water typically covers the floodplain throughout the year, making scour holes difficult to locate visually. To determine the location and extent of clear-water contraction and pier scour, it was necessary to wade the area under the bridge and probe with a rod to determine the deepest area of scour (fig. 8). Clear-water contraction scour, as shown in the previous investigation (Benedict, 2003), typically is located beneath the bridge (outside of the abutment-scour area) with the deepest contraction scour occurring close to the roadway centerline (fig. 9). Therefore, by wading and probing the swampy floodplain under the bridge, the maximum clear-water contraction-scour depth can be located. Although it is difficult in the field setting to isolate the various components of scour (abutment, contraction, and pier) with complete confidence, the following steps were taken to isolate clear-water contraction scour from the other scour components. At many sites, the deeper, clear-water abutment-scour hole that forms close to the bridge abutment could be readily distinguished from clear-water contraction scour that occurs beyond the abutment scour region (fig. 9). At such sites, this pattern provided a means for isolating clear-water contraction scour from abutment scour. When this pattern was not obvious, judgment was required to identify a reasonable place to measure the clear-water contraction scour component. This measurement typically was made at some distance from the bridge abutment where abutment scour

Figure 8. Probing in standing water to determine clear-water scour depth at structure 342050111100 on U.S. Route 501, crossing the Little Pee Dee River floodplain in Marion County, South Carolina. *(Photograph by U.S. Geological Survey, South Carolina Water Science Center, December 3, 2002)*

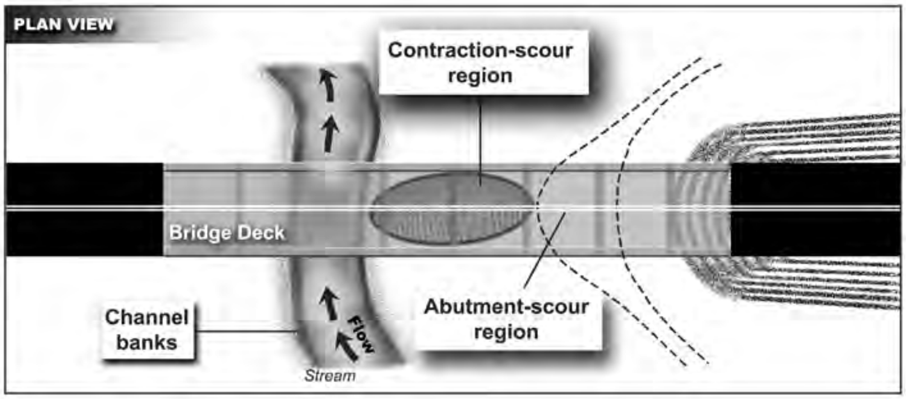


Figure 9. Plan view of regions of clear-water abutment and contraction scour (modified from Benedict, 2003).

likely would not occur. Additionally, to isolate the component of clear-water contraction scour from pier scour, the point of measurement was selected at some distance away from nearby piers. Based on these attempts to isolate the various components of scour, the measured contraction-scour depths in this study should be viewed, in general, as the contraction-scour component only, with negligible influence from the components of pier and abutment scour.

After finding the point of maximum contraction-scour depth, the lateral and longitudinal location of that point in reference to the bridge was documented by hand measurements, and the reference surface and low point of the contraction-scour hole were surveyed to determine the scour depth. The reference surface used to determine the contraction-scour depth was the average, undisturbed floodplain elevation in the area of the scour hole. In general, surveyed ground elevations of the upstream and downstream floodplain just outside the area affected by scour were used to determine the reference surface.

In the case of pier scour, the scour hole typically is located close to the pier (figs. 10–12). Therefore, pier scour at a given site can be determined by probing around each pier or pile while wading under the bridge. The maximum pier scour for both the left and right floodplain typically was located and measured at a given site and included in the database. After finding the pier or pile bent with the maximum pier-scour depth, its lateral and longitudinal location in reference to the bridge were documented by hand measurements, and the scour depth from the reference surface to the low point of the pier-scour hole was surveyed or, in some cases, determined by hand measurements. The reference surface used to determine the pier-scour depth was the average streambed elevation at the top edge of the pier-scour hole. Piers or piles located in large abutment-scour holes were not investigated because the pier-scour holes typically were obscured by the large abutment-scour hole. At many bridges, the pier or pile geometry was identical for all supports (fig. 13), and only one measurement of the maximum pier-scour depth was located and recorded on the left and right overbanks. However, some sites had piers or piles with different column widths (fig. 14), and the maximum pier-scour depth for each pier or pile geometry (on the left and right overbank) generally was recorded. In addition to measuring the scour depth at the pier or pile with the maximum scour, the width of the scour hole perpendicular to the flow also was measured (fig. 12). Although it is difficult in the field setting to separate with complete confidence the various components of scour, attempts were made to isolate clear-water pier scour from the components of abutment and contraction scour. Therefore, measured pier-scour depths in this study should be viewed as the pier-scour component only, with negligible influence from the other components of scour.

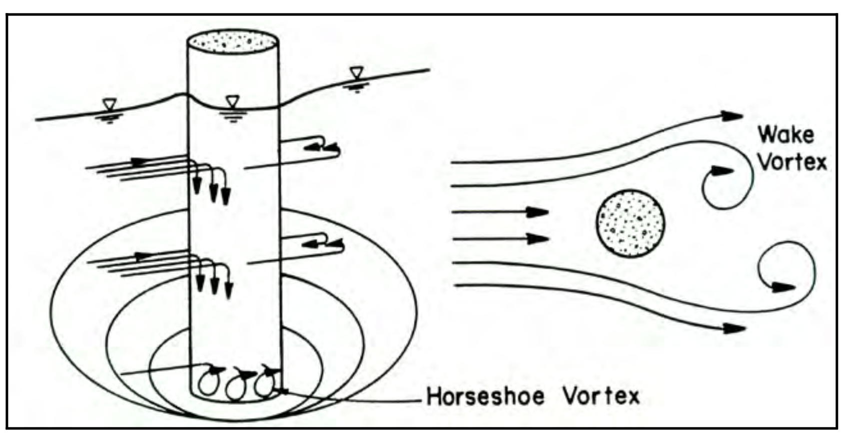


Figure 10. Illustration of scour at a cylindrical pier (from Richardson and Davis, 2001).

Figure 11. Typical pier-scour hole at structure 342050110800 on U.S. Route 501, crossing the Little Pee Dee River floodplain in Marion County, South Carolina. *(Photograph by the U.S. Geological Survey, South Carolina Water Science Center, December 3, 2002)*

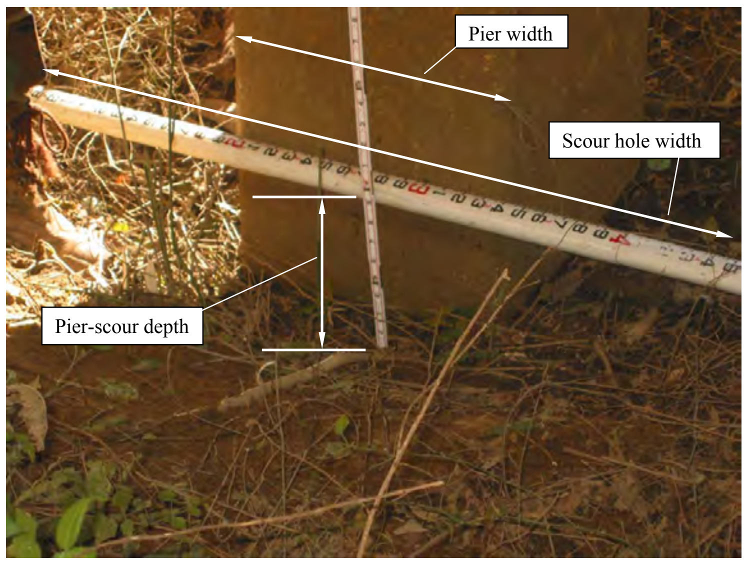


Figure 12. Typical geometry of pier-scour hole at structure 212007621100 on U.S. Route 76, crossing the Great Pee Dee River floodplain in Florence County, South Carolina. *(Photograph by the U.S. Geological Survey, South Carolina Water Science Center, December 2, 2002)*

Figure 13. Concrete pile bent with common pile dimension at structure 182007800200 on U.S. Route 78, crossing Polk Swamp in Dorchester County, South Carolina. *(Photograph by the U.S. Geological Survey, South Carolina Water Science Center, November 26, 1996)*

Figure 14. Composite bent at structure 262050103200 on U.S. Route 501, crossing the Waccamaw River in Horry County, South Carolina (from Benedict, 2003). *(Photograph by the U.S. Geological Survey, South Carolina Water Science Center, June 13, 2000)*

If sediment samples at a particular bridge were collected and analyzed in the previous investigation (Benedict, 2003), no additional samples were collected. However, if a site was not included in the previous investigation, then sediment samples were collected. These samples typically were obtained by using a 2-inch (in.)-diameter polyvinyl chloride (PVC) drive tube that allowed the collection of submerged bed material. In the case of contraction scour, bed samples generally were collected from the low point of the scour hole and from the upstream reference surface. In the case of pier scour, the upstream sample taken for contraction scour was assumed to be representative of the sediment at the pier prior to any scour. A description of each sediment sample was documented in the field notes, and the samples were analyzed for grain-size distribution.

Data Collection in the Piedmont

Clear-water abutment and contraction scour in the Piedmont were investigated previously (Benedict, 2003), and the primary focus in the Piedmont for the current investigation was clear-water pier scour. In contrast to the Coastal Plain, clear-water pier scour in the Piedmont occurred on the overbank areas where scour holes typically were dry. Under these conditions, the pier or pile with the deepest scour on the left and right overbank could be determined by visual inspection. After finding the pier or pile bent with the maximum pier-scour depth, scour data were collected using similar procedures as those used at Coastal Plain sites. Data collected in the Piedmont included (1) the lateral and longitudinal location of the pier or pile in reference to the bridge, (2) the scour-hole depth, and (3) the scour-hole width. As with the Coastal Plain sites, only the maximum pier-scour depth for a given pier or pile geometry on each overbank was measured. The reference surface used to determine the pier-scour depth was the average bed elevation at the top edge of the pier-scour hole. Piers or piles located in the large abutment-scour holes were not investigated because the pier-scour holes typically were obscured by the large abutment-scour hole. As in the Coastal Plain, the measured pier-scour depths in this study represent the pier-scour component only, isolated from the components of contraction and abutment scour.

Development of the Predicted Bridge-Scour Database

Predicted scour was computed at each bridge for the 100-year flow and, where available, for the maximum historic flow. Methods and equations described in HEC-18 (Richardson and Davis, 2001) were used to calculate predicted clear-water pier and contraction scour. The hydraulic variables required for these equations were obtained from the Water-Surface-PROfile model (Shearman, 1990). (For the remainder of the report the Water Surface-PROfile model will be referred to as WSPRO or the WSPRO model.) Computer programs were written to automate the extraction of hydraulic data from the WSPRO output files and to calculate predicted scour. Predicted scour depths and variables required to compute these depths are in a database available at https://pubs.er.usgs.gov/publication/sir20055289.

Estimating Hydraulic Data

As noted previously, data collected for this study represent maximum clear-water scour depths for the life of a bridge rather than scour produced by a unique flow event. The limitation of such data is that measured scour cannot be associated with the hydraulic conditions that produced the scour. Because many of the scour-prediction equations are driven by hydraulic properties, such as flow depth and velocity, direct verification of these equations was limited in this study. In an attempt to minimize this limitation, the one-dimensional WSPRO model was used for each bridge to gain insights about hydraulic conditions during large flows. Because the magnitude of historic floods was not known at all of the sites, the 100-year flow was modeled at all bridges as a common flood condition. The maximum historic flows also were modeled at the 34 bridge crossings, which included 51 bridges. Twenty-seven of the 51 bridges were multiple-bridge openings and 24 were single-bridge openings. Hydraulic data generated from WSPRO were used in the scour-prediction equations to make a limited comparison of predicted and measured scour. Hydraulic properties from WSPRO and scour-prediction variables were entered in a database and these data, along with field data, were used to investigate relations that may help explain scour in South Carolina. For more details on standard techniques for developing the WSPRO models and their limitations, refer to Benedict (2003).

Estimates of the 100-Year Flow

The 100-year flow, recommended in HEC-18 (Richardson and Davis, 2001) as a standard condition for predicting potential scour, was used as a common flow at all sites. Predicted scour based on the 100-year flow was compared with the measured scour to evaluate the reasonableness of the HEC-18 methods. In addition, the hydraulic and scour-prediction data generated with the 100-year flow were used to investigate relations of scour in South Carolina.

The 100-year flow for rural basins was computed using the flood-frequency equations and methods presented in Feaster and Tasker (2002). (In the previous investigation, Benedict (2003) used the rural flow equations from Guimaraes and Bohman (1992)). Limited streamgage data in the areas surrounding York and Chester Counties indicate that flood-frequency trends in this region are better represented by equations developed for the North Carolina Piedmont (Pope and others, 2001), and Feaster and Tasker (2002) recommend that rural flows in this region (fig. 1) be estimated with the North Carolina equations. The North Carolina Piedmont equations can give significantly larger peak-flow magnitudes than the South Carolina equations. This should be kept in mind when reviewing sites in this region. Fifteen bridges in the current study were influenced by the high-flow region (table 3). Flows for urban drainage basins with an impervious area greater than 10 percent of the basin were computed using the urban-runoff equations presented in Bohman (1992). Two sites have impervious areas exceeding 10 percent of the drainage basin (table 4).

Table 3. Bridges influenced by the high-flow region in the Piedmont of South Carolina.

[SCDOT, South Carolina Department of Transportation; SC, South Carolina Route; SR, Secondary Road; I, Interstate Highway; U.S., United States Route]

County	Road	Stream	SCDOT structure number	Reference number (figure 1 and Appendix 2)
Cherokee	SC 5	Buffalo Creek	114000500200	11
Cherokee	SR 348	Buffalo Creek	117034800100	12
Chester	I–77	Fishing Creek	121007710700	13
Chester	SC 72	Sandy River	124007200200	14
Chester	SC 97	Turkey Creek	124009700100	15
Chester	SC 97	Rocky Creek	124009700800	16
Chester	SC 215	Sandy River	124021500200	17
Chester	SC 223	Fishing Creek	124022300100	18
Chester	SC 901	Rocky Creek	124090100200	19
Fairfield	I–77	Little Wateree Creek	201007710600	35[a]
Fairfield	I–77	Big Wateree Creek	201007710700	36
Fairfield	U.S. 21	Dutchmans Creek	202002100200	37[a]
York	SC 97	Bullocks Branch	464009700300	116
York	SC 322	Fishing Creek	464032200300	117
York	SR 721	Taylors Creek	467072100100	118

[a]Site is not in the high-flow region, but flows at the site are thought to be similar to or influenced by the high-flow region.

Table 4. Bridges with impervious areas of the drainage basin exceeding 10 percent.

[SCDOT, South Carolina Department of Transportation; mi^2, square mile; SC, South Carolina Route]

County	Road	Stream	SCDOT structure number	Drainage area (mi^2)	Impervious area of drainage basin, in percent	Reference number (figure 1 and Appendix 2)
Spartanburg	SC 146	Enoree River	424014600100	127	13	107
Spartanburg	SC 296	Enoree River	424029600100	119	14	108

Historical Flows

Although many bridges in this study had no record of historic flows, streamgage data were available at or near 51 bridges and were used to estimate maximum flows during the life of the bridge (table 2). The maximum historic flows were estimated using data obtained from USGS gaging stations or from floods documented by the USGS using indirect methods for flow computation. Particular focus was given to recent floods that occurred in October 1990, October 1992, August 1995, and September 1999 when flows often equaled or exceeded the 100-year flow. (For more details on the estimate of maximum historic flows, refer to the section "Assumption of Large Floods.") These maximum historic flows were used in the WSPRO model to estimate hydraulic conditions during these floods. The hydraulic variables then were used in predictive equations to estimate scour for the maximum flows and were compared with the measured scour. In addition, the hydraulic and scour-prediction data generated with the maximum historic flows were used to investigate relations that may help explain scour in South Carolina.

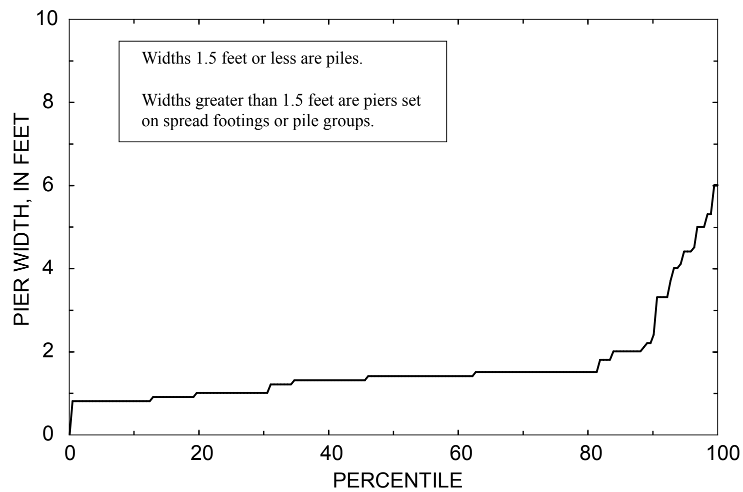


Figure 15. Distribution of pile and pier widths for selected bridges in the Coastal Plain and Piedmont of South Carolina.

Predicted Clear-Water Pier Scour

Pile bents are the primary foundation at approximately 80 percent of the bridges studied in this investigation (fig. 15). Pile bents consist of a row of piles driven into the ground and interconnected by a bent cap at the top of the piles (fig. 16) that provides support for the bridge deck. The three types of piles observed in this study were round timber, steel H, and square concrete (figs. 17, 18, 13, respectively). The widths of these piles varied from 0.8 to 1.5 ft.

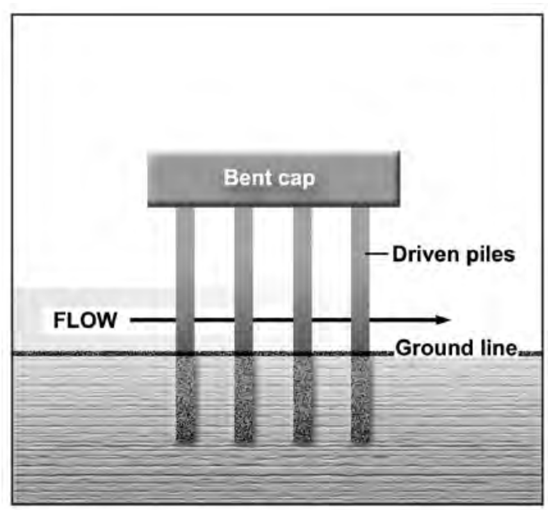


Figure 16. Generalized profile of bridge pile bent (from Benedict, 2003).

Figure 17. Timber pile bent at structure 194023000500 on S.C. Route 230, crossing Horne Creek in Edgefield County, South Carolina (from Benedict, 2003). *(Photograph by the U.S. Geological Survey, South Carolina Water Science Center, February 6, 1997)*

Figure 18. Steel H-pile bent at structure 467072100100 on Road S-721, crossing Taylors Creek in York County, South Carolina (from Benedict, 2003). *(Photograph by the U.S. Geological Survey, South Carolina Water Science Center, January 29, 1997)*

Another type of bridge foundation is a pier supported on spread footings or pile groups (figs. 19, 20), which support about 20 percent of the bridges in this study (fig. 15). The piers generally are larger than piles and range in width from 1.8 to 6.0 ft. On bridges that had been widened to accommodate additional traffic lanes, it was common to find a combination of piers and piles forming a composite bent to support the bridge. Composite bents typically have piers supporting the original structure with piles added upstream and downstream from the old piers to support the newly added lanes (figs. 14, 21). Although a pile bent and pier are structurally different bridge supports, the scour processes are the same, and the local scour that occurs at either support will be called pier scour throughout the report.

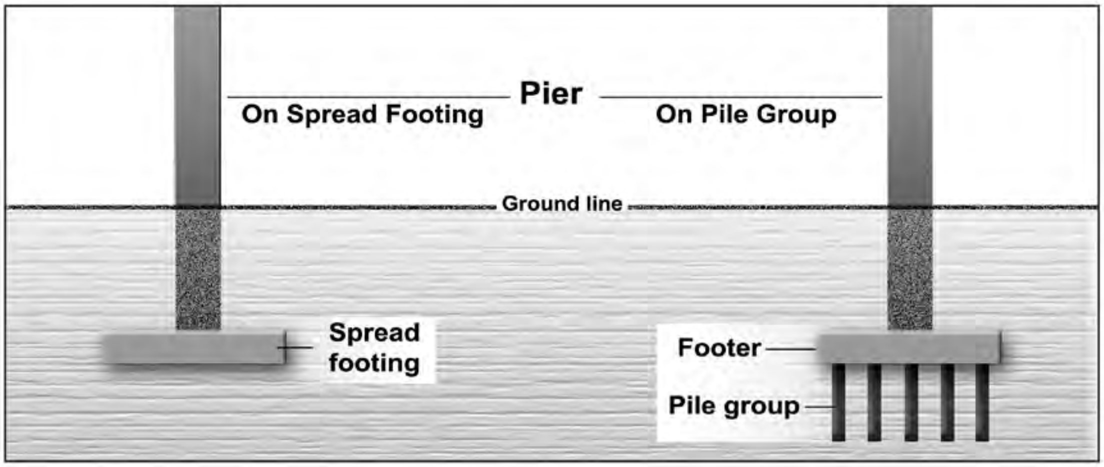


Figure 19. Generalized profile of pier on spread footing and pile group (from Benedict, 2003).

Figure 20. Pier at structure 262050103100 on U.S. Route 501 Business, crossing the Waccamaw River in Horry County, South Carolina (from Benedict, 2003). *(Photograph by the U.S. Geological Survey, South Carolina Water Science Center, June 13, 2000).*

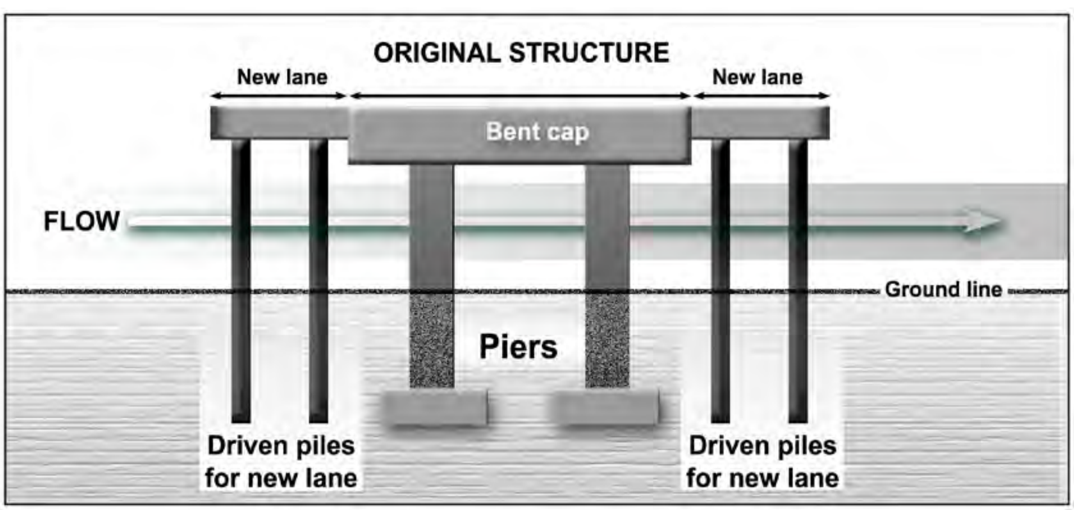


Figure 21. Generalized profile of composite bent (from Benedict, 2003).

Predicted pier scour was computed using the HEC-18 (Richardson and Davis, 2001) equation:

$$\frac{y_s}{b} = 2.0K_1K_2K_3K_4\left[\frac{y_1}{b}\right]^{0.35} Fr_1^{0.43}, \qquad (1)$$

where

y_s is the predicted pier-scour depth, in feet;
b is the pier width, in feet;
K_1 is the dimensionless correction coefficient for pier-nose shape;
K_2 is the dimensionless correction coefficient for flow angle of attack;
K_3 is the dimensionless correction coefficient for streambed conditions;
K_4 is the dimensionless correction coefficient for streambed armoring;
y_1 is the approach-flow depth, in feet; and
Fr_1 is the approach-flow Froude number defined as

$$Fr_1 = V_1/\ (gy_1)^{0.5};$$

where

V_1 is the mean approach velocity, in feet per second; and
g is the acceleration of gravity, in square feet per second.

When applying this equation to compute local scour around piers and pile bents, the following assumptions and methods were used. In general, the width of the pier or pile was determined by using the pier or pile dimension parallel with the bridge face opening and perpendicular to the direction of flow. For composite bents with columns of varying widths (figs. 14, 21), the width of the column where the pier-scour measurement was taken was used to represent the pier width in the HEC-18 equation. Most bridges in this study had piers or piles that were constant in width along the vertical axis. Several bridges, however, had piers that diminished in width as elevation increased. In such cases, the pier width at the ground line was used in the HEC-18 equation. Although the pier or pile bent length is not used directly in the HEC-18 equation, it is required to determine the correction coefficient for flow angle of attack. For pile and composite bents, the pier length was determined by summing the length of each pile or pier parallel with the direction of flow. For solid piers, the pier dimension parallel with the flow was used to represent the pier length.

The correction coefficient for pier-nose shape, K_1, was obtained from HEC-18 (Richardson and Davis, 2001). Pile bents with square piles were assumed to have the shape of a square-nosed pier, whereas pile bents with circular piles were treated as a group of cylinders. The correction coefficient for flow angle of attack, K_2, also was obtained from HEC-18. To determine this factor, an estimate must be made of the high-flow angle of attack. This angle typically is based on visualizing the flow patterns during high-flow conditions and, as such, has a measure of subjectivity. Field inspections and USGS topographic maps were used to estimate the high-flow angle of attack with weight typically given to the topographic map. In general, a single flow angle of attack was determined for each bridge crossing and applied to all piers at that bridge. For Coastal Plain swamps, backwater upstream from the bridge commonly causes flood flows to pass relatively straight through the bridge opening irrespective of the bridge orientation to the floodplain. In such cases, the flow angle of attack was considered to be zero.

In the case of multiple columns, HEC-18 recommends that if the spacing between the columns is 5 pier widths or greater, the correction coefficient for the flow angle of attack, K_2, should not exceed 1.2. This recommendation is specific to cylindrical columns. In the current study, however, it was applied to pile bents with cylindrical or square piles (figs. 13, 17, 18). Therefore, if the spacing between piles was equal to or greater than 5 times the pile width, the K_2 skew correction coefficient was limited to 1.2. The application of this limit to multiple columns with square geometry was based on observed trends in the field that indicated little or no influence from adjacent piles when the piles were spaced approximately 5 or more pier widths apart. Multiple column bents at bridges that had not been widened typically had uniform column spacings that were 5 pier widths or greater (figs. 13, 17, 18) and the skew correction coefficient could be limited to 1.2. However, at bridges that had been widened, column spacings were typically irregular (figs. 14, 21). In the case of irregular column spacings, the smallest column spacing and largest column width were used to determine if the spacing between the columns was greater than or equal to 5 pier widths, therefore limiting the skew correction coefficient to 1.2. (There was one exception to this application at U.S. Route 21 crossing Dutchmans Creek in Fairfield County where only two of the eight piles were less than 5 pier widths apart. The interaction between these columns, however, was considered minimal, and the skew correction coefficient was limited to 1.2.)

Because this study primarily focused on the occurrence of clear-water pier scour in the floodplain, the streambed conditions at piers and pile bents were assumed to be clear water for all cases. Therefore, the correction coefficient for streambed conditions, K_3, was set to 1.1 for all pier-scour computations. The smallest median grain size (D_{50}) required for applying the streambed armoring correction coefficient, K_4, is 2 millimeters (mm). The largest D_{50} for all bridges in the study was 0.99 mm with an average D_{50} of 0.2 mm. Therefore, the effects of streambed armoring on pier scour were considered negligible, and the correction coefficient, K_4, was set to 1.0 for all computations of pier scour.

To calculate the Froude number at a given pier, the stream-tube algorithm within the WSPRO model was applied to the bridge cross section to obtain estimates of the flow velocity and depth. This algorithm divides the bridge cross section into 20 stream tubes of equal conveyance and computes the flow area and the average velocity within each tube. The stream tube that corresponds to the location of a given pier or pile bent was selected, and the velocity and depth associated with that tube were used to compute the Froude number for the pier or pile bent of interest.

When computing predicted scour at piers with footings, HEC-18 (Richardson and Davis, 2001) recommends adjusting the streambed elevation at a pier to account for the predicted contraction scour. If a footing is exposed, based on this adjusted bed elevation, special considerations must be made for computing scour at this pier. In general, observed trends in the field confirmed that piers in this study rarely had exposed footings due to construction constraints or contraction scour. Based on these trends, the special considerations for exposed footings were assumed unnecessary when computing predicted pier scour. Predicted scour was computed only for piers where field measurements of scour were collected, and the scour-prediction variables were stored in the pier-scour database. For further details on the variables stored in the predicted pier-scour database, see appendix 1.

Predicted Clear-Water Contraction Scour

Clear-water contraction scour occurs where upstream bed sediments are not transported through a contracted section. This condition may occur when velocities upstream from a contraction are insufficient to transport bed materials from the upstream reach into the contraction. Clear-water scour conditions may be enhanced further by dense vegetation that limits sediment transport along the streambed, regardless of upstream flow velocities. As discussed previously in the section "Clear-Water Scour Conditions," both conditions typically prevail in the floodplains and swamps of South Carolina, making them good candidates for clear-water scour. On streams with well-defined channels, low velocities and dense vegetation in the floodplain create clear-water scour conditions on the bridge overbanks (fig. 22). Likewise, low velocities and thick vegetation in Coastal Plain swamps produce clear-water scour conditions across the entire bridge opening (fig. 23). These types of clear-water contraction scour were a primary focus of this study.

For computing predicted clear-water contraction scour, HEC-18 (Richardson and Davis, 2001) recommends the use of

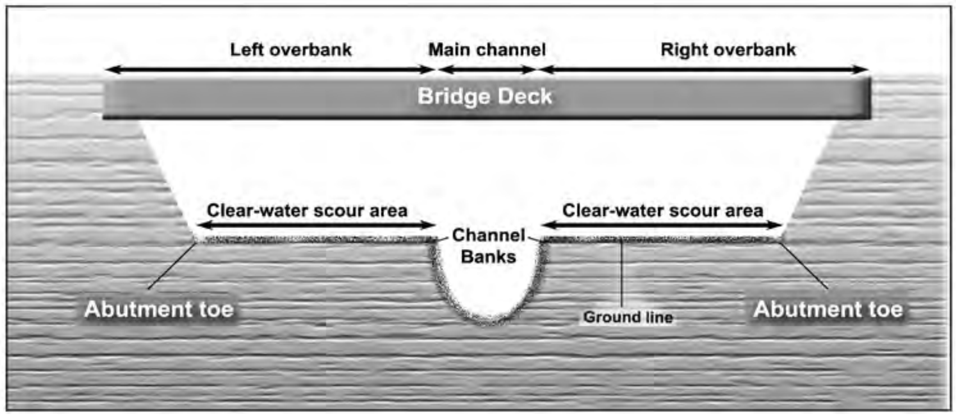


Figure 22. Generalized profile of typical bridge with well-defined low-flow channel, showing areas of clear-water scour (from Benedict, 2003).

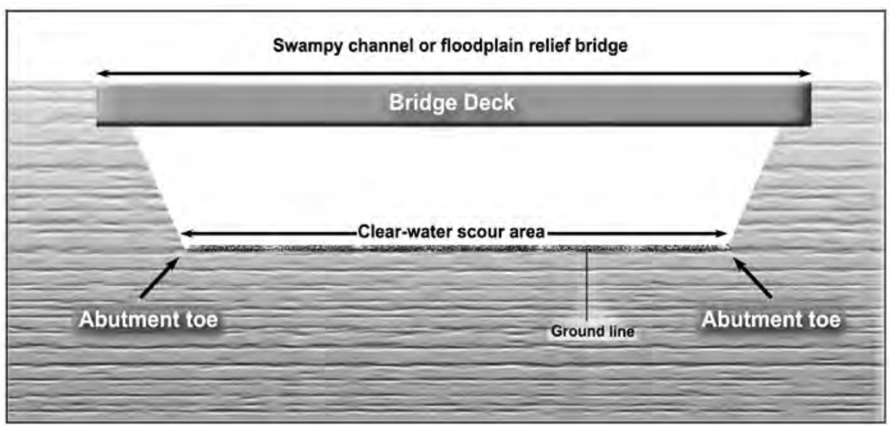


Figure 23. Generalized bridge cross section for a swampy channel or floodplain relief bridge, showing area of clear-water scour (from Benedict, 2003).

a modified version of Laursen's (1963) equation for clear-water scour at long contractions and is defined as

$$y_2 = \left[\frac{K_u Q^2}{D_m^{\frac{2}{3}} W^2}\right]^{\frac{3}{7}} \quad (2)$$

and

$$y_s = y_2 - y_1, \quad (3)$$

where

y_2 is the average depth of flow in the contracted section after the occurrence of contraction scour, in feet;

K_u is the units coefficient and is 0.0077 for English units;

Q is the flow associated with the contraction width W, in cubic feet per second;

D_m is the diameter of the smallest non-transportable particle in the streambed material at the contracted section, in feet, and is defined as $D_m = 1.25D_{50}$;

W is the width of the contracted section adjusted by subtracting the pier width(s) within the section, in feet;

y_s is the average scour depth in the contracted section, in feet;

y_1 is the average depth of flow in the contracted section prior to contraction scour, in feet; and

D_{50} is the median grain size of the streambed material, in feet.

For sites with well-defined low-flow channels, the left and right overbanks were the areas of the bridge opening where clear-water contraction scour occurred. Therefore, predicted scour was computed at each overbank. The contracted width for a given overbank is defined as the distance from the abutment toe to the channel bank (fig. 22). The flow across the overbank was determined by prorating the total flow through the bridge by the ratio of conveyance within the overbank to that of the entire bridge cross section. The average depth of flow prior to the occurrence of contraction scour was obtained by dividing the flow area at the overbank by the overbank width. For Coastal Plain sites with swampy channels, the entire channel at the bridge opening has clear-water contraction scour (fig. 23). In this case, the contracted width was defined as the distance from the left abutment toe to the right abutment toe, and procedures defined above were used to determine the other variables. The D_{50} was determined from a grain-size analysis of a sediment grab sample. To obtain a representation of the pre-scour sediments, the sample was taken upstream from the contraction, outside the limits of any scour.

Predicted scour depths and the variables used to compute these depths were stored in the predicted clear-water contraction-scour database and related to the 100-year flow or maximum historic flow. For further details on the stored variables, see appendix 1.

Development of the South Carolina Pier-Scour Envelope Curve

The frequent use of envelope curves to understand scour trends in the laboratory indicates that this approach also can be used to understand scour trends in the field. To develop an envelope curve that displays the range and trend of scour, it is important to use a dominant explanatory variable in that envelope curve. The following sections review selected variables that have been shown to influence pier scour in the laboratory setting and investigate their influence on pier scour in South Carolina. Based on these findings, an appropriate explanatory variable was selected for developing the South Carolina pier-scour envelope curve, and the envelope curve with its limitations is described.

Variables Influencing Pier Scour

Local bridge scour is the erosion of streambed material from around flow obstructions, such as piers and abutments. The mechanism that causes the erosion is the combined effect of flow acceleration and the resulting vortexes that are induced by the obstructions (Richardson and Davis, 2001). In the case of piers, three principal flow features that contribute to the development of scour were identified in laboratory studies. These include down flow at the face of the pier, the horseshoe vortex at the bottom of the pier, and the wake vortexes downstream from the pier (Melville and Coleman, 2000; Richardson and Davis, 2001; fig. 10). The down flow acts like a vertical jet eroding sediments at the pier face. The eroded sediments then are transported by the horseshoe vortex past the pier and into the area of the wake vortexes. Melville and Coleman (2000) describe the wake vortexes as vacuum cleaners that can erode bed sediments downstream from the pier as well as continue the downstream transportation of the sediments eroded by the down flow. The interaction of these flow patterns creates a scour hole at a pier that is located close to the pier base.

Numerous laboratory studies have been conducted of the variables that influence pier scour. Melville and Coleman (2000) summarized the laboratory findings and described the effect of selected variables, including the velocity of approaching flow, the depth of approaching flow, sediment characteristics, pier geometry, pier alignment with flow, and flow duration. Because conditions in the field can be substantially different from the simplified conditions of the laboratory, direct application of laboratory results to the field may not be justified. However, it is reasonable to assume that the qualitative trends in the laboratory also will be observed in the field; therefore, laboratory investigations likely provide valuable insights for understanding pier scour under field conditions. With this assumption in mind, a brief description of selected laboratory findings and how they relate to pier scour in South Carolina is presented. Because the variable pier width was used to develop the South Carolina pier-scour envelope curve, the influence of pier width on pier-scour depth will be reviewed in the section “Pier Width and the South Carolina Pier-Scour Envelope Curve.”

Justification for Using the 100-Year Flow to Estimate Hydraulics

Hydraulic variables associated with field measurements were estimated using the one-dimensional model, WSPRO. Because only 51 of the 116 bridges in this investigation have historical flow estimates, the theoretical 100-year flow was used as a common condition to gain insights about the hydraulics that may occur during a large flood. Because the hydraulic variables for the 100-year flow can be different from the actual hydraulic conditions that created the measured scour, scour trends based on the 100-year flow hydraulics or comparisons of field measurements with predicted scour estimated with the 100-year flow can include error. To gain perspective of the potential error, 15 bridges were randomly selected and the 25-, 50-, 100-, and 200-year flows were simulated with the WSPRO model. A comparison of the average 100-year flow velocity and depth at the bridge with the same variables for the 25-, 50-, and 200-year flows (fig. 24) indicates that the 100-year flow hydraulic variables are relatively close to the hydraulic variables for the 25- to 200-year flows. In the case of the 50- and 200-year flows, the 100-year flow hydraulic variables had magnitude differences of 0.3 ft/s or less for the average velocity and 1.3 ft or less for the average depth. In the case of the 25-year flow, the 100-year flow hydraulic variables had magnitude differences of 0.7 ft/s or less for the average velocity and 2.4 ft or less for the average depth. These differences are relatively small and indicate that the 100-year flow hydraulic variables are good approximations of the hydraulic conditions of flow that fall within the range of the 25- to 200-year flows. Because it is probable that streamflow at 80 percent of the bridges in this investigation has equaled or exceeded the 25-year flow (see report section, “Assumption of Large Floods”), the 100-year flow hydraulics provide a good approximation of the hydraulic conditions that produced the measured scour at many of the bridges.

To understand the relative effect of errors in hydraulic variables on the computation of predicted scour, the hydraulic variables for the same 15 selected sites were used to compute predicted pier scour for cylindrical piers 1.5-ft and 6-ft wide using the HEC-18 pier-scour equation (Richardson and Davis, 2001). (The computation of predicted pier scour is based on the average velocity and depth within the bridge opening rather than a local velocity and depth based on flow distribution within the bridge opening at a specific pier. While the computations with local hydraulic variables would provide a more refined estimate of predicted scour at a specific pier, the scour computations based on the average hydraulics provide understanding of the relative changes in predicted pier scour as the hydraulics vary.) A comparison of the predicted pier scour for the 25-, 50-, and 200-year flows with predicted pier scour for the 100-year flow (fig. 25) indicates that the differences are relatively small. In the case of the 1.5-ft wide pier, predicted pier scour for the 25-, 50-, and 200-year flows varied from the predicted pier scour for the 100-year flow by 0.3 ft or less (12 percent or less; fig.25A). In the case of the 6-ft wide pier, predicted pier scour for the 25-, 50-, and 200-year flows varied from the predicted pier scour for the 100-year flow by 0.8 ft or less (again, 12 percent or less; fig. 25B). These differences are relatively small indicating that predicted pier scour computed with the 100-year flow hydraulics is a good approximation of the predicted pier scour resulting from flows in the range of the 25- to 200-year flows. Based on these trends, it is apparent that the use of 100-year flow as a common flow condition at all sites provides a good indication of the hydraulics that likely created the scour with the potential for only a small error in the predicted pier scour for most sites. Some sites likely will deviate from the trends shown in figure 24, and use of the 100-year flow will provide a poor estimate of the hydraulic conditions that created the pier scour. This is most likely to occur at sites where extreme floods have occurred.

The trends displayed in figure 25 have an important implication regarding field measurements of scour. Although the HEC-18 pier-scour equation may not provide the exact pier-scour depth that can occur in the field setting, it is reasonable to assume that it will provide some indication of the relative change in pier-scour depth as flow increases from the 25- to the 100-year flow. In South Carolina, because the relative change in predicted pier scour between the 25- and 100-year flows is small (12 percent or less), it is reasonable to assume that a field measurement of pier scour at a site where only a 25-year flow has occurred will be a good approximation of the pier scour that will occur during the 100-year flow. Because approximately 80 percent of the bridges in this study have had flows equal to or exceeding the 25-year flow, it is reasonable to assume that pier-scour measurements at these sites will be good indicators of scour that can result from the 100-year flow. This assumption, along with the known maximum historic flows at 51 of the bridges (table 2) strongly supports the theory that measured pier scour in this investigation provides a good approximation of the range and trend of pier scour that can be anticipated for 100-year flows.

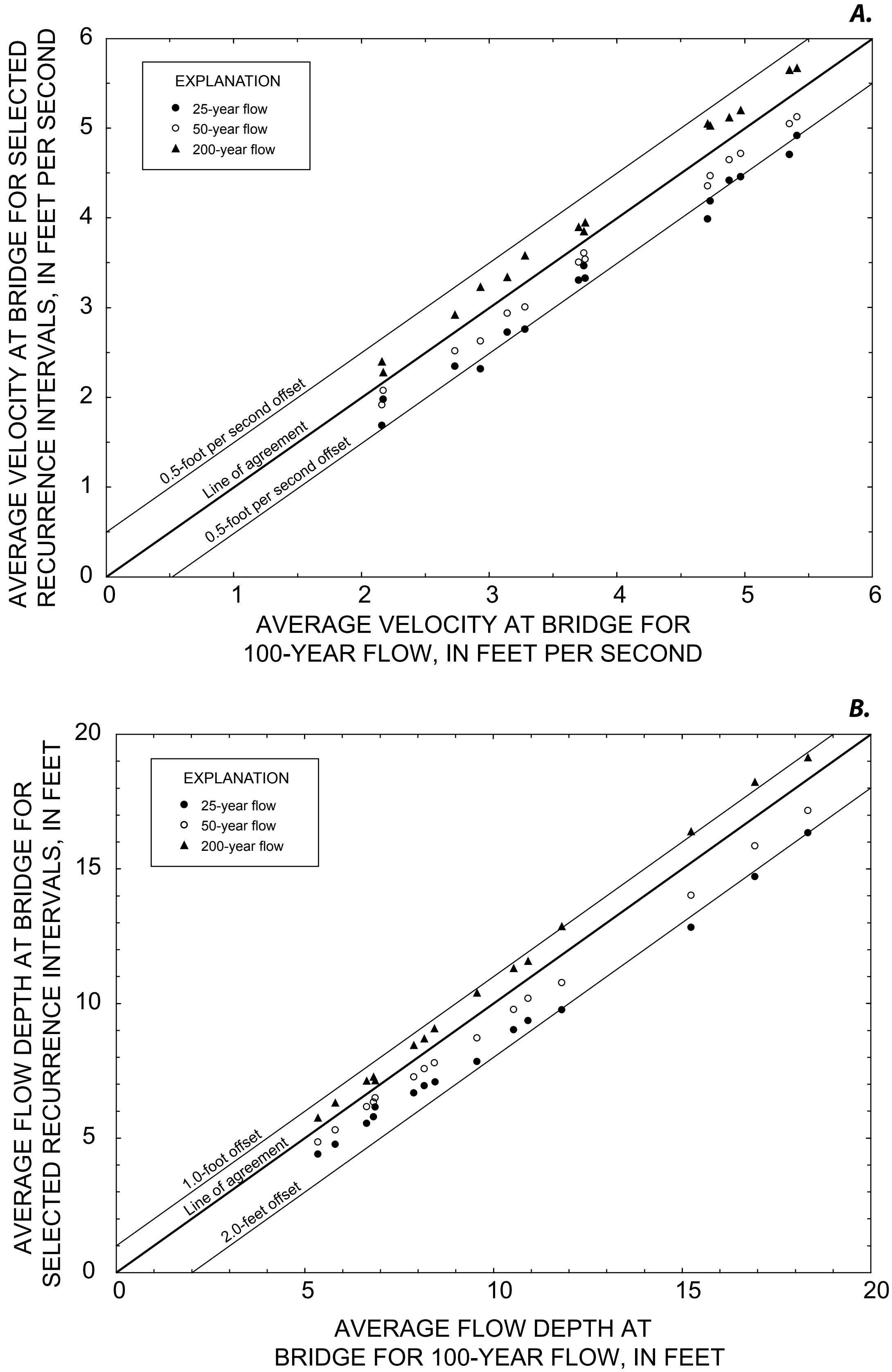
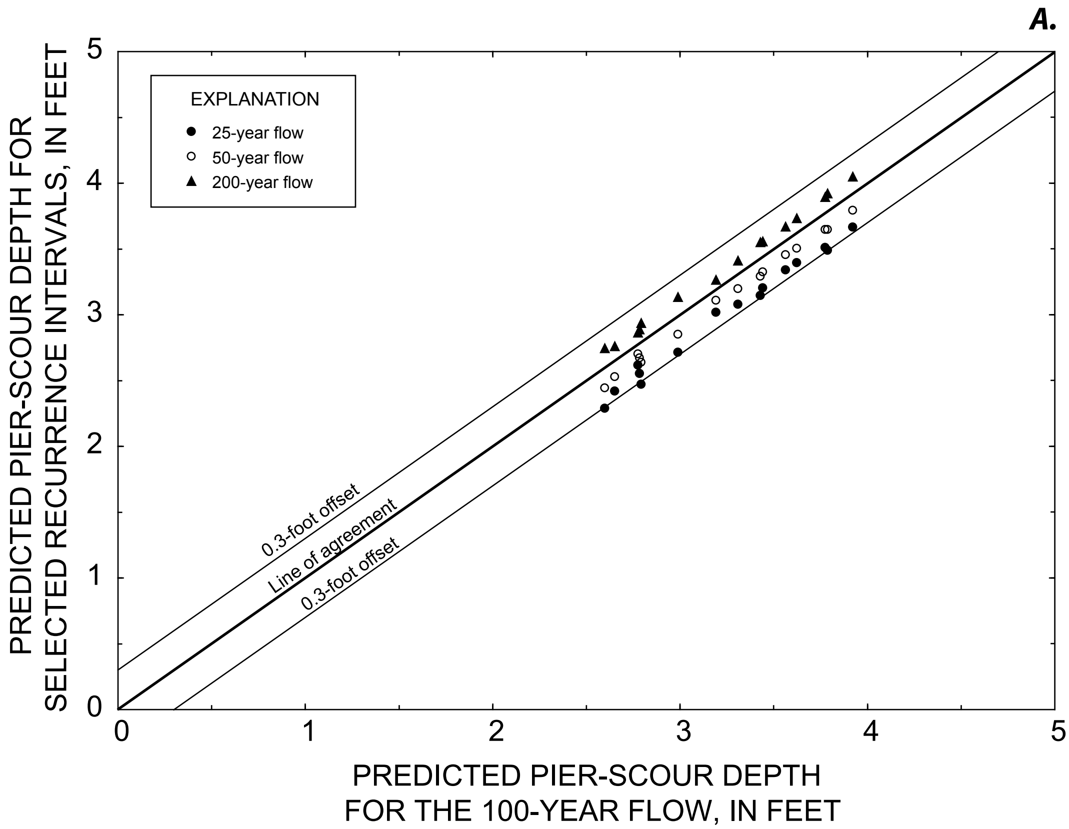


Figure 24. Relation of (A) the average velocity at the bridge and (B) the average flow depth at the bridge for the 25-, 50-, and 200-year flows with respect to the 100-year flow at selected bridges in South Carolina.



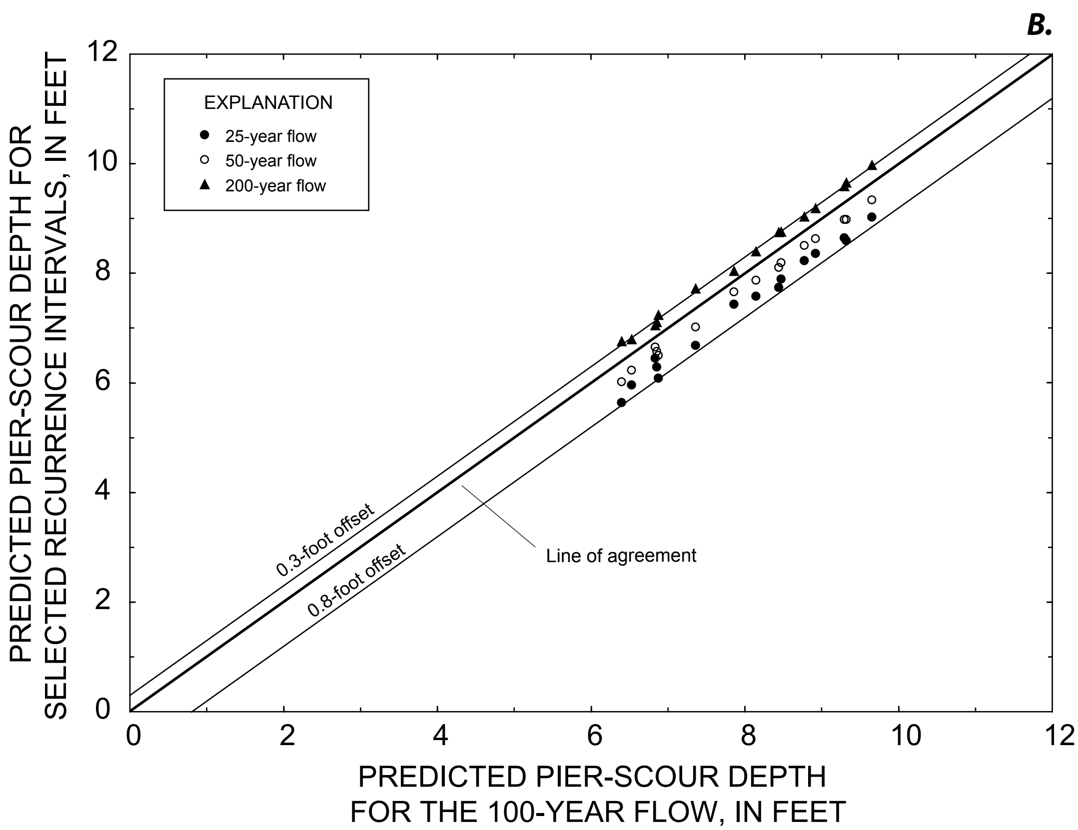


Figure 25. Relation of theoretical pier scour for the 25-, 50-, and 200-year flows with respect to the 100-year flow at selected bridges in South Carolina for (A) a pier width of 1.5 feet and (B) a pier width of 6 feet.

Time and Flow Duration

Laboratory investigations indicate that flow duration can be an important factor in the development of scour holes. Under live-bed conditions (sediments being transported into the scour area), scour reaches equilibrium scour depths much more rapidly than under clear-water conditions (no sediment transport into the scour area; fig. 26; Richardson and Davis, 2001). (Mueller (1996) cites laboratory studies of pier scour in which equilibrium scour depths for live-bed conditions were reached in hours in contrast to days for clear-water scour.) In the field setting, streamflows may peak and recede within hours rather than days. Therefore, if peak streamflows have relatively short durations, it is unlikely that clear-water equilibrium scour depths can be achieved in one flood event; rather, a succession of floods may be required to achieve this end (Richardson and Davis, 2001). Because the probability of multiple 100-year flows occurring during the life of a bridge is low, it is improbable that the clear-water equilibrium scour depth associated with the 100-year flow can be achieved at such sites.

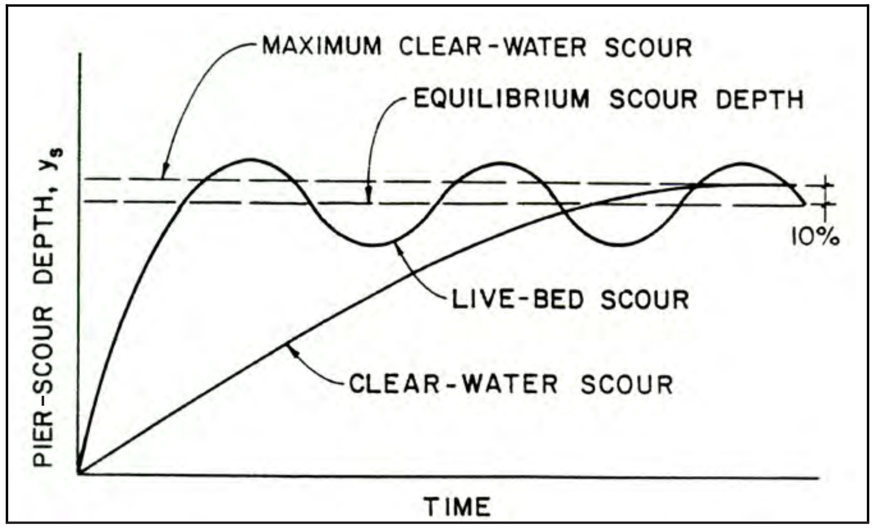


Figure 26. Generalized relation of pier-scour depth to time (from Richardson and Davis, 2001).

Because clear-water pier scour is the primary focus of this investigation, it is important to gain some understanding of flow durations under field conditions in South Carolina. Using a hypothetical 200-mi^2 basin and regionalized dimensionless hydrographs (Bohman, 1990) for the 100-year-flow, Benedict (2003) presented the contrast between flow hydrographs in the Coastal Plain and Piedmont of South Carolina (fig. 27), which shows that the Piedmont hydrograph is much shorter in duration than the Coastal Plain hydrograph. Therefore, scour depths for a single flood in the Coastal Plain are more likely to approach equilibrium scour depths for peak flows than are those in the Piedmont.

Assuming that the hydrograph duration for 95 percent of the 100-year flow (fig. 28) represents the duration of the peak flow, Benedict (2003) concluded that South Carolina drainage basins less than 2,000 mi^2 are unlikely to have sustained flow durations of 2 days for flows approaching the 100-year magnitude. Drainage areas for bridges used in the current study range from 11.5 to 12,990 mi^2, and 88 percent of the drainage areas are less than 2,000 mi^2. Because of the shorter flow durations associated with drainage basins less than 2,000 mi^2, scour depths collected in this study likely will not have reached equilibrium scour depths as associated with scaled laboratory studies of clear-water scour. In support of this conclusion, Melville and Coleman (2000) note that under field conditions "where clear-water scour conditions exist, the equilibrium depth of scour may be too conservative."

These observations highlight the fact that equilibrium scour depth is a laboratory-derived concept that often will not fully represent clear-water scour trends in the field. Basin characteristics in conjunction with regional hydrology strongly influence flow duration, which in turn determines if equilibrium scour depths can be achieved. The characteristics of many basins will promote flow durations insufficient to allow clear-water scour to achieve equilibrium scour depth

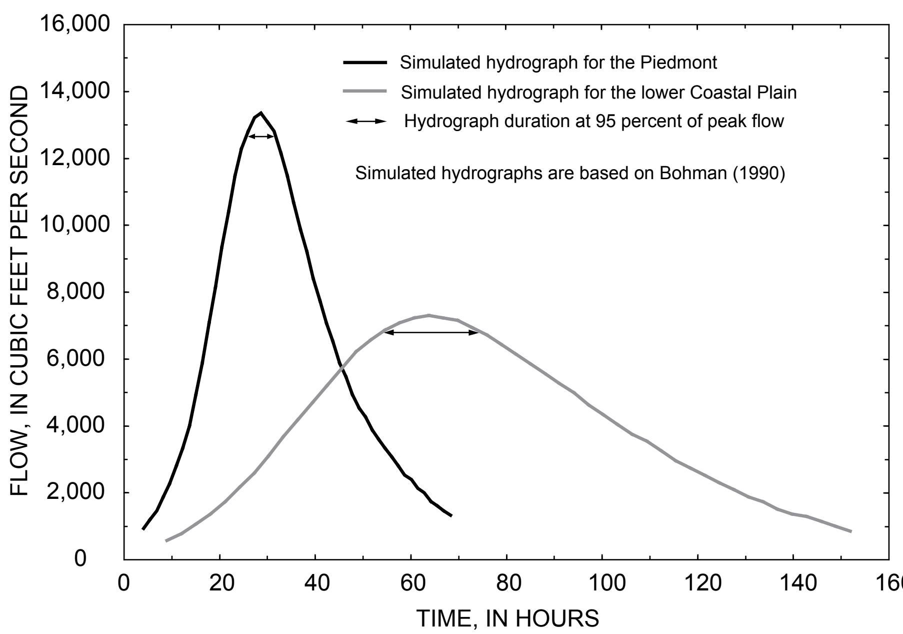


Figure 27. Simulated 100-year-flow hydrographs for 200-square mile basins in the Piedmont and lower Coastal Plain Physiographic Provinces of South Carolina (from Benedict, 2003).

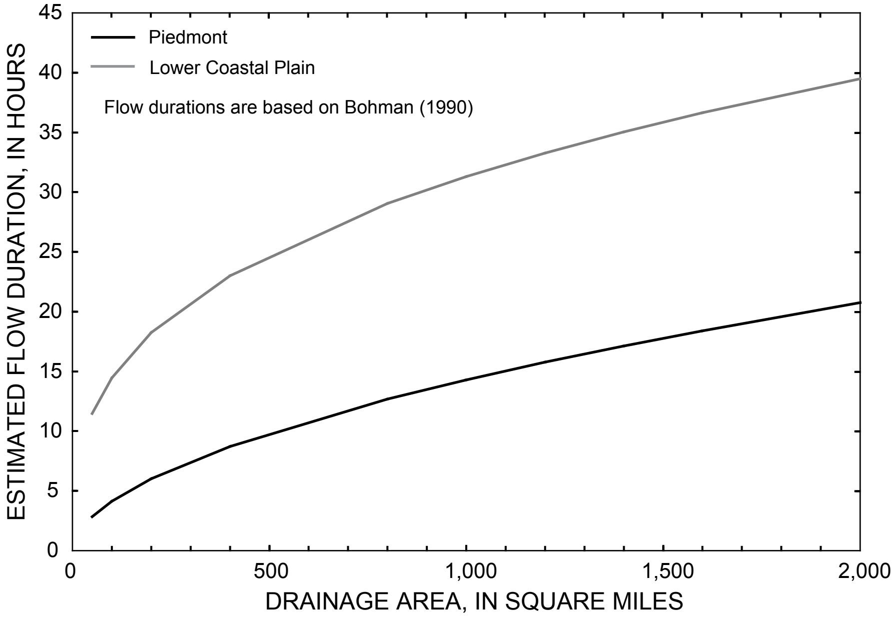


Figure 28. Hydrograph durations at 95 percent of the peak flow estimated from simulated 100-year-flow hydrographs for various basin sizes in the Piedmont and lower Coastal Plain Physiographic Provinces of South Carolina (from Benedict, 2003).

associated with steady-flow laboratory investigations. The scour depths in the field, however, will achieve a type of equilibrium depth that is a function of the flow characteristics of the basin. The basin equilibrium depth for clear-water scour can be less than equilibrium depth under steady-flow conditions. However, the basin equilibrium depth represents typical values of scour in the field providing insight to anticipated scour depths at other sites with similar characteristics.

To provide perspective on the relation of pier-scour depth and peak-flow duration for field data, a graphical relation of measured pier-scour depth and the estimated peak-flow duration for the 100-year flow in the Coastal Plain and Piedmont of South Carolina is shown in figure 29A and B, respectively. The peak-flow duration for each site was estimated by using methods presented in Bohman (1990), assuming that the hydrograph duration at 95 percent of the 100-year flow represents the duration for the 100-year peak flow. Because pier width is a dominant explanatory variable, data were grouped by selected pier widths. As can be seen in figure 29, there is a large scatter of data within the pier-width categories making it difficult to identify any strong correlation between flow duration and measured scour. However, in the case of pier widths less than or equal to 1.5 ft (which includes approximately 80 percent of the measured pier scour in this study), the upper bound of measured scour appears to be relatively flat, indicating that flow duration likely has only minor influence on clear-water pier-scour depths in South Carolina. It is noteworthy that all estimated peak-flow durations for the simulated hydrographs are less than 22 hours; most durations are less than 14 hours for the Coastal Plain (fig. 29A) and less than 7 hours for the Piedmont (fig. 29B). Such flow durations likely are insufficient to achieve equilibrium scour depths associated with laboratory investigations that run for several days or longer.

Flow Velocity

Results of laboratory investigations indicate that clear-water pier-scour depths increase with increasing approach velocity (Dongol, 1993; Melville and Coleman, 2000; Richardson and Davis, 2001). A typical laboratory relation showing the general influence of flow velocity on equilibrium scour depths for uniform sediments is shown in figure 30. The shape of this curve is slightly different for non-uniform sediments because of the effect of streambed armoring, but the trends are similar. The vertical axis in figure 30 represents the relative scour, which is defined as the equilibrium scour depth normalized by the pier width. The horizontal axis represents flow intensity, which is defined as the ratio of the average approach-flow velocity to the critical average velocity required to initiate motion of a given sediment. For clear-water scour conditions, the flow intensity is less than 1; the transition from clear-water to live-bed scour occurs when the flow intensity equals 1. (For non-uniform sediments, the breakpoint between clear-water and live-bed scour is at flow intensities greater than 1.) The curve in the area of clear-water scour has a relatively steep slope, indicating that small changes in approach velocity can produce relatively large changes in scour depth when other variables remain constant.

The laboratory data used to develop the original HEC-18 pier-scour equation (Richardson and others, 1991) include 102 measurements from Chabert and Engeldinger (1956) and 19 unpublished measurements from Colorado State University. Only 75 of the 121 measurements were used in figure 31 to show the laboratory relation of relative scour depth to flow intensity. (Laboratory data were provided by J.S. Jones, Federal Highway Administration, written commun, October 2003.) The patterns in figure 31 are similar to those depicted in the generalized graph in figure 30, with a relatively steep slope in the region of clear-water scour and a flatter slope beyond the envelope curve peak. (The peak for the relative scour depth for the envelope curves in figure 31 does not occur at the expected flow intensity of 1, but rather near a value of 1.3. Information provided with the data was insufficient to determine the reason for this trend.) It is noteworthy that distinct envelope curves are shown for the data grouped by the pier widths of 50, 75, and 100 mm (fig. 31). This demonstrates the strong influence of pier width on pier-scour depth, leading researchers to conclude that pier width is a dominant explanatory variable (Mueller, 1996; Melville and

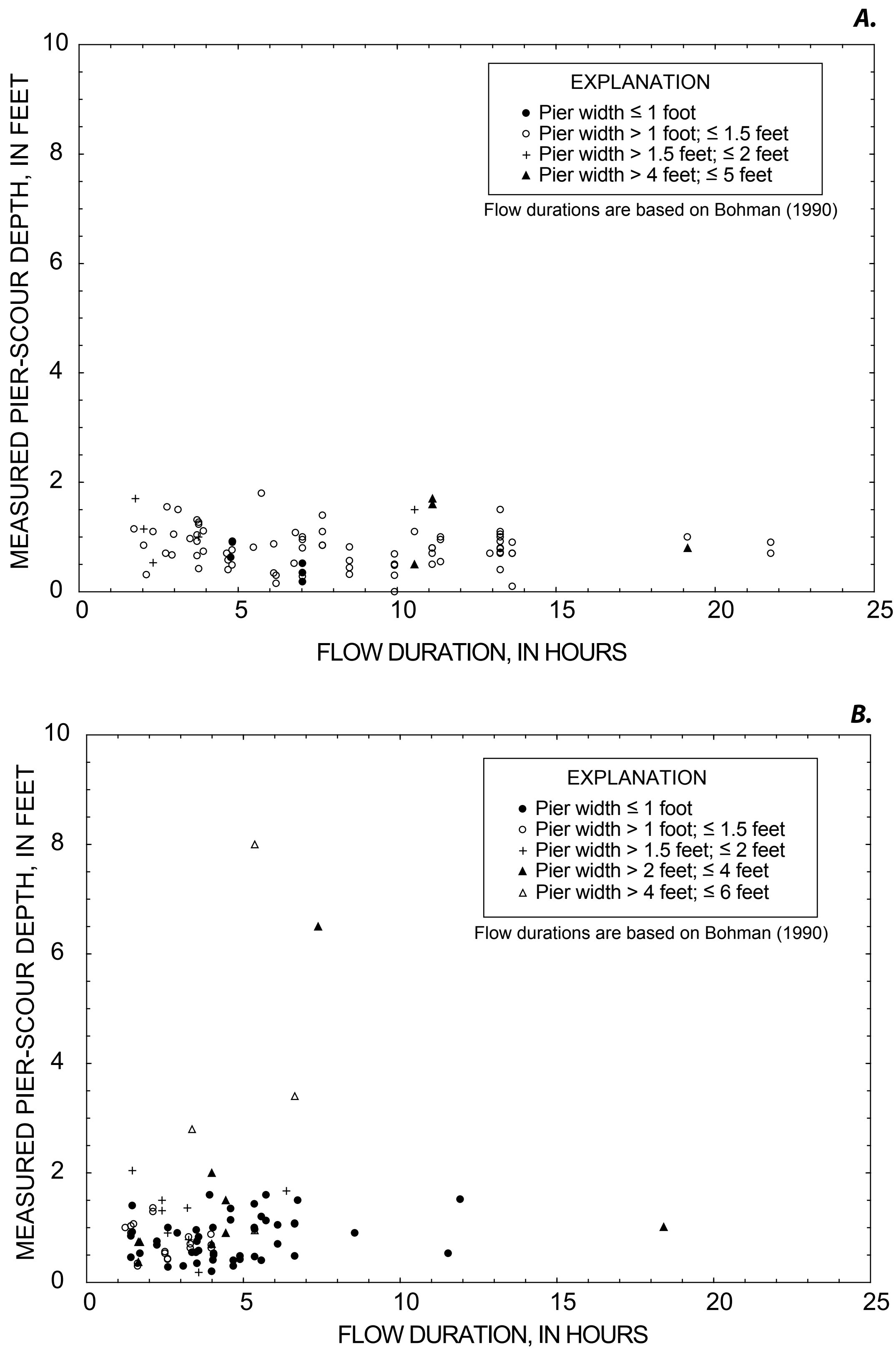


Figure 29. Relation of measured clear-water pier-scour depth to the estimated 95 percent peak flow duration for the 100-year flow at selected sites in the (A) Coastal Plain and (B) Piedmont Physiographic Provinces of South Carolina.

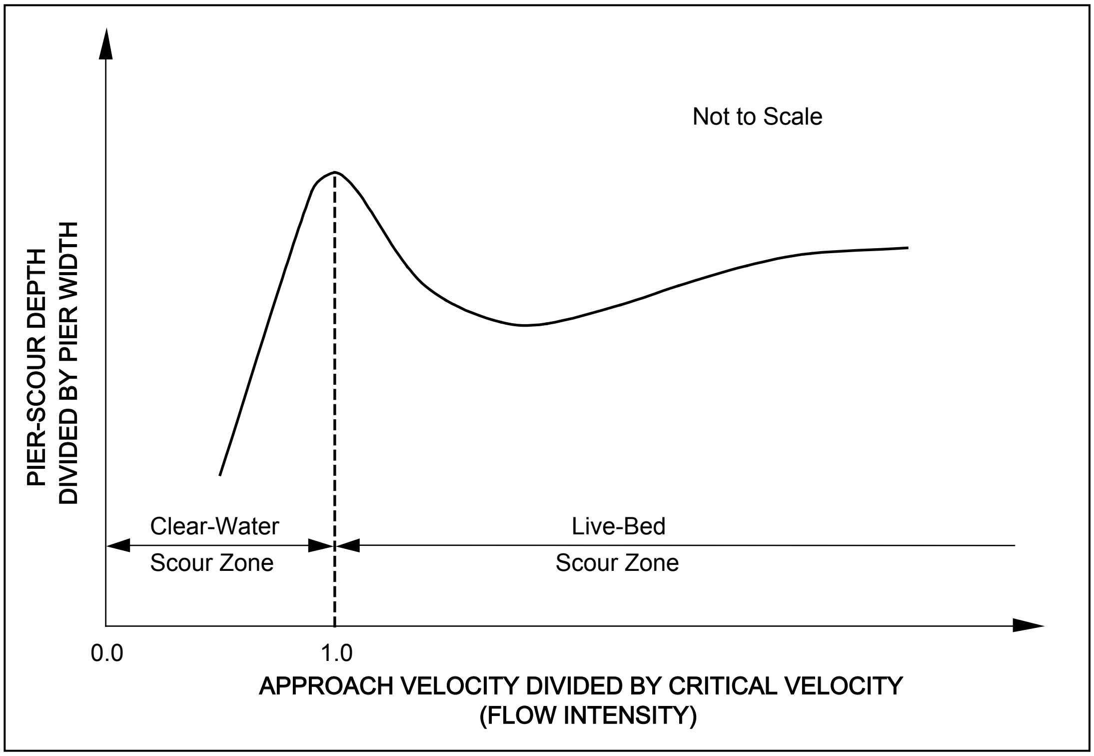


Figure 30. Generalized relation of flow intensity to relative pier scour based on laboratory investigations (from Dongol, 1993).

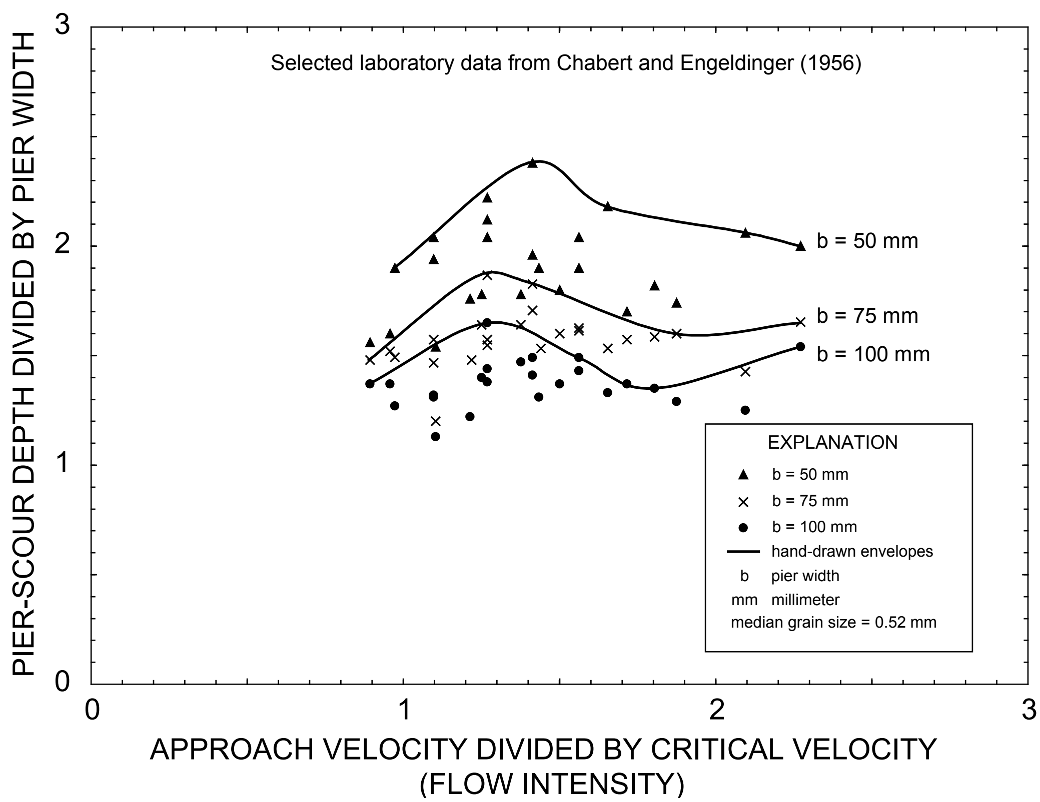


Figure 31. Relation of flow intensity to relative pier scour for selected laboratory data used to develop the original HEC-18 pier-scour equation (Richardson and others, 1991).

Coleman, 2000). The trends in figure 31 indicate that as pier width increases, relative pier scour decreases. This initially may seem counter intuitive, because one expects (and the data clearly demonstrate) that as pier width increases, scour depth also increases. Figure 31, however, displays the relation for relative scour, not actual scour, and the trend for relative scour is the reverse of actual scour. For example, the maximum relative scour for the 50- and 100-mm piers in figure 31 is 2.4 and 1.6, respectively. When the relative scour is converted to actual scour by multiplying by the respective pier widths, the actual scour for the 50- and 100-mm piers is 120 and 160 mm, respectively. To avoid potential confusion, it is important to keep the distinction between relative and actual scour in mind when reviewing the results of this investigation.

The relation of relative scour and flow intensity for field data in the current investigation is shown in figure 32 along with the laboratory data used to develop the original HEC-18 equation. The flow intensity for the field data was estimated by determining the ratio of the modeled 100-year approach-flow velocity and the critical velocity for soil conditions at each bridge. Determining critical velocity for natural soils that have some measure of consolidation and cohesion is difficult. Therefore, two different methods for estimating critical velocity (and, in turn, flow intensity) were used, including (1) the HEC-18 critical velocity equation for loose-grain sediments (Richardson and Davis, 2001) and (2) permissible velocities for natural soils as discussed in Fortier and Scobey (1926). These methods are described briefly below and the results for both methods are presented in figure 32.

The critical velocity equation published in HEC-18 (Richarson and Davis, 2001) was developed from laboratory data for loose-grained sediments and tends to underestimate critical velocities for floodplain soils of South Carolina. The HEC-18 critical velocity equation provides relatively large flow intensities for the South Carolina field data (fig. 32A), with approximately 90 percent of flow intensities exceeding 1 and a flow intensity as high as 7.2. Because all field data are clear-water scour in nature, flow intensities theoretically should be 1 or less. Therefore, the trends in figure 32A indicate that the HEC-18 critical velocity equation may yield critical velocities that are too low for the floodplain soils of South Carolina.

To reduce the potential for underprediction of critical velocity for South Carolina floodplain soils, permissible velocities as described in Fortier and Scobey (1926) also were used (fig. 32B). Permissible velocities are estimates of the maximum velocity that a given type of natural soil can sustain without severe erosion. Because natural soils are often consolidated and have some measure of cohesion, they can sustain a higher velocity than the critical velocities associated with loose-grain sediments. Permissible velocities, therefore, may better represent incipient motion conditions for the natural floodplain soils of South Carolina. However, the use of permissible velocities is subjective, introducing some measure of error in its application. In general, a permissible velocity of 2.5 ft/s adjusted for flow depth was used to represent the critical velocity for the sandy floodplain soils of the Coastal Plain and a permissible velocity of 5 ft/s adjusted for flow depth was used to represent the critical velocity for the clayey floodplain soils of the Piedmont. Although the estimates of critical velocity, using permissible velocities, may provide more reasonable estimates for flow intensities, there is certainly error in these estimates and this should be kept in mind when reviewing the relation in figure 32B.

Although the flow intensities for the field data in figure 32 introduce some error in the relation, there are some trends worth noting. First, regardless of the method used to estimate critical velocity, the envelope curves of the laboratory and field data in figure 32 have similar shapes. The upper bound steeply rises to a peak near a flow intensity of 1 and then decreases as flow intensity exceeds the peak. This similarity shows that the field data follow the general anticipated trend for pier scour, indicating that the collected field data likely will provide a reasonable representation of pier-scour trends in South Carolina.

Secondly, the relative scour for the majority of the laboratory data was greater than 1, while the relative scour for the majority of the field data was less than 1. The average relative scour for the laboratory data was 1.4, while the average relative scour for the field data was 0.7. Perhaps a primary reason for this trend stems from the difference in soil conditions. The laboratory soils were loose-grain sediments that are more easily eroded than the natural floodplain soils in the field. Additionally, the use of smaller pier widths in laboratory experiments, which tends to yield higher relative scour than the wider pier widths associated with the field data also may promote this trend. Because the HEC-18 pier-scour equation (eq. 1) was developed from the laboratory data, which has a higher relative scour than the field data in this investigation, it is likely that the HEC-18 equation will overpredict clear-water pier scour for many sites in South Carolina. This is a desirable trend for design as long as overprediction is not excessive.

To provide some perspective on the relation of pier-scour depth and approach velocity for non-normalized field data, the relation of pier-scour depth to the average 100-year flow velocity approaching the pier was examined for the Coastal Plain (fig. 33A) and Piedmont (fig. 33B) of South Carolina. Because pier width is a strong explanatory variable, data were grouped by selected pier widths. Additionally, because 80 percent of the field data have pier widths of 1.5 ft or less, hand-drawn envelope curves of those data are included. The envelope curves represent an upper-bound potential of pier-scour depth for pier widths of 1.5 ft or less over the given range of flow velocity. It is interesting to note that the envelope curves shown in figure 33 indicate that clear-water pier-scour potential is at its lowest for smallest velocities, and increases as velocity increases over a range of approximately 0.5 to 2 ft/s. Additionally, the rate-of-change in the upper bound of pier scour is greatest over this range of velocity, indicating that flow velocity within this range is a strong explanatory variable having a large influence on the upper bound of pier-scour depth for pier widths of 1.5 ft or less.

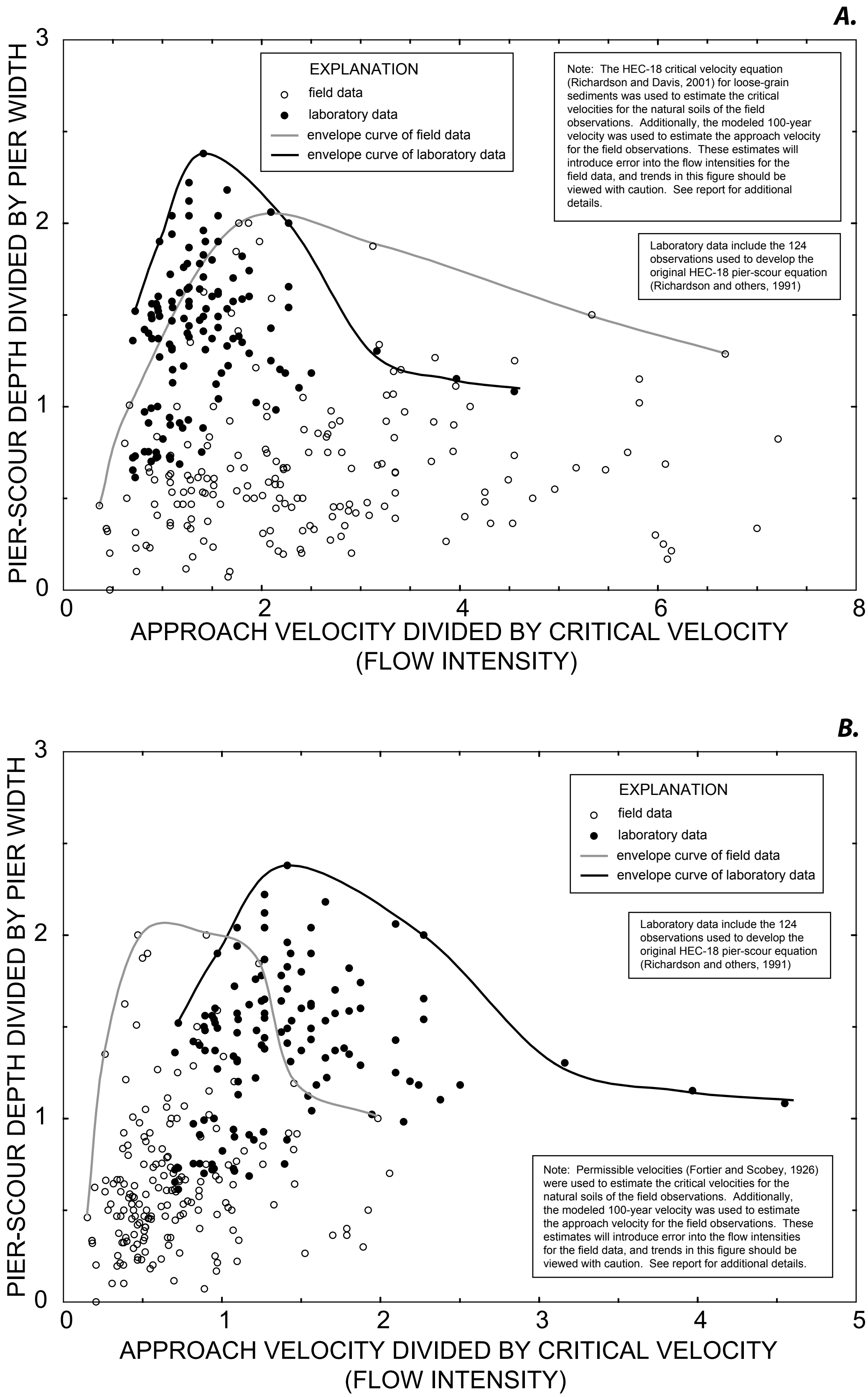
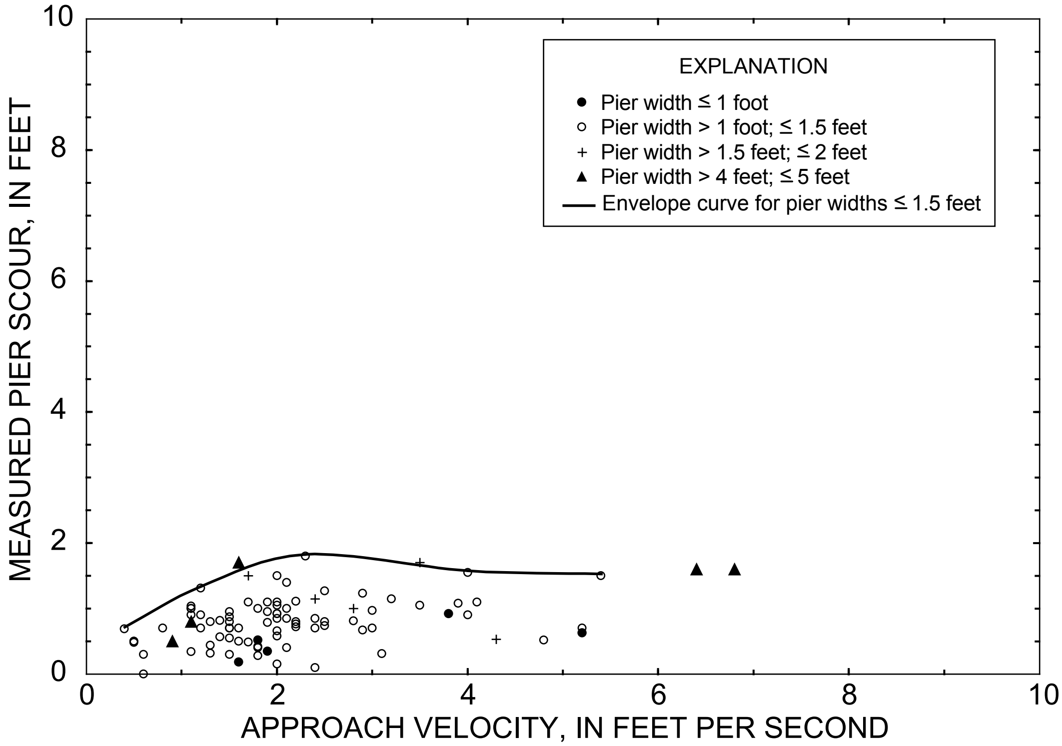


Figure 32. Relation of flow intensity to relative pier-scour for selected laboratory data and field data in South Carolina, using (A) loose-grain critical velocities and (B) permissible velocities to determine flow intensities for the field data.

A.



B.

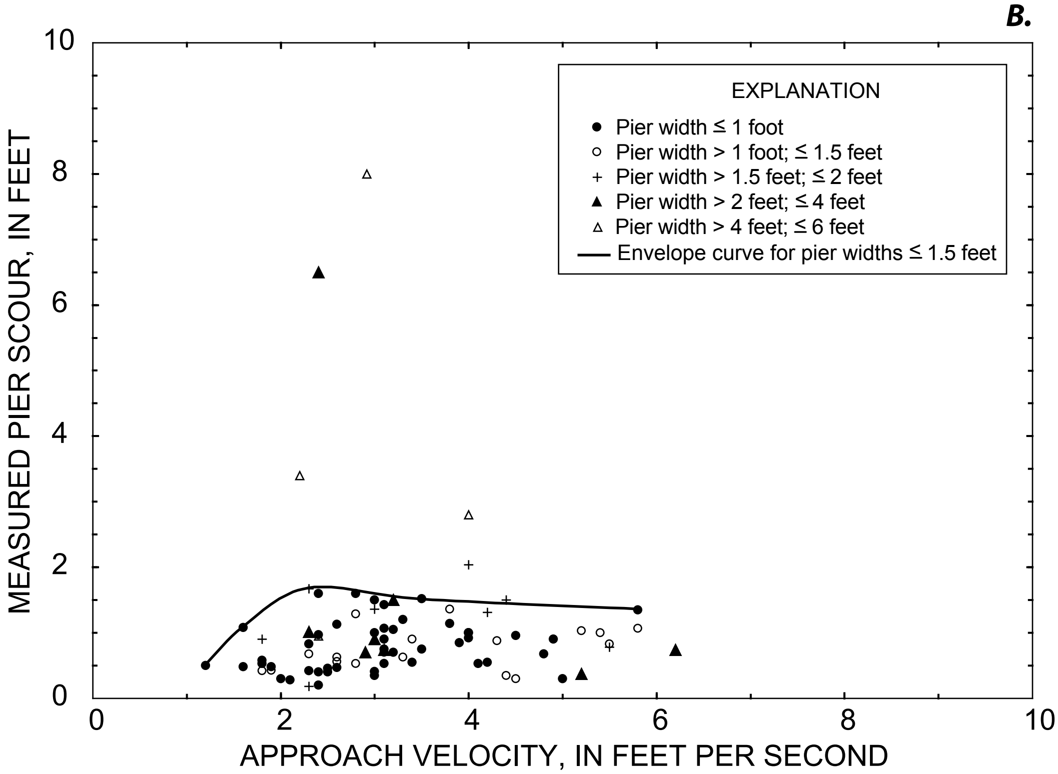


Figure 33. Relation of measured clear-water pier-scour depth to the 100-year flow approach velocity at selected sites in the (A) Coastal Plain and (B) Piedmont Physiographic Provinces of South Carolina.

When flow velocity reaches approximately 2 ft/s, the potential for scour is at its maximum and remains relatively constant as velocity increases beyond this value. This indicates that when flow velocity exceeds 2 ft/s, it becomes a weak explanatory variable having little influence on the upper bound of pier-scour depth for pier widths of 1.5 ft or less. The trends shown in figure 33 can be useful in understanding the potential for scour for various flow velocities. When velocities are below 2 ft/s (which is more common in parts of the lower Coastal Plain of South Carolina), the potential for pier scour will be diminished for pier widths of 1.5 ft or less. In contrast, when flow velocity is greater than 2 ft/s, the potential for scour is at a maximum and, generally, will not increase with increasing velocity. The measured data for pier widths greater than 1.5 ft are sparse, making it difficult to draw strong conclusions about the influence of velocity at such piers. However, it is reasonable to assume that similar trends exist for pier widths greater than 1.5 ft, and some anecdotal observations are noted later in this report section.

The USGS National Bridge Scour Database (NBSD) [http://water.usgs.gov/osw/techniques/bs/BSDMS/index.htm accessed February 6, 2004] is a compilation of bridge-scour field measurements made at 91 sites in 19 states. The NBSD contains 503 pier-scour measurements and typically includes measurements of the hydraulic conditions that existed at the time the scour measurement was made. These data were a valuable resource to help verify the pier-scour trends in the current investigation. (The NBSD contains some pier-scour measurements that were influenced by complicating field conditions such as debris, tides, complex pier geometry, remnant scour holes, and severe skews. Therefore, the data must be carefully reviewed and screened when used in an analysis.) Pier-scour measurements with pier widths less than or equal to 1.5 ft, no influence from debris, and sediment sizes and velocities in the range of the South Carolina data were selected from the NBSD. From this selection process, 50 measurements of pier scour were obtained; however, 43 of these measurements were affected by pier skew. In order to remove the effect of pier skew, the skew coefficient from the HEC-18 pier-scour equation (Richardson and Davis, 2001) was computed and applied to each measurement. The skew coefficient for the HEC-18 pier-scour equation is likely conservative (Ettema and others, 1998); therefore, scour depths from the NBSD adjusted for skew will likely produce underestimates of non-skewed pier-scour depths. Regardless of this limitation, the NBSD data are an independent source of data that provide some insights regarding the reasonableness of the South Carolina data.

The relation of pier-scour depth and approach-flow velocity for the selected NBSD data was examined along with the envelope curves for the Coastal Plain and Piedmont of South Carolina (fig. 34). The measured scour associated with the skewed pier data is shown in figure 34A, and approximately 80 percent of the data falls within or near the envelope curves, indicating that even the measurements affected by skew fall near the range of the South Carolina data. The measured scour adjusted to remove the effects of skew is shown in figure 34B, and the majority of the NBSD data fall within the South Carolina envelope curves. The similarities in the range and trend of the South Carolina and NBSD data indicate that the South Carolina data are reasonable and represent the general trend of clear-water pier scour in South Carolina.

It is also interesting to note that the Coastal Plain envelope curve shown in figure 34 is slightly higher than the Piedmont envelope curve. This slight difference is likely a result of the cohesive soils of the Piedmont, which are more scour resistant than the sandy soils of the Coastal Plain. The upper bound of the two envelope curves are very similar, however, indicating that (1) the influence of velocity on the upper bound of scour is similar between the two regions and (2) the influence of the Piedmont cohesive soils on the upper bound of scour is minimal. This trend does not mean that cohesive soils do not influence scour, but that within the Piedmont region, where soil cohesion varies, the data that form the upper bound are very similar to those of the Coastal Plain. However, at Piedmont sites where soil cohesion is very high, pier scour will likely be less than that of the sandy soils of the Coastal Plain. Because the upper bound trends are similar between the Piedmont and Coastal Plain, it is probably reasonable to combine data from these two regions when developing envelope curves for pier scour.

Figures 32 and 33 show the trend of the upper bound of pier scour for selected data in the Piedmont and Coastal Plain of South Carolina. Upper bound trends are frequently used in laboratory investigations and are of particular interest to the current investigation because of the focus on developing envelope curves for evaluating scour in South Carolina. However, it is also instructive to look at the average trends of the data to gain insights regarding the influence of a particular variable on pier scour. Using data for pier widths of 1.5 ft or less (as shown in figures 33 and 34), figure 35 shows the relation of pier-scour depth and approach-flow velocity for the NBSD (with the effect of skew removed) and the South Carolina data. The general scatter and trend lines for the Coastal Plain, Piedmont, and NBSD data are very similar, again indicating that the range and trend of the South Carolina data conform well with other field data and, therefore, should reasonably represent the general trend of scour in South Carolina. Because the trends of the South Carolina data are similar to those of the NBSD data which includes measured velocities rather than modeled velocities, use of the 100-year flow for estimating hydraulic conditions that may have created the measured scour in South Carolina is a reasonable approach. The trend lines in figure 35 are relatively flat (approximately 0.1 ft increase in scour per 1 ft/s increase in velocity), indicating that over the range of data in this investigation, the influence of velocity on pier-scour depths in South Carolina is small when pier widths are less than or equal to 1.5 ft.

The trend lines in figure 35 are understandably different from the upper bound envelope curves of figures 33 and 34. However, the trend lines and upper bound envelope curves both indicate that the influence of velocity on pier-scour

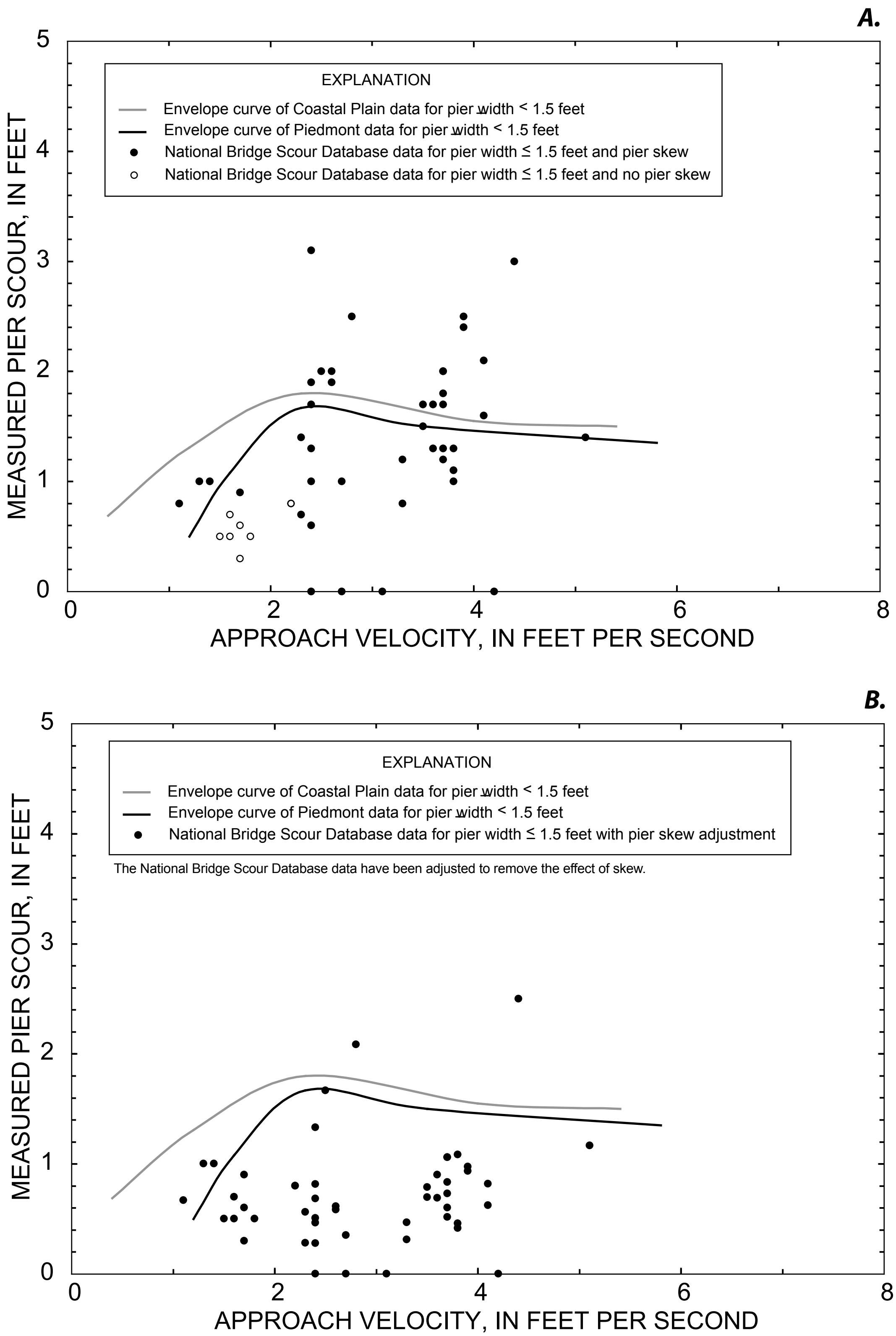


Figure 34. Relation of measured clear-water pier-scour depth to the approach velocity for field data from the National Bridge Scour Database with (A) skewed pier data and (B) data adjusted to remove the effect of skew, compared to the envelope curves of the South Carolina field data.

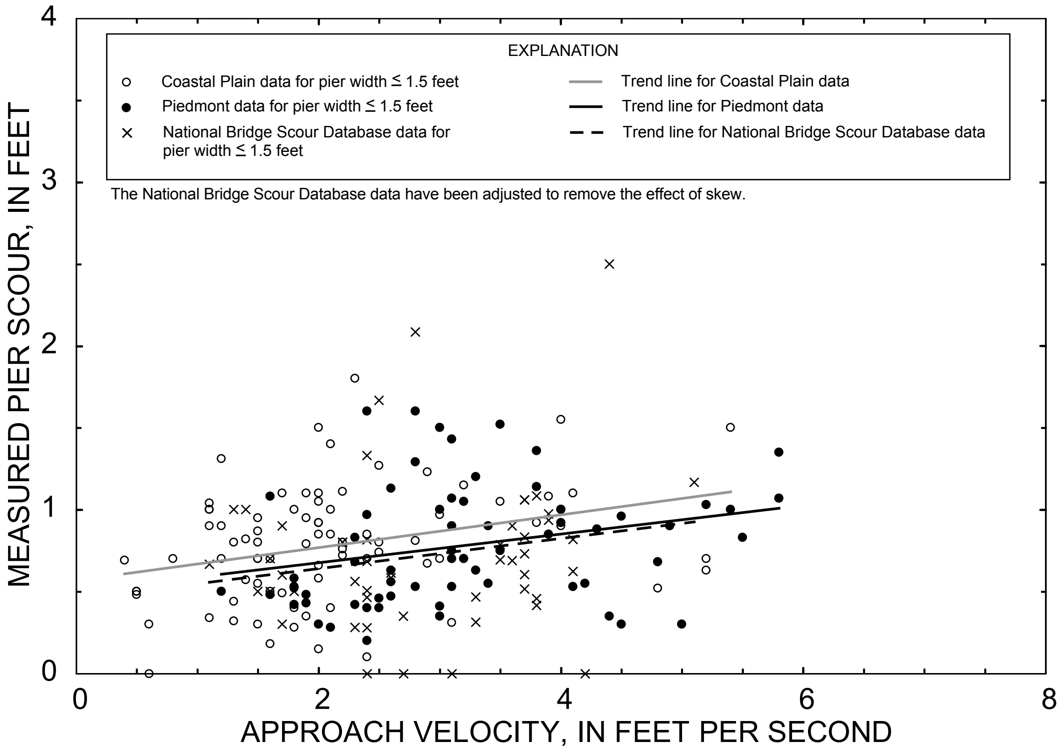


Figure 35. Relation of measured clear-water pier-scour depth to the approach velocity, and trend lines for selected field data from the National Bridge Scour Database and from the Coastal Plain and Piedmont Physiographic Provinces of South Carolina.

depths is small over much of the velocity range. In the case of the upper bound envelope curves, the influence of velocity on pier scour is more pronounced when velocities are approximately 2 ft/s or less, but beyond this value (where much of the data falls) the influence is small. Similarly, the trend lines of figure 35 (which represent the average trend over the full range of velocity) indicate that the influence of velocity on pier scour is generally small.

The measured data for pier widths greater than 1.5 ft are sparse, making it difficult to draw strong conclusions about the influence of velocity at such piers. It is reasonable to assume, however, that larger piers function in a manner similar to smaller piers where the rate of increase in the upper bound of scour is more pronounced for relatively small velocities and then diminishes significantly at some larger velocity. To provide perspective on the trend for the upper bound of scour at wider piers, selected data from the NBSD for pier widths ranging from 3.5 to 6 ft are displayed in figure 36, along with selected South Carolina data for pier widths ranging from 3.3 to 6 ft. (The NBSD data in figure 36 include measurements with no skew, no significant effect from debris, and median sediment sizes ranging from fine to medium sand.) The data in figure 36 have a larger upper bound for pier-scour depth than that of figure 33, which primarily can be attributed to larger pier widths. However, there are similar trends between the envelope curves of figure 33 and 36 in that the rate-of-change in the upper bound of pier scour is more sensitive to changes in velocity for relatively small velocities (approximately 2 ft/s and less) and less sensitive for velocities greater than 2 ft/s. It also is worth noting that the range of the South Carolina scour data is similar to that of the NBSD, indicating the range of the South Carolina data for scour at piers wider than 1.5 ft is reasonable.

Anecdotal evidence indicating the effects of flow velocity on pier-scour depths at wider piers can be seen by comparing pier scour at structure 304004900400 on S.C. Route 49 crossing the Enoree River with pier scour at structure 262050103100 on U.S. Route 501 crossing the Waccamaw River. Large flows have occurred at these bridges. The Waccamaw River had a peak flow near the 100-year flow at the U.S. Route 501 crossing in 1999, and the Enoree River had flow approximately 2 times the 100-year flow at the S.C. Route 49 crossing in 1995 (table 2). During the 1999 flood, the U.S. Route 501 crossing of the Waccamaw River had a 5-ft wide pier with a measured scour depth of 0.5 ft and an estimated approach-flow velocity of 0.9 ft/s. In contrast, the S.C. Route 49 crossing of the Enoree River had a 4-ft wide pier with a measured scour depth of 6.5 ft and an estimated approach-flow velocity of 6.6 ft/s during the 1995 flood. From these two data points, it could be concluded that there is a linear relation between velocity and pier-scour depth where pier-scour depth increases proportionally with velocity. However, the envelope curves of figures 33 and 36 indicate that the upper-bound potential for pier scour varies with velocity having lower

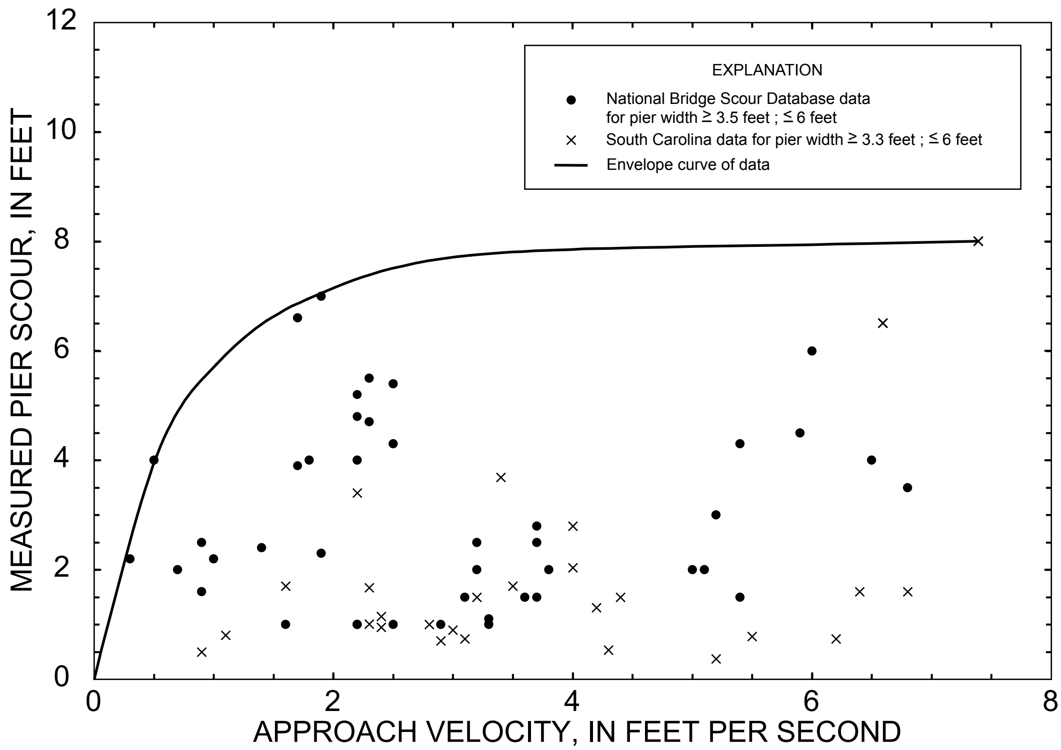


Figure 36. Relation of measured clear-water pier-scour depth to the approach velocity for selected data from the National Bridge Scour Database and the South Carolina field data for pier widths greater than 1.5 feet but less than or equal to 6 feet.

scour potential but a large rate-of-change at flow velocities of about 2 ft/s or less, and a relatively constant (and maximum) potential beyond about 2 ft/s. Although pier-scour data from the U.S. Route 501 crossing of the Waccamaw River does not fall along the envelope curve in figure 36, it is in the area of the curve where flow velocities are less than 2 ft/s and the potential of the upper bound of scour is diminished, providing some justification for the small pier-scour depth at this site. In contrast, the larger pier-scour depth for the S.C. Route 49 crossing of the Enoree River is in the area of the envelope curve (fig. 36) where flow velocities are greater than 2 ft/s and the potential for scour is at its maximum. It should be kept in mind that factors other than velocity have likely contributed to the development of scour at these sites, making it difficult to isolate the effect of flow velocity on pier scour at wide piers. The comparison, however, provides some understanding of how small flow velocities less than 2 ft/s, such as those of the Waccamaw River, tend to produce smaller amounts of scour at pier widths greater than 1.5 ft, in contrast to larger scour depths, such as those of the Enoree River, where flow velocities are relatively large.

Flow Depth

Results of laboratory investigations indicate that clear-water pier-scour depths increase at a diminishing rate with increasing flow depth (Dongol, 1993; Melville and Coleman, 2000). The relation between the relative scour depth (equilibrium scour depth normalized by the pier width) and relative flow depth (flow depth normalized by pier width) for laboratory data used to develop the original HEC-18 pier-scour equation (Richardson and others, 1991) is shown in figure 37. Because pier width is a strong variable influencing scour, the data are grouped by pier width, and envelope curves were hand drawn for each pier-width group. The envelope curves in figure 37 indicate that the upper-bound potential for relative scour is diminished for shallow flow depths (relative flow depths from approximately 0.5 to 2). However, the rate-of-change in the upper bound of relative scour is greatest over this same range. For deeper flows (relative flow depth greater than approximately 2), the potential for relative scour approaches a maximum and remains relatively constant as depth increases. This indicates that for flow depths greater than approximately 2 times the pier width, the potential for pier scour is near a maximum and flow depth will have little influence on pier-scour depth. Based on this trend, researchers have suggested that when relative flow depths are approximately 4 or greater, the influence of flow depth on pier-scour depths is negligible. Breusers and others (1977) suggest a relative flow depth of 3; Chiew (1984) suggests a value of 4; and Melville and Coleman (2000) suggest a value of 5.

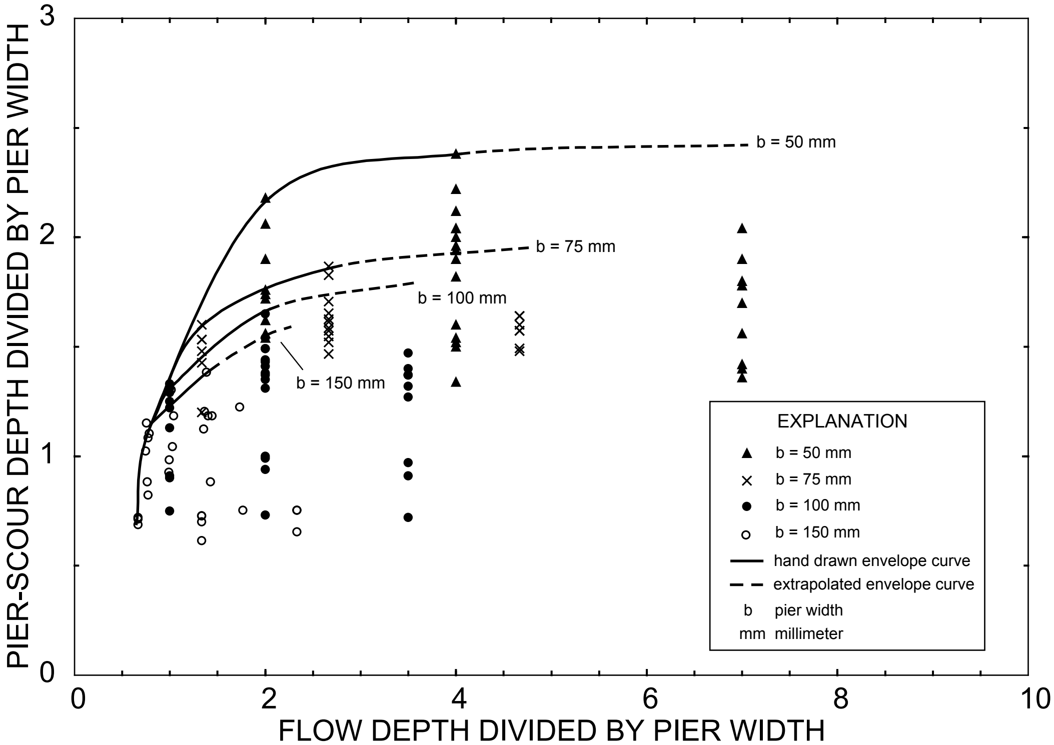


Figure 37. Relation of relative flow depth to relative pier scour for laboratory data used to develop the original HEC-18 pier-scour equation (Richardson and others, 1991).

The relation of relative scour to relative flow depth for laboratory data used to develop the original HEC-18 pier-scour equation (Richardson and others, 1991) and for field data from the current investigation was evaluated (fig. 38). The relative flow depth for the field measurements was estimated by taking the ratio of the modeled 100-year approach-flow depth to the pier width. The envelope curves encompassing the laboratory and field data have similar shapes as well as similar upper bounds. This similarity indicates that the field data follow the general anticipated trends for pier scour, which further indicates that the collected field data reasonably represent the pier-scour trends in South Carolina. The range of relative flow

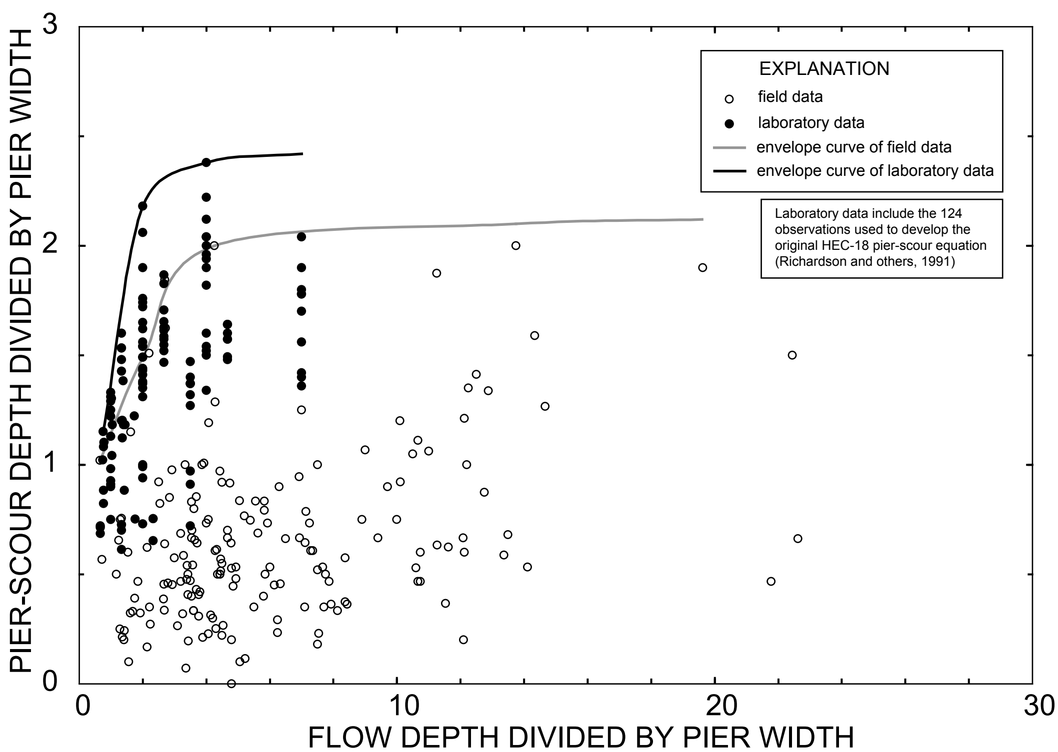


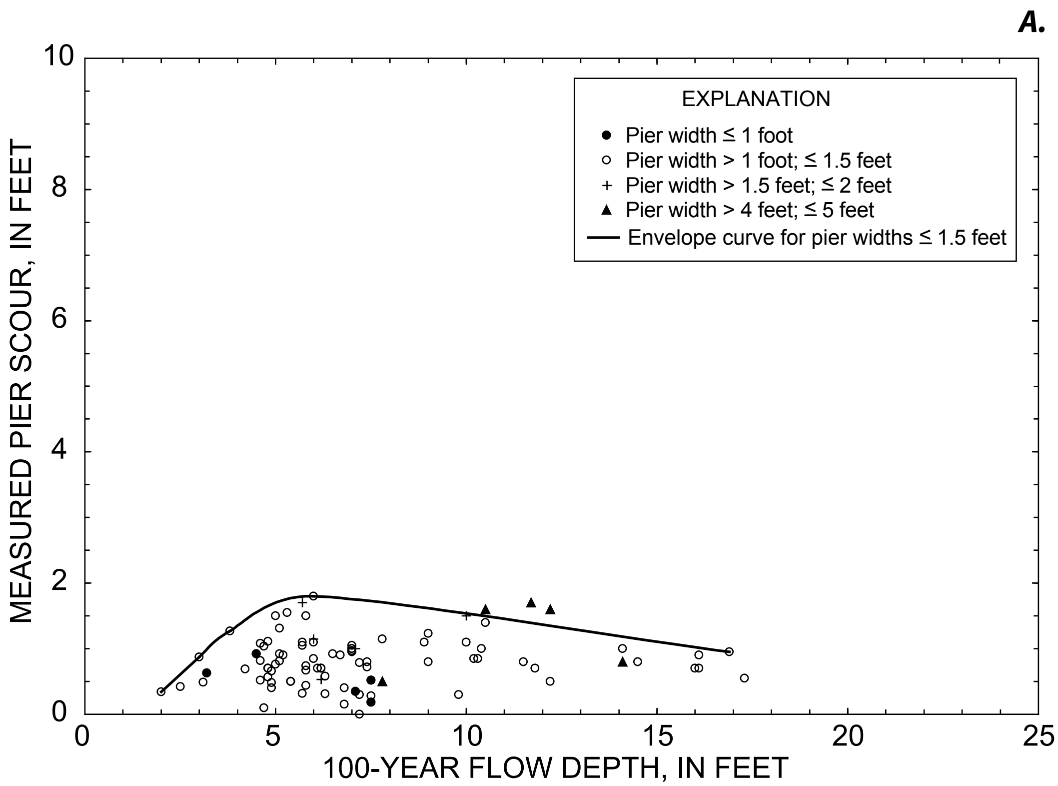
Figure 38. Relation of relative flow depth to relative pier scour for laboratory data used to develop the original HEC-18 pier-scour equation (Richardson and others, 1991) and field data in South Carolina.

depths for field data is much larger than the range of laboratory data, indicating that the laboratory data do not encompass the relative flow depths found in the field. However, because relative flow depths greater than about 4 have negligible influence on pier-scour depths, this deficiency in the laboratory data may be moderate in nature. Approximately 62 percent of the field measurements in this study had relative flow depths of 4 or greater, indicating that many sites in South Carolina have flow depths during large flows that provide the maximum potential for pier-scour depth and yet have minimal influence on the change in that potential as flow depth increases.

To provide perspective on the relation of pier-scour depth and approach-flow depth for non-normalized field data, the relation of pier-scour depth and the average 100-year approach-flow depth for the Coastal Plain and Piedmont of South Carolina were examined (fig. 39). Because pier width is a strong explanatory variable, data were grouped by selected pier widths. Additionally, because 80 percent of the observed pier widths were 1.5 ft or less, a hand-drawn envelope curve of these measurements is included. (In figure 39B, there are relatively few pier-scour measurements with pier widths between 1 and 1.5 ft, and the more numerous measurements with pier widths less than or equal to 1 ft form the boundary of the envelope curve. However, all measurements for pier widths of 1.5 ft or less are encompassed by the envelope curve.) It is interesting to note that the envelope curves of the measured scour for pier widths of 1.5 ft or less indicate that rate-of-change in the upper bound of scour depth is more strongly influenced by flow depths over a range of approximately 1 to 5 ft in the Coastal Plain and 1 to 3 ft in the Piedmont. When these ranges are exceeded, the influence of flow depth on the rate-of-change in the upper bound of pier-scour depth diminishes. This trend can be useful in understanding the potential for scour under various hydraulic conditions. When flow depths are below about 5 ft (and pier widths are 1.5 ft or less), the potential for pier scour is diminished, and for depths greater than 5 ft, the potential for pier scour tends to be at its maximum. The measured data for pier widths greater than 1.5 ft is sparse making it difficult to develop upper-bound envelope curves for pier widths greater than 1.5 ft, and conclusions about the influence of flow depths at such piers cannot be made.

The relation of pier-scour depth and flow depth for selected NBSD data, as described previously in the section “Flow Velocity,” along with the envelope curves for the Coastal Plain and Piedmont of South Carolina (fig. 39) was examined (fig. 40). The majority of the NBSD data (with the effect of skew removed) fall within the South Carolina envelope curves, indicating that the range and trend of the South Carolina data are reasonable and can be used to understand general scour patterns in South Carolina.

The trend of the upper bound of pier scour for selected data in the Piedmont and Coastal Plain of South Carolina is an important relation for understanding the potential for pier scour for various flow depths (fig. 39). However, it is also instructive to look at the average trends of the data to gain insights regarding the influence of a flow depth on pier-scour depth. Using data for pier widths of 1.5 ft or less, figure 41 shows the relation of pier-scour depth and flow depth for the NBSD (with the effect of skew removed) and South Carolina data. The trend lines in figure 41 are relatively flat (the steepest line having approximately 0.02 ft increase in scour depth per 1 ft increase in flow depth), indicating that over the range of flow depth in this investigation, the influence of flow depth on pier-scour depths in South Carolina is small when pier widths are less than or equal to 1.5 ft.



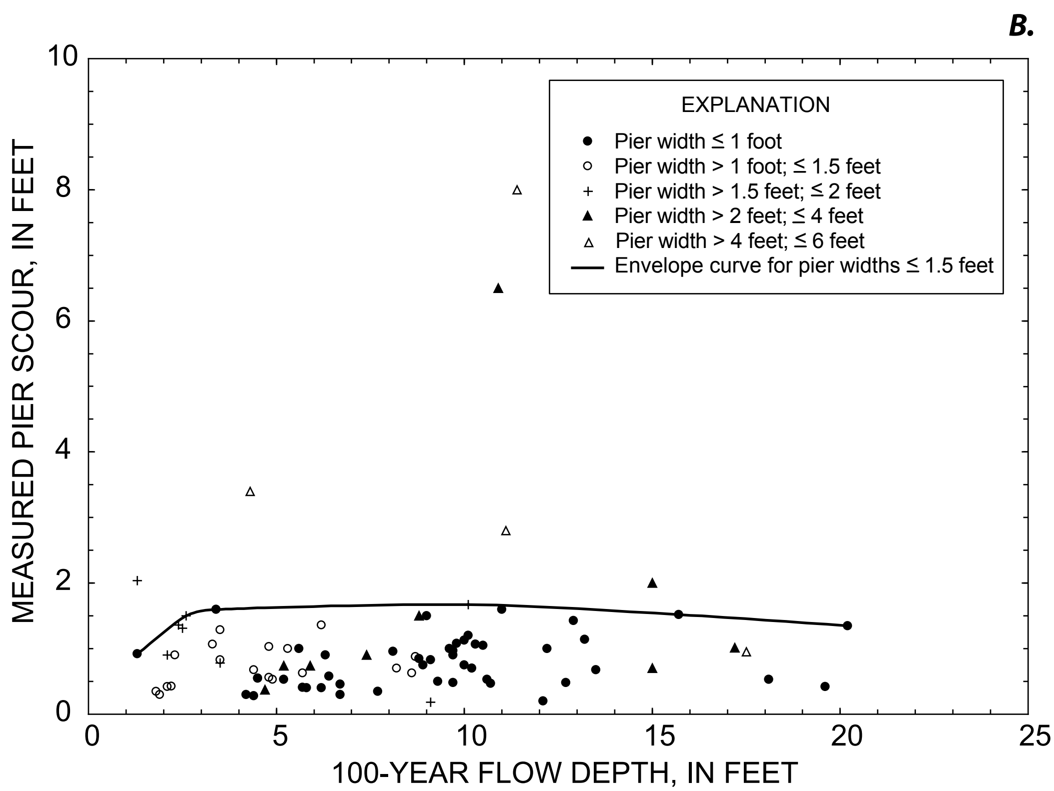


Figure 39. Relation of measured clear-water pier-scour depth to the approach depth for the 100-year flow at selected sites in the (A) Coastal Plain and (B) Piedmont Physiographic Provinces of South Carolina.

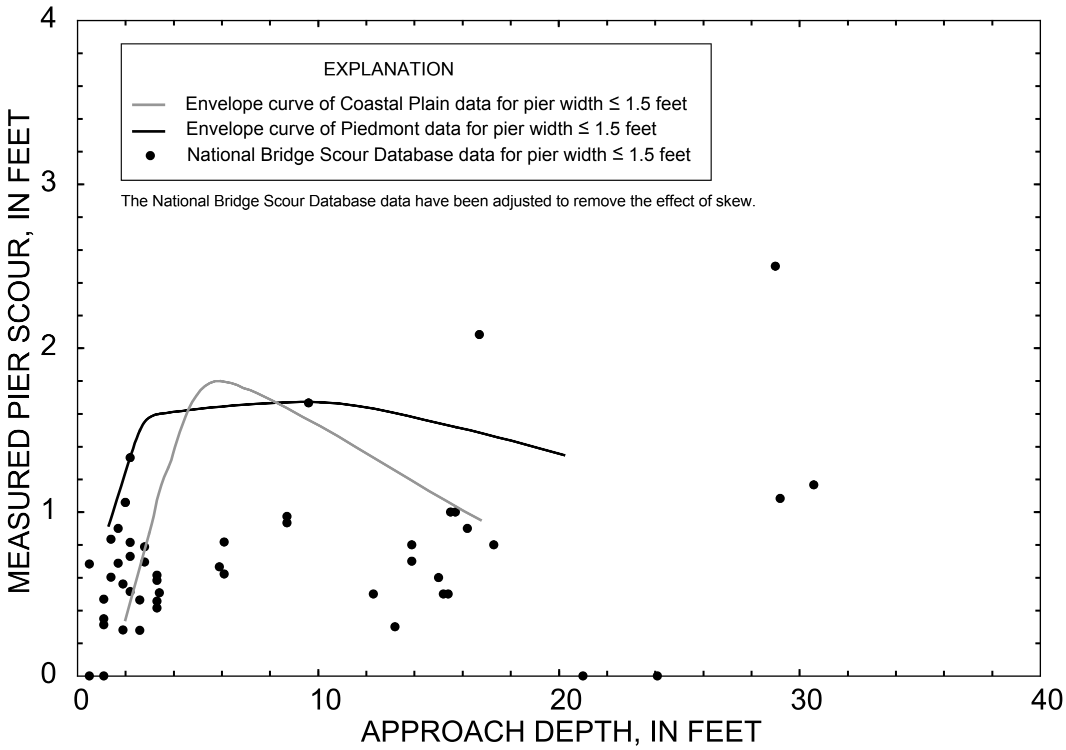


Figure 40. Relation of measured clear-water pier-scour depth to the approach flow depth for selected field data from the National Bridge Scour Database compared to the envelope curves of the South Carolina field data.

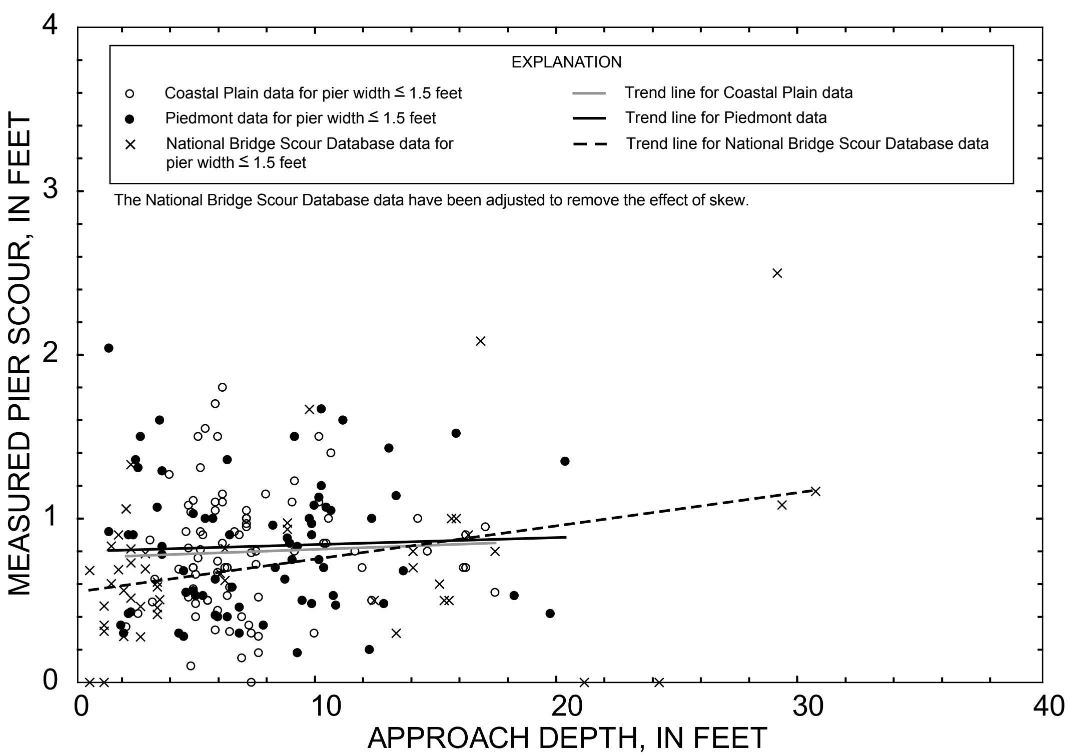


Figure 41. Relation of and trend lines for measured clear-water pier-scour depth to the approach flow depth for selected field data from the National Bridge Scour Database and from the Coastal Plain and Piedmont Physiographic Provinces of South Carolina.

Sediment Size

A typical laboratory relation from Melville and Coleman (2000) shows the general influence of sediment coarseness on equilibrium scour depths for uniform sediments (fig. 42). The shape of this curve is similar for non-uniform sediments, with the maximum scour being lower because of the effect of streambed armoring. The vertical axis in figure 42 represents the relative scour, and the horizontal axis represents the inverse of the relative grain size. Relative grain size is defined as the ratio of the median grain size (D_{50}) to the pier width (b). Figure 42 indicates that when the dimensionless ratio of b/D_{50} is less than about 50, the grain size is relatively coarse and pier-scour depths will be diminished. In contrast, when b/D_{50} is about 50 or greater, the grain size is relatively fine and the potential for scour is at a maximum. Figure 42 also indicates that when b/D_{50} is 50 or greater, the relative scour remains constant and variation in grain size (within the relatively fine sediment range) has no influence on equilibrium scour depths. These two trends have important implications for understanding the effect of sediment size on pier scour in South Carolina. Because sediment sizes in South Carolina are typically fine, the potential for scour (with respect to the influence of sediment size) is at its maximum and is relatively constant even when sediment size varies. Because the potential for scour in relatively fine sediments is constant regardless of grain size, sediment size is a weak explanatory variable for pier scour in South Carolina and can be neglected when developing relations or envelope curves that help explain pier-scour trends.

The laboratory data used to develop the original HEC-18 pier-scour equation (Richardson and others, 1991) included measurements for three grain sizes only: 0.24, 0.26, and 0.52 mm. Based on this limited range of grain size, a dimensionless plot of the laboratory data similar to figure 42 does not indicate strong trends. It is worth noting that the dimensionless variable b/D_{50} for the laboratory data ranges from 63 to 577, indicating that the sediments are relatively fine, providing the maximum potential for scour but having negligible influence on equilibrium scour depths as the sediment size varies.

Although soils in South Carolina are not uniform and commonly have some degree of cohesion, the dimensionless variable b/D_{50} provides some insight into the effect of sediment size on pier-scour depths in South Carolina. For the 177 measurements of pier scour in this study collected at 116 bridges, D_{50} ranges from 0.004 to 0.99 mm, pier widths range from 0.8 to 6 ft, and the dimensionless variable b/D_{50} ranges from 616 to 1.79×10^5. The range of b/D_{50} greatly exceeds 50, indicating that soils in South Carolina are relatively fine, providing conditions that promote the maximum potential

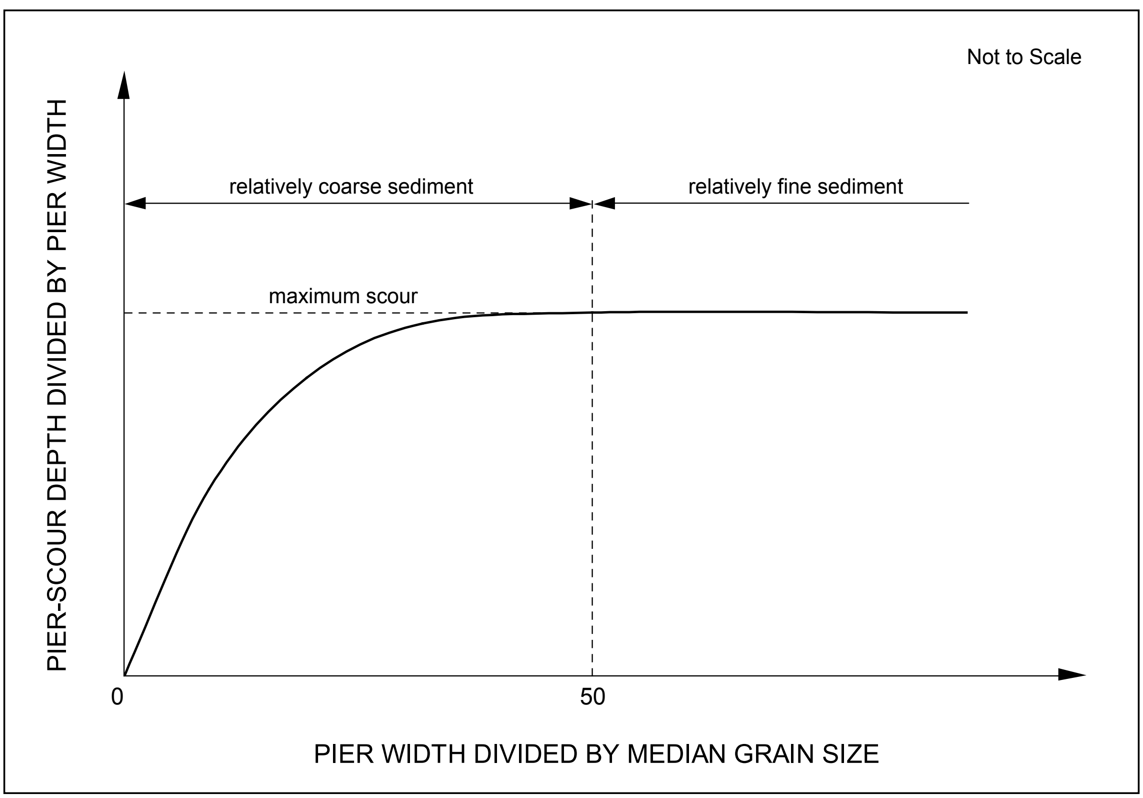


Figure 42. General relation of relative sediment size to relative pier scour based on laboratory investigations (from Melville and Coleman, 2000).

for scour. Additionally, because the potential for scour in relatively fine sediments is constant (see relatively fine sediment range in figure 42) and does not vary as grain size varies, it is appropriate to assume sediment size will not be a strong explanatory variable for pier scour in South Carolina and, therefore, can be neglected when developing relations or envelope curves that help explain clear-water pier scour in South Carolina.

To provide perspective on the relation of pier-scour depth and median grain size for non-normalized field data, the relation of pier-scour depth and the median grain size for surface soils in the Coastal Plain and Piedmont of South Carolina was examined (fig. 43). Because pier width is a strong explanatory variable, data were grouped by selected pier widths. Additionally, because 80 percent of the pier widths in the field measurements were 1.5 ft or less, a hand-drawn envelope curve of these measurements is included. The envelope curves of the measured scour for pier widths of 1.5 ft or less are flat, indicating that the fine sediment sizes typically found in South Carolina have negligible effect on the upper bound of scour. It is also noteworthy to compare the envelope curves for the Coastal Plain and Piedmont of South Carolina (fig. 43). The Piedmont envelope curve is slightly lower than the Coastal Plain envelope curve, but the minimal difference indicates that the cohesive soils of the Piedmont do not have a significant effect on the upper bound of pier-scour depths for pier widths 1.5 ft or less. The measured data for pier widths greater than 1.5 ft are sparse, and trends cannot be discerned. However, the dimensionless variable b/D_{50} for the larger piers is greater than 50, indicating that the scour at these sites is independent of the sediment size.

This finding is for uniform sediments; therefore, the effects of streambed armoring that are often encountered with graded soils may influence scour depths at a given site. If the largest grain size of a graded sediment is too small to armor the streambed, however, the effect of gradation is insignificant. Because sediments in South Carolina are relatively fine, streambed armoring likely is minimal and does not strongly influence scour depths.

Pier Shape

Laboratory studies indicate that pier shape can influence scour depths; pier shapes that are more streamlined (round and sharp nosed) tend to create smaller scour depths than square-shaped piers. When a pier is uniform in shape in the vertical direction, which is typical in South Carolina, Melville and Coleman (2000) note that the influence of shape is relatively insignificant, and a square-nosed pier produces a scour depth about 10 percent greater than that of a round-shaped pier of the same width. To account for this phenomenon, the HEC-18 pier-scour equation (Richardson and Davis, 2001) includes a correction coefficient for shape—round piers have a coefficient of 1, sharp-nosed piers have a coefficient of 0.9, and square-nosed piers have a coefficient of 1.1. The influence of pier shape becomes negligible when pier skew is greater than 5 degrees, and a shape coefficient of 1 can be used under such circumstances (Melville and Coleman, 2000; Richardson and Davis, 2001).

In this study, 17 of the 177 measurements of pier scour have round-shaped piers; the remainder have square-shaped piers. The limited number of pier shapes other than square does not allow much to be concluded about the effect of pier shape on scour depths for the South Carolina data. In an extensive look at pier-scour data from the NBSD, however, Mueller and Wagner (2005) concluded that “pier shape does not affect the depth of scour in the field as much as it does in the laboratory.” Based on this observation it is reasonable to assume that pier shape does not have a strong influence on pier-scour depths in South Carolina. From a practical view, pile bents are the most common type of bridge support in South Carolina and typically have pier widths of 1.5 ft or less. (In the current study, 80 percent of the pier widths are 1.5 ft or less.) These data indicate that scour depths at such piers have an upper bound of about 2.75 ft. If the shape-correction coefficient for square piers is applied to this scour depth (2.75 ft), it will increase by 10 percent or about 0.28 ft. This correction is small and could be neglected without adverse effect on the estimation of scour. As will be discussed later, the effect of pier shape on scour depths can be neglected in pier-scour computations without adversely influencing predictions of clear-water pier-scour depths.

Pier Skew

Laboratory investigations indicate that alignment of the pier to flow (pier skew) can significantly influence scour depths; as the aspect ratio of pier length to pier width increases, the influence of pier skew also increases. The influence of pier skew is attributed to the increase of the effective frontal width of the pier as the pier skew becomes larger. When the length-to-width aspect ratio is large, small changes in pier skew can increase significantly the effective frontal width of the pier, thus increasing pier-scour depth. The laboratory data indicate that with large skews and aspect ratios, pier-scour depths can be as much as 6 times those of unskewed piers. In the case of a round-shaped pier, the aspect ratio is 1 and pier skew has no effect.

Pile bents (figs. 16, 17, 18) are very common bridge supports in South Carolina, and evaluating the influence of pier skew at such supports is more complicated than evaluating the influence of a single uniform pier. A typical pile has a length-to-width aspect ratio near 1; therefore, the effect of pier skew on an individual pile is small. However, when piles are aligned in close proximity to one another, as in the case of a pile bent, the surrounding piles potentially influence scour depths. This influence diminishes as the spacing between the piles increases. If the spacing is relatively close, the interaction between piles can be strong, and scour depth increases with increasing pier skew. Using laboratory data, Melville and

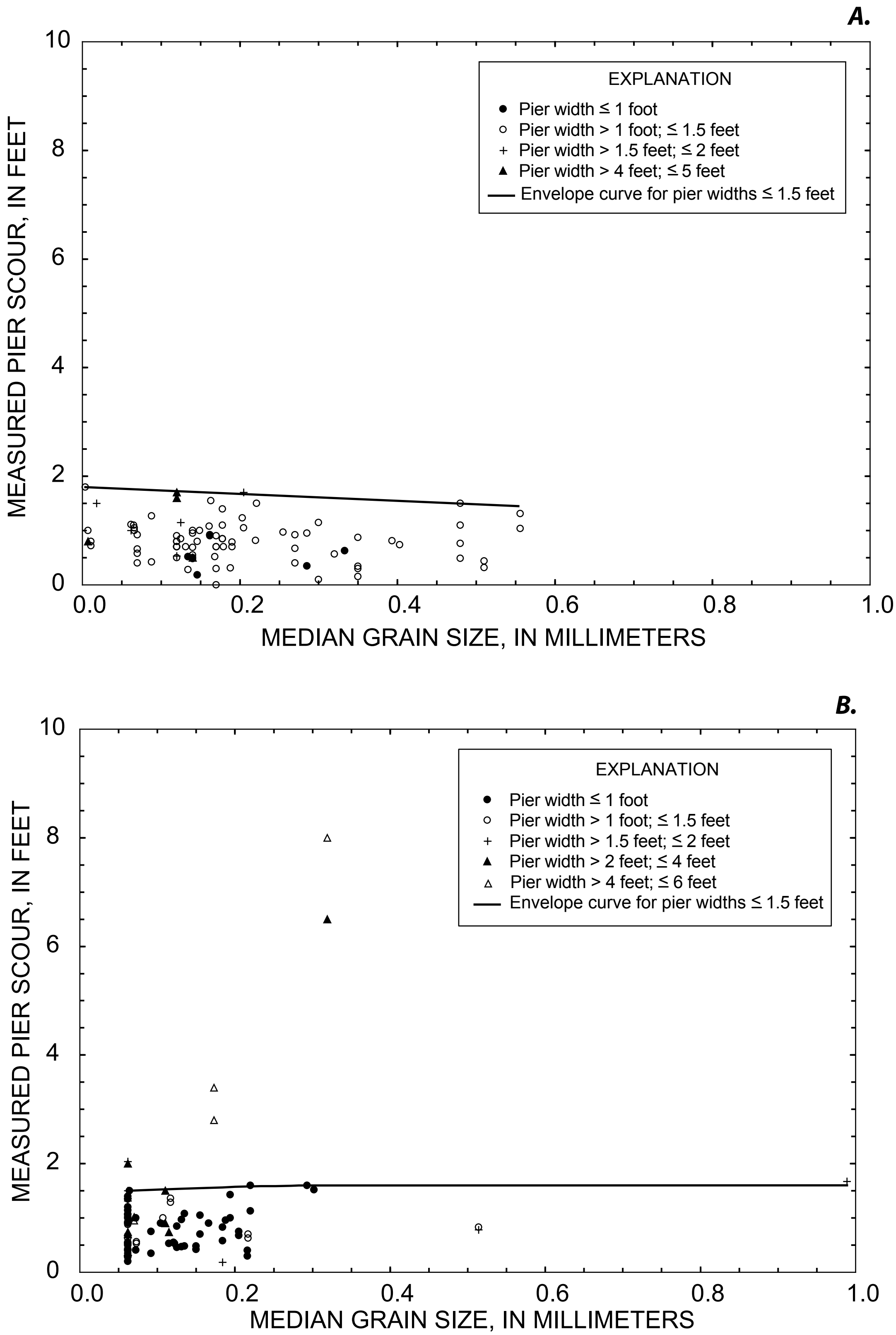


Figure 43. Relation of measured clear-water pier-scour depth to median grain size at selected sites in the (A) Coastal Plain and (B) Piedmont Physiographic Provinces of South Carolina.

Coleman (2000) tabulated pier-skew correction coefficients for pile bents (table 5) with selected pile spacing (measured from center-to-center of pile) and skews. At a pile-spacing to pile-width ratio of 10, there is no influence from the surrounding piles (table 5). As this ratio diminishes, the influence from surrounding piles increases and the effect of pier skew is stronger. However, the influence of pier skew on a pile bent is always smaller (and often significantly so) than on a comparable solid pier. To account for this diminished scour depth at skewed pile bents, the HEC-18 (Richardson and Davis, 2001) recommends that if piles or multiple columns are spaced 5 pier widths or greater apart, the pier-skew correction coefficient should be limited to 1.2 times the local scour of a single column. For spacings less than 5 pier widths, the standard skew-correction coefficient is to be applied. This method will be more conservative than using the coefficients in table 5.

Table 5. Pier-skew correction coefficients for pile bents (from Melville and Coleman, 2000).

Pile spacing to pier width ratio	Pier skew less than 5 degrees	Pier skew between 5 and 45 degrees	Pier skew equal to 90 degrees
2	1.12	1.40	1.20
4	1.12	1.20	1.10
6	1.07	1.16	1.08
8	1.04	1.12	1.02
10	1.00	1.00	1.00

In the current study, 169 of the 177 pier-scour measurements are associated with pile bents or multiple-column bents with column spacings ranging from 1 to 10.5 pier widths. Most of the column spacings less than 5 pier widths are associated with bridge widenings, where new piles were driven in close proximity to the old structure supports (figs. 14, 21), resulting in relatively small spacings between some of the columns. However, the typical pile bent in South Carolina, not associated with a bridge widening, (figs. 13, 17, 18) has spacings between the piles that are approximately 5 pier widths or greater, which limits the maximum pier-skew correction coefficient to 1.2. (In the current investigation there were only 24 pier-scour measurements that were taken at pile bents with no bridge widening. These bents had pile spacings that range from 4.3 to 9.4 pier widths with an average of 6.6.) Field data from this study indicate that for typical pile bents, individual scour holes tend to develop at each pile with limited to no interaction from neighboring piles. This trend indicates that even when a pile bent is skewed to the approaching flow, individual and independent scour holes develop at each pile and the effect of pier skew is minimal and perhaps negligible. This field trend indicates that the HEC-18 recommendation to limit the pier-skew correction coefficient to 1.2 at pile bents with pile spacings 5 pier widths or greater is reasonable and, perhaps, somewhat conservative. The limited field data also indicate that this may apply to pile spacings as small as 4.3 pier widths for piers 1.5 ft wide or less, but this should be applied with caution. (As noted later, the correction coefficient for skew, in many cases, can be neglected for pile bents typically found in South Carolina.)

Of the 177 measurements of pier scour in this investigation, 127 have no pier skew; the remaining 50 measurements have pier skews ranging from 4 to 22 degrees with an average of 13 degrees. Of these 50 pier-scour measurements, 45 have pier skews associated with pile bents or multiple column bents where the effect of skew is significantly diminished from that of a comparable solid pier. In addition, the pier-skew values used in this investigation were based primarily on the bridge orientation to the floodplain and channel and are not based on actual measurements of flow direction during high flows. Because the selected skew angles represent more of the general skew at a bridge rather than the specific skew at a particular pier, it is likely that the selected pier-skew angles have some error associated with them. These data limitations in the South Carolina database must be kept in mind when attempting to evaluate the effect of pier skew within the South Carolina data.

To provide perspective on the relation of pier-scour depth and the effect of pier skew on pile bents and multiple-column bents in the South Carolina field data, the relation of pier-scour depth and pier width in data grouped by bents with and without pier skew was examined (fig. 44). The range and scatter of the skewed and non-skewed bents are similar, indicating that there is not a large difference between the groups. Interestingly, the trend lines indicate that bents with pier skew tend to have slightly higher scour depths than bents without pier skew. For practical purposes, however, the difference in the trend lines is small, indicating it is probably reasonable to combine skewed and non-skewed data in the South Carolina database when developing upper-bound envelope curves for pier scour.

Pier Width and The South Carolina Pier-Scour Envelope Curve

Researchers agree that pier-scour depth is strongly related to pier width. According to HEC-18 (Richardson and Davis, 2001), “Pier width has a direct influence on depth of local scour. As pier width increases, there is an increase in scour depth.” Melville and Coleman (2000) reported, “…the depth of scour at a pier is strongly dependent on the width of the pier.” After analyzing 224 field measurements of pier scour from the NBSD, Mueller (1996) concluded, “…pier width shows the strongest correlation with pier scour.” Although other variables influence pier-scour depth (flow velocity, flow depth, sediment size, flow alignment, and pier shape), the previous analysis indicates that these influences often are small for field conditions in South Carolina. If, however, pier width is a strong explanatory variable for pier-scour depth, it can be used as the primary explanatory variable in the development of simple envelope curves that display the upper-bound trend of scour in South Carolina. Such envelope curves are simple but

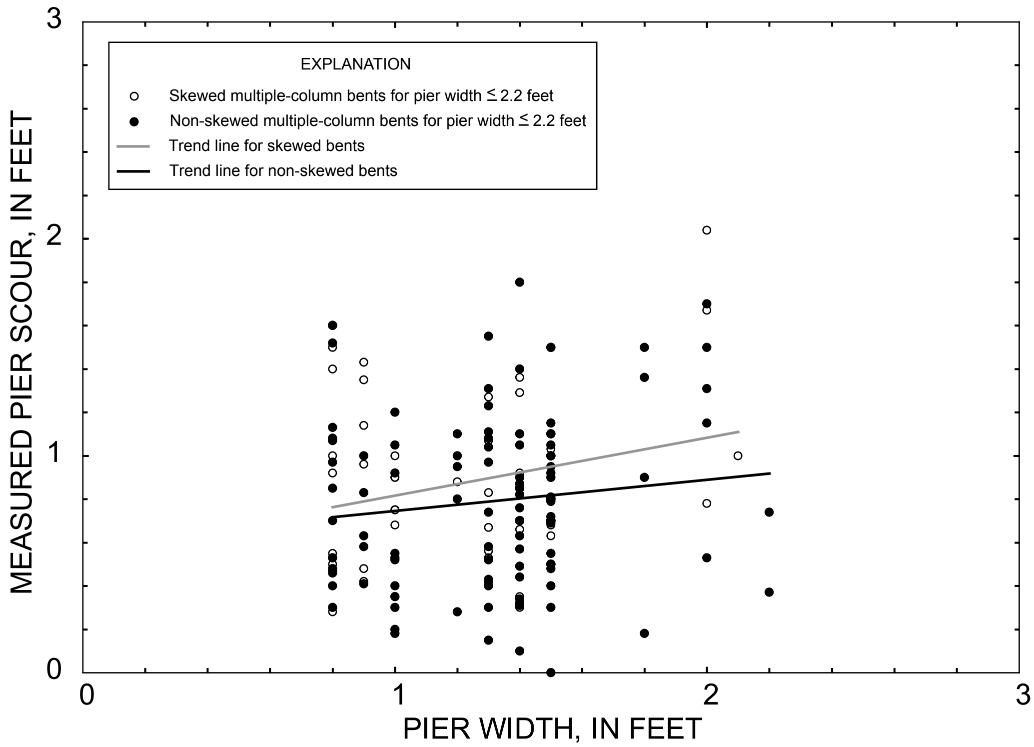


Figure 44. Relation of measured clear-water pier-scour depth to pier width associated with multiple-column bents with and without pier skews at selected sites in South Carolina.

useful tools that can be used to evaluate the potential for scour, evaluate the reasonableness of predicted scour, and in general help practitioners develop judgment regarding the range and trend of scour under given field conditions. The following section reviews the upper-bound relation of pier width to pier-scour depth in laboratory and field data and describes the development of the South Carolina pier-scour envelope curve along with its applications and limitations.

Envelope Curves for Laboratory and Field Data

The relation of pier-scour depth to pier width for laboratory data is shown in figure 45. The relation includes data used to develop the original HEC-18 pier-scour equation (Richardson and others, 1991) and laboratory data from Melville and Chiew (1999). The data from Melville and Chiew (1999) are scour depths for clear-water pier-scour experiments made in relatively fine sediments with long flow durations (average duration of 2.7 days with a maximum duration of 10.4 days) to assure that scour depths reached equilibrium conditions. Figure 45A shows the relation of pier width to scour depth and indicates that the upper bound of scour increases with increasing pier width. An exception to this trend occurs for pier widths of 0.66 ft where the upper bound of scour decreases. This decrease likely can be attributed to the small flow intensities associated with these measurements, which range from 0.46 to 0.82. Because scour depth is sensitive to flow intensities less than 1 (fig. 30), the smaller flow intensities will tend to reduce pier-scour depth. In contrast, the measurements in the Melville and Chiew (1999) data that form the upper bound in figure 45 have flow intensities of 0.9 or greater. If laboratory tests had been made at flow intensities of 0.9 or greater for pier widths of 0.66 ft, the upper bound for this pier width likely would approach that of the envelope curve in figure 45. It is interesting to note that the envelope curve for the data from Melville and Chiew (1999) plots above the envelope curve for the HEC-18 data and has a much steeper slope (see equations on fig. 45A). This, in part, may be attributed to the long flow durations in the Melville and Chiew (1999) experiments; therefore, the envelope curve for the Melville and Chiew (1999) data may better represent an upper bound for equilibrium clear-water pier-scour depths.

Relative scour (scour depth divided by pier width) is a dimensionless variable that commonly is used by laboratory investigators to display the trends of pier-scour depth in relation to pier width. Figure 45B shows the laboratory relation of pier width to relative scour and indicates that the upper bound of relative scour decreases with increasing pier width. The envelope curves in this plot were developed by dividing the envelope-curve equations in figure 45A by pier width. The trends of the envelope curves for relative scour (fig. 45B) indicate that as pier width increases, relative scour asymptotically approaches a constant. This trend is more evident in figure 46B where the laboratory envelope curves were extended. The envelope curves for the HEC-18 data and the Melville and Chiew (1999) data have asymptotic limits for

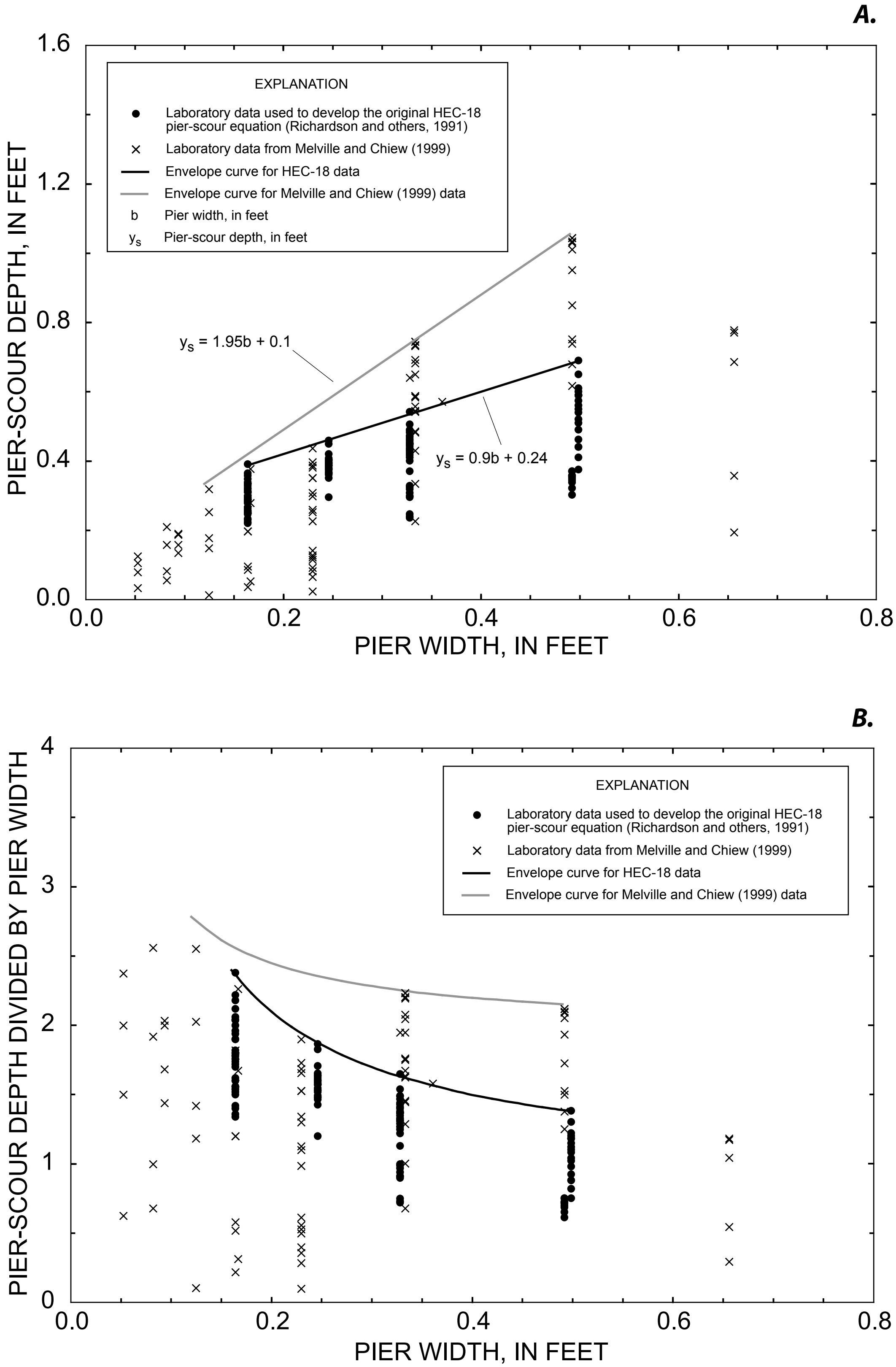


Figure 45. Relation of pier width to (A) measured scour depth, and (B) relative scour for selected laboratory data.

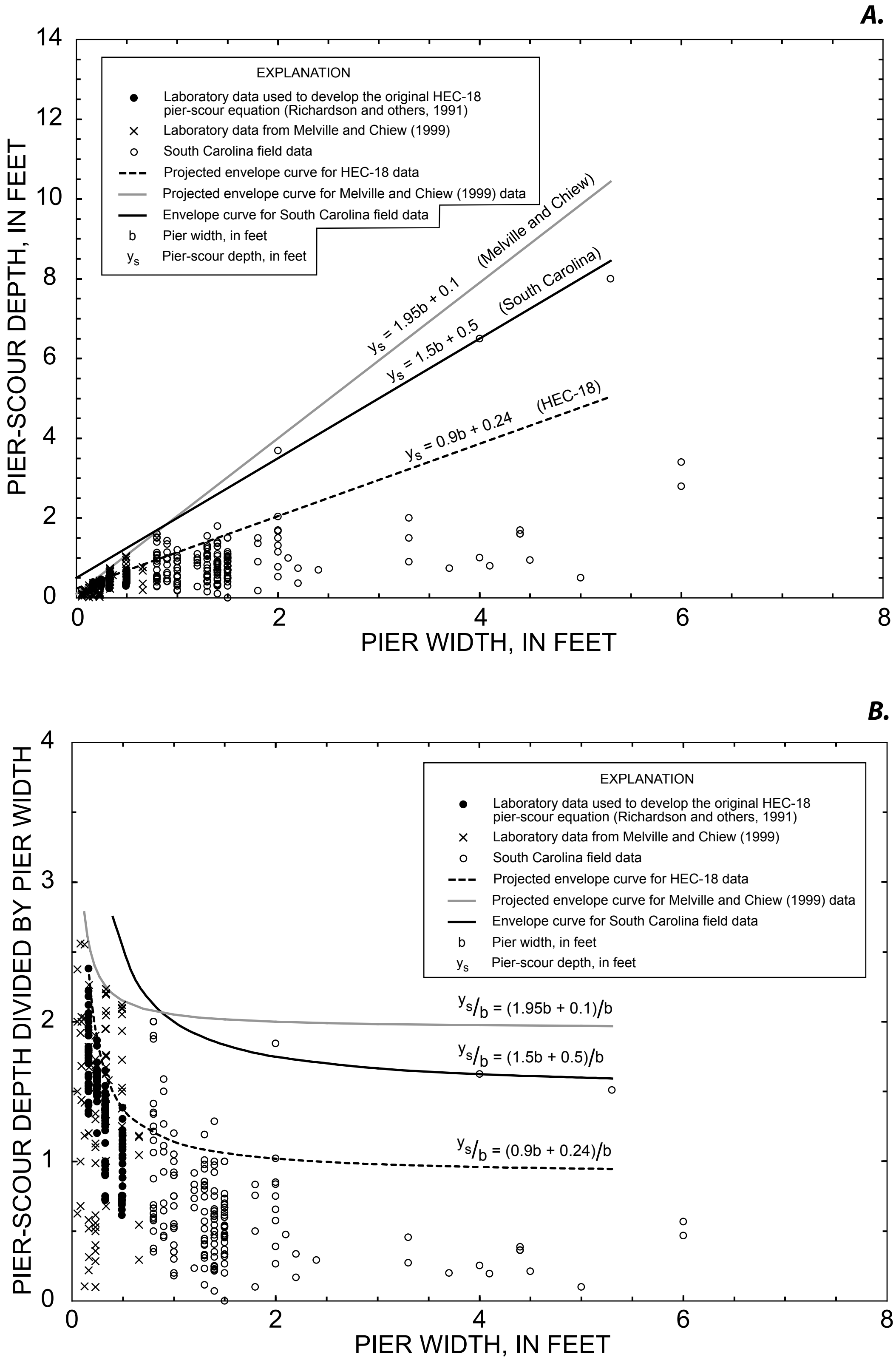


Figure 46. Relation of pier width to (A) measured scour depth, and (B) relative scour for selected data from laboratory investigations and field data from selected sites in South Carolina.

relative scour of 0.9 and 1.95, respectively. Again, the difference in the limiting values between the two laboratory data sets can, in part, be attributed to the long flow durations in the Melville and Chiew (1999) experiments.

To provide perspective on how the trends of the South Carolina field data compare to the trends of the laboratory data, the laboratory and field data were plotted for analysis (fig. 46). The envelope curves of the laboratory data were extended to show how they compare with the envelope curve of the field data. (The extension of the laboratory envelope curves exceeds the range of the laboratory data by approximately 1,000 percent, and the excessive extrapolation would be considered inappropriate for many engineering applications. Although the extrapolated laboratory curves are useful in comparing with the field data, the excessive extrapolation highlights potential problems with scaling laboratory data to the field.) The trends in the field data are very similar to those of the laboratory data, and it is interesting to note that the envelope curve of the field data is close to that of the Melville and Chiew (1999) data. This similarity in envelope curves indicates that the upper bound of the South Carolina data is reasonable and can be used to evaluate upper bounds of pier scour for field conditions in South Carolina.

Because the South Carolina envelope curve is lower than the Melville and Chiew (1999) envelope curve, one might question whether the South Carolina envelope curve is a reasonable upper bound. The distinctions between laboratory and field conditions, however, can account, in large measure, for these differences. The Melville and Chiew (1999) data represent scour in loose grained, unconsolidated soils with very long flow durations. In contrast, the field data represent scour in natural soils that have some measure of consolidation and cohesion, and peak-flow durations much smaller than those of the laboratory (refer to section “Time and Flow Duration”). Although there are differences, the relative closeness of the South Carolina field envelope curve to the Melville and Chiew (1999) envelope curve provides strong support that it is a good indicator of the upper bound of scour for piers in South Carolina.

To further determine whether the upper bound of the South Carolina data is reasonable for field conditions, the South Carolina data were plotted along with selected NBSD data (fig. 47). The NBSD data include 46 measurements that were selected for their similarity to the South Carolina data, including measurements with no skew, insignificant influence from debris, fine to medium sands, and pier widths 6 ft or less. The majority of the selected NBSD data fall within the South Carolina envelope curves and only five measurements slightly exceed the envelope curve. The pier-scour data in the NBSD, in general, represent live-bed pier scour in the main channels that tend to have higher velocities, longer flow durations, and more loose-grained sediments and, therefore, increased scour potential. In contrast, the South Carolina data represent scour in the floodplain (outside of the main channel) where soils typically have some measure of consolidation and cohesion, velocities are lower, and flow durations shorter than those of the main channel. Under these conditions, it is reasonable that the South Carolina data tend to have a lower upper bound than the NBSD data. Although there are five NBSD measurements that exceed the South Carolina envelope curve, the magnitude is relatively small, providing strong support that the South Carolina envelope curve is reasonable as an upper bound for clear-water pier scour in the floodplains of South Carolina.

Because much of the pier-scour data collected during this investigation cannot be associated with the flow conditions that created the scour, it may be argued that the upper-bound envelope curve of the South Carolina data is questionable. However, the data include 51 bridges where known maximum historic flows have occurred, and at most of these bridges, flows near or exceeding the 100-year flow magnitude have occurred (table 2). The South Carolina data and envelope curves identifying the 78 pier-scour measurements from the 51 bridges with known maximum historic flows are shown in figure 48. The envelope curves are defined by sites that have had large flows, and it is noteworthy that the sites from the 1995 flood along the Enoree River (when flows significantly exceeded the 100-year flow) form the upper bound. This indicates that the South Carolina pier-scour envelope curve, with pier width as an explanatory variable, represents scour trends resulting from large floods.

The South Carolina pier-scour data and envelope curves were plotted for sites in the Coastal Plain and Piedmont of South Carolina (fig. 49). It is important to note the trends of the data within these two regions. Figure 49 clearly shows that the Piedmont data form the envelope curve. For pier widths 2 ft or less, the upper bound of the data is similar in both regions, indicating that for pier widths of 2 ft or less, the South Carolina envelope curve will provide a reasonable upper bound for both regions. However, for pier widths greater than 2 ft, the pier-scour depths in the Coastal Plain are significantly below the upper bound of the Piedmont data. This, in part, may be attributed to insufficient data in the Coastal Plain for piers exceeding 2 ft in width and additional data could possibly indicate that the upper bound does more closely approach the Piedmont data. Additionally, the trend in figure 49 may be attributed to lower velocities in the Coastal Plain, which may not allow scour depths at many sites to approach those of the higher velocities in the Piedmont. This trend also can be seen in figure 50 in which data from the 1995 flood along the Enoree River in the Piedmont and data from the 1999 flood along the Waccamaw River in the Coastal Plain are plotted. The pier-scour measurements along the Enoree River (fig. 50) had estimated velocities ranging from 2.9 to 12.1 ft/s during the 1995 flood. In contrast, pier-scour measurements along the Waccamaw River (fig. 50) had estimated velocities ranging from 0.4 to 1.7 ft/s during the 1999 flood and pier-scour depths are significantly lower than those of the Enoree River. The trends in figure 50 indicate that the South Carolina envelope curve will often be too high for piers in the Coastal Plain, and notably so when piers exceed 2 ft in width. Developing a separate envelope curve for the Coastal Plain could be beneficial in minimizing the potential

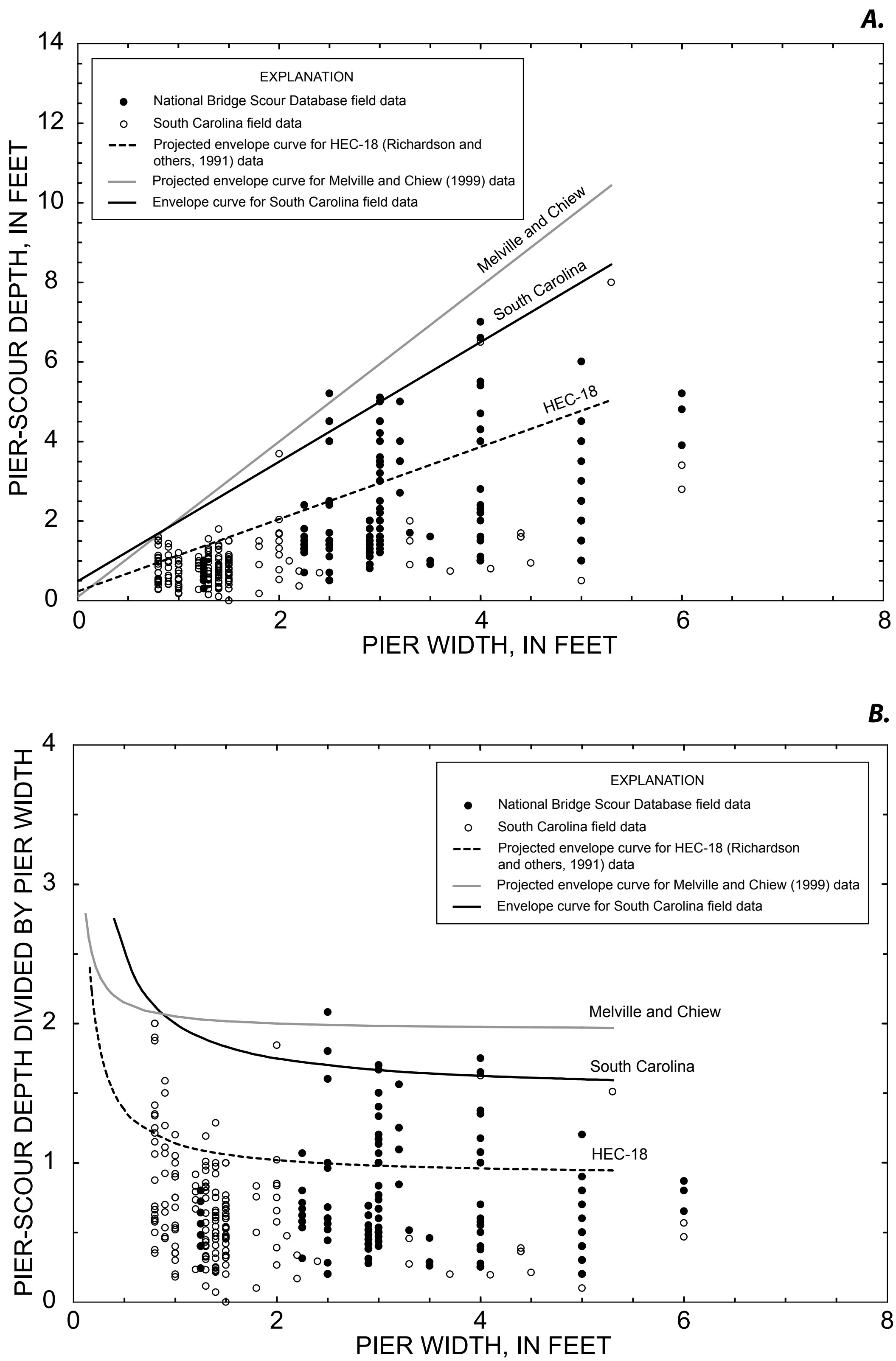


Figure 47. Relation of pier width to (A) measured scour depth, and (B) relative scour, for selected data from the National Bridge Scour Database and field data from selected sites in South Carolina.

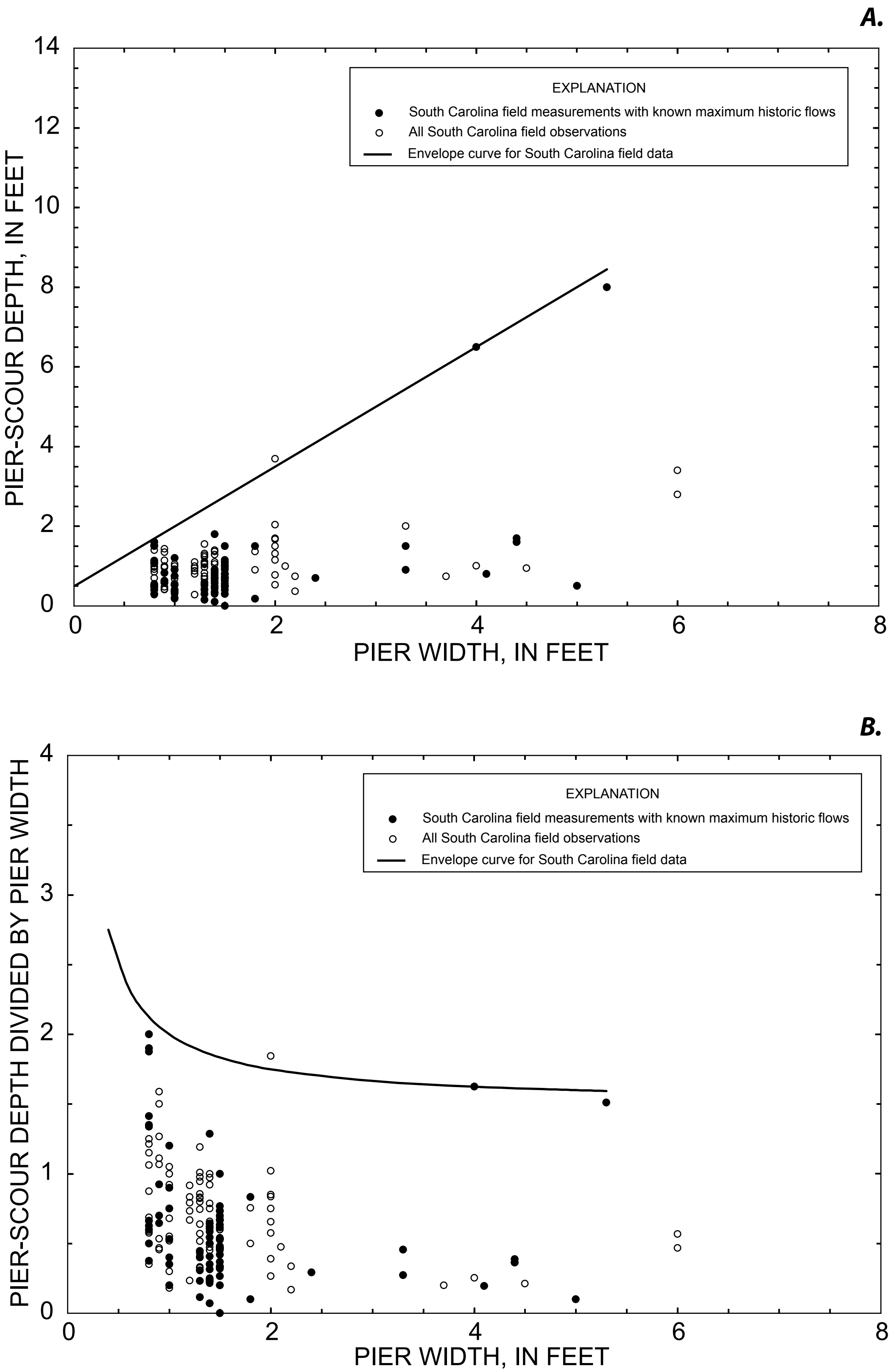


Figure 48. Relation of pier width to (A) measured scour depth, and (B) relative scour for selected sites with known maximum historic flows in South Carolina.

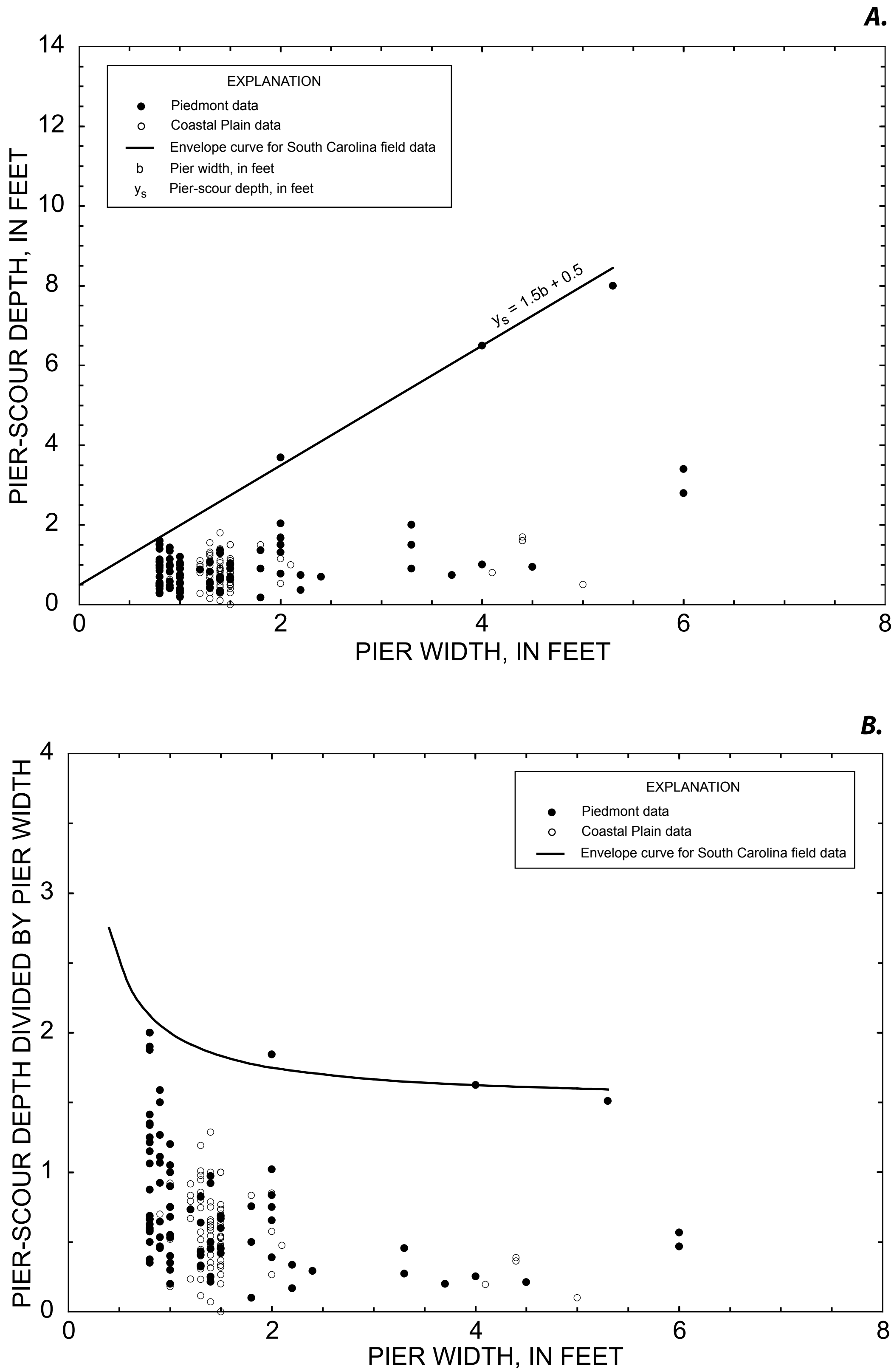


Figure 49. Relation of pier width to (A) measured scour depth, and (B) relative scour for selected sites in the Coastal Plain and Piedmont Physiographic Provinces of South Carolina.

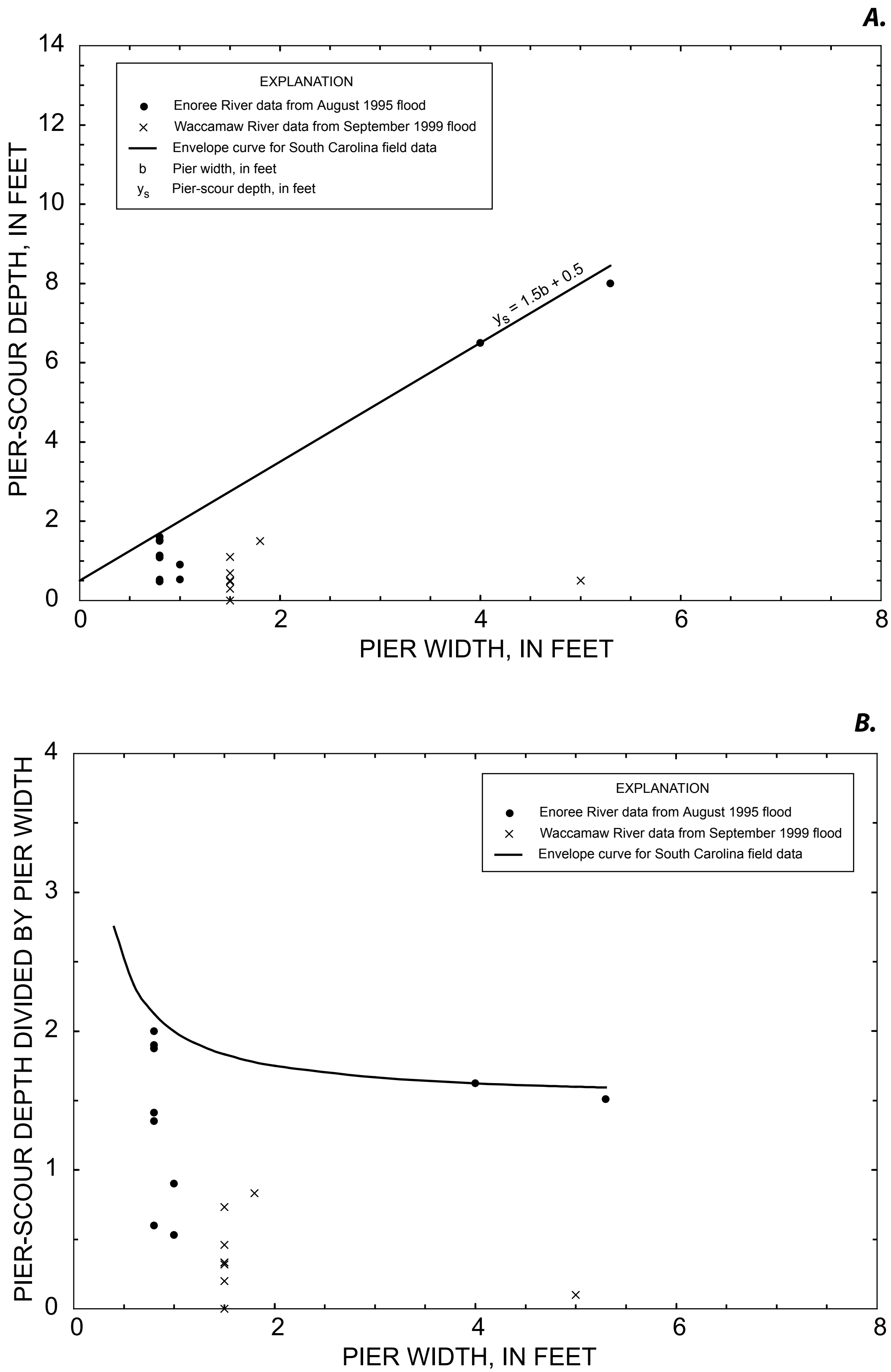


Figure 50. Relation of pier width to (A) measured scour depth, and (B) relative scour for selected sites on the Enoree and Waccamaw Rivers in South Carolina.

for excessive evaluation of the upper bound of scour at wide piers in the Coastal Plain. However, the data in this study are insufficient to confidently develop a distinct envelope curve for the Coastal Plain, and the envelope curve encompassing data from both regions is recommended to evaluate pier-scour potential in South Carolina.

It should be kept in mind that the envelope curve in figure 46A does not imply that all piers in South Carolina eventually will have scour depths near the upper bound of the envelope curve. Each site has unique field and hydraulic characteristics that determine the progression and the limit of scour depth. At many sites, these characteristics will prevent scour depths from reaching the upper bound envelope curve. (This is highlighted by the differing scour depths for similar pier widths along the Enoree and Waccamaw Rivers as described in the preceding paragraph.) The South Carolina pier-scour envelope curve (fig. 46A) represents an upper bound of pier scour that infrequently will be exceeded. Thus, if one is evaluating scour for a 1-ft-wide pier in field conditions similar to those of the current investigation, the envelope curve indicates that pier-scour for such a pier infrequently will exceed a value of 2 ft. Therefore, the envelope curve can be used to quickly evaluate the potential for scour for a given pier width, as well as evaluate the reasonableness of predicted scour. In general, the South Carolina pier-scour envelope curve will help practitioners develop judgment regarding the range and trend of scour under given field conditions.

Equation for the South Carolina Pier-Scour Envelope Curve

The equation for the South Carolina pier-scour envelope curve is as follows:

$$y_s = 1.5b + 0.5, \quad (4)$$

where,

y_s is the upper bound of local pier-scour depth, in feet; and

b is pier width, in feet.

This equation can be applied to round- and square-shaped piers, and to pile bents with moderate skews (approximately 20 degrees or less) with spacings between piles of approximately 5 pier widths or greater. When skews are thought to influence scour (such as for solid piers), a correction coefficient as defined in HEC-18 (Richardson and Davis, 2001) can be applied to equation 4. Equation 4 was developed for pier widths of 6 ft or less and is not recommended for use outside the limits of the South Carolina data.

Using equation 4 initially may appear to be an oversimplified approach to evaluating pier scour; however, a literature review by Breusers and others (1977) identified four equations by various researchers in which pier width was the only explanatory variable for local pier scour. These four equations emphasize the fact that pier width strongly influences scour depth, thus developing an equation that includes pier width as the only explanatory variable is not unreasonable. Interestingly, one of the four equations identified by Breusers and others (1977) is very similar to equation 4. The equation was developed by Breusers (1965) and cited by Breusers and others (1977) as follows, with variables previously defined:

$$y_s = 1.4b. \quad (5)$$

(Mueller (1996) and Mueller and Wagner (2005) also used equation 5 as part of their analysis comparing measured and predicted pier scour for selected scour-prediction equations, and results indicated that the equation rarely underpredicted but on occasion provided excessive overpredictions.) Although equation 5 is not recommended as an evaluation tool for clear-water pier scour in South Carolina, it is very similar to equation 4 (having almost the same slope with a slightly different y-intercept) and indicates that equation 4 is reasonable. Additionally, HEC-18 (Richardson and Davis, 2001) recommends an upper bound for pier scour at round-nosed piers aligned with the flow. The equations for the upper bound are as follows, with variables previously defined:

$$y_s = 2.4b \quad \text{for Froude numbers } \le 0.8 \quad (6)$$

and

$$y_s = 3.0b \quad \text{for Froude numbers } > 0.8. \quad (7)$$

Equations 6 and 7 are based on laboratory data, where upper bounds of scour in the loose-grain sediments of the laboratory will likely exceed those of the floodplain soils of South Carolina. Additionally, the laboratory data represent scour resulting from small pier widths (typically 1 ft or less) that will tend to have larger relative scour values and, therefore, envelope curves with higher upper bounds. The effects of loose-grain sediments and small pier widths provide some explanation of why the HEC-18 and South Carolina upper bound equations differ, indicating that the trends of equation 4 are not unreasonable.

Evaluation of Selected Methods for Predicting Clear-Water Pier Scour in South Carolina

When designing new bridges or evaluating existing bridges for scour, it is important to have evaluation tools that consistently provide conservative yet realistic estimates for scour. The current methods for predicting scour, as described in HEC-18 (Richardson and Davis, 2001), are in need of field verification and possible modifications to increase accuracy. Additionally, there is need to provide tools derived from field data to help practitioners develop judgment regarding the range and trend of scour within a given region and to evaluate the reasonableness of predicted scour. The following report sections (1) evaluate the performance of the HEC-18 pier-scour equation (Richardson and Davis, 2001), (2) present and evaluate a modified equation for predicting clear-water pier scour for field conditions in South Carolina, and (3) evaluate the performance of the South Carolina pier-scour envelope curve.

The HEC-18 Pier-Scour Equation

To predict pier-scour depth for clear-water scour conditions, Richardson and Davis (2001) recommend using the HEC-18 pier-scour equation (eq. 1) that initially was derived from laboratory data for noncohesive sediments and later was modified with correction coefficients to account for coarse sediments and wide piers. Using hydraulic variables estimated from the WSPRO model with the 100-year flow at all sites (116 bridges) and the known maximum historic flow at 51 bridges (table 2), predicted pier-scour depths were computed using the HEC-18 pier-scour equation. Predicted pier-scour depths computed for the 100-year flow can be compared with measured pier-scour depths, as shown in figure 51A. The data are grouped by regional location (Coastal Plain and Piedmont) and by broad categories for pier width and skew correction coefficients. When pier widths are 2 ft or less and the skew correction coefficient is 1.3 or less, the equation appears to provide reasonable values of predicted scour with no underprediction. For piers wider than 2 ft and(or) with skew correction coefficients exceeding 1.3, however, overprediction can be excessive. Excessive overprediction occurs more frequently in the Piedmont than in the Coastal Plain. This can be attributed to the higher incident of skew correction coefficients near a value of 2 for Piedmont sites and higher velocities in the Piedmont that tend to increase predicted scour. Only two underpredictions for scour occurred, both at the S.C. Route 49 crossing of the Enoree River (fig. 51A). At this site, the peak flow during the 1995 flood was approximately twice that of the 100-year flow magnitude. When the 1995 peak flow is used (which more appropriately represents the flow conditions that created the measured scour) to evaluate predicted scour (fig. 51B), underprediction no longer occurs at this site.
In figure 51B, the predicted pier-scour depth computed at sites with known maximum historic flows (table 2) can be compared with the measured pier-scour depth. The data are grouped in a similar manner as in figure 51A, and the trends are similar. (There are only two measurements that have skew coefficients that are greater than 1.3 and are noted on the graph.) Data in figure 51 indicate that use of the HEC-18 pier-scour equation infrequently underpredicts measured scour but, on occasion, can excessively overpredict measured scour. Similar trends were observed in the investigation by Mueller and Wagner (2005), in which 266 pier-scour measurements from the NBSD were compared with predicted scour from selected prediction equations, including the HEC-18 pier-scour equation. Based on the results of the current investigation and those of Mueller and Wagner (2005), it is reasonable to conclude that using the HEC-18 pier-scour equation generally provides conservative estimates of pier scour that, at times, are excessive. The South Carolina data show that excessive overprediction frequently occurs when pier widths exceed 2 ft and(or) when skew coefficients exceed approximately 1.3.
To minimize excessive overprediction in South Carolina, the HEC-18 pier-scour equation likely needs modifications to the correction coefficients for skew and pier width, which is addressed in the following section.

The South Carolina Modified Pier-Scour Equation

The original form of the HEC-18 pier-scour equation (Richardson and others, 1991), called the Colorado State University (CSU) equation, is as follows, with variables defined previously in equation 1:

$$\frac{y_s}{y_1} = 2.0K_1K_2\left[\frac{y_1}{b}\right]^{0.35}Fr_1^{0.43}, \tag{8}$$

If the pier is cylindrical, the K_1 and K_2 correction coefficients for pier shape and skew are set to a value of 1, and the equation can be simplified as follows:

$$\frac{y_s}{y_1} = 2.0\left[\frac{y_1}{b}\right]^{0.35}Fr_1^{0.43}. \tag{9}$$

Through mathematical manipulation, equation 9 can be arranged in the following format:

$$\frac{y_s}{y_1} = 2.0\left[\left(\frac{b}{y_1}\right)^3 Fr_1^2\right]^{0.215}, \tag{10}$$

Equation 10 is the format of a power function, $y = cx^d$, where y is the response variable, c is a coefficient, x is the explanatory variable, and d is an exponent. In equation 10, the response variable is y_s/y_1, the coefficient has a value of 2, the explanatory variable is

$$\left[\left(\frac{b}{y_1}\right)^3 Fr_1^2\right],$$

and the exponent has a value of 0.215.
Power functions often are used in linear regression analysis of logarithmically transformed data. Using logarithmically transformed data for laboratory measurements used to develop the original HEC-18 pier-scour equation and the South Carolina pier-scour data, the relation of the power function in equation 10 is shown in figure 52. (Hydraulic variables for the field data were estimated with the WSPRO model for the 100-year flow.) The laboratory data represent cylindrical piers, and there is no need to make adjustments for skew or pier

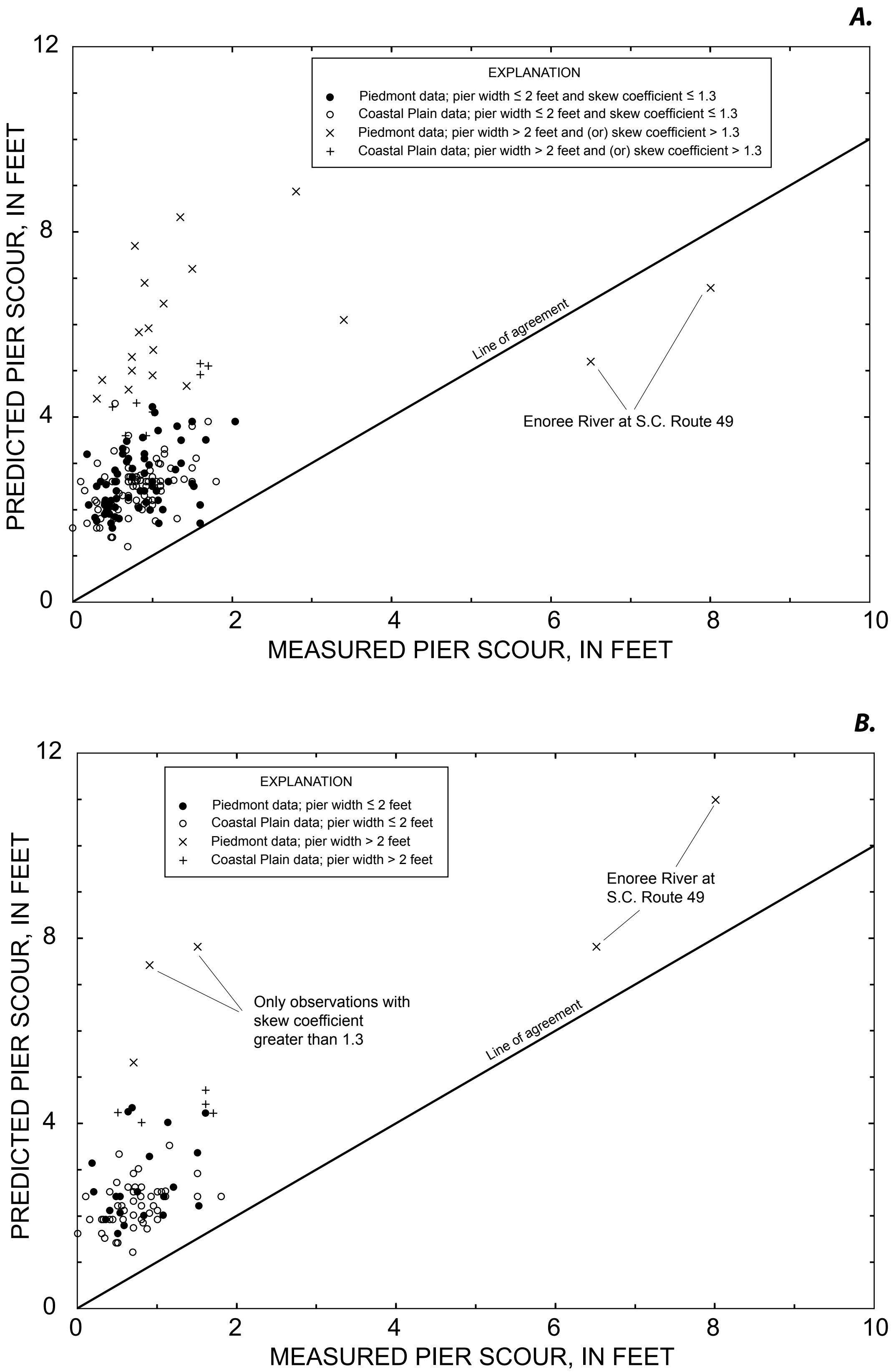


Figure 51. Relation of measured clear-water pier-scour depth to predicted pier-scour depths for (A) the 100-year flow, and (B) known maximum historic flows at selected sites in South Carolina. (Predicted pier scour was calculated with the HEC-18 equation (Richardson and Davis, 2001).)

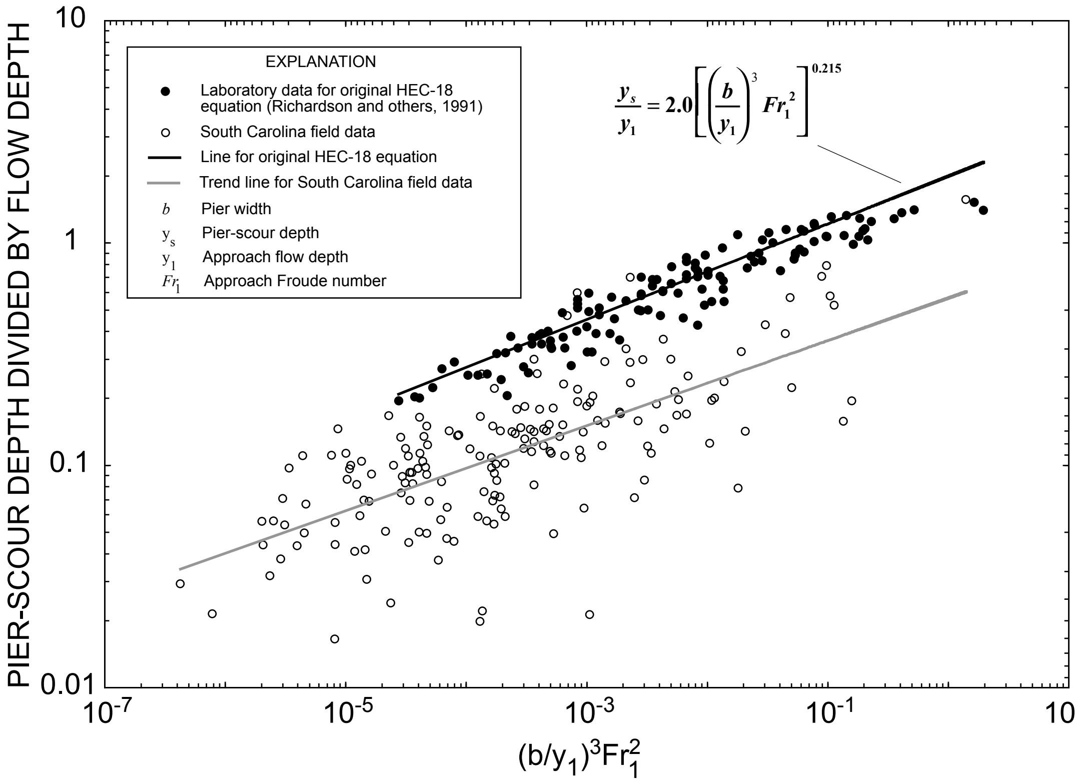
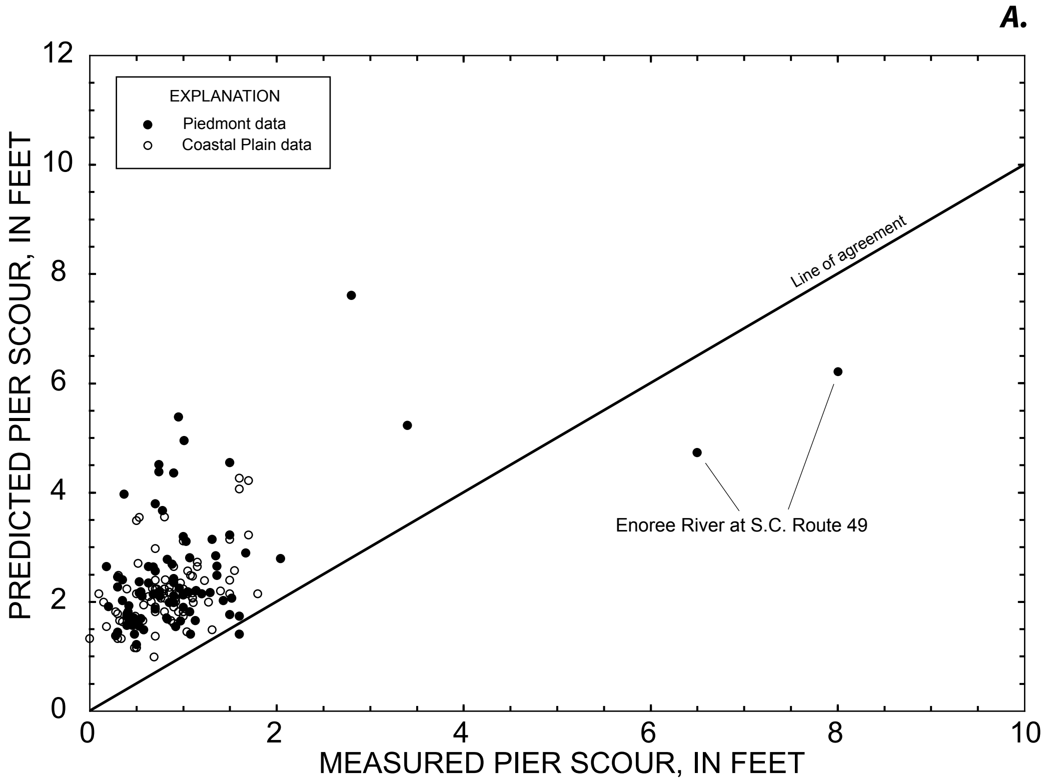


Figure 52. Relation of relative scour to the dimensionless variable, $(b/y_1)^3 Fr_1^2$, for laboratory data used to develop the original HEC-18 pier-scour equation (Richardson and others, 1991) and data from selected sites in South Carolina.

shape. The trend line through the laboratory data represents the original regression line developed for the CSU equation.

The South Carolina data have a much larger scatter than that of the laboratory data in figure 52; however, the slope of the trend line is very similar to that of the laboratory data, indicating that the trends in the South Carolina data are reasonable. It is interesting to note that all but three of the South Carolina measurements of pier scour plot below the line of the CSU equation. Two of the measurements above the line are pier-scour measurements at the S.C. Route 49 crossing of the Enoree River. When hydraulic variables from the 1995 flood are used instead of those from the 100-year flow, the data plot below the line. This trend indicates that equation 10 (which neglects the correction coefficients for pier shape, skew, sediment coarseness, and wide piers) will perform well in predicting clear-water pier-scour depths for field conditions in South Carolina. The relation of measured scour to predicted scour using equation 10 with hydraulic variables for the 100-year flow is shown in figure 53A. (Note: As mentioned previously, scour prediction at S.C. Route 49 crossing the Enoree River is best represented by hydraulics for the 1995 flood as displayed in figure 53B. Therefore, the underprediction of this site using the 100-year flow hydraulics as shown in figure 53A does not properly represent the trend of equation 10.) In figure 53B, the relation and trends are similar to those shown in figure 53A but include the hydraulics for the known maximum historic flows at the 51 bridges listed in table 2. When the trends of equation 10 (fig. 53) are compared with the trends of equation 1 (fig. 51), it is evident that equation 10 improves overall prediction by reducing excessive overprediction and maintaining minimal underprediction.

To test the performance of equation 10 on field data other than those collected in South Carolina, the equation was applied to all 503 pier-scour measurements from the NBSD. In figure 54A, measured and predicted pier scour were plotted for the NBSD, indicating that underprediction is minimal. Of the 503 measurements in the NBSD, 41 are underpredicted by equation 10, with a maximum underprediction of 2.9 ft, a minimum underprediction of 0.1 ft, and an average underprediction of 1 ft. The NBSD contains many pier-scour measurements that were influenced by complicated field conditions, including debris, tides, complex pier geometry, remnant scour holes developed by larger flows than those at the time of measurement, and severe skews exceeding 45 degrees. Even with these complicated conditions included (fig. 54A), use of equation 10 results in minimal underprediction. When NBSD data are screened to remove measurements with complicating factors, as mentioned above, and pier widths are limited to 6 ft or less to correspond to the pier widths of the South Carolina data, the underprediction for the 323 screened measurements is significantly reduced (fig. 54B), further indicating that equation 10 can be used to help reduce excessive overpredic-



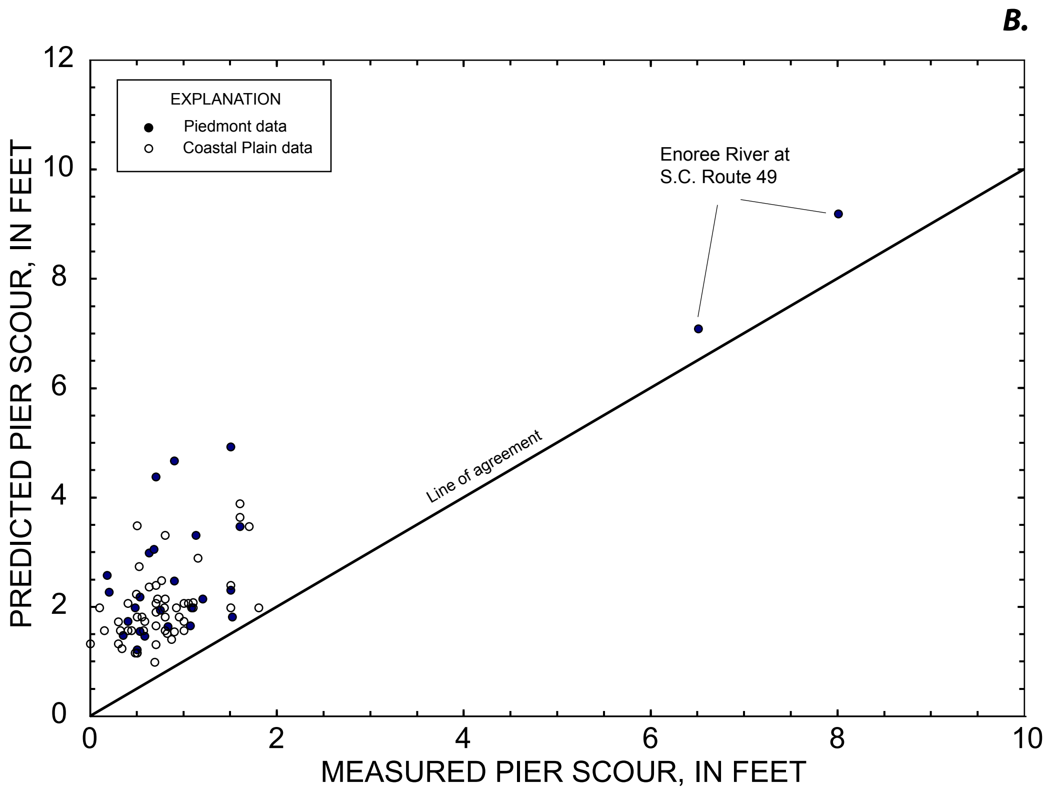


Figure 53. Relation of measured clear-water pier-scour depth to the predicted pier-scour depth for (A) the 100-year flow, and (B) known maximum historic flows at selected sites in South Carolina. (Predicted pier scour was calculated with the HEC-18 pier-scour equation (Richardson and Davis, 2001) with all correction coefficients set to 1.)

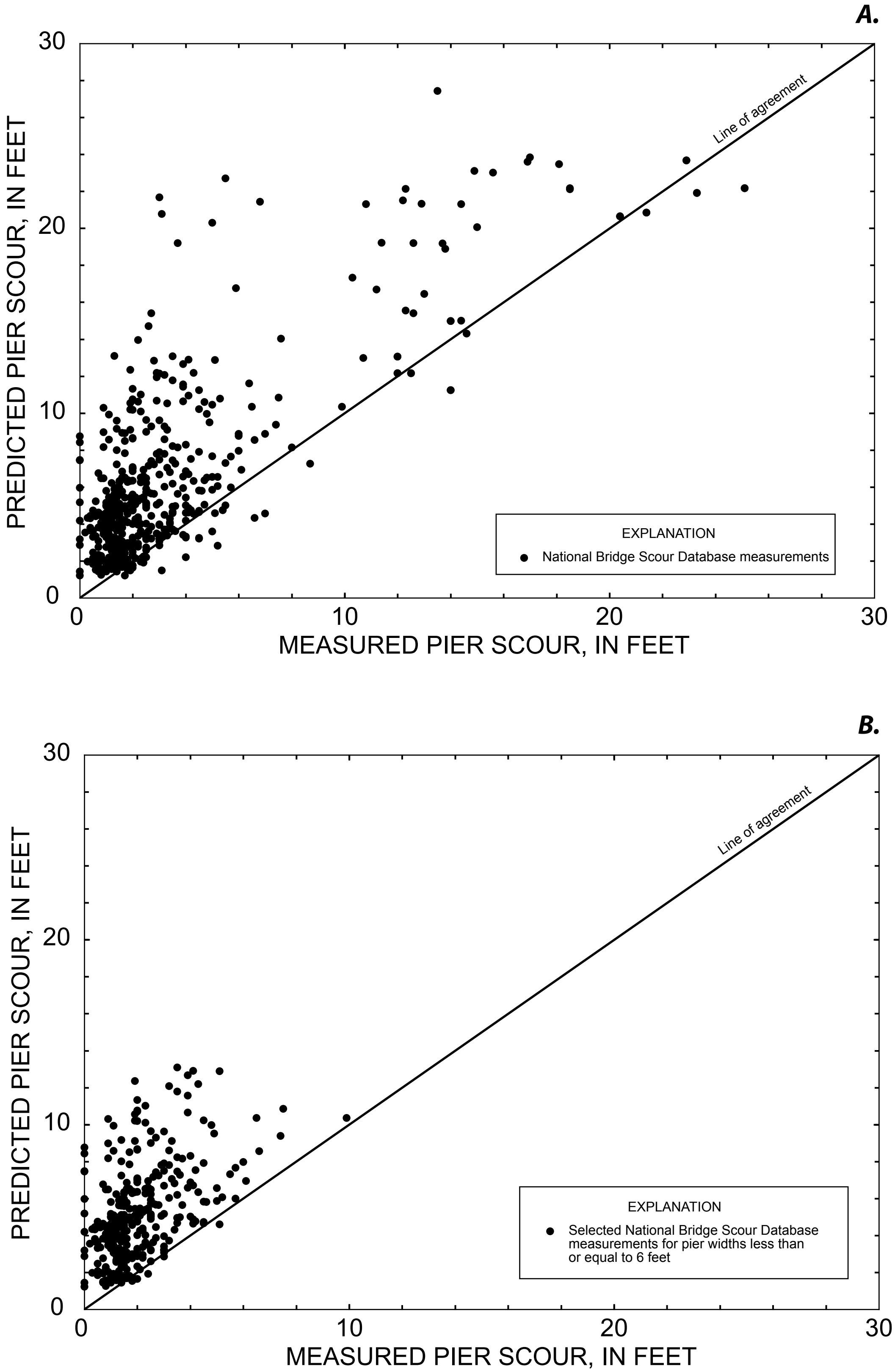


Figure 54. Relation of measured clear-water pier-scour depth to the predicted pier-scour depth for (A) all data in the National Bridge Scour Database, and (B) selected data with pier widths less than or equal to 6 feet. (Predicted pier scour was calculated with the HEC-18 pier-scour equation (Richardson and Davis, 2001) with all correction coefficients set to 1.)

tion and maintain minimal underprediction.

A review of the trends of equation 10 for the South Carolina data indicates that the larger overpredictions displayed in figure 53 can be associated with piers exceeding 2 ft in width. This trend indicates that it may be possible to include a pier-width correction for equation 10 to improve predictions at wider piers. Figure 55 shows the relation of pier width to prediction error (predicted scour minus measured scour) for equation 10 using hydraulics for the 100-year flow and the known maximum historic flow for the South Carolina data. (Note: The error for Route S.C. 49 crossing the Enoree River based on the 100-year flow pier-scour computations were excluded from this plot, because the August 1995 flood, which exceeds the 100-year flow, best represents the error of the equation.) The trend lines indicate that error magnitude increases with pier width. (It is interesting to note that a plot of the selected NBSD data used to develop figure 54 yields a similar trend line as in figure 55, indicating that the trend of increasing error with increasing pier width is not unique to the South Carolina data. This trend also is evident in the investigation of the NBSD by Mueller and Wagner (2005).) The envelope curve that encompasses the lower bound of the data in figure 55 provides a conservative adjustment for pier width that can be applied to equation 10 as follows:

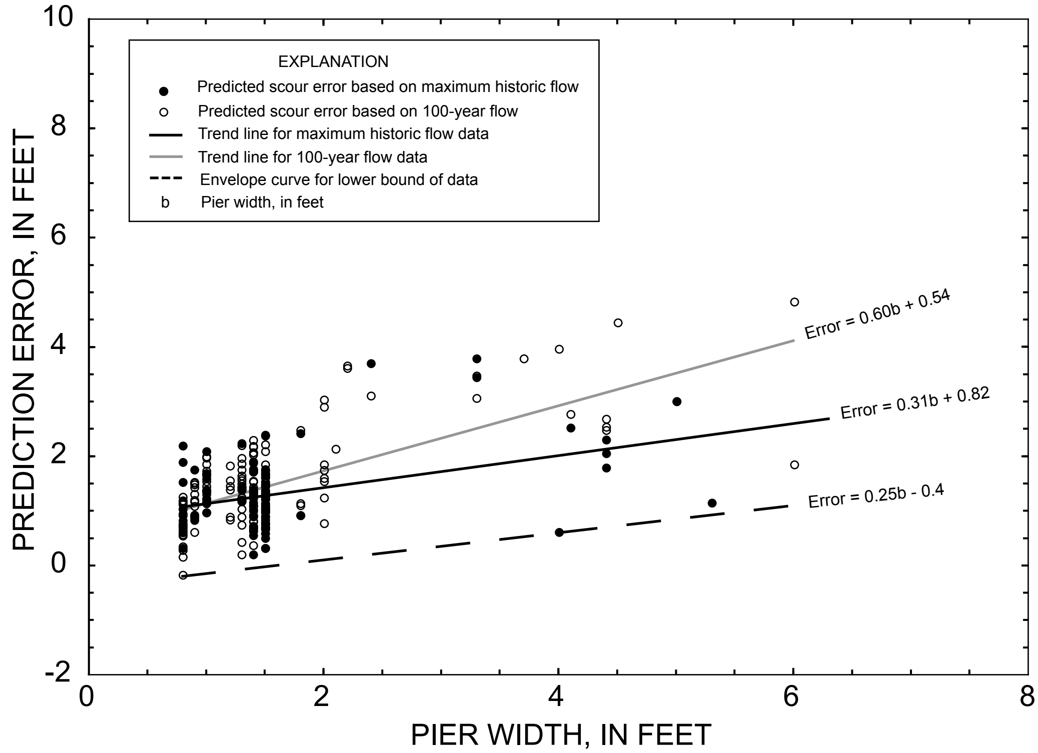


Figure 55. Relation of pier width to prediction error using hydraulic variables for the 100-year flow and known maximum historic flows at selected sites in South Carolina. (Predicted pier scour was calculated with the HEC-18 pier-scour equation (Richardson and Davis, 2001) with all correction coefficients set to 1.)

$$\frac{y_s}{y_1} = 2.0\left[\left(\frac{b}{y_1}\right)^3 Fr_1^2\right]^{0.215} - \frac{(0.25b - 0.4)}{y_1}, \qquad (11)$$

with all variables previously defined.

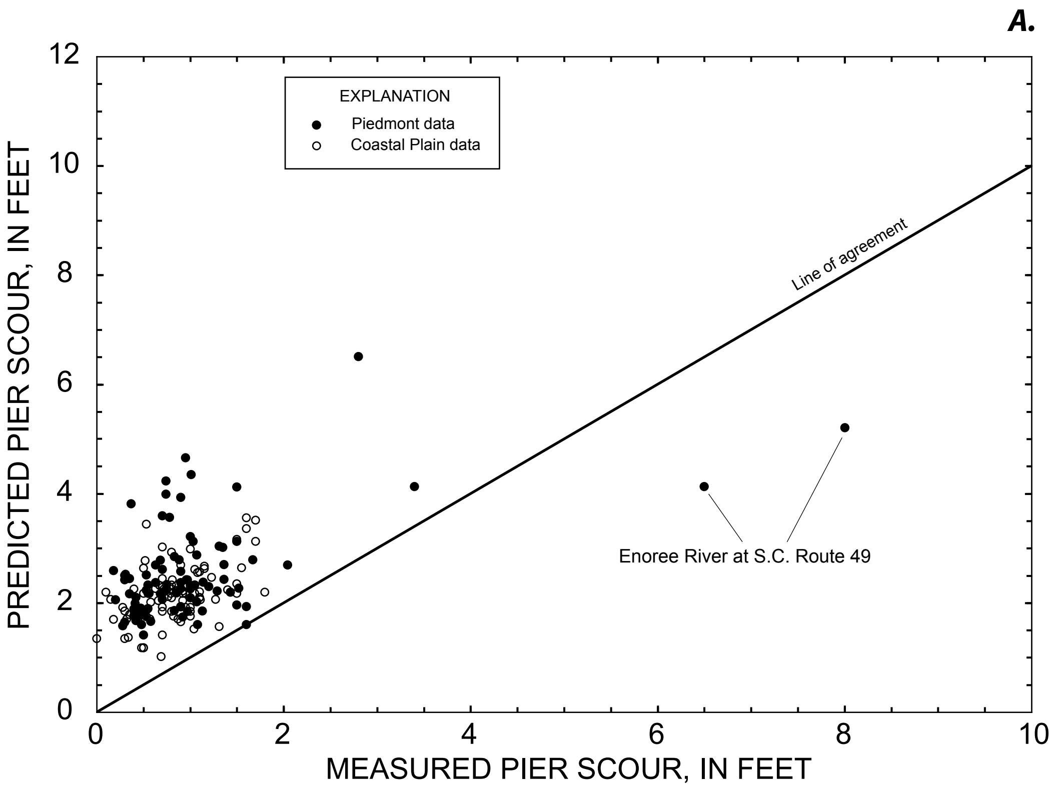
This adjustment slightly increases predicted scour for pier widths less than 1.6 ft but reduces predicted scour for pier widths greater than 1.6 ft. Equation 11 can be simplified and rearranged in the following format to directly solve for pier-scour depth:

$$y_s = 2.0 y_1^{0.35} b^{0.65} Fr_1^{0.43} - (0.25b - 0.4), \qquad (12)$$

with all variables previously defined.

Equation 12 will be called the South Carolina modified pier-scour equation for the remainder of the report. Because equation 12 was developed for pier widths of 6 ft or less under field conditions in South Carolina, it should not be applied to sites outside of these conditions.

The relation of measured scour to predicted scour using the South Carolina modified pier-scour equation (eq. 12) with hydraulic variables for the 100-year flow was plotted for the South Carolina data (fig. 56A). (Again, it should be noted that the underprediction at S.C. Route 49 crossing the Enoree River is erroneous and is best represented by hydraulics for the 1995 flood as displayed in figure 56B.) In figure 56B, a similar relation is shown using hydraulics for the known maximum historic flows at the 51 bridges listed in table 2, and the trends are similar to those in figure 56A. When the trends of equation 12 (fig. 56) are compared with the trends of equation 10 (fig. 53), it is evident that equation 12 moderately reduces the overprediction associated with pier widths greater than 2 ft. In addition, because of the slight increase for predicted scour at pier widths less than 1.6 ft, the minor underprediction of one measurement in figure 53A was removed.



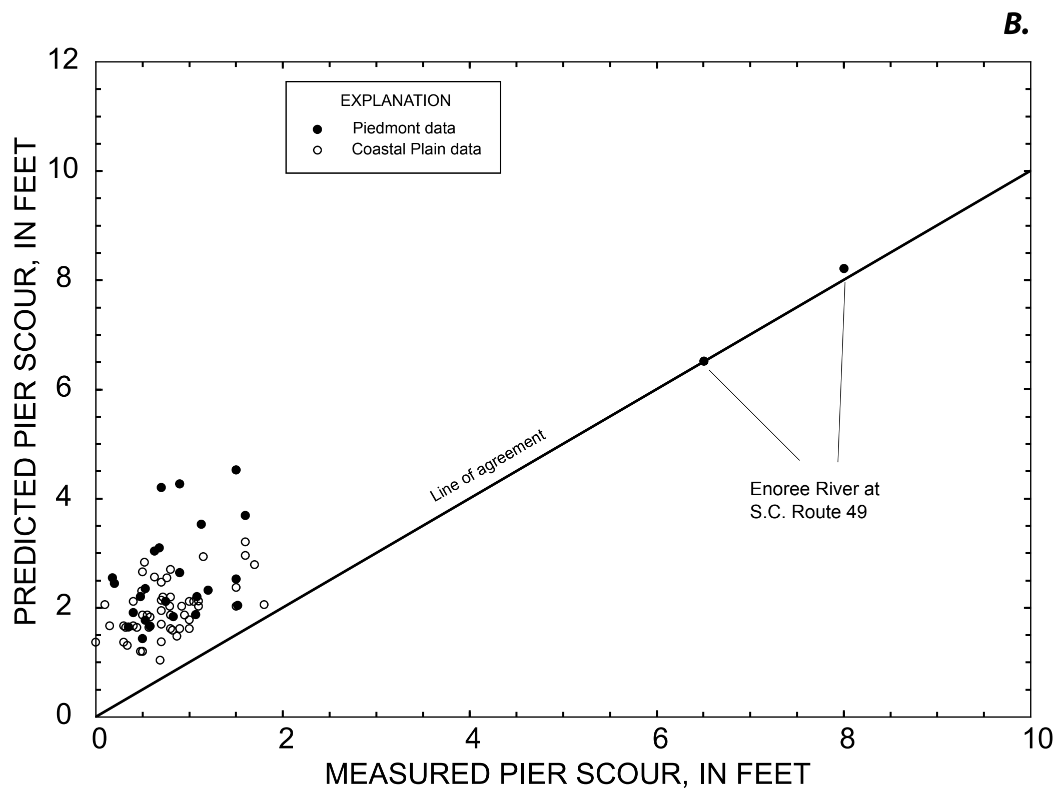


Figure 56. Relation of measured clear-water pier-scour depth to the predicted pier-scour depth for (A) the 100-year flow, and (B) known maximum historic floods at selected sites in South Carolina. (Predicted pier scour was calculated with the South Carolina modified pier-scour equation.)

To validate the South Carolina modified pier-scour equation (eq. 12), selected data from the NBSD were used for verification. The selected NBSD data used to test equation 12 (the same data used in fig. 54B) included 323 pier-scour measurements excluding sites affected by debris, sites with skews greater than 45 degrees, sites influenced by tides, sites with incomplete data, sites with remnant scour holes apparently produced by larger flows than at the time of flow measurement, and sites with pier widths greater than 6 ft. The pier width was limited to 6 ft because that was the limit of the South Carolina data. Additionally, large pier widths in the NBSD often are associated with complex pier geometries, making it difficult to determine the pier width that should be used when evaluating potential scour. In figure 57, the relation of measured and predicted scour for equation 12 using the selected data from the NBSD is plotted, indicating that underprediction is minimal. Of the 323 selected measurements, 15 were underpredicted by equation 12, with a maximum underprediction of 0.9 ft, a minimum underprediction of 0.1 ft, and an average underprediction of 0.3 ft. The similar results for the South Carolina and NBSD data indicate that the South Carolina modified pier-scour equation (eq. 12) is a viable equation for evaluating potential clear-water pier scour in South Carolina for pier widths of 6 ft or less.

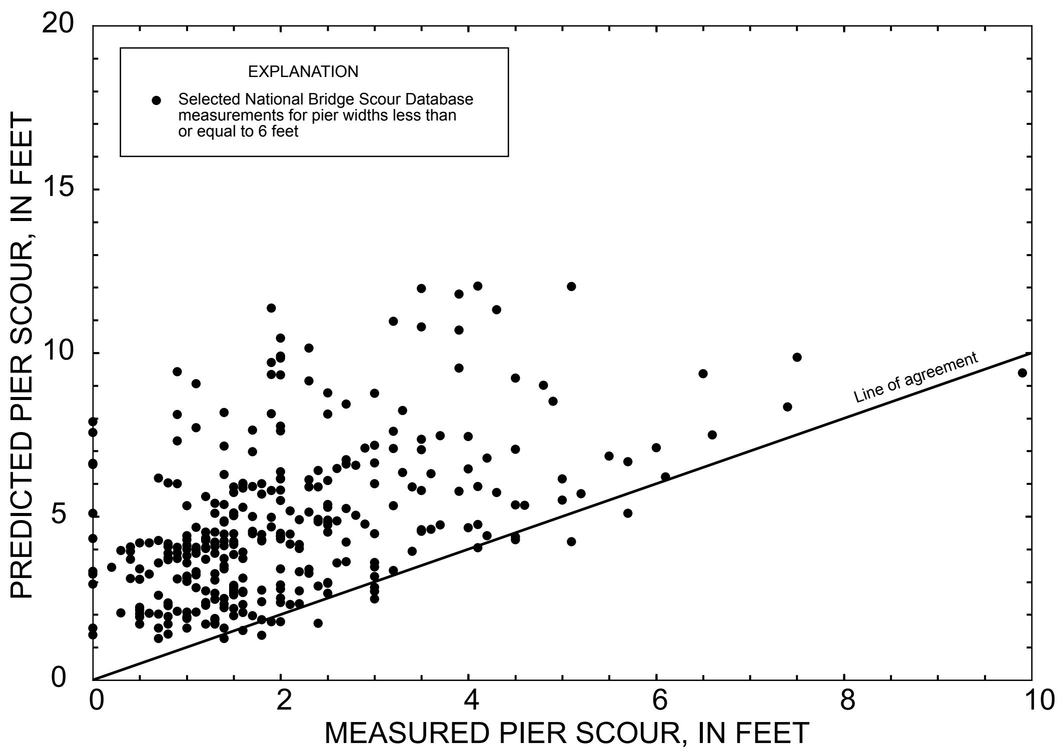


Figure 57. Relation of measured clear-water pier-scour depth to the predicted pier-scour depth for selected sites from the National Bridge Scour Database. (Predicted pier scour was calculated with the South Carolina modified pier-scour equation.)

The South Carolina Pier-Scour Envelope Curve

The South Carolina pier-scour envelope curve (fig. 46A; eq. 4) can be used to evaluate potential scour in South Carolina. Because the envelope curve represents the upper bound of observed data it will tend to overpredict pier scour, and at times this overprediction may be excessive. The relation of measured to predicted scour, determined with the South Carolina envelope curve (eq. 4), is plotted in figure 58 and indicates that overprediction can be as large as 7.5 ft. The relation of pier width to prediction error for equation 4 is shown in figure 59. The trend indicates that the South Carolina pier-scour envelope curve provides conservative, yet reasonable estimates of pier-scour depth for pier widths approximately 2 ft or less. However, larger overpredictions are often associated with pier widths exceeding 2 ft.

To validate the South Carolina pier-scour envelope curve (eq. 4) as a tool for obtaining quick, yet conservative estimates of pier-scour depth, all data from the NBSD (503 measurements) were used for verification. Pier width for these data range from 0.95 to 18.1 ft with measured pier-scour depth ranging from 0 to 25.1 ft. The relation of measured to predicted scour, based on equation 4, is shown in figure 60. Of the 503 measurements of pier scour, 26 were underpredicted by equation 4, with a maximum underprediction of 3.6 ft, a minimum underprediction of 0.1 ft, and an average underprediction of 1.1 ft. Overprediction was excessive at times, with a maximum value of 23.5 ft, and an average of 4.1 ft. Figure 60 also displays selected NBSD data that exclude measurements with complicating factors and limit pier width to 6 ft or less (same data used in figures 54B and 57). Of the 323 selected pier-scour measurements, 7 were underpredicted by equation 4, with a maximum underprediction of 1.3 ft, a minimum underprediction of 0.1 ft, and an average underprediction of 0.5 ft. Overprediction was not as excessive, having a maximum value of 8 ft and an average value of 3.6 ft. The trends in figure 60 indicate that the South Carolina pier-scour envelope curve (eq. 4) will, in general, provide quick and conservative estimates of pier-scour

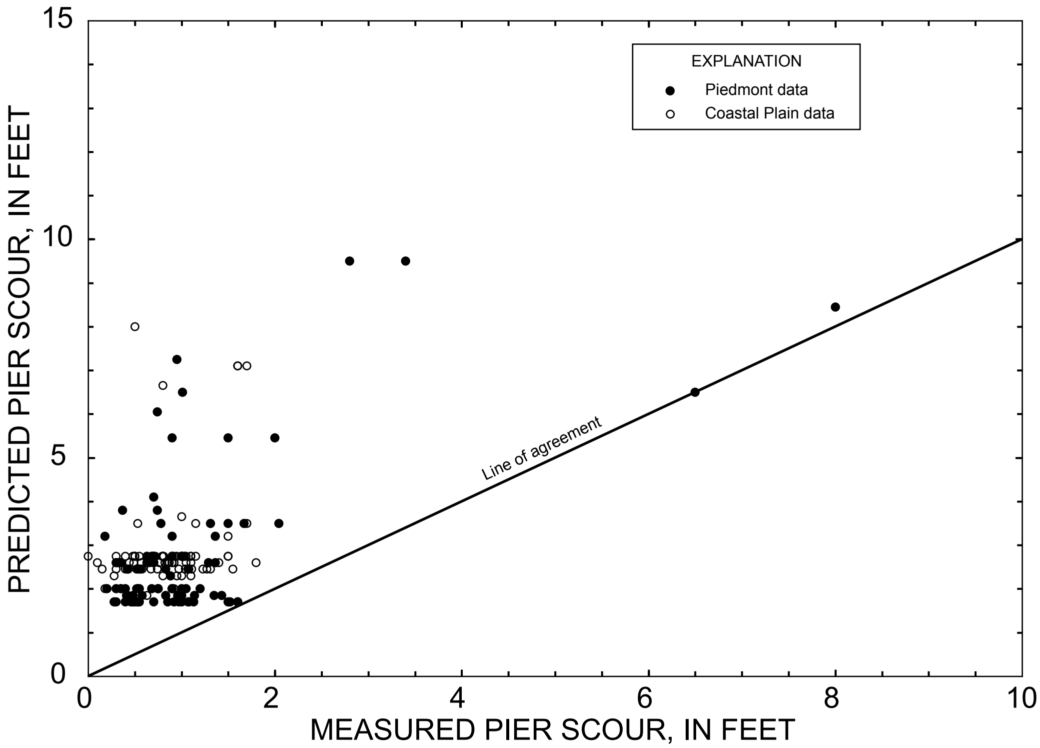


Figure 58. Relation of measured to predicted clear-water pier-scour depth for selected sites in the Coastal Plain and Piedmont Physiographic Provinces of South Carolina. (Predicted pier scour was calculated with the South Carolina pier-scour envelope-curve equation.)

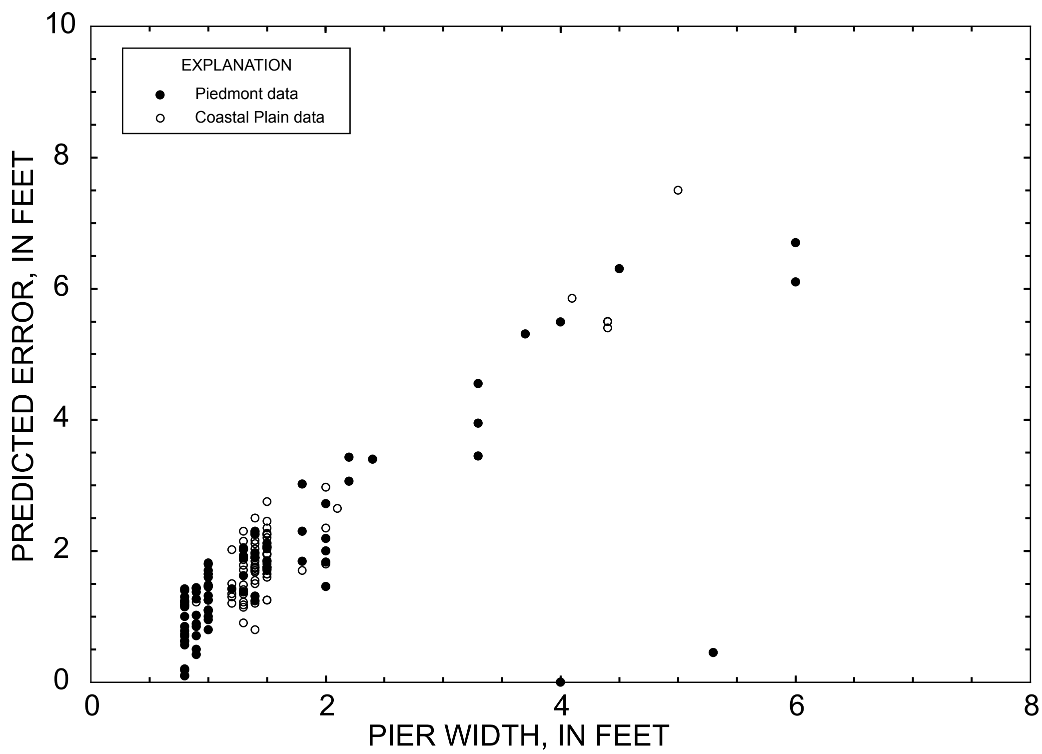


Figure 59. Relation of pier width to predicted error for clear-water pier-scour depth using the South Carolina pier-scour envelope-curve equation for selected sites in the Coastal Plain and Piedmont Physiographic Provinces of South Carolina.

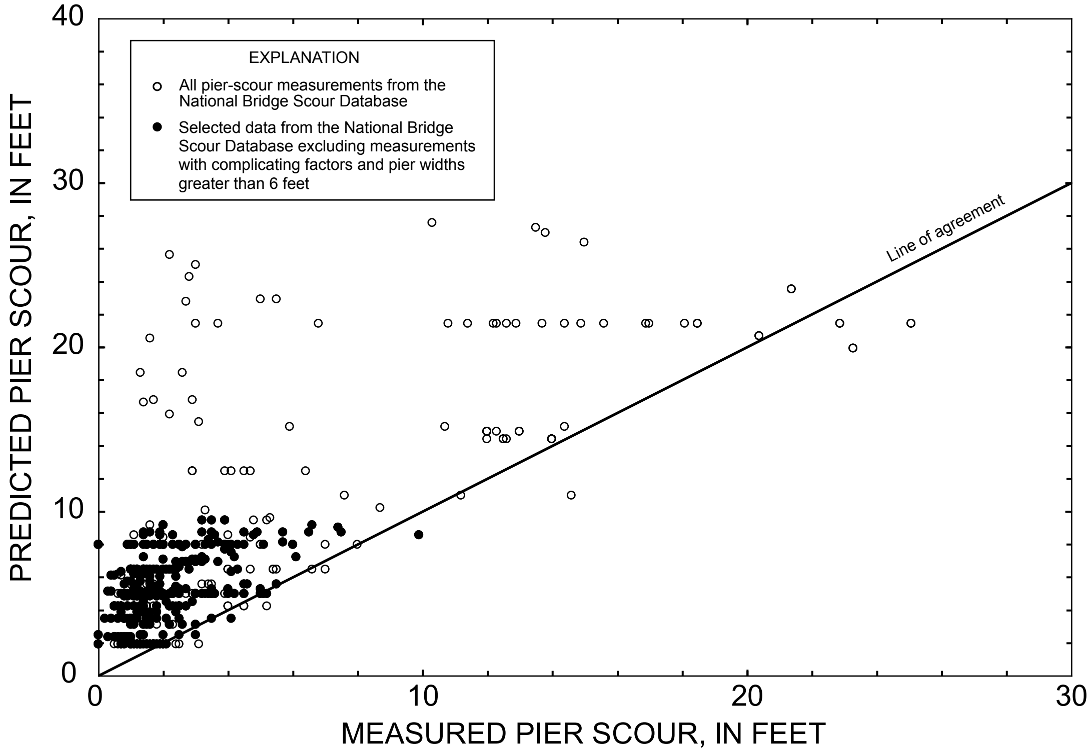


Figure 60. Relation of measured to predicted pier scour using the South Carolina pier-scour envelope-curve equation for all data in the National Bridge Scour Database.

depth with infrequent and minimal underprediction but at times excessive overprediction. Based on the range of the NBSD data, the equation appears to be applicable for pier widths as wide as 18.1 ft, but performs better when pier widths are 6 ft or less.

The relation of pier width to predicted scour for the HEC-18 pier-scour equation (Richardson and Davis, 2001; eq. 1), the South Carolina modified pier-scour equation (eq. 12), and the South Carolina pier-scour envelope curve (eq. 4) was plotted in figure 61. (The relation of measured scour with predicted scour for each of these equations (1, 12, and 4) is displayed in figures 51, 56, and 58, respectively.) Predicted scour in figure 61 was

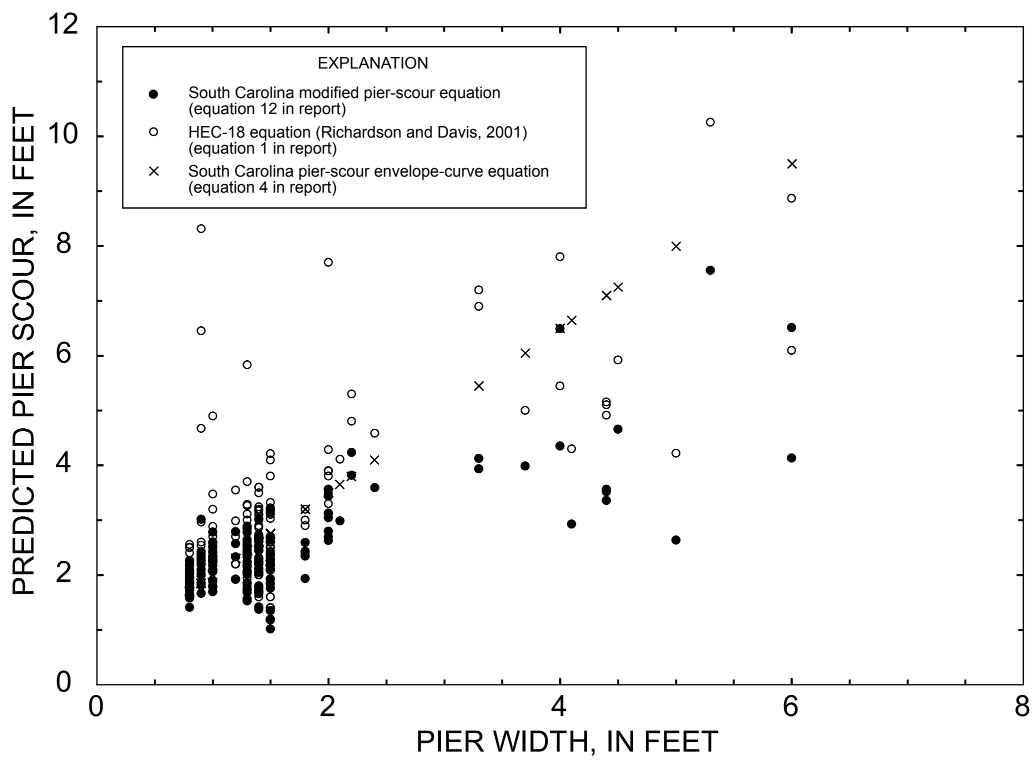


Figure 61. Relation of pier width to predicted pier-scour depth based on the South Carolina pier-scour envelope-curve equation, the HEC-18 pier-scour equation (Richardson and Davis, 2001), and the South Carolina modified pier-scour equation for selected sites in the Coastal Plain and Piedmont Physiographic Provinces of South Carolina.

based on the 100-year flow with the exception of the data for Route S.C. 49 crossing the Enoree River. The predicted pier scour at this bridge is best represented by the calculations with the maximum-historic flow and was used in the figure. It should be kept in mind that the predicted scour for all three equations as displayed in figure 61 equals or exceeds the measured scour. Therefore, the equation that displays the smallest predicted value for a given pier is performing the best for that particular pier. Additionally, the predicted values are plotted against pier width, providing some understanding of the performance of each equation for selected pier widths. For pier widths of 2 ft or less, figure 61 indicates that the South Carolina pier-scour envelope curve (eq. 4) and the South Carolina modified pier-scour equation (eq. 12) perform well and in general minimize some of the excessive overpredictions of the HEC-18 equation (eq. 1). However, for pier widths greater than about 2 ft, the data indicate that the South Carolina modified pier-scour equation (eq. 12) will provide better estimates of scour that minimize the overprediction that is associated with the South Carolina pier-scour envelope curve (eq. 4) and the HEC-18 equation (eq. 1).

Guidance For Evaluating Pier-Scour Depth in South Carolina

Using the findings of this investigation, the following guidance is provided for evaluating potential clear-water pier-scour depths in South Carolina. Scour prediction is an imprecise science, and the practitioner must rely on judgment when making a final estimate of pier scour. Current scour-prediction methods along with the field evaluation tools developed in this investigation should be used to make such evaluations. The scour-prediction methods developed in this investigation for evaluating scour are empirical, and application of the methods should be limited to sites with similar characteristics to those used in this investigation. Additionally, the envelope curves were developed from a limited sample of bridges in the Coastal Plain and Piedmont, and it is possible that scour depths could exceed the envelope curves. Therefore, it may be prudent to apply a safety factor to the envelope curves. Clear-water pier-scour data collected in this investigation were limited to pier widths of approximately 6 ft and less. Thus, guidance will be separated into categories for pier widths less than or equal to 6 ft and for pier widths greater than 6 ft.

Evaluating Scour Depth at Pier Widths Less Than or Equal to 6 Feet

For pier widths less than or equal to 6 ft, empirical methods developed in this investigation can be used to help evaluate the potential for clear-water pier scour. It is important, however, to initially determine if the site of interest has characteristics similar to those used in this study. This can be accomplished by comparing characteristics at the site of interest to those presented in tables 6 and 7 and figures 2 and 3 that display the range and trend of selected site characteristics for this investigation. If site conditions are similar to those used in this investigation, the following guidance can be applied. It is important that the methods presented in this report only be applied to clear-water scour conditions in the floodplains of South Carolina. These methods should not be used to evaluate pier scour in the main river channels. Additionally, the methods presented in this report are not intended for tidally influenced sites, for bridges in pressure flow, or for sites where pier scour may be influenced by debris.

Table 6. Range of selected characteristics for 87 measurements of clear-water pier scour collected at 53 bridges in the Piedmont of South Carolina.

[mi^2, square mile; ft/ft, feet per foot; ft/s, feet per second; ft, feet; mm, millimeter]

Characteristic	Minimum value	Median value	Maximum value
Drainage area (mi^2)	10.7	81.5	1,620[a]
Channel slope determined from topographic map (ft/ft)	0.00015	0.0012	0.0029
100-year average approach velocity at pier face (ft/s)[b]	1.3	3.1	6.2
100-year average approach depth at pier face (ft)[b]	1.0	5.4	14.6
Pier width (ft)	0.8	1	6
Pier skew (degrees)	0	0	22
Median grain size (mm)	less than 0.062	0.105	0.990
Observed pier-scour depth (ft)	0	0.85	8

[a]Approximately 94 percent of the study sites in the Piedmont have drainage areas less than 400 mi^2 (fig. 3).

[b]Values were estimated from the one-dimensional water-surface profile model, WSPRO (Shearman, 1990).

Table 7. Range of selected characteristics for 92 measurements of clear-water pier scour collected at 63 bridges in the Coastal Plain of South Carolina.

[mi^2, square mile; ft/ft, feet per foot; ft/s, feet per second; ft, feet; mm, millimeter]

Characteristic	Minimum value	Median value	Maximum value
Drainage area (mi^2)	26.3	586	13,000[a]
Channel slope determined from topographic map (ft/ft)	0.00007	0.00033	0.00092
100-year average approach velocity at pier face (ft/s)[b]	0.4	1.9	5.4
100-year average approach depth at pier face (ft)[b]	2	6.3	17.3
Pier width (ft)	0.9	1.4	5
Pier skew (degrees)	0	0	20
Median grain size (mm)	less than 0.062	0.162	0.556
Observed pier-scour depth (ft)	0.0	0.8	1.8

[a]Approximately 80 percent of the study sites in the Coastal Plain have drainage areas less than 1,420 mi^2 (fig. 3).

[b]Values were estimated from the one-dimensional water-surface profile model, WSPRO (Shearman, 1990).

For quick evaluations of potential pier scour without the use of hydraulic data, the South Carolina pier-scour envelope curve (eq. 4) can be used. Equation 4 will provide reasonable estimates of scour without excessive overprediction for pier widths of approximately 2 ft or less. When pier width increases beyond approximately 2 ft, the potential for excessive overprediction increases. If the larger pier-scour predictions associated with pier widths greater than 2 ft are acceptable for practical purposes, there is no need for refinement of the estimated clear-water pier-scour depth. However, if a refinement is desired, the South Carolina modified pier-scour equation (eq. 12) can be used. Predicted scour from equations 4 and 12 should be compared, and the smallest value should be selected as the best estimate of clear-water pier-scour depth. After estimating potential scour by using equation 4 or 12, the South Carolina Clear-Water Pier- and Contraction-Scour Database (SCPCSD; appendix 1) and the NBSD should be queried for comparison sites that can be used to evaluate the reasonableness of the estimated scour. After evaluating pier-scour depth based on methods developed in this investigation, it would be prudent to compare this evaluation with the HEC-18 equation (Richardson and Davis, 2001) before making a final estimate of scour potential. The South Carolina pier-scour envelope curve was developed using field data from sites with flows approaching the 100-year flow and, therefore, should not be used to evaluate clear-water pier-scour depths for extreme conditions, such as the 500-year flow.

Special consideration must be given to piers that are skewed to the approaching flow. For solid piers, the skew-correction coefficient used in the HEC-18 pier-scour equation (Richardson and Davis, 2001) should be applied to equations 4 and(or) 12. Judgment then must be used to select the most appropriate estimate of scour at the skewed pier. In the case of pile bents or multiple columns that have moderate skews (approximately 20 degrees or less) and spacings between piles of approximately 5 pier widths or greater, equations 4 and(or) 12 can be used with no adjustment for skew. When skews or pile spacings exceed these limits, the HEC-18 pier-scour equation (Richardson and Davis, 2001) can be used with the recommended correction for skew. An alternative may be to use the HEC-18 pier-scour equation with the skew-correction coefficients as recommended by Melville and Coleman (2000) and listed in table 5. This likely would minimize the potential for overprediction associated with skewed pile bents, but judgment should be used to assure that the predicted values are reasonable.

Evaluating Scour Depth at Pier Widths Greater Than 6 Feet

When pier widths in South Carolina exceed 6 ft, the methods presented in this report are not applicable. It is particularly important that equation 12 not be used, because the adjustment for pier width can produce excessive adjustments at very wide piers and, thus, underpredictions. The plotted data in figure 60 indicate that the South Carolina pier-scour envelope curve (eq. 4) performs fairly well with the NBSD data when pier width exceeds 6 ft; therefore, it is reasonable to assume that equation 4 can be used in South Carolina for pier widths greater than 6 ft. It is anticipated, however, that this equation may provide some excessive overprediction for wide piers. Other alternatives for evaluating scour at pier widths greater than 6 ft are the HEC-18 equation (eq. 1) or equation 10. The performance of equation 10 with data from the NBSD (fig. 54) suggests that this is a good alternative to the HEC-18 equation; however, caution and judgment should be used in selecting a final value for predicted scour. Because the effect of skew on wide piers can be substantial, the skew correction coefficient for the HEC-18 pier-scour equation (Richardson and Davis, 2001) should be applied to equation 10. After estimating potential scour, the NBSD should be queried for comparison sites that can be used to evaluate the reasonableness of the estimated scour.

Evaluating Scour-Hole Top Width

In addition to evaluating pier-scour depth at a given site, it is important to consider the scour-hole geometry and location. Both laboratory and field data indicate that pier-scour holes are located in close proximity to the pier (figs. 10–12) and for practical purposes can be assumed to be symmetrical about the pier. HEC-18 (Richardson and Davis, 2001) recommends the following equation for estimating the top width of a pier-scour hole.

$$TW = 4y_s + b \tag{13}$$

where

TW is the top width of the scour hole, in feet;
y_s is the pier-scour depth, in feet; and
b is the pier width, in feet.

The relation of measured scour-hole top width to predicted scour-hole top width estimated with equation 13 and the observed scour depth in the Coastal Plain and Piedmont of South Carolina was plotted (fig. 62). The HEC-18 equation (eq. 13) for estimating the top width of a pier-scour hole frequently underpredicts the measured top width. Figure 63

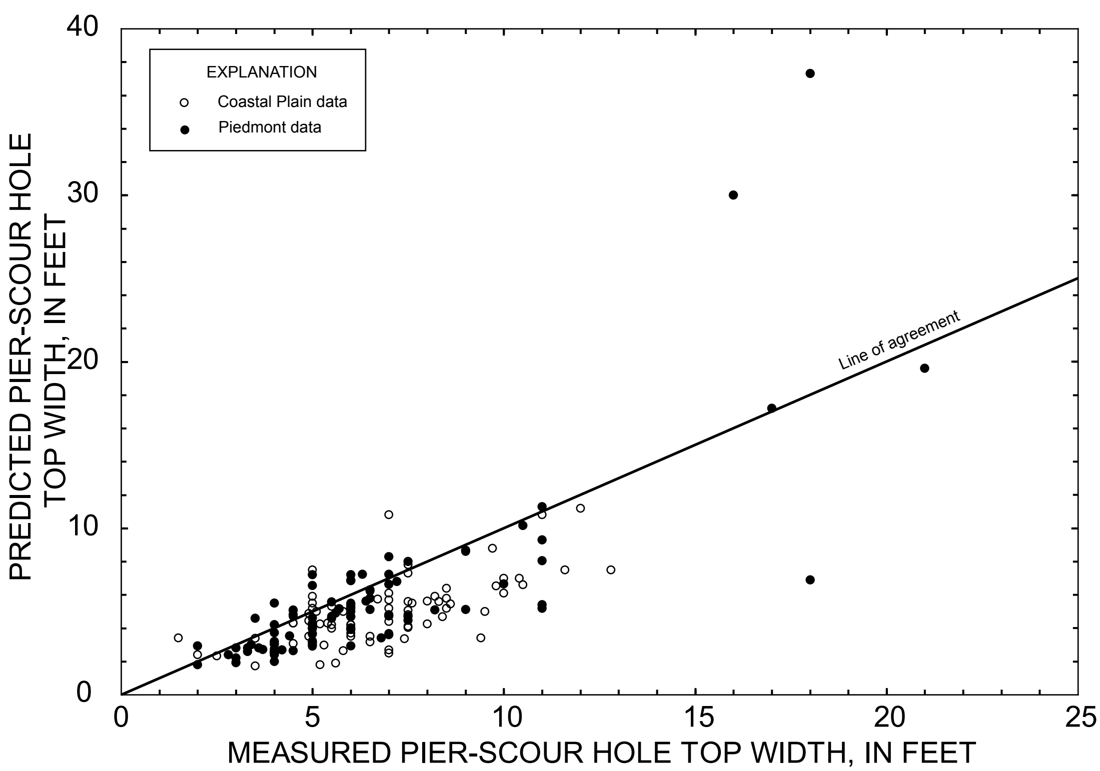


Figure 62. Relation of measured scour-hole top width to predicted scour–hole top width based on the HEC-18 equation (Richardson and Davis, 2001) for selected sites in the Coastal Plain and Piedmont Physiographic Provinces of South Carolina.

shows the relation of the measured pier-scour depth to measured top width minus the pier width (*TW–b*). The pier width was subtracted from the top width to determine if the value of 4 used as the slope in equation 13 is reasonable. Neglecting the two outliers on the far right of figure 63, and forcing the y-intercept to 0, the trend line through the data has a slope of 5.1. Therefore, it may be more reasonable to modify equation 13 as follows:

$$TW = 5.1y_s + b, \quad (14)$$

with variables previously defined. Equation 14 is an average line and will have an under-prediction rate of about 50 percent; therefore, it should be used with caution. Additionally, data used to develop equation 14 are sparse beyond a pier-scour depth of about 2 ft, and the equation should be used with caution for scour depths exceeding this value.

The relation of pier width to scour-hole top width for the measured pier scour in the Coastal Plain and Piedmont of South Carolina was plotted (fig. 64). The trend line through the data indicates that the top width of the pier-scour hole increases with pier width. The upper-bound envelope

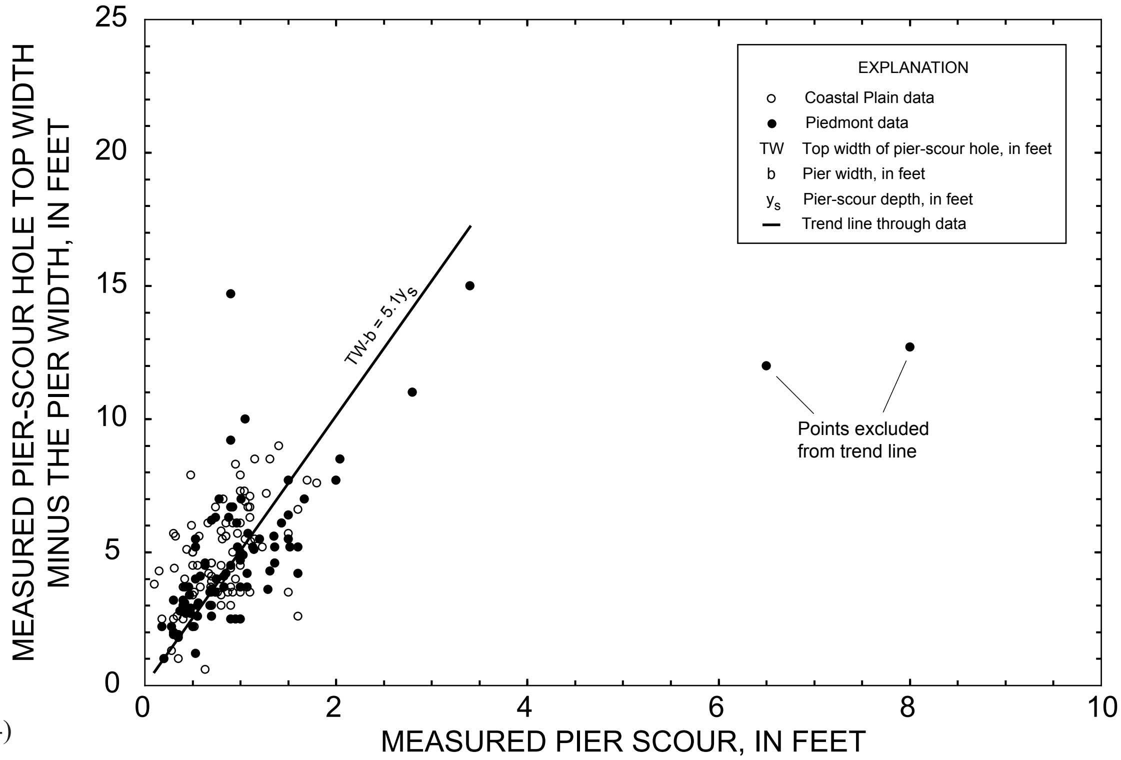


Figure 63. Relation of measured pier-scour depth to scour-hole top width minus the pier width for selected sites in the Coastal Plain and Piedmont Physiographic Provinces of South Carolina.

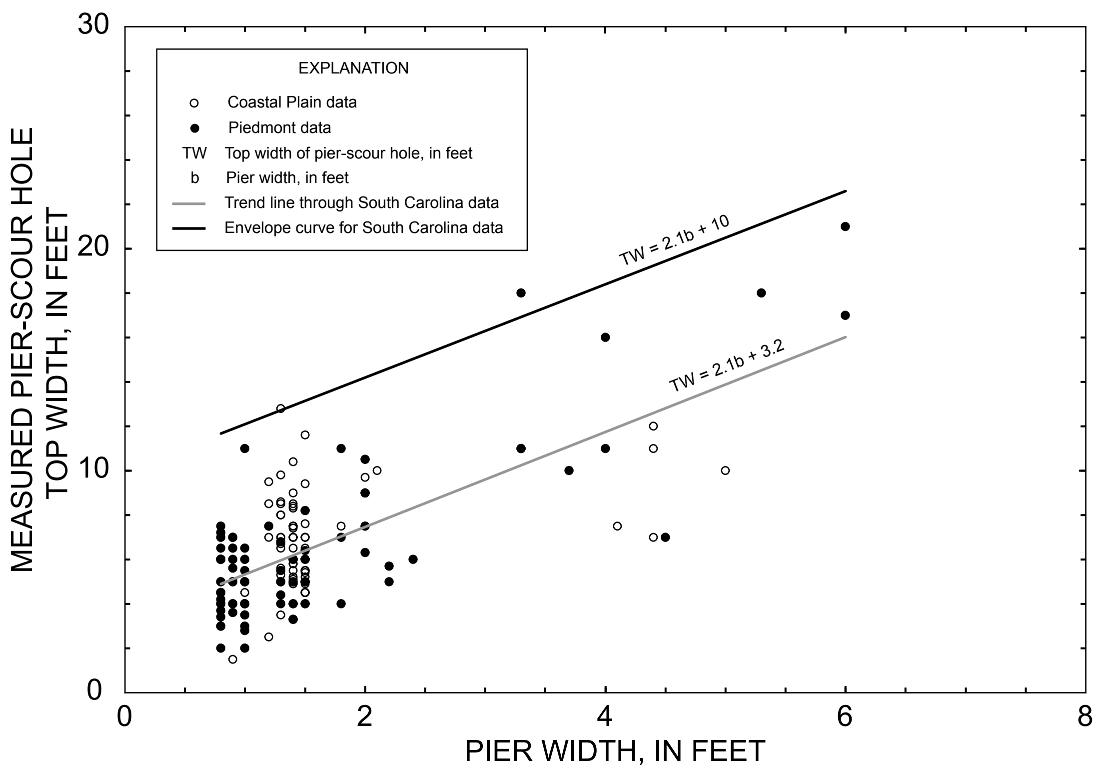


Figure 64. Relation of pier width to scour-hole top width for selected sites in the Coastal Plain and Piedmont Physiographic Provinces of South Carolina.

curve was developed by arbitrarily shifting the trend line to encompass most of the data (fig. 64). The trend line can be used to estimate an average top width of the scour hole for pier widths 6 ft or less. For a more conservative estimate of the top width, the upper-bound envelope curve can be used. Because data are sparse for pier widths beyond 2 ft, the method shown in figure 64 should be used with caution for estimating scour-hole top widths beyond this value.

Development of the South Carolina Clear-Water Contraction-Scour Envelope Curves

The previous investigation of clear-water contraction scour (Benedict, 2003) identified weaknesses in the current scour-prediction methods and developed a field-data envelope curve for improved evaluation of clear-water contraction scour in the Piedmont of South Carolina. Using an approach similar to Benedict (2003), contraction-scour data collected in the Coastal Plain of South Carolina were analyzed and several envelope curves that display the range and trend of clear-water contraction scour in South Carolina were investigated. The contraction-scour data in the Piedmont and Coastal Plain have similar trends, and the envelope curves developed in this investigation represent the upper bound of scour for both regions. Following is a description of the field data, a comparison of measured and predicted scour using the HEC-18 equation (Richardson and Davis, 2001), a description of several envelope curves, and guidance for evaluating the potential for clear-water contraction scour in the Coastal Plain and Piedmont of South Carolina.

Clear-Water Contraction Scour in the Coastal Plain and Piedmont

Densely vegetated floodplains in combination with small flow velocities (see previous section “Clear-Water Scour Conditions”) promote clear-water scour at bridge overbanks in the Coastal Plain and Piedmont of South Carolina. The primary mechanism that causes contraction scour under clear-water scour conditions typically is associated with increased flow velocities generated by bridge contraction. Although approaching velocities on the upstream floodplain are insufficient to transport bed sediments into the bridge (fig. 4), the contraction of flow forced by the bridge causes increased flow velocities within the bridge opening that, in turn, erode bed material at the bridge. As contracted flow leaves the bridge opening, it expands, velocities begin to decrease, and the potential for scour diminishes. Thus, clear-water contraction scour on bridge overbanks typically occurs in close proximity to the bridge.

In the current and previous (Benedict, 2003) investigations, observed clear-water contraction scour on bridge overbanks typically was limited to beneath the bridge and commonly formed a shallow, parabolic depression running parallel to the bridge and perpendicular to the flow (fig. 9). The area of clear-water scour varies slightly for bridges crossing streams with well-defined channels and bridges crossing floodplains or swamps with poorly defined channels. In the former case, the area of clear-water scour typically occurs between the abutment toe and the top of the channel bank (fig. 22). In the latter case, the area of clear-water scour typically occurs between the abutment toes (fig. 23). When a bridge has long embankments that block approaching flows, it is common for relatively deep clear-water abutment-scour holes to develop in close proximity to the abutment toes, with smaller clear-water contraction scour depths developing beyond this area (fig. 9). An example of this pattern can be seen at structure 274000300200 on S.C. Route 3 crossing Cypress Creek in Jasper County (fig. 65). The left and right abutment-scour holes are 10.8 and 14.4 ft deep, respectively, and are located in close proximity to the abutment toes. Beyond the abutment-scour holes, the clear-water contraction-scour depth is significantly less, with a maximum depth of approximately 2 ft. Clear-water contraction-scour depths for the current and previous (Benedict, 2003) investigations were collected in the clear-water contraction-scour area, as defined in figure 9, outside the area of abutment scour.

A total of 64 measurements of clear-water contraction scour were collected at 53 bridges in the Coastal Plain of South Carolina, including 8 bridges in the upper Coastal Plain and 45 in the lower Coastal Plain. Scour depths ranged from 0 to 3.9 ft with a median scour depth of 1.8 ft. Overbank widths (clear-water contraction scour area as defined in figs. 22 and 23) ranged from 51.7 to 4,100 ft with a median width of 238 ft. The soils of the Coastal Plain generally are sandy (with occasional clayey soils), and the D_{50} for the 53 bridges ranged from less than 0.0002 to 0.0018 ft (less than 0.062 to 0.56 mm) with a median of 0.00056 ft (0.17 mm). Table 8 lists the range of selected characteristics associated with the 64 clear-water contraction-scour measurements. Values for hydraulic data were estimated with the WSPRO model using the 100-year flow.

A total of 75 measurements of clear-water contraction scour were collected at 52 bridge sites in the Piedmont of South Carolina, including 16 bridges in the high-flow region (fig. 1) (Guimaraes and Bohman, 1992). Scour depths ranged from 0 to 4.5 ft with a median scour depth of 0.8 ft. Overbank widths ranged from 7 to 684 ft with a median width of 71 ft. The floodplain soils of the Piedmont generally are clayey with varying degrees of cohesion, and the D_{50} for the 52 bridges ranged from less than 0.0002 to 0.0032 ft (less than 0.062 to 0.99 mm) with a median of 0.0003 ft (0.09 mm). Table 9 lists the range of selected characteristics associated with the 75 clear-water contraction-scour measurements. Values for hydraulic data were estimated with the WSPRO model using the 100-year flow.

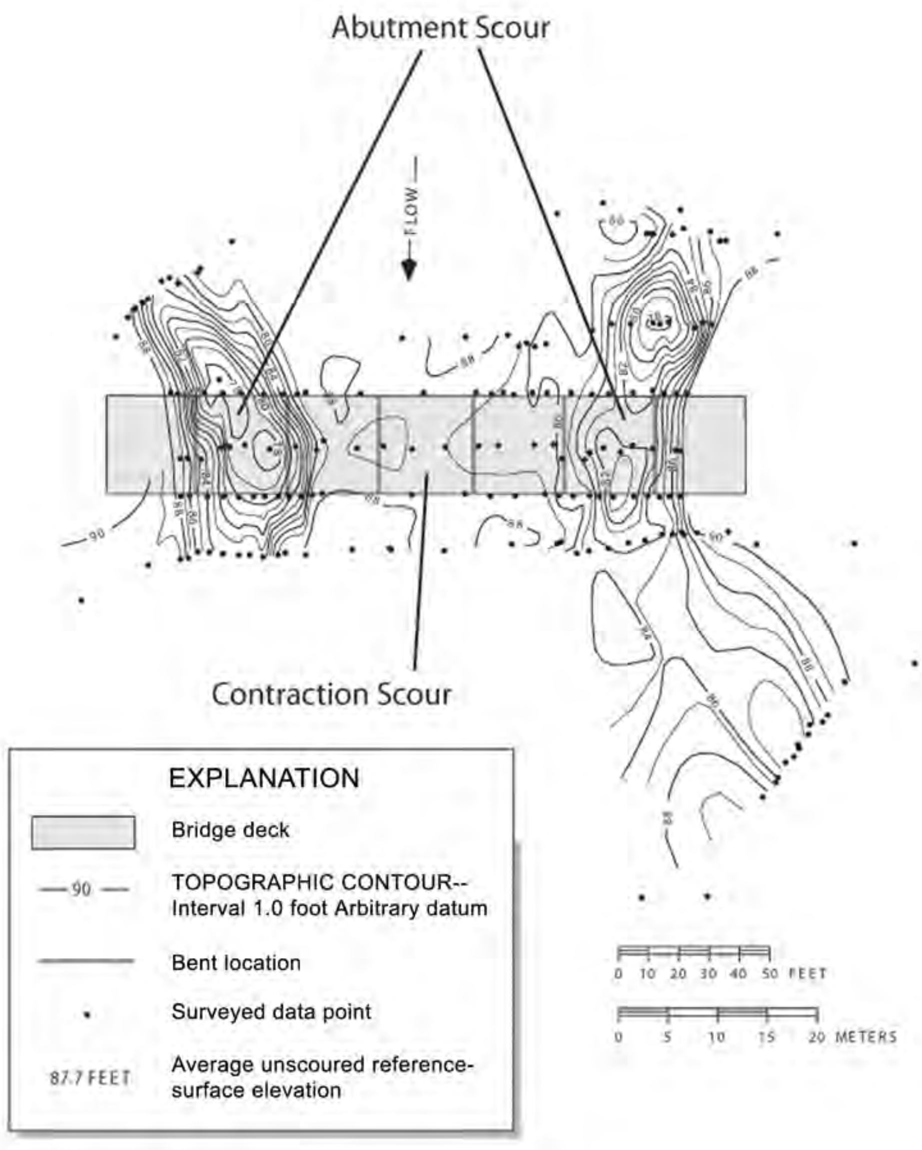


Figure 65. Example of clear-water abutment- and contraction-scour areas at structure 274000300200 on S.C. Route 3 crossing Cypress Creek in Jasper County (December 9, 1996).

Table 8. Range of selected characteristics for 64 measurements of clear-water contraction scour collected at 53 bridges in the Coastal Plain of South Carolina.

[mi², square mile; ft/ft, feet per foot; ft/s, feet per second; ft, feet; mm, millimeter]

Characteristic	Minimum value	Median value	Maximum value
Drainage area (mi^2)	26.3	586	13,000[a]
Channel slope determined from topographic map (ft/ft)	0.00007	0.00037	0.00092
100-year flow average overbank velocity at the bridge (ft/s)[b]	0.5	2.0	6.7
100-year flow average overbank depth at the bridge (ft)[b]	4.2	6.6	17.9
Median grain size (mm)	Less than 0.062	0.17	0.56
Observed clear-water contraction-scour depth (ft)	0.0	1.8	3.9

[a]Approximately 80 percent of the study sites in the Coastal Plain have drainage areas less than 1,420 mi^2 (fig. 3).

[b]Values were estimated from the one-dimensional water-surface profile model, WSPRO (Shearman, 1990).

Table 9. Range of selected characteristics for 75 measurements of clear-water contraction scour collected at 52 bridges in the Piedmont of South Carolina (modified from Benedict, 2003).

[mi², square mile; ft/ft, feet per foot; ft/s, feet per second; ft, feet; mm, millimeter]

Characteristic	Minimum value	Median value	Maximum value
Drainage area (mi^2)	11	81	1,620[a]
Channel slope determined from topographic map (ft/ft)	0.00015	0.0012	0.0029
100-year flow average overbank velocity at the bridge (ft/s)[b]	1.7	3.1	5.8
100-year flow average overbank depth at the bridge (ft)[b]	1.3	7.9	20.5
Median grain size (mm)	less than 0.062	0.09	0.99
Observed clear-water contraction-scour depth (ft)	0.0	0.8	4.5

[a]Approximately 97 percent of the study sites in the Piedmont have drainage areas less than 400 mi^2 (fig. 3).

[b]Values were estimated from the one-dimensional water-surface profile model, WSPRO (Shearman, 1990).

Comparison of Measured and Predicted Contraction-Scour Depths

To predict potential contraction-scour depth for clear-water scour conditions, HEC-18 (Richardson and Davis, 2001) recommends the use of a modified version of Laursen's (1963) equation (eq. 2), which was derived for noncohesive sediments. The relation of measured to predicted clear-water contraction-scour depths for selected sites in the Coastal Plain of South Carolina is shown in figure 66. Predicted scour was calculated using the 100-year flow and the maximum historic flow, and trends for both flow conditions are similar. A large scatter can be seen in figure 66 around the line of agreement with frequent overprediction. Although overprediction is desirable for design purposes, the trends in figure 66 indicate that the HEC-18 clear-water contraction-scour equation can be excessive at times. Of greater concern, however, is the frequent underprediction (approximately 30 percent) produced by the equation, highlighting the need to use judgment in the application of the equation. The HEC-18 equation predicts the average clear-water contraction scour across the area of anticipated scour; however, the field data represent the maximum observed scour within this region. This difference between observed maximum and predicted average scour may, in part, account for some of the underprediction. Additionally, the estimate of the median grain size may be in error because of the difficulty of obtaining a representative sample under field conditions (Benedict 2003; Mueller and Wagner, 2005).

To better understand the trends of the HEC-18 clear-water contraction-scour equation (Richardson and Davis, 2001), field data from the Piedmont of South Carolina (Benedict, 2003), the NBSD, and Hayes (1996) were plotted with the Coastal Plain data (fig. 67). Published data for field measurements of clear-water contraction scour are limited, and most of the data identified by Mueller and Wagner (2005) were used in figure 67. The Piedmont data include 75 measurements in clayey soils, and the flow conditions associated with the measurements were estimated with the 100-year flow. (Benedict (2003) used the clear-water contraction scour equation from Richardson and others (1993) which is slightly different from the equation in Richardson and Davis (2001). In the current investigation, predicted clear-water contraction scour for the Piedmont was recomputed using the Richardson and Davis (2001) equation. Therefore, predicted clear-water contraction scour for Piedmont sites in this investigation will be slightly different from Benedict (2003).) Although underprediction was infrequent for Piedmont data, the overprediction was often excessive. This most likely can be attributed to the smaller grain sizes and larger flow velocities associated with the Piedmont sites. The data from the NBSD and Hayes (1996) included 35 measurements at 10 sites in Alaska, Maryland, Montana, and Ohio all having sediments in the gravel-to-

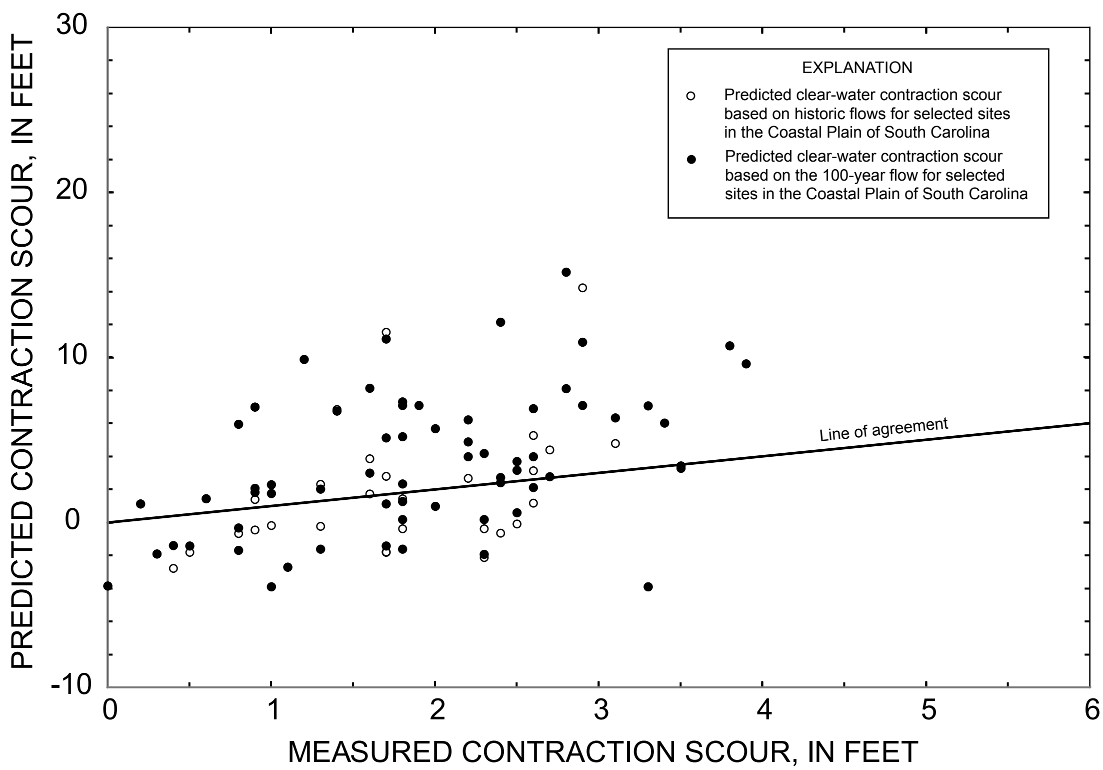


Figure 66. Relation of measured clear-water contraction-scour depth to predicted contraction-scour depth for the 100-year flow and known maximum historic floods at selected sites in the Coastal Plain Physiographic Province of South Carolina. (Predicted clear-water contraction scour was calculated with the HEC-18 equation (Richardson and Davis, 2001).)

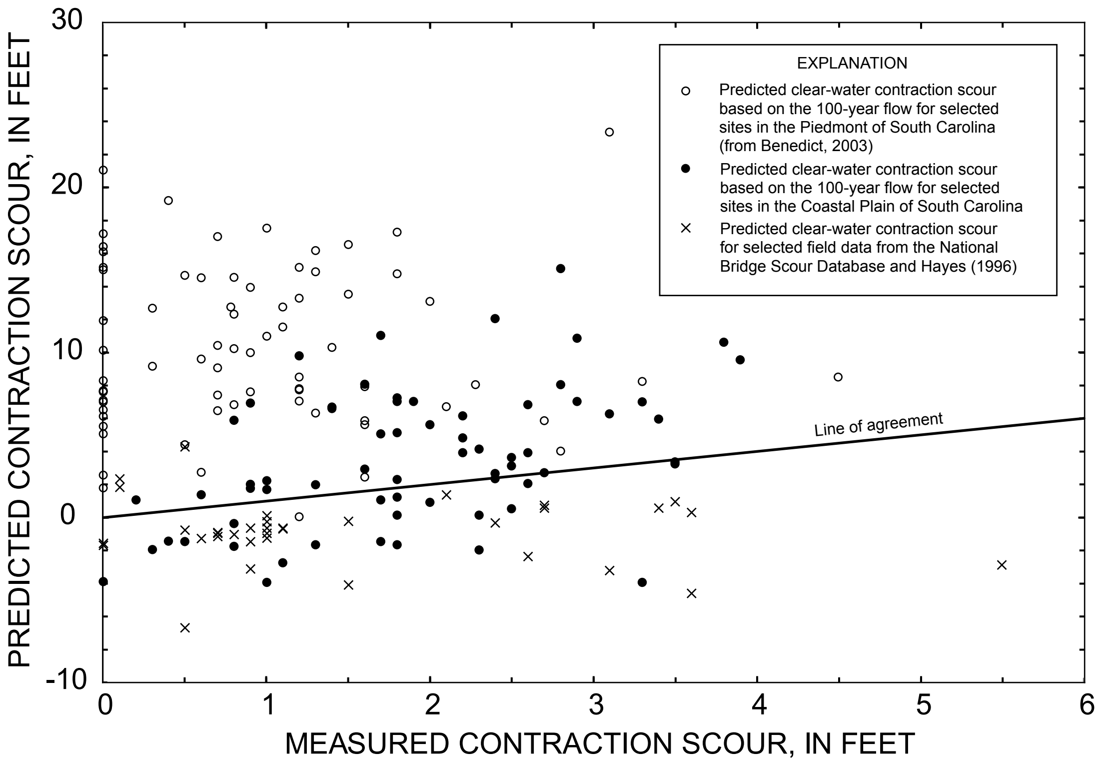


Figure 67. Relation of measured clear-water contraction-scour depth to predicted contraction-scour depth for selected sites in the Coastal Plain and Piedmont Physiographic Provinces of South Carolina, and from the National Bridge Scour Database and Hayes (1996). (Predicted clear-water contraction scour was calculated with the HEC-18 equation (Richardson and Davis, 2001).)

cobble range except one that had sandy soil. Flows associated with these scour measurements were based on concurrent flow measurements. All but three of the measurements from the NBSD and Hayes (1996) data are underpredicted by the HEC-18 equation. This trend most likely can be attributed to the larger grain sizes associated with these data.

The data in figure 67 were used to plot the relation of the prediction error (predicted scour minus observed scour) to the average flow velocity through the clear-water scour area (figs. 22 and 23) and the median grain size (fig. 68). The trends in this figure give some insights for when underprediction and overprediction with the HEC-18 clear-water contraction-scour equation (Richardson and Davis, 2001) can be anticipated. In figure 68A, there is a discernable relation between velocity and prediction error in the data for the Coastal Plain and Piedmont of South Carolina. Underprediction occurs when flow velocities are approximately 2.5 ft/s or less, and overprediction tends to increase as velocity increases beyond this value. For the NBSD and Hayes (1996) data, underprediction occurs for most measurements and no discernible trend occurred between prediction error and velocity. In figure 68B, however, there is a discernible relation between the median sediment grain size and prediction error, indicating that underprediction associated with the NBSD and Hayes (1996) measurements can be attributed to the large sediment sizes at these sites. For median grain sizes of approximately 0.00328 ft (1 mm) or less, the upper bound of the prediction error tends to increase with decreasing grain size. For median grain sizes greater than 0.00328 ft (1 mm), underprediction dominates. These trends can be used to judge the reasonableness of predicted scour when using the HEC-18 equation to estimate clear-water contraction scour in the Piedmont and Coastal Plain of South Carolina. The trends in figures 66 through 68, however, suggest that the HEC-18 equation (Richardson and Davis, 2001) is a poor predictor for clear-water contraction scour at Coastal Plain and Piedmont sites and that a different method is needed to estimate clear-water contraction scour in these regions.

Development of the Contraction-Scour Envelope Curve Using the Geometric-Contraction Ratio

The geometric-contraction ratio (m) is an indicator of the severity of flow contraction created by a bridge and is defined as $m = 1 - b/B$, where B is the approach-flow width, in feet, and b is the bridge-opening width, in feet. In general, as the geometric-contraction ratio increases, the flow velocity through a bridge opening rises, increasing the potential for scour. Therefore, it is reasonable to expect an increase in clear-water contraction-scour depth with increasing geometric-contraction ratios. Using this concept, Benedict (2003) plotted clear-water contraction scour and the 100-year flow geometric-contraction ratio for data measured in the Piedmont of South Carolina and developed an envelope curve for

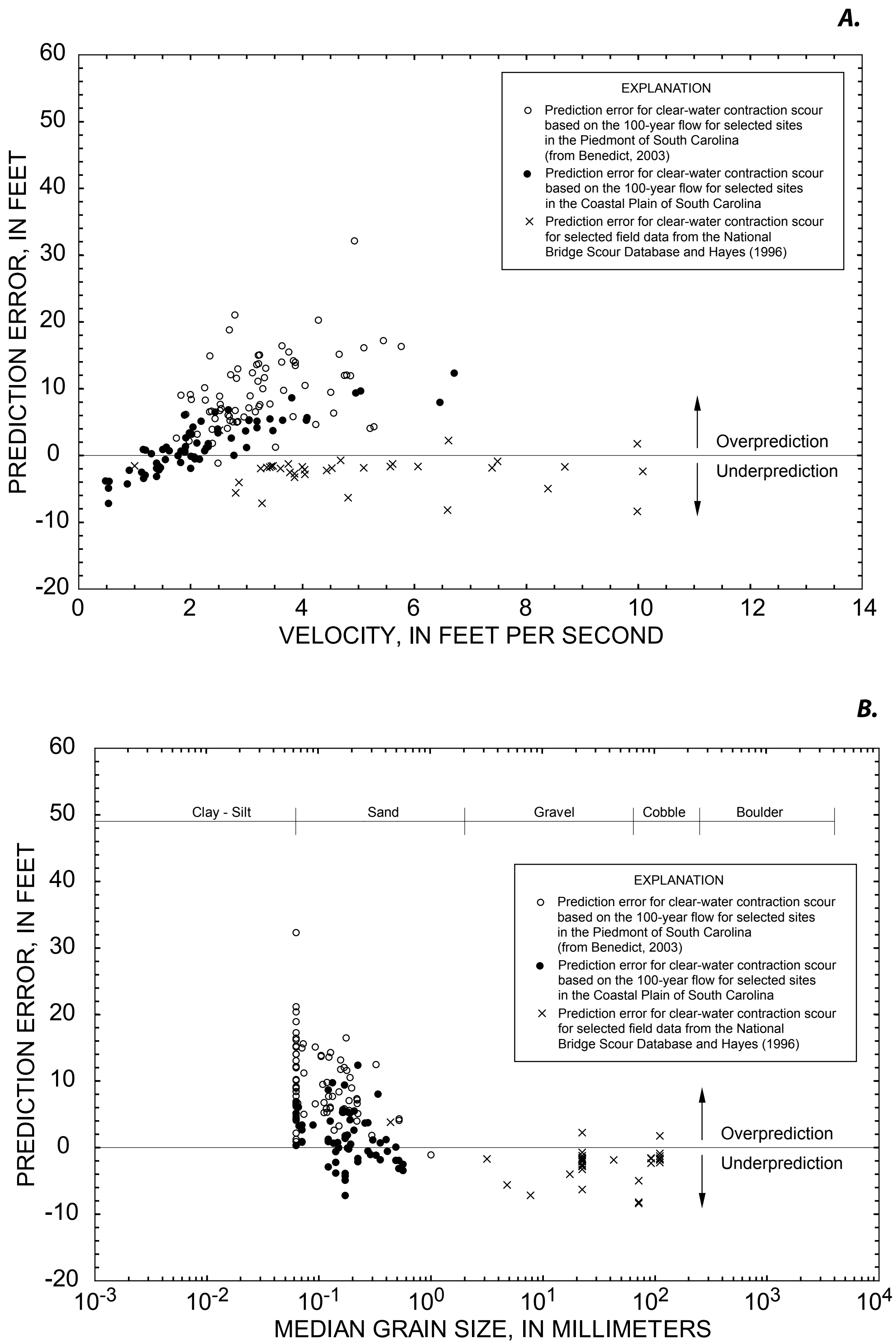


Figure 68. Prediction error for clear-water contraction-scour depths at selected sites from South Carolina, the National Bridge Scour Database, and Hayes (1996), compared with (A) the average flow velocity in the contraction, and (B) the median grain size. (Prediction error is defined as predicted minus observed clear-water contraction scour.)

evaluating the upper bound of clear-water contraction scour in that region. Clear-water contraction-scour data collected in the Coastal Plain for the current study are shown in figure 69 with the Piedmont data and envelope curve (Benedict, 2003). (The envelope-curve equation used in figure 69 is slightly modified from that published in Benedict (2003) but is essentially the same equation.) The geometric-contraction ratio for the 100-year flow for all measurements is plotted in figure 69A, and the trends for the smaller set of data where the maximum historic flow was known (table 2) is plotted in figure 69B. (Note: The geometric-contraction ratio was determined from the WSPRO model.) The trends of the data for the 100-year flow and the maximum historic flow are similar, indicating that the trends based on the 100-year flow, which is a substantially larger data set, are reasonable. Additionally, the envelope curve encompasses measured scour resulting from large floods (fig. 69B), indicating that the envelope curve represents the upper bound of scour that may result from large flows at sites with similar regional characteristics. It is interesting that all of the Coastal Plain data falls within the Piedmont envelope curve and that the upper limits of the data fall in close proximity to that envelope curve. This trend indicates that the envelope curve developed for the Piedmont is also a reasonable envelope curve for the Coastal Plain and can be used to evaluate potential clear-water contraction scour in both regions. The equation associated with the envelope curve is as follows:

$$y_s = -6m^2 + 10m + 0.6, \quad (15)$$

where

y_s is the upper limit of the range for anticipated clear-water contraction-scour depth, in feet; and

m is the 100-year-flow geometric-contraction ratio; where $m = 1 - b/B$, with variables previously defined.

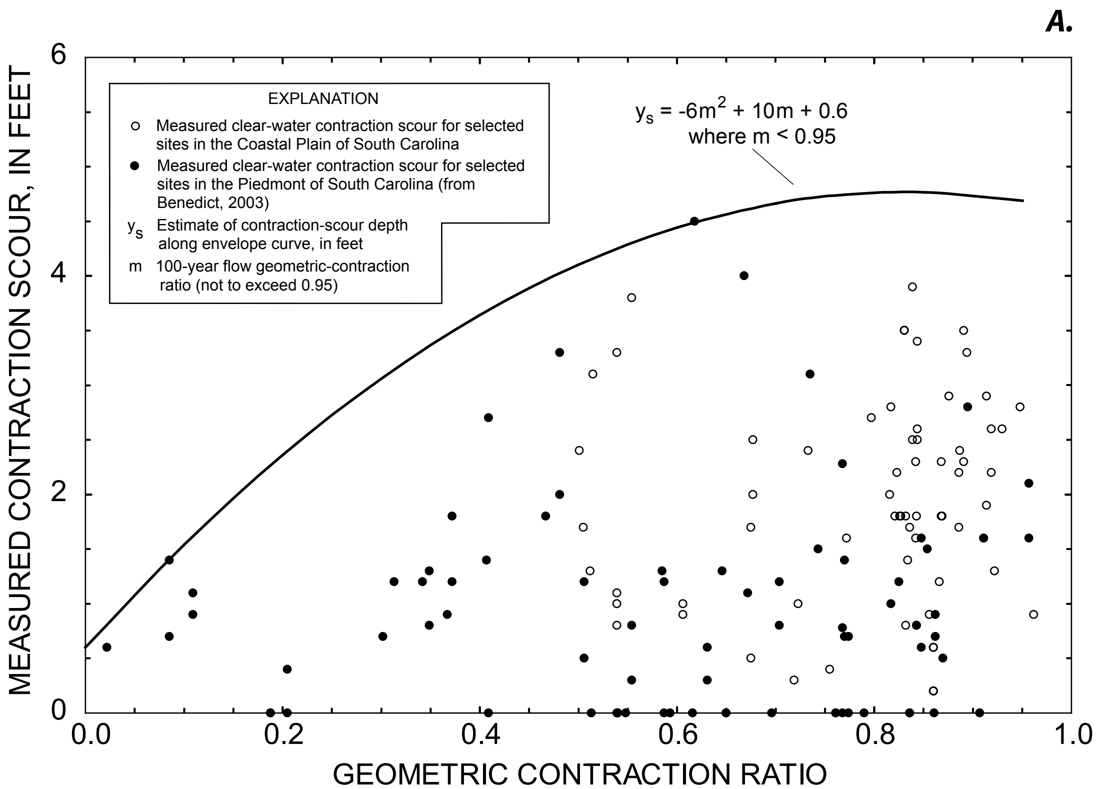
Based on the limits of the Piedmont data, Benedict (2003) recommended limiting the application of the envelope-curve equation to geometric-contraction ratios less than or equal to 0.85. Because of the additional data from the Coastal Plain that exceeds this value, it seems reasonable to increase this limit to 0.95. However, because data are sparse for geometric-contraction ratios between 0.9 and 0.95, the equation should be used with caution within this range.

Development of the Contraction-Scour Envelope Curve Using Flow Velocity

Flow velocity is known to be an important factor that influences the development of clear-water contraction scour. In order to evaluate the potential for scour, many researchers have used the concept of critical velocity, which is a threshold flow velocity at which sediments of a given size begin to erode. The determination of critical velocity for a given grain size has been defined, in large measure, by laboratory experiments using loose-grain sediments of uniform size. As Vanoni (1977) notes, the point of initial motion for a given sediment is difficult to discern even in the laboratory setting; therefore, discrepancies exist in the published literature that define the critical velocity for a given grain size. This problem is compounded when the concept of critical velocity, as defined in the laboratory, is applied to a natural soil that is nonuniform and often has some measure of cohesion and consolidation.

Use of the HEC-18 clear-water contraction-scour equation (Richardson and Davis, 2001) is dependent on the concept of critical velocity to evaluate potential scour. The equation assumes that as the streambed within the contraction is lowered by scour, the average velocity will decrease. Contraction scour will continue until the average flow velocity is equal to the critical velocity. Therefore, the equation estimates the clear-water contraction-scour depth by determining how much the streambed within the contraction must be lowered in order for the average flow velocity to equal the critical velocity. This is a very simple and useful concept for understanding the process of scour within a contraction. The difficulty, however, is determining the appropriate critical velocity for the given soil. The HEC-18 equation estimates the critical velocity by using an empirical equation derived from laboratory experiments of uniform loose-grain sediments. As such, it is likely to misrepresent the true critical velocity associated with natural soils. This is certainly one of the reasons why the HEC-18 clear-water contraction-scour equation (Richardson and Davis, 2001) poorly predicts measured scour (figs. 66, 67).

Despite the difficulties associated with properly applying the HEC-18 clear-water contraction-scour equation, it is still useful in understanding the role that velocity has in the development of contraction scour. Figure 70 shows the relation of measured clear-water contraction scour to the average overbank flow velocity for selected sites in the Coastal Plain (current investigation) and the Piedmont (Benedict, 2003) of South Carolina. Figure 70A is based on the average overbank flow velocity for the modeled 100-year flow for all measurements, and figure 70B represents the trends for the smaller data set of known maximum historic flows (table 2). The trends of the data for the 100-year flow and the maximum historic flow are similar, indicating that the trends based on the 100-year flow (a substantially larger data set) are reasonable. Additionally, the envelope curve encompasses measured scour resulting from large floods (fig. 70B), indicating that the curve represents the upper bound of scour that may result from large flows at sites with similar regional characteristics. It is interesting to note that much of the Piedmont data are below a scour depth of 1.5 ft, while much of the Coastal Plain data are above 1.5 ft. This is likely attributed to the cohesive soils of the Piedmont that are more scour resistant than the sandy soils of the Coastal Plain. Additionally, most of the flow velocities for the Piedmont are greater than 2 ft/s while a number of Coastal Plain sites have velocities less than 2 ft/s. This



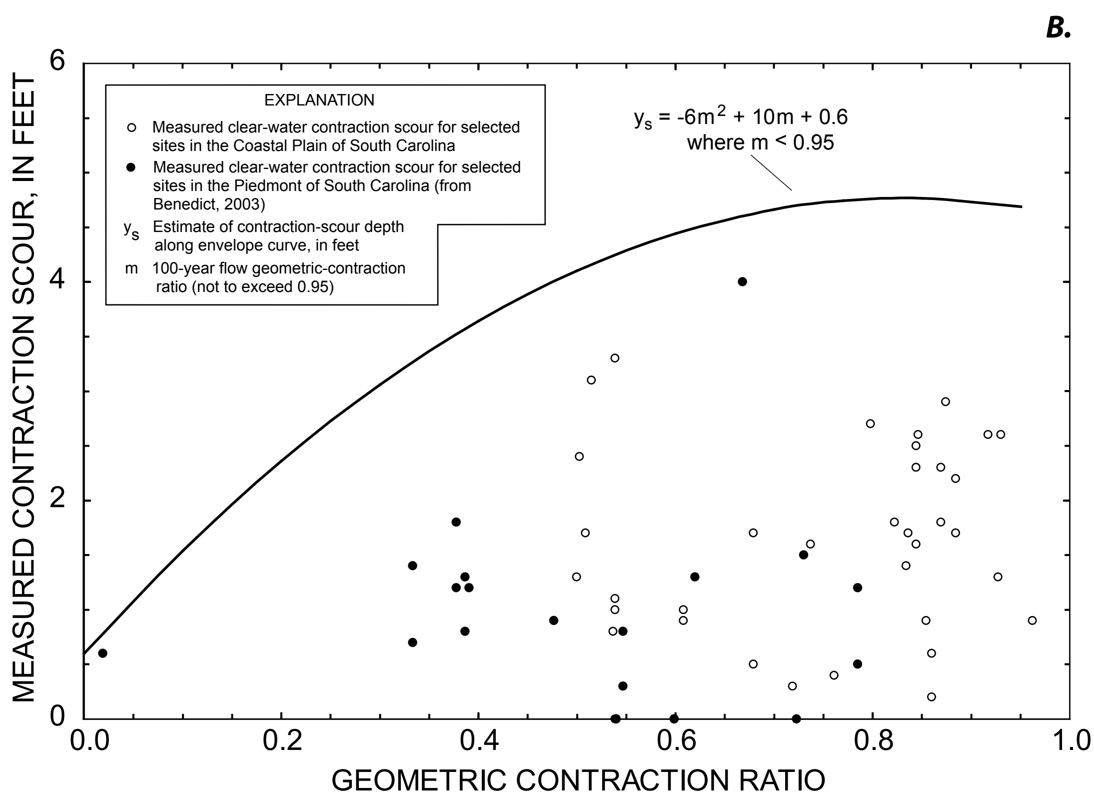
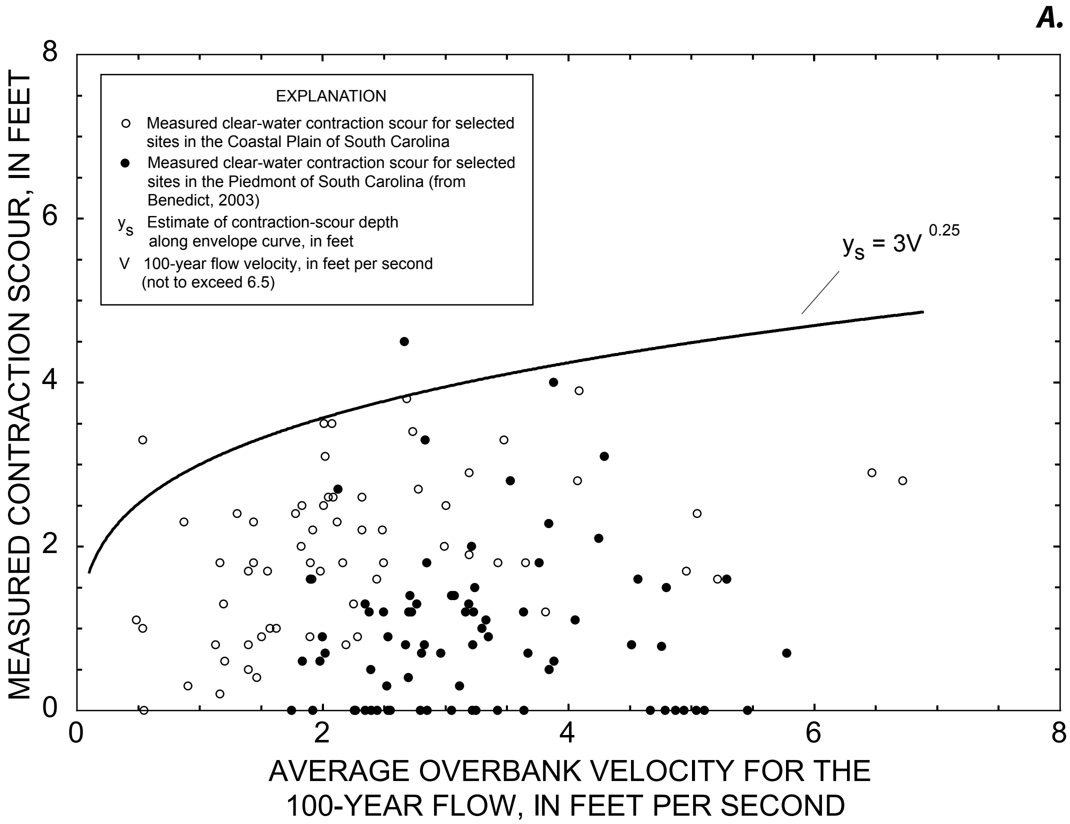


Figure 69. Relation of measured clear-water contraction-scour depths to the geometric contraction ratio at selected sites in the Coastal Plain and Piedmont Physiographic Provinces of South Carolina for (A) the 100-year flow and (B) the maximum historic flows.



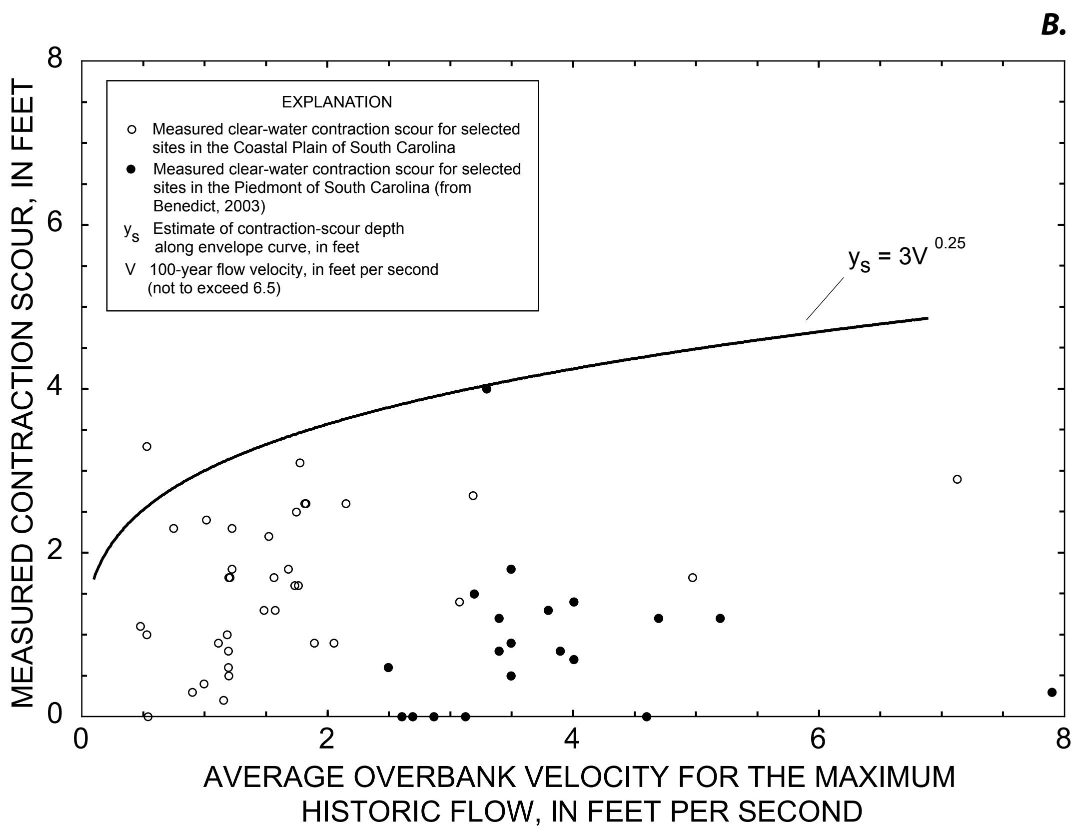


Figure 70. Relation of measured clear-water contraction-scour depths to the average overbank flow velocity at selected sites in the Coastal Plain and Piedmont Physiographic Provinces of South Carolina for (A) the 100-year flow and (B) the maximum historic flows.

highlights the low-gradient streams that are often associated with the Coastal Plain. Although there are distinct differences in the Piedmont and Coastal Plain contraction-scour data, the trends of the upper-bound data for both regions are similar, indicating that the development of a single envelope curve is reasonable.

The envelope curve displayed in figure 70 is arbitrarily drawn to encompass the majority of the measured clear-water contraction-scour data. The two outliers were intentionally excluded because abutment scour appeared to influence the magnitude of the measured clear-water contraction scour. The excluded data are located at structure 264002220300 on S.C. Route 22 crossing the Waccamaw River in Horry County and structure 204020000500 on S.C. Route 200 crossing the Wateree Creek in Fairfield County. The envelope curve displays a sharp increase of scour depth as velocity increases to about 1.5 ft/s, with a smaller rate of increasing scour beyond this point. This pattern is consistent with other types of scour (pier and abutment) as shown in figs. 33, 34 (Benedict, 2003, fig. 40) where the rate of scour is high for low velocities and then approaches a limit as velocity increases.

The envelope curve in figure 70 can be used to evaluate the upper bound of potential clear-water contraction scour within the Piedmont and Coastal Plain of South Carolina. The equation associated with the envelope curve is as follows:

$$y_s = 3V^{0.25}, \quad (16)$$

where

y_s is the upper limit of the range for anticipated clear-water contraction-scour depth, in feet; and

V is the average 100-year-flow velocity, in feet per second, on the overbank at the bridge where scour is anticipated.

The largest velocity associated with the data in figure 70 is approximately 7 ft/s; therefore, equation 16 should not be applied beyond this limit. Because data are sparse beyond a velocity of 4.5 ft/s, caution and judgment should be used when applying the equation to velocities greater than 4.5 ft/s.

Comparison and Limitations of the Contraction-Scour Envelope Curves

The envelope curves in figures 69 and 70 represent an upper limit of measured scour, and when used to evaluate clear-water contraction scour, overprediction of scour depth is likely for various site conditions; however, the envelope curve will not exceed the upper bound of measured field data, which is approximately 4.5 ft. In contrast, predicted scour computed with the HEC-18 equation (Richardson and Davis, 2001) can have frequent underprediction as well as excessive over-prediction (at times exceeding 20 ft; fig. 67), indicating that the South Carolina contraction-scour envelope curves often will provide more reasonable evaluations of potential clear-water contraction-scour depths in both the Piedmont and Coastal Plain of South Carolina. To understand how each envelope curve performs, measured clear-water contraction scour is compared to predicted scour that was estimated with the South Carolina contraction-scour envelope curves (fig. 71). The comparison indicates that both envelope curves provide conservative estimates of clear-water contraction scour and either one could be used in evaluating the upper bound of clear-water contraction scour in South Carolina. However, the envelope curve that uses the flow velocity (fig. 70) as the explanatory variable provides conservative estimates of scour that often are slightly lower than scour estimates based on the envelope curve that uses the geometric contraction ratio (fig. 69) as the explanatory variable. If the practitioner is interested in obtaining the smallest estimate of the upper bound of scour, it would be reasonable to select the smallest value of contraction scour from the two envelope curves. If a more conservative estimate of the upper bound of contraction scour is desired, then the largest value from the two envelope curves may be selected.

The evaluation of clear-water contraction scour using the South Carolina contraction-scour envelope curves should be limited to sites having similar characteristics to sites used in this study and in Benedict (2003). To assist in this evaluation, characteristics of Coastal Plain sites can be compared to those listed in table 8 and shown in figures 2 and 3 that display the range and trend of characteristics for Coastal Plain sites used in this investigation. Characteristics of Piedmont sites can be compared to those listed in table 9 (current investigation) and shown in figures 2 and 3 of Benedict (2003). The limitations of the envelope curves that were described in the previous sections should be carefully followed, and caution should be used when characteristics at a bridge approach the limits of the site characteristics used to develop the envelope curves. Because the envelope curves were developed from a limited sample of bridges in the Coastal Plain and Piedmont, scour depths could exceed the envelope curves. It may be prudent, therefore, to apply a safety factor to the envelope curves. When using the envelope curves, it is critical to properly estimate the average 100-year flow velocity on the overbank of anticipated scour and the 100-year geometric-contraction ratio. For guidance on how the 100-year flow velocities were determined in this investigation, refer to the section, “Predicted Clear-Water Contraction Scour.” To ensure that the geometric-contraction ratio is properly evaluated, various sources of data should be reviewed, including but not limited to topographic maps, hydraulic models, road plans, and field measurements. The envelope curves in figures 69 and 70 were developed using field data from sites with flows approaching the 100-year flow and, therefore, should not be used to evaluate clear-water contraction-scour depths for extreme conditions, such as the 500-year flow.

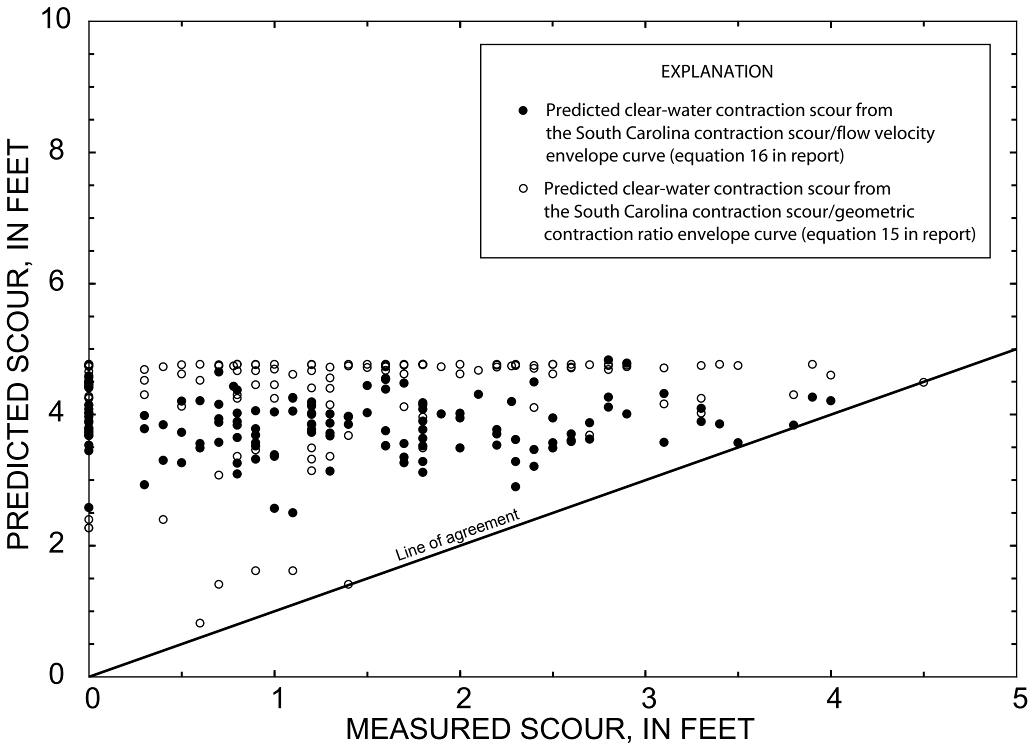


Figure 71. Relation of measured to predicted clear-water contraction-scour based on the South Carolina clear-water contraction-scour envelope-curve equations.

Selecting a Reference Surface for Clear-Water Contraction Scour

In this study, the average, undisturbed floodplain elevation in the clear-water contraction-scour region was used to determine contraction-scour depth. This reference surface should be used when evaluating clear-water contraction scour with the South Carolina contraction-scour-depth envelope curves (figs. 69, 70). This reference surface can be determined by reviewing floodplain elevations from SCDOT road and bridge plans, surveyed cross sections, and(or) site-visit observations. In many cases, the floodplain in the region of clear-water contraction scour is relatively flat, and estimating an average floodplain elevation is not a difficult task. However, there can be cases where the floodplain slopes substantially in the lateral and(or) longitudinal direction making the determination of a reference surface more difficult. In such cases, judgment should be applied, bearing in mind that lower reference-surface elevations will produce lower scour-hole elevations.

Pier Scour Within Clear-Water Contraction-Scour Areas

Because of the shallow nature of clear-water contraction-scour holes, it generally was possible to distinguish the area of pier scour from the area of clear-water contraction scour. When collecting data at these sites, scour around piers generally was not included in the measurement of clear-water contraction scour. Thus, the envelope curves in figures 69 and 70 represent contraction scour only and not total scour. These envelope curves can be used to evaluate anticipated ranges of clear-water contraction scour in Coastal Plain and Piedmont overbanks, but judgment must be used to account for any additional scour created by piers and pile bents. Guidance previously given in this report can be used to evaluate potential clear-water pier scour on the overbanks of the Coastal Plain and Piedmont.

A potential threat to overbank piers that is not addressed in this study is channel widening (see Benedict (2003) for more details). Channel widening can undermine overbank piers that are located near a channel bank. One should be aware of this potential problem and use judgment when evaluating scour at overbank piers or bents near channel banks.

Estimate of Clear-Water Contraction-Scour Hole Location

Scour hole patterns for clear-water contraction scour in the Coastal Plain are similar to those found in the previous investigation of clear-water contraction scour in the Piedmont (Benedict, 2003). In general, the shape of clear-water contraction-scour holes in the overbank region consisted of shallow parabolic depressions running perpendicular to flow (figs. 9, 65) and covering most of the overbank region unaffected by abutment scour. In general, the low point of the scour hole was in close proximity to the roadway centerline, and all observations were located beneath the bridge deck. The left and right lateral extent of the clear-water contraction scour typically began at the edge of the abutment-scour hole and ran toward the bank. Scour depths over the lateral extent of the scour hole varied; however, observed scour was measured at the deepest area over the lateral extent.

Data are insufficient to predict the exact location of the deepest part of clear-water contraction-scour holes. The data indicate, however, that it is reasonable to assume that scour will occur under the bridge and will extend laterally from the edge of the abutment-scour hole to the channel bank. The range of anticipated contraction-scour depths can be evaluated using one of the clear-water contraction-scour envelope curves (figs. 69, 70). The upper limit of this range can be applied across the entire overbank region from the edge of the abutment-scour hole to the channel bank. Because the edge of the abutment-scour hole is a limiting boundary for the clear-water contraction scour, abutment scour at the bridge should be evaluated first. For guidance on evaluating abutment scour, refer to Benedict (2003).

The South Carolina Clear-Water Pier- and Contraction-Scour Database

Selected data from this study have been compiled into the South Carolina Clear-Water Pier- and Contraction-Scour Database (SCPCSD) and can be viewed using Microsoft Access. (Note: Clear-water contraction-scour measurements from the Piedmont Physiographic Province collected in a previous investigation (Benedict, 2003) are included in this database.) The SCPCSD includes photographs, selected field data, variables used to compute predicted scour, predicted scour depths, limited basin characteristics, limited soil data, and selected hydraulic data estimated with the WSPRO model. These raw data were compiled in various data tables in the database, and automated forms have been developed to allow extraction of selected data for a bridge of interest. Appendix 1 contains a description of the SCPCSD automated forms, raw data tables, and variable definitions.

The SCPCSD was developed using Microsoft Access 2000. The electronic file for the database requires approximately 1.2 gigabytes of computer storage and is available at https://pubs.er.usgs.gov/publication/sir20055289. To install the database, the file “SCPCSD.mdb” should be copied from the Web site to the user’s directory of choice. After copying the file to the user’s computer, the properties of the file should be changed from “Read-only” by (1) right clicking on the file and selecting “Properties” on the popup menu, (2) deselecting the “Read-only” option in the Properties menu box, and then (3) clicking “OK” at the bottom of the menu box. To invoke the SCPCSD, the file “SCPCSD.mdb” should be opened in Access. Upon opening this file, the form selection menu box with the heading, “The South Carolina Clear-Water Pier and Contraction Scour Database” will appear. This menu box lists the five automated forms described in appendix 1, including pier-scour data for the 100-year flow, pier-scour data for the historic flow, contraction-scour data for the 100-year flow, contraction-scour data for the historic flow, and a form that displays photographs for each bridge. The pier- and contraction-scour forms will display the predicted scour for the 100-year or historic flow along with field measurements of scour and selected site information. The forms are invoked by clicking on the appropriated button in the form selection menu box. Near the top right corner of each form there is a drop-down menu designated by a menu button with an arrow pointing downward. Clicking on this button will produce a list of bridges or scour observations included in this study. From this list, the user can select a bridge or scour observation of interest. Once a specific bridge or scour observation has been selected, the form will automatically retrieve the data.

The SCPCSD is a valuable tool for use in investigating clear-water pier scour and contraction scour. For one who is evaluating scour at bridges in South Carolina, the SCPCSD provides a tool for making site comparisons. Sites under investigation but not included in the current study can be compared with sites in the SCPCSD to gain insights about the range of anticipated scour depths. The SCPCSD also provides a source of data to evaluate various methods for predicting clear-water pier and contraction scour. Most equations for predicting scour are driven by hydraulic variables, such as flow depth and velocity. These variables can be extracted from the SCPCSD and used in various equations to compute predicted scour depths. The predicted scour depths can then be compared with measured scour and the field-data envelope curves to evaluate the chosen equation’s performance. (Hydraulic data in the SCPCSD may need to be manipulated to obtain specific variables required for a given predictive equation.)

One should keep in mind that the hydraulic data in the SCPCSD were generated from a model and, therefore, do not necessarily represent the flow conditions that created the measured scour. As a result, some error is likely to be introduced into the comparison of predicted and measured scour because of inaccuracies in the hydraulic data. However, the numerous data points in the SCPCSD will allow such comparisons to show the general trends of a predictive equation and will provide some indication of the equation’s performance.

The SCPCSD provides only limited information at each study site and, therefore, cannot be relied on to provide a complete understanding of the sites. If more detailed information is required to understand conditions at a given site, other data sources should be consulted, such as topographic maps and bridge plans. Under certain circumstances, site visits may be required to gain a full appreciation of the measured scour and the conditions that created it.

Summary

The U.S. Geological Survey in cooperation with the South Carolina Department of Transportation collected measurements of clear-water pier scour and clear-water contraction scour at 116 bridges in the Piedmont and Coastal Plain of South Carolina. The 177 measurements of clear-water pier-scour depth ranged from 0 to 8 feet, and the 64 measurements of clear-water contraction-scour depth in the Coastal Plain ranged from 0 to 3.9 feet. The collected data represent the maximum clear-water pier-scour and(or) clear-water contraction-scour depths that have occurred at selected bridges since construction. Although flow conditions creating the measured scour are not known for most sites, evidence indicates that approximately 80 percent of the bridges may have had at least one event equal to or exceeding the 25-year flow. Fifty-one of these sites had documented maximum historic flows, and 50 of these sites had flows equal to or exceeding the 25-year flow magnitude; 21 sites had flows equal to or exceeding the 100-year flow magnitude. Because the collected data include a number of sites where relatively large flows have occurred, the data should provide a reasonable range for anticipated scour depths at bridges with similar site characteristics.

To gain insights into hydraulic conditions that may have created the measured scour, hydraulic models were developed for each site using the one-dimensional step-backwater model, WSPRO. Because the magnitude of historic peak flows was not known at all sites, the 100-year flow was modeled as a common flood. In addition, known maximum historic flows were modeled at 51 bridges. Hydraulic data generated from the WSPRO model were used to compute predicted-scour with methods presented in HEC-18. A comparison of predicted and measured scour showed that predicted pier-scour depths generally exceeded the measured scour depths and at times were excessive. A comparison of predicted and measured scour for clear-water contraction-scour depths, showed that predicted scour typically exceeded the measured scour and at times was excessive, but occasionally underpredicted observed contraction scour.

Modeled hydraulic data, predicted scour data, and field data were compiled into a database and were used to investigate relations that may help explain scour in South Carolina. Pier-scour field data were compared with dimensionless relations for laboratory data. This comparison showed that the range of dimensionless variables used in laboratory investigations was similar to the range of the dimensionless variables for field data in South Carolina. This implies that pier-scour relations derived from laboratory data will likely have some applicability to field conditions in South Carolina.

Variables determined to be influential in creating pier scour in laboratory studies were investigated to understand their influence on the South Carolina field data. Many of these variables appeared to be insignificant under field conditions found in South Carolina. The strongest explanatory variables for pier scour in South Carolina appeared to be pier width and approach velocity.

The investigation identified several envelope curves in the field data that could be used for evaluating reasonable ranges of pier-scour depth in South Carolina. These envelope curves include pier width as the primary explanatory variable. The envelope curves are simple to apply and can be used to obtain a quick evaluation of the upper bound of pier scour in South Carolina. Additionally, a modified version of the HEC-18 pier-scour equation was developed to reduce some of the excessive predictions for pier scour associated with the correction coefficient for skew and wide piers.

The investigation also identified several envelope curves within the field data that could be used for evaluating reasonable ranges of clear-water contraction-scour depths in the Coastal Plain and Piedmont of South Carolina. The envelope curves include the geometric-contraction ratio and flow velocity as the primary explanatory variables. The envelope curves show that clear-water contraction-scour depth increases as the explanatory variables increase and eventually approaches a limit of about 4.5 ft. The envelope curves are simple to apply and are an improvement over the current methods for predicting clear-water contraction scour on overbanks in the Coastal Plain and Piedmont of South Carolina.

Although the methods presented offer some improvement over existing theoretical methods for predicting scour in South Carolina, the limitations of these empirical methods are important. The methods should not be used outside the range of data for which they were developed.

Data for each bridge have been compiled into a database that includes photographs, measured scour depths, predicted scour depths, limited basin characteristics, limited soil data, and estimated hydraulic data. The database can be used to compare studied sites with unstudied sites to evaluate the potential for scour at the unstudied sites. In addition, the database provides a large source of field data that can be used to evaluate the performance of various theoretical methods for predicting clear-water pier and contraction scour.

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Appendix 1

Explanation of variables in the South Carolina clear-water pier- and contraction-scour database

Explanation of variables in the South Carolina clear-water pier- and contraction-scour database

Data for this investigation have been compiled into a database, including photographs, figures, observed scour depths, predicted scour depths, limited basin characteristics, limited soil data, and theoretical hydraulic data and can be viewed using Microsoft Access[1]. (Note: Clear-water contraction-scour data from the Piedmont Physiographic Province collected in a previous investigation (Benedict, 2003) are included in this database.) The South Carolina Clear-Water Pier- and Contraction-Scour Database (SCPCSD) provides automated forms that can be used to view data for a given site. The raw data also can be viewed in tabular format. Although most data for a given site can be viewed through the report formats, some data can only be viewed in the raw data tables. Blank data entries that appear in the reports or raw data tables indicate that data are not applicable or are missing. Following is a list and brief description of the automated forms that are in the SCPCSD.

(1) Bridge Information

Includes site location information, bridge length, construction history, bridge age, drainage area, and channel slope.

(2) Clear-Water Pier-Scour Data for the 100-Year Flow

Includes field measurements of scour, predicted pier scour for the 100-year flow based on the HEC-18 equation (Richardson and Davis, 2001), variables used to compute predicted scour, and selected site information.

(3) Clear-Water Pier-Scour Data for the Historic Flow

Includes field measurements of scour, predicted pier scour for the historic flow based on the HEC-18 equation (Richardson and Davis, 2001), variables used to compute predicted scour, and selected site information. (Note: Many sites in the database do not have known historic flows.)

(4) Clear-Water Contraction-Scour Data for the 100-Year Flow

Includes field measurements of scour, predicted contraction scour for the 100-year flow based on the HEC-18 equation (Richardson and Davis, 2001), variables used to compute predicted scour, and selected site information.

(5) Clear-Water Contraction-Scour Data for the Historic Flow

Includes field measurements of scour, predicted contraction scour for the historic flow based on the HEC-18 equation (Richardson and Davis, 2001), variables used to compute predicted scour, and selected site information. (Note: Many sites in the database do not have known historic flows.)

(6) Photographs

Includes photographs and captions for most sites.

There are five raw data tables in the SCPCSD; a brief description of each table and the associated variables follows. The headings for the following sections correspond with the table names in the database and are listed in alphabetical order. The pier- and contraction-scour forms for the 100-year and historic flows have identical variables, and variable values will change only for the predicted scour because of the change in estimated flow conditions. It should be kept in mind that hydraulic variables in the database are estimates obtained from the WSPRO (Shearman, 1990) model, and errors could exist within these estimates.

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[1]Any use of trade, product, or firm names is for descriptive purposes only and does not imply endorsement by the U.S. Government.

Bridge_Information Table

This table provides basic site information, including bridge identification, location, limited basin characteristics data, construction dates, South Carolina Department of Transportation (SCDOT) bridge-plan file numbers, and bridge age. The variables are defined below:

bridgeno — SCDOT bridge identification number

county — county in which the bridge is located

long — longitude of bridge, in degrees, minutes, seconds

lat — latitude of bridge, in degrees, minutes, seconds

province — physiographic province in which the bridge is located

road — road type and number

stream — name of stream

drainagearea — drainage area at bridge, in square miles

channel_slope — channel slope at the bridge as determined from U.S. Geological Survey (USGS) 7.5-minute series topographic map, in feet per foot

bridgelength — bridge length, in feet

bridgeconstdate — calendar year in which bridge was originally constructed

bridgeplannumber — SCDOT road plans file number from which construction date was estimated

widened — indicates if bridge has been widened since original construction date

widendate — calendar year when bridge was widened

widenplannumber — SCDOT road plans file number from which widening date was estimated

bridgeage — age of bridge in 2002; if bridge was widened, an attempt was made to assess if the construction at the time of widening disturbed the area of scour; if the assessment indicated that the area of scour was disturbed, the age was based on the widening date, otherwise the age was based on the original construction date. (For measurements of clear-water contraction-scour in the Piedmont (Benedict, 2003), the bridge age is based on the year 1996.)

oldbridge — indicates if an old bridge was in place (but removed) at the time of the original construction of the existing bridge

oldbridgedata — calendar year in which the old structure was constructed

Contraction_Scour_Q100 and Contraction_Scour_QHIS Tables

These tables include field measurements of clear-water contraction scour, predicted contraction scour for the 100-year (Q100) and maximum historic (QHIS) flow based on the HEC-18 equation (Richardson and Davis, 2001), variables used to compute predicted scour, and selected site information. For more details about the computation of pier scour refer to the "Predicted Clear-Water Contraction Scour" section of the report. The variables are defined below:

county — county in which the bridge is located

road — road type and number

stream — name of stream

bridgeno — SCDOT bridge identification number

flow_index — identifies the flow used in the predicted scour computation as the Q100 or QHIS

scour_type — identifies the type of scour as either pier or contraction scour

predicted_contraction_scour — predicted clear-water contraction-scour depth computed using the HEC-18 equation (Richardson and Davis, 2001), in feet

contracted_flow — flow in the contracted section, in cubic feet per second

contracted_width — width of contracted section, in feet

contracted_flow_depth — average flow depth at the contracted section, in feet

contracted_d50mm — the D_{50} based on a grab sample at each site, in millimeters; for sites with a D_{50} less than 0.062 millimeter, the D_{50}, was typically set to 0.062 millimeter

cumm_pier_width — the cumulative pier width within the contracted section, in feet

contracted_velocity — flow velocity in the contracted section, in feet per second

multi_bridge — identifies if the bridge is a multiple bridge or not

twin_bridge — identifies if the bridge is a twin bridge or not

location — identifies overbank location as either the left or right overbank as determined by an observer looking downstream or as a swampy channel with no well-defined low-flow channel

survey_date — date of field measurement, in month/day/year

measured_scour — maximum clear-water scour depth referenced to the average floodplain elevation in the region of the observed scour, in feet

estimated_infill — the amount of infill at the low point of the scour hole, in feet

quality_infill — subjective indicator of the quality of the measured infill

soil_type — a subjective indicator of the general surface soils in the unscoured region of the observed scour; this information is not necessarily an indicator of the measured grain size and should be viewed with caution; following is a description of each class:

clay – a relatively cohesive soil
sand – a sandy soil with relatively low cohesion
layered – alternating layers of clay and sand
mix – a mixture of sand and clay

bridge_age — age of bridge in 2002; if bridge was widened, an attempt was made to assess if the construction at the time of widening disturbed the area of scour; if the assessment indicated that the area of scour was disturbed, the age was based on the widening date, otherwise the age was based on the original construction date. (For measurements of clear-water contraction-scour in the Piedmont (Benedict, 2003), the bridge age is based on the year 1996.)

stream_slope — channel slope at the bridge as determined from USGS 7.5-minute series topographic map, in feet per feet

drainage_area — drainage area at bridge, in square miles

province — physiographic province in which the bridge is located

contraction_ratio — geometric-contraction ratio determined from WSPRO model (Shearman, 1990)

bridge_avg_flow_velocity — flow velocity in bridge opening, in feet per second

bridge_length — bridge length, in feet

latitude — latitude of bridge, in degrees, minutes, seconds

longitude — longitude of bridge, in degrees, minutes, seconds

comments — general comments related to scour observation

Pier_Scour_Q100 and Pier_Scour_QHIS Tables

These tables include field measurements of clear-water pier scour, predicted pier scour for the (Q100) and QHIS flow based on the HEC-18 equation (Richardson and Davis, 2001), variables used to compute predicted scour, and selected site information. For more details about the computation of pier scour refer to the "Predicted Clear-Water Pier Scour" section of the report. The variables are defined below:

county — county in which the bridge is located

road — road type and number

stream — name of stream

bridgeno — SCDOT bridge identification number

flow_index — identifies the flow used in the predicted scour computation as the Q100 or QHIS

scour_type — identifies the type of scour as either pier or contraction scour

station — station of pier from left end of bridge as determined by an observer looking downstream

predicted_pier_scour — predicted clear-water pier-scour depth computed using the HEC-18 equation (Richardson and Davis, 2001), in feet

pier_flow_depth — average approach flow depth at the pier, in feet

pier_flow_velocity — approach flow velocity at the pier, in feet per second

pier_width — width of the pier, in feet

pier_length — length of the pier, in feet

skew_angle — the skew of the pier to the approaching flow, in degrees

k1 — the dimensionless correction factor for pier nose shape

k2 — the dimensionless correction factor for flow angle of attack

k3 — the dimensionless correction factor for streambed conditions

k4 — the dimensionless correction factor for streambed armoring

pier_Froude_number — the approach flow Froude number

multi_bridge — identifies if the bridge is a multiple bridge or not

twin_bridge — identifies if the bridge is a twin bridge or not

location — identifies location of pier-scour measurement location as either the left or right overbank as determined by an observer looking downstream

bent_number — identifies the bent number from the SCDOT plans

survey_date — date of field measurement, in month/day/year

measured_scour — maximum clear-water pier-scour depth referenced to the average ground elevation at the top of the pier-scour hole in close proximity to the pier, in feet

pier_shape — shape of the pier

pier_material — material from which pier is made

field_pier_width — width of the pier as measured in the field, in feet

multi_column — identifies if pier has multiple columns

number_columns — number of columns in multiple-column pier

max_column_width — the largest column width in a multiple-column pier, in feet

min_column_width — the smallest column width in a multiple-column pier, in feet

max_spacing — the largest spacing between columns in a multiple-column pier, in feet

min_spacing — the smallest spacing between columns in a multiple-column pier, in feet

scour_hole_width — the width of the pier-scour hole perpendicular to flow, in feet

soil_type — a subjective indicator of the general surface soils in the unscoured region of the observed scour; this information is not necessarily an indicator of the measured grain size and should be viewed with caution; following is a description of each class:

clay – a relatively cohesive soil
sand – a sandy soil with relatively low cohesion
layered – alternating layers of clay and sand
mix – a mixture of sand and clay

D50mm — the D_{50} based on a grab sample at each site, in millimeters; for sites with a D_{50} less than 0.062 millimeter, the D_{50}, was typically set to 0.062 millimeter

bridge_age — age of bridge in 2002; if bridge was widened, an attempt was made to assess if the construction at the time of widening disturbed the area of scour; if the assessment indicated that the area of scour was disturbed, the age was based on the widening date, otherwise the age was based on the original construction date.

stream_slope — channel slope at the bridge as determined from USGS 7.5-minute series topographic map, in feet per feet

drainage_area — drainage area at bridge, in square miles

province — physiographic province in which the bridge is located

contraction_ratio — geometric-contraction ratio determined from WSPRO model (Shearman, 1990)

bridge_avg_flow_velocity — flow velocity in bridge opening, in feet per second

bridge_length — bridge length, in feet

latitude — latitude of bridge, in degrees, minutes, seconds

longitude — longitude of bridge, in degrees, minutes, seconds

comments — general comments related to scour observation

Selected References

Benedict, S.T., 2003, Clear-water abutment and contraction scour in the Coastal Plain and Piedmont Provinces of South Carolina, 1996–99: U.S. Geological Survey Water-Resources Investigations Report 03–4064, 137 p.

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Shearman, J.O., 1990, User’s manual for WSPRO—A computer model for water-surface profile computations: Federal Highway Administration, Report No. FHWA–IP–89–027, 175 p.

Appendix 2

South Carolina bridge-scour study sites and reference numbers for figures 1 and 7.

Appendix 2. South Carolina bridge-scour study sites and reference numbers for figures 1 and 7.
(Note: At twin bridge crossings only the structure number for the North or East bound bridge is provided.)

I, Interstate Highway; SR, Secondary Road; U.S., United States Route; S.C., South Carolina Route]

Reference number for figures 1 and 7	County	Road	Stream	Structure number
1	Aiken	I–20	South Edisto River	021002021200
2	Anderson	I–85	Brushy Creek	041008511200
3	Anderson	SR 263	Rocky River	047026300100
4	Bamberg	U.S. 321	South Edisto River Swamp	052032100300
5	Bamberg	U.S. 321	South Edisto River Swamp	052032100400
6	Bamberg	U.S. 321	South Edisto River	052032100500
7	Bamberg	U.S. 321	South Edisto River RR	052032150311
8	Bamberg	U.S. 321	South Edisto River RR	052032150411
9	Bamberg	U.S. 321	South Edisto River RR	052032150511
10	Barnwell	S.C. 39	South Edisto River	064003900200
11	Cherokee	S.C. 5	Buffalo Creek	114000500200
12	Cherokee	SR 348	Buffalo Creek	117034800100
13	Chester	I–77	Fishing Creek	121007710700
14	Chester	S.C. 72	Sandy River	124007200200
15	Chester	S.C. 97	Turkey Creek	124009700100
16	Chester	S.C. 97	Rocky Creek	124009700800
17	Chester	S.C. 215	Sandy River	124021500200
18	Chester	S.C. 223	Fishing Creek	124022300100
19	Chester	S.C. 901	Rocky Creek	124090100200
20	Chesterfield	S.C. 9	Thompson Creek	134000900400
21	Chesterfield	S.C. 109	Thompson Creek	134010900100
22	Clarendon	U.S. 521	Ox Swamp	142052100300
23	Colleton	S.C. 63	Little Salkehatchie River	154006300400
24	Colleton	S.C. 63	Little Salkehatchie River	154006300500
25	Colleton	S.C. 63	Little Salkehatchie River	154006300600
26	Colleton	S.C. 63	Little Salkehatchie River	154006300700
27	Colleton	S.C. 641	Willow Swamp	154064100200
28	Darlington	I–20	Jeffries Creek	161002020400
29	Darlington	U.S. 52	Black Creek	162005200300
30	Dillon	I–95	Great Pee Dee River	171009530100
31	Dillion	S.C. 41	Little Pee Dee River	174004100200
32	Dillon	S.C. 41	Buck Swamp	174004107100
33	Dorchester	I–26	Four Hole Swamp	181002620300
34	Edgefield	S.C. 230	Horne Creek	194023000500
35	Fairfield	I–77	Little Wateree Creek	201007710600
36	Fairfield	I–77	Big Wateree Creek	201007710700

Appendix 2. South Carolina bridge-scour study sites and reference numbers for figures 1 and 7. — Continued
(Note: At twin bridge crossings only the structure number for the North or East bound bridge is provided.)

I, Interstate Highway; SR, Secondary Road; U.S., United States Route; S.C., South Carolina Route]

Reference number for figures 1 and 7	County	Road	Stream	Structure number
37	Fairfield	U.S. 21	Dutchmans Creek	202002100200
38	Fairfield	U.S. 21	Big Wateree Creek	202002100400
39	Florence	U.S. 52	Lynches Lake	212005200100
40	Florence	U.S. 76	Sparrow Swamp	212007600400
41	Florence	U.S. 76	Great Pee Dee River	212007621100
42	Florence	U.S. 301	Sparrow Swamp	212030100400
43	Florence	U.S. 378	Lynches River	212037800900
44	Florence	U.S. 378	Big Swamp	212037801000
45	Florence	S.C. 41	Lynches River	214004100200
46	Florence	S.C. 41	Lynches River Swamp	214004100300
47	Florence	S.C. 41	Lynches River RR	214004100311
48	Florence	S.C. 41	Lynches River Swamp	214004100400
49	Florence	S.C. 41	Lynches River RR	214004100411
50	Georgetown	U.S. 701	Yauhannah Lake	222070100400
51	Greenville	S.C. 417	Horse Pen Creek	234041700200
52	Greenville	SR 68	Reedy River	237006800100
53	Greenwood	S.C. 246	Wilson Creek	244024600200
54	Hampton	U.S. 601	Coosawhatchie River	252060100300
55	Hampton	S.C. 363	Coosawhatchie River	254036300100
56	Hampton	SR 13	Whippy Swamp	257001300500
57	Horry	U.S. 378	Little Pee Dee River	262037800100
58	Horry	U.S. 378	Little Pee Dee Swamp	262037800200
59	Horry	U.S. 501	Waccamaw River	262050103100
60	Horry	U.S. 501	Waccamaw River	262050103200
61	Horry	U.S. 501	Waccamaw River	262050103300
62	Horry	U.S. 501	Waccamaw River	262050105200
63	Horry	U.S. 501	Little Pee Dee River	262050110100
64	Horry	U.S. 701	Great Pee Dee River	262070100100
65	Horry	S.C. 22	Waccamaw River	264002220200
66	Horry	S.C. 22	Waccamaw River	264002220300
67	Horry	S.C. 22	Waccamaw River	264002220400
68	Jasper	U.S. 278	Cypress Creek	272027800100
69	Jasper	S.C. 3	Cypress Creek	274000300200
70	Jasper	SR 87	Coosawhatchie River	277008700100
71	Kershaw	U.S. 1	Little Lynches River	282000100500
72	Kershaw	U.S. 521	Granneys Quarter Creek	282052100900

Appendix 2. South Carolina bridge-scour study sites and reference numbers for figures 1 and 7. — Continued
(Note: At twin bridge crossings only the structure number for the North or East bound bridge is provided.)

I, Interstate Highway; SR, Secondary Road; U.S., United States Route; S.C., South Carolina Route]

Reference number for figures 1 and 7	County	Road	Stream	Structure number
73	Kershaw	S.C. 97	White Oak Creek	284009700300
74	Kershaw	S.C. 97	Granneys Quarter Creek	284009700400
75	Laurens	S.C. 49	Enoree River	304004900400
76	Laurens	S.C. 72	Duncan Creek	304007201100
77	Laurens	SR 36	Reedy River	307003600200
78	Laurens	SR 102	Little River	307010200100
79	Laurens	SR 112	Enoree River	307011200100
80	Laurens	SR 263	Enoree River	307026300100
81	Lee	U.S. 401	Scape Ore Swamp	312040100100
82	Lexington	S.C. 113	North Edisto River	324011300100
83	McCormick	S.C. 67	Cuffeytown Creek	334006700100
84	Marion	U.S. 76	Great Pee Dee River	342007620100
85	Marion	U.S. 378	Little Pee Dee Swamp	342037800800
86	Marion	U.S. 501	Little Pee Dee River	342050110700
87	Marion	U.S. 501	Little Pee Dee River	342050110800
88	Marion	U.S. 501	Little Pee Dee River	342050110900
89	Marion	U.S. 501	Little Pee Dee River	342050111000
90	Marion	U.S. 501	Little Pee Dee River	342050111100
91	Newberry	U.S. 176	Indian Creek	362017600400
92	Newberry	U.S. 176	Kings Creek	362017600500
93	Newberry	S.C. 34	Little River	364003400300
94	Newberry	S.C. 121	Saluda River	364012100101
95	Newberry	SR 45	Enoree River	367004500100
96	Newberry	SR 81	Enoree River	367008100200
97	Oconee	U.S. 76	Coneross Creek	372007620500
98[a]	Oconee	S.C. 11	Colonels Fork Creek	374001100500
99	Oconee	S.C. 183	Coneross Creek	374018300200
100	Orangeburg	U.S. 301	Four Hole Swamp	382030110800
101	Orangeburg	S.C. 453	Four Hole Swamp (Br 1)	384045300200
102	Richland	U.S. 321	Crane Creek	402032100200
103	Saluda	U.S. 378	Red Bank Creek	412037800100
104	Saluda	U.S. 378	Little Saluda River	412037800200
105	Spartanburg	U.S. 29	South Tyger River	422002900100
106[a]	Spartanburg	U.S. 176	Lawsons Fork Creek	422017620900
107	Spartanburg	S.C. 146	Enoree River	424014600100
108	Spartanburg	S.C. 296	Enoree River	424029600100

Appendix 2. South Carolina bridge-scour study sites and reference numbers for figures 1 and 7. — Continued
(Note: At twin bridge crossings only the structure number for the North or East bound bridge is provided.)

I, Interstate Highway; SR, Secondary Road; U.S., United States Route; S.C., South Carolina Route]

Reference number for figures 1 and 7	County	Road	Stream	Structure number
109	Spartanburg	SR 62	South Tyger River	427006200500
110	Spartanburg	SR 118	Enoree River	427011800001
111	Spartanburg	SR 242	South Tyger River	427024200200
112	Union	S.C. 56	Enoree River	444005600100
113	Union	SR 22	Enoree River	447002200100
114	Williamsburg	S.C. 261	Paisley Swamp	454026100300
115	Williamsburg	SR 16	Johnson's Creek	457001600100
116	York	S.C. 97	Bullocks Branch	464009700300
117	York	S.C. 322	Fishing Creek	464032200300
118	York	SR 721	Taylors Creek	467072100100

[a] Limited data were collected at this site; however, site was excluded from investigation because of complicating site conditions.

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