

Prepared in cooperation with the Missouri Department of Transportation

Potential-Scour Assessments and Estimates of Scour Depth Using Different Techniques at Selected Bridge Sites in Missouri

MISSOURI STATE HIGHWAY DEPARTMENT



Scientific Investigations Report 2004–5213

U.S. Department of the Interior U.S. Geological Survey

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By Richard J. Huizinga and Paul H. Rydlund, Jr.

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Conversion	Factors	and Datum
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Multiply	Ву	To obtain					
	Length						
inch (in.)	2.54	centimeter (cm)					
inch (in.)	25.4	millimeter (mm)					
foot (ft)	0.3048	meter (m)					
	Area						
square foot (ft ²)	929.0	square centimeter (cm ²)					
square foot (ft ²)	0.09290	square meter (m ²)					
square mile (mi ²)	259.0	hectare (ha)					
square mile (mi ²)	2.590	square kilometer (km ²)					
	Flow rate						
foot per second (ft/s)	0.3048	meter per second (m/s)					
cubic foot per second (ft^3/s)	0.02832	cubic meter per second (m^3/s)					
Hydraulic gradient							
foot per mile (ft/mi)	0.1894	meter per kilometer (m/km)					
	Unit discharge*						
cubic foot per second per foot [(ft ³ /s)/ ft]	0.09290	cubic meter per second per meter [(m ³ /s)/m]					
	Acceleration						
foot per second squared (ft/s ²)	0.3048	meter per second squared (m/s^2)					

Temperature in degrees Celsius (°C) may be converted to degrees Fahrenheit (°F) as follows: °F = $(1.8 \times °C) + 32$

Temperature in degrees Fahrenheit (°F) may be converted to degrees Celsius (°C) as follows:

°C = (°F - 32) / 1.8

Vertical coordinate information is referenced to the North American Vertical Datum of 1988 (NAVD 88).

*Unit discharge: The standard unit for unit discharge is cubic foot per second per foot [(ft³/s)/ft]. In this report, the mathematically reduced form, foot squared per second (ft²/s), is used for convenience.

By Richard J. Huizinga and Paul H. Rydlund, Jr.

Abstract

The evaluation of scour at bridges throughout the state of Missouri has been ongoing since 1991 in a cooperative effort by the U.S. Geological Survey and Missouri Department of Transportation. A variety of assessment methods have been used to identify bridges susceptible to scour and to estimate scour depths. A potential-scour assessment (Level 1) was used at 3,082 bridges to identify bridges that might be susceptible to scour. A rapid estimation method (Level 1+) was used to estimate contraction, pier, and abutment scour depths at 1,396 bridge sites to identify bridges that might be scour critical. A detailed hydraulic assessment (Level 2) was used to compute contraction, pier, and abutment scour depths at 398 bridges to determine which bridges are scour critical and would require further monitoring or application of scour countermeasures.

The rapid estimation method (Level 1+) was designed to be a conservative estimator of scour depths compared to depths computed by a detailed hydraulic assessment (Level 2). Detailed hydraulic assessments were performed at 316 bridges that also had received a rapid estimation assessment, providing a broad data base to compare the two scour assessment methods. The scour depths computed by each of the two methods were compared for bridges that had similar discharges. For Missouri, the rapid estimation method (Level 1+) did not provide a reasonable conservative estimate of the detailed hydraulic assessment (Level 2) scour depths for contraction scour, but the discrepancy was the result of using different values for variables that were common to both of the assessment methods. The rapid estimation method (Level 1+) was a reasonable conservative estimator of the detailed hydraulic assessment (Level 2) scour depths for pier scour if the pier width is used for piers without footing exposure and the footing width is used for piers with footing exposure. Detailed hydraulic assessment (Level 2) scour depths were conservatively estimated by the rapid estimation method (Level 1+) for abutment scour, but there was substantial variability in the estimates and several substantial underestimations.

Introduction

Scour is the removal of the channel bed and bank material by flowing water, and is the leading cause of bridge failures in the United States (Richardson and Davis, 2001). Total scour is divided into three primary components: general scour, which refers to long-term geomorphicological processes that cause degradation (lowering), aggradation (filling), or lateral shifting of the natural channel; contraction scour, which refers to the erosion of material that occurs when the cross-sectional flow area of the stream is reduced or contracted; and local scour, which refers to the localized erosion of material caused by flow vortex action that forms near bridge piers and abutments (Richardson and others, 1991, 1993; Richardson and Davis, 1995, 2001; Holnbeck and Parrett, 1997). Although scour processes continually are at work, the processes are accelerated during high-flow conditions, and the potential for scour-related problems at a bridge tend to increase during such events.

History of Scour Studies in Missouri

In 1988, the Federal Highway Administration (FHWA) recommended that "every bridge over a scourable stream, whether existing or under design, should be evaluated as to its vulnerability to floods in order to determine the prudent measures to be taken for its protection" (U.S. Federal Highway Administration, 1988). In 1991, the U.S. Geological Survey (USGS) and the Missouri Department of Transportation (MoDOT) began a cooperative study to accomplish two goals: (1) to develop and use a suitable screening process to identify "scour-susceptible" bridges, and (2) for bridges identified as "scour-susceptible," perform a detailed hydraulic evaluation and estimate values of scour using accepted hydraulic techniques. The hydraulic information can be used by MoDOT for possible implementation of countermeasures. The term "scoursusceptible" describes a bridge that is deemed potentially unstable because abutment and/or pier foundations have the potential to be undermined by erosion of the channel bed or banks (U.S. Federal Highway Administration, 1988).

The FHWA categorizes scour assessments into three levels, depending on complexity (Lagasse and others, 1991):

- Level 1: A qualitative analysis of stream characteristics, simple geomorphic concepts, and other qualitative indicators to assess the scour potential of a bridge.
- Level 2: An application of hydrologic, hydraulic, and sediment transport concepts to determine quantitative scour-depth estimates.
- Level 3: An application of mathematical or physical modeling studies (used only for investigation of highly complex situations, because of the additional expense and time required).

All three of these levels were used in the study in Missouri: Level 1-type potential-scour assessments (hereinafter referred to as "Level 1") were used to meet the first goal of the study, and Level 2-type detailed hydraulic assessments (hereinafter referred to as "Level 2") were used to meet the second goal of the study, with a quasi-Level 3 analysis used at one hydraulically complex site described in Huizinga and Rydlund (2001).

The Level 1 assessments identified 1,327 bridges as scoursusceptible. As a way to reach the second goal of the study and to meet time-frame requirements established by the FHWA, a rapid estimation method (hereinafter referred to as "Level 1+") developed by the USGS in Montana (Holnbeck and Parrett, 1997) was used to evaluate bridges deemed scour-susceptible by the Level 1 assessment to identify bridges that might be scour critical. In 2002, 104 bridges were identified that had not been assessed for scour-susceptibility; these bridges received a Level 1+ assessment without a Level 1 assessment. In all, Level 1 assessments were performed at 3,082 bridges, Level 1+ assessments were performed at 1,396 bridges, and Level 2 assessments were performed at 398 bridges (of which 316 also received a Level 1+ assessment). The number of bridges that received each type of assessment, grouped by MoDOT district and county in Missouri, are shown in table 1.

Table 1. Number of bridges that received potential-scour (Level 1), rapid estimation (Level 1+), and detailed hydraulic

 (Level 2) assessments, grouped by Missouri Department of Transportation (MoDOT) district.

[--, no bridges]

County	Bridges that received	Bridges that received Bridges that received	
(fig. 1)	Level 1 assessment	Level 1+ assessment	Level 2 assessment
MoDOT District 01			
Andrew	27	14	7
Atchison	51	14	7
Buchanan	45	4	1
Caldwell	23	11	4
Clinton	11	1	1
Daviess	37	21	11
De Kalb	32	16	7
Gentry		29	18
Harrison	53	19	8
Holt	47	9	7
Nodaway	58	24	13
Worth	13	8	3
Total	397	170	87
MoDOT District 02			
Adair	32	11	4
Carroll	58	30	17
Chariton	48	11	11
Grundy	27	19	7
Howard	28	4	1
Linn	53	15	8
Livingston	40	17	11
Macon	33	8	5
Mercer	23	14	9
Putnam	28	14	6
Randolph	17	3	2
Saline	39	1	2
Schuyler	28	6	3
Sullivan	31	18	10
Total	485	171	96

Table 1. Number of bridges that received potential-scour (Level 1), rapid estimation (Level 1+), and detailed hydraulic

 (Level 2) assessments, grouped by Missouri Department of Transportation (MoDOT) district.—Continued

[--, no bridges]

County (fig. 1)	Bridges that received Level 1 assessment	Bridges that received Level 1+ assessment	Bridges that received Level 2 assessment
MoDOT District 03			
Audrain	24	9	6
Clark	18	5	5
Knox	22	8	3
Lewis	19	4	3
Lincoln	32	9	2
Marion	15	7	
Monroe	16	9	1
Montgomery	18	9	4
Pike	16	7	4
Ralls	14	6	
Scotland	28	8	7
Shelby	18	6	3
Warren	19	8	4
Total	259	95	42
MoDOT District 04			
Cass	26	2	
Clay	49	19	5
Henry	27	10	3
Iackson	57	15	9
Johnson	42	5	1
I afavette	53	3	
Platte	55 47	18	7
Pav	36	10	6
Total	337	83	31
Total	551	05	51
MoDOT District 05			
Benton	3	4	
Boone	35	16	1
Callaway	32	19	6
Camden	12	2	
Cole	19	12	11
Cooper	26	10	5
Gasconade	17	7	3
Maries	10	7	2
Miller	13	8	5
Moniteau	5	2	2
Morgan	3	2	1
Osage	16	12	2
Pettis	17	13	6
Total	208	114	44
MoDOT District 06			
Franklin	37	11	2
Tefferson	45	21	$\frac{2}{2}$
St Charles		13	2
St. Charles	25 60	10	10
St. Louis City	1	+7 0	12
Total	+ 171	06	

Table 1. Number of bridges that received potential-scour (Level 1), rapid estimation (Level 1+), and detailed hydraulic

 (Level 2) assessments, grouped by Missouri Department of Transportation (MoDOT) district.—Continued

County	Bridges that received	Bridges that received	Bridges that received
(fig. 1)	Level 1 assessment	Level 1+ assessment	Level 2 assessment
MoDOT District 07			
Barry	5	4	
Barton	10	1	
Bates	30	9	1
Cedar	8	2	
Dade	5	3	
Iasper	34	14	
Lawrence	27	17	2
McDonald	19	17	1
Newton	19	10	
St Clair	8	8	
Vernon	33	8	
Total	108	03	<u></u>
10141	170	75	-
Christian	10	4	
Christian	10	4	
Dallas	5	/	1
Douglas	l	l	
Greene	18	13	
Hickory	2	1	
Laclede	12	3	1
Ozark	7	5	1
Polk	7	3	
Stone	6	3	
Taney	7	6	
Webster	6	3	1
Wright	6	3	1
Total	87	52	5
MoDOT District 09			
Carter	8	3	1
Crawford	10	3	
Dent	17	1	1
Howell	17	7	
Iron	10	7	1
Oregon	15	12	3
Phelps	13	4	
Pulaski	11	1	
Revnolds	18	5	
Ripley	23	16	5
Shannon	11	6	
Texas	11	4	
Washington	9	9	
Total	173	78	
Total	175	70	11
MoDOT District 10			
Bollinger	26	16	2
Butler	20 77	33	2
Cane Girardeau	59	32	6
Dunklin	83	67	8
Madison	11	3	
Mississinni	45	1	
THOUSON PPI	15	1	

 Table 1. Number of bridges that received potential-scour (Level 1), rapid estimation (Level 1+), and detailed hydraulic

 (Level 2) assessments, grouped by Missouri Department of Transportation (MoDOT) district.—Continued

[--, no bridges]

County (fig. 1)	Bridges that received Level 1 assessment	Bridges that received Level 1+ assessment	Bridges that received Level 2 assessment
MoDOT District 10—	Continued		
New Madrid	118	88	16
Pemiscot	83	33	
Perry	15	13	6
St. Francois	15	11	2
Ste. Genevieve	22	19	1
Scott	57	16	3
Stoddard	113	85	7
Wayne	43	27	5
Total	767	444	58
Grand Total	3,082	1,396	398

Purpose and Scope

The purpose of this report is to describe potential-scour assessments and estimates of scour depth at selected bridge sites in Missouri. Results of the Level 1, Level 1+, and Level 2 assessments performed during a 13-year study in Missouri are presented. This report describes the methods used to obtain Level 1+ and Level 2 scour depth estimates, and compares the results of the two methods at 316 bridges.

In the comparison, scour depths determined by the Level 1+ method are compared against depths determined by the Level 2 method (the reference base). Depths determined by the Level 1+ method that are less than depths determined by the Level 2 method are therefore referred to as "underestimates" in this report, whereas depths greater than the Level 2 estimates are referred to as "overestimates."

Study Area

The study area includes the state of Missouri, which contains 114 counties and the city of St. Louis. MoDOT has divided the State into 10 highway districts (fig. 1). Missouri has a total area of 69,674 square miles, and consists of parts of three major physiographic regions: the Central Lowlands, the Ozark Plateaus, and the Mississippi Alluvial Plain (Fenneman, 1938) (fig. 1).

Approximately 45 percent of the study area is in the Central Lowlands region, predominantly in the northwestern part of the State, north of the Missouri River (fig. 1). Elevations in this region range from 600 to 1,200 feet above the North American Vertical Datum of 1988 (NAVD 88), and the area has numerous wide, flat valleys incised by rivers (Alexander and Wilson, 1995; Hauck and Nagel, 2003). This region is made up of loess or clayey glacial till soils overlying sedimentary shales, limestones, and dolomites. Streambeds in this region primarily are made up of very fine gravels, sands, and cohesive soils.

Approximately 45 percent of the study area is in the Ozark Plateaus region in the south-central part of the State (fig. 1).

This region typically is wooded, rugged, and has deep narrow valleys separated by sharp ridges. Elevations in this region range from about 800 to 1,700 feet above NAVD 88 (Alexander and Wilson, 1995; Fenneman, 1938; Hauck and Nagel, 2003). It primarily is composed of limestone and dolomite layers interspersed with sandstone and shale layers, overlying igneous rocks that in one location protrude through the layers to form the St. Francois Mountains in the southeastern part of the state. The limestone and dolomite layers contain cherty material that often is present in the streambeds in this region.

In the extreme southeast part of the State is the Mississippi Alluvial Plain region (fig. 1). About 10 percent of the State's area is in this region, which consists of well-drained alluvial deposits that are relatively flat, with elevations ranging from 200 to 300 feet above NAVD 88 (Alexander and Wilson, 1995; Hauck and Nagel, 2003). Streams in this region are very low gradient, and are incised in alluvial deposits from the Mississippi River. Numerous dredged ditches drain this area, and many of the natural channels have been straightened and leveed. Within the Mississippi Alluvial Plain region is Crowley's Ridge, which has the streamflow characteristics of the Ozark Plateaus physiographic region.

Potential-Scour Assessments (Level 1)

A potential-scour assessment (Level 1) is a methodology by which specific characteristics of a bridge site are documented and the potential for damaging scour to occur is qualitatively determined. The bridge site is examined for evidence of past scour, such as existing scour holes around the bridge piers or abutments, mass wasting of the banks at or near the bridge, or slumped riprap on the abutments or on the banks near the bridge. The evidence of past scour is a key indicator that the site experiences scour and would warrant a more detailed evalua-



Figure 1. Highway districts and major physiographic regions of Missouri.

tion. The site also is examined for characteristics that might increase the potential for scour to occur, such as the flow angle of attack, the percentage of channel and flood-plain constriction caused by the bridge and road approach embankments, the presence of piers in the main channel, or the presence of debris rafts at piers. The various characteristics are assigned a value that pertains to the propensity of the characteristic to exacerbate or minimize scour at the site. Forms used to collect bridge site data, the characteristics examined with regards to observed scour and potential scour, and the values assigned to these characteristics in Missouri are documented in Huizinga and Waite (1994). Level 1 assessments were made at 3,082 bridges in Missouri, as shown in figure 2.

The values assigned to the evidence of past scour are combined into a total observed-scour index. Similarly, the values assigned to the characteristics that affect the scour potential are combined into a total potential-scour index. These two indices were used to qualitatively determine whether or not a bridge was susceptible to scour. Computed index values for potential and observed scour for each bridge evaluated in the Level 1 assessment are provided in table 2, on the compact disk at the back of this report. If a bridge had an observed-scour index of 3 or higher, or a potential-scour index of 40 or higher, it became a candidate for further evaluation; otherwise, it was classified as not susceptible to scour. Occasionally a bridge that was classified as not susceptible to scour received further evaluation, as noted in the remarks column of table 2.

Estimates of Scour Depth Using Different Techniques

The Level 1 assessment indicated that 1,327 bridges would require a more detailed evaluation for scour, based on the computed potential- and observed-scour index values. Upon completion of the Level 1 work, detailed hydraulic assessments (Level 2's) had been completed at 35 bridges that the Level 1 assessment indicated were most susceptible to scour, as well as 2 overflow bridges that the Level 1 assessment indicated were not susceptible to scour, but were in proximity to 1 of the 35 bridges. A rapid estimation method (Level 1+) developed by Holnbeck and Parrett (1997) was utilized to further evaluate the remaining 1,292 bridges deemed scour-susceptible based on the Level 1 assessment, and approximate scour depths were calculated. Sites that had scour depths greater than defined values were then assessed with a detailed hydraulic assessment (Level 2) using the guidelines set forth in Hydraulic Engineering Circular No. 18 (HEC-18) (Richardson and Davis, 1995, 2001).

Rapid Estimation Method (Level 1+)

The rapid estimation method (Level 1+) is a method by which pertinent data can be collected about a bridge site during a brief visit, from which scour depths can be estimated. It is not intended to replace the more detailed Level 2 method, but it is considered useful for limited-detail bridge scour assessments. This method was chosen to assess the 1,292 bridges that required a more detailed evaluation of scour as determined by the Level 1 potential- and observed-scour index values that had not yet received a Level 2 assessment (fig. 3). In 2002, MoDOT identified 104 bridges that had not been previously assessed for scour-susceptibility, and these bridges also received a Level 1+ assessment (fig. 3).

The Level 1+ method uses calculated scour-depth and hydraulic data from Level 2 assessments from 10 states, including Missouri. Relations between significant hydraulic or physical variables and calculated scour depths were developed from the Level 2 assessments. Surrogate variables that could easily be measured or estimated in the field were substituted for more complex hydraulic variables required in the Level 2 analyses, and the scour equations were simplified to utilize the surrogate variables to estimate scour depths. The Level 1+ method is designed to overestimate scour depths, compared to the Level 2 method, by using envelope curves to define the relation rather than best-fit curves. An exhaustive explanation of the development, implementation, and limitations of the Level 1+ method is contained in Holnbeck and Parrett (1997), but two special variables and additional computation requirements needed to apply the method, as well as the general procedure of the method as applied in Missouri, are discussed in the following sections.

Special Variables Needed for Application in Missouri

Two variables discussed in Holnbeck and Parrett (1997) that are presumed to vary regionally are the average main channel velocity at the bridge, V_2 , and the difference in water-surface elevation from the approach section to the downstream face of the bridge, Δh . The first regional variable, the average main-channel velocity, V_2 , is critical in the determination of scour, and is determined by taking the total discharge through the bridge opening, Q_2 , divided by the total flow area at the bridge, A_2 . However, to enable comparison of widely varying ranges of bridge span and discharge, Holnbeck and Parrett advocate converting the total discharge to a unit discharge, q_2 , by dividing the total discharge, Q_2 , by the estimated top width of flow at the downstream face of the bridge opening (adjusted for any skewness to flow and for effective pier width), W_2 . The average velocity is determined using an equation that defines the relation between the unit discharge, q_2 , and the average velocity, V₂. Holnbeck and Parrett (1997) developed this relation for Montana using data from Level 2 analyses done in Montana and Colorado, which share similar physiography.

In Missouri, Level 2 analyses had been completed at only 35 sites before the Level 1+ assessment, and all of these sites were in the Central Lowlands region. To avoid any physiographic regional bias in the Level 1+ assessments, gage information was used to develop the relation for V_2 . However, discharge measurements at gages typically are not of sufficient

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Figure 2. Distribution of potential-scour assessment (Level 1) sites in Missouri.



Figure 3. Distribution of rapid estimation method (Level 1+) sites in Missouri.

magnitude to be comparable to the flows used in the Level 1+ assessment. Therefore, the slope and area for 23 gages in the Central Lowlands region and 29 gages in the Ozark Plateaus region found in Alexander and Wilson (1995) were used to estimate the 100- and 500-year peak discharges at the gages using the regional regression equations. Discharge measurements at each gage were used to approximate the average velocity at the 100-year and 500-year discharges, and this information was used to develop a relation that estimates the average main-channel velocity from the unit discharge at the gage for the Central Lowlands and Ozark Plateaus physiographic regions:

$$V_2 = 0.931 q_2^{0.395}$$
 for the Central Lowlands, and (1)

$$V_2 = 0.950 q_2^{0.399} \text{ for the Ozark Plateaus}$$
(2)

where

- V_2 is the average main channel velocity, in feet per second (ft/s);
- q_2 is the unit discharge per foot of width, in cubic feet per second per foot (ft³/s/ft) or square feet per second (ft²/s), computed as Q_2/W_2 ;
- Q_2 is the total discharge through the bridge, in cubic feet per second (ft³/s); and
- W_2 is the estimated adjusted top width of flow, in feet. Equations 1 and 2 have coefficients of determination (R^2) of 0.36 and 0.49, respectively. An average main-channel velocity equation was not determined for the Mississippi Alluvial Plain region because of sparse gage data in this region. Equation 1 for the Central Lowlands was used for sites in the Mississippi Alluvial Plain region.

The second regional variable, the difference in water-surface elevation from the approach section to the downstream face of the bridge, Δh , requires information about the water-surface elevation at the approach section, typically one bridge length upstream from the bridge. Information on approach conditions typically is not available from discharge measurements at gages; therefore, the Level 2 assessments performed at 35 sites in Missouri before the Level 1+ assessments were used to develop a relation between the average velocity at the bridge, V_2 , and Δh , which was determined to be:

$$\Delta h = 0.030 V_2^2 + 0.03 \tag{3}$$

where

- Δh is the change in water-surface elevation from the approach section to the downstream face of the bridge, in feet; and
- V_2 is the average main channel velocity determined from equation 1 or 2, in feet per second (ft/s).

Equation 3 has a coefficient of determination (R^2) of 0.52. Although all of the 35 Level 2 assessments were in the Central Lowlands physiographic region, equation 3 was used to determine Δh in all three physiographic regions, because the Level 2 assessments were the only source for these data in Missouri before the Level 1+ assessments. However, as will be shown later, equation 3 provides a reasonable estimation of Δh throughout Missouri.

Determination of Average Bed Location for Referencing Average Depth

The flow area at the bridge is controlled by the geometry of the bridge opening, and the average velocity, V_2 , would be represented by the "average" flow area, A_2 , for a given discharge. For a rectangular bridge opening, the average flow area

$$A_2 = b_{\rm adj} * y_2 \tag{4}$$

where

 A_2 is the average area of flow, in square feet (ft²);

- b_{adj} is the clear span of the bridge, b, adjusted for skew, computed as $b * \cos \theta$, in feet;
 - θ is the flow angle of approach of the bridge, or skew, in degrees; and
- y_2 is the depth of flow at the downstream bridge face, in feet.

In the case of a rectangular bridge opening, equation 1 or 2 can be used to directly determine the depth of flow, because

$$Q_2 = V_2 A_2$$

and, therefore

$$q_2 = V_2 y_2.$$
 (5)

However, as noted in Holnbeck and Parrett (1997), when a bridge has spill-through abutments, the bridge opening can be approximated by a trapezoidal shape and an iterative procedure must be followed to determine the actual flow width. This process is further complicated when the abutments are set back from the channel so that flow is conveyed in both the mainchannel and setback areas of the bridge opening, as is the case at a majority of the bridges assessed in Missouri.

Therefore, when each site was visited, the "average bed", of the channel at the bridge had to be determined. As shown in figure 4, the average bed of the bridge opening was chosen so that the average flow area would approximate a rectangle; therefore, equation 1 or 2 could be used to determine the depth of flow at the bridge, y_2 . Although the location of the average bed is dependent upon the depth of flow, it typically was chosen such that the area below the average bed in the channel approximately equaled the area above the average bed on the setback areas of the bridge (fig. 4). In this way, the average flow area is approximated as a rectangle, and equation 5 could be used. The water-surface elevation was determined using the average depth of flow on the setbacks and on the approach flood plains were determined relative to the water surface.

Results obtained using the average flow approximation technique indicated depths determined by this method were rea-



Figure 4. Definition sketch of average flow area, average bed, and average flow depth as used in rapid estimation assessment (Level 1+) in Missouri.

sonable for most of the sites assessed by the Level 1+ method. However, as will be shown later, at sites that had substantial setback areas relative to the channel area, the approximation technique resulted in an "average" channel flow depth that was substantially less than the depth of flow in the channel used in the Level 2 assessments, which resulted in lower main channel contraction scour depths in the Level 1+ assessments compared to the Level 2 assessments. Similarly, bridges with substantial setback areas tended to have lower average velocities in Level 1+ assessments than bridges with small setback areas, which resulted in lower pier scour depth estimates in the Level 1+ assessments compared to the Level 2 assessments.

Modification of Discharge

MoDOT requested that scour depth estimates be made for two discharges: the 100-year peak discharge, Q_{100} , (the discharge that has a 100-year recurrence interval, or a 1 percent chance of occurring in any given year), and a second discharge equal to the lesser of the 500-year peak discharge, Q_{500} , or the discharge that causes impending overtopping of the road embankments adjacent to the bridge, Q_{imp} , (impendent discharge). The two flood discharges were generically labeled Q_a and Q_b , and the scour depth estimates were associated with the part of the total flood discharge that passed through the bridge opening, labeled Q_{2a} and Q_{2b} . The bridges being assessed were plotted on a map, and the contributing drainage basin was determined for each. The area and slope of each basin were obtained, and the 100-year and 500-year peak discharges were determined for each site based on the regression equations described in Alexander and Wilson (1995). Generally, the calculated 100year discharge was used as the first flood discharge, Q_a ; the calculated 500-year discharge was used as the second discharge, Q_b ; and the discharges through the bridge, Q_{2a} and Q_{2b} , were equal to the total discharges, Q_a and Q_b . Occasionally, however, modifications to the calculated 100-year and 500-year discharges were required to find the discharge through the bridge. Characteristics of the site that would warrant modification of the discharges were: adjacent road embankments that would experience road overflow; the presence of additional bridges (for example, bridges designed for flood-plain overflow, or bridges over secondary channels that were not hydraulically isolated from the bridge in question); or, bridges located in the Mississippi Alluvial Plain physiographic region.

Road Overflow

When the potential for road overtopping existed at a site, the following procedure was followed:

- 1. The depth of flow relative to the average bed of the channel at the bridge associated with impendent road overtopping, y_{2imp} , was determined at the highest point of the road cross section (typically the center line or the high side of a tilted curve) at the lowest section of road embankment.
- 2. The top width of flow under the bridge associated with

that depth, b_{imp} , was determined and adjusted for skew as

$$(b_{\rm imp})_{\rm adi} = b_{\rm imp} * \cos \theta.$$
 (6)

3. The unit discharge associated with the impendent discharge was calculated using equation 5

$$q_2 = V_2 y_2$$

or,

$$y_2 = q_2 / V_2. (7)$$

Replacing V_2 using equation 1 for the Central Lowlands,

$$y_2 = q_2 / (0.931 q_2^{0.395})$$

and simplifying yields

$$q_{2\text{imp}} = (0.888 \ y_{2\text{imp}}^{1.653})$$
 for the Central Lowlands. (8)

Similarly, replacing V_2 using equation 2 for the Ozark Plateaus and simplifying yields

$$q_{2\text{imp}} = (0.918 \ y_{2\text{imp}}^{1.664})$$
 for the Ozark Plateaus. (9)

4. Using the unit discharge from equation 8 or 9 and the adjusted top width of flow from equation 6, the impendent discharge was determined as

$$Q_{\rm imp} = q_{\rm 2imp} * (b_{\rm imp})_{\rm adj}.$$
 (10)

It was assumed that the total impendent discharge also passed through the bridge, or $Q_{2imp} = Q_{imp}$.

- 5. The two discharges used to estimate scour depths at the site were assigned using the following logic:
 - a. if Q_{imp} is less than Q_{100} , Q_a was assumed to be Q_{100} , and

$$Q_{2a} = Q_{100} - {}^{1}/_{2}(Q_{100} - Q_{imp});$$
(11)

 Q_b and Q_{2b} were assumed to be Q_{imp} .

- b. if Q_{imp} was between Q_{100} and Q_{500} , Q_a and Q_{2a} were assumed to be Q_{100} , and Q_b and Q_{2b} were assumed to be Q_{imp} .
- c. if Q_{imp} was greater than Q_{500} , Q_a and Q_{2a} were assumed to be Q_{100} , and Q_b and Q_{2b} were assumed to be Q_{500} .

Presence of Additional Bridges

When an additional bridge was present, separate assessments were performed for the primary bridge in question and the additional bridge(s). To determine the part of the flood discharges to assign to each bridge, the approximate conveyance of each bridge was determined. Conveyance is a component of the Manning's uniform-flow equation (Chow, 1959), and is defined as

$$K = \frac{1.49}{n} A R^{2/3} \tag{12}$$

where

- *K* is the conveyance of the section, in cubic feet per second (ft³/s);
- *n* is the Manning's roughness coefficient for the section (Chow, 1959; Barnes, 1967; Arcement and Schneider, 1989);
- A is the cross-sectional area of the section, in square feet (ft^2) ; and
- *R* is the hydraulic radius of the section, in feet.

The *approximate* conveyance of each bridge opening was determined using the average flow area of the bridge opening, assuming the depth and hydraulic radius were both approximated by the distance from the average bed to the low steel of the bridge deck, or

$$K_i = \frac{1.49}{n} (b \cdot y_{\rm ls}) y_{\rm ls}^{2/3}$$

or, simplifying

$$K_i = \frac{1.49}{n} b \cdot y_{\rm ls}^{5/3} \tag{13}$$

where

- K_i is the approximate conveyance of the individual bridge *i*, in cubic feet per second (ft³/s);
- *n* is the Manning's roughness coefficient averaged for the total bridge opening;
- b is the total bridge length (skew is ignored), in feet; and
- y_{ls} is the distance from the average bed to the low steel of the bridge deck, in feet.

A conveyance ratio for each bridge was established, using

$$\gamma_i = K_i / K_{\text{tot}} \tag{14}$$

where

- γ_i is the conveyance ratio of the individual bridge *i*;
- K_i is the approximate conveyance of the individual bridge *i*, in cubic feet per second (ft³/s); and
- K_{tot} is the sum of the approximate conveyances of all the individual bridges, in cubic feet per second (ft³/s).

The two flood discharges, Q_a and Q_b , and the discharges passing through the bridge, Q_{2a} and Q_{2b} , were modified by the conveyance ratio of each bridge to get the flood discharges and bridge discharges for each bridge. The upstream flood plain was divided into sections to be associated with each bridge, and each bridge was examined separately.

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When a combination of road overflow and additional bridges existed at a site, the discharge modification for road overflow was performed first, and the resulting Q_a , Q_b , Q_{2a} , and Q_{2b} were then distributed to the various bridges using the calculated conveyance ratios.

Sites Located in the Mississippi Alluvial Plain Region

Because of the low gradients and general lack of substantial relief in the Mississippi Alluvial Plain region of the state, the region is drained by a complex system of interconnected dredged ditches. The main ditches often are leveed, but many of the tributary ditches are not. During floods, the smaller tributary ditches will overtop their banks and spread over wide areas, with minimal flow velocities (except near bridges). Furthermore, roadways in this area often are raised slightly above the level of the surrounding land, and will cross several ditches that, while separated physically by substantial distance, are hydraulically connected to one another during floods. The regional regression equations in Alexander and Wilson (1995) use drainage area as the only variable for determination of discharge in the Mississippi Alluvial Plain. However, the equations for this region were derived from gaging stations on the main ditches, whereas determination of the contributing drainage area of a particular bridge on the tributary ditches is difficult because of the hydraulic connectivity of these ditches in floods.

Therefore, only the discharge that caused impending road overtopping, Q_{imp} , was used to compute the scour depths for a majority of the sites located in the Mississippi Alluvial Plain region. The impendent discharge was determined using the techniques described in the Road Overflow section and the Central Lowlands equation (eq. 8) was used to determine the depth of flow. For bridges located at or near a gaging station on one of the principal rivers or main ditches, two discharges were used as in the other regions. Bridges located in the Crowley's Ridge area of the Mississippi Alluvial Plain region (fig. 1) were considered to be in the Ozark Plateaus region, and scour depths were computed for two discharges.

General Procedure of Level 1+ Assessments in Missouri

The following general procedure was followed to assess bridges using the Level 1+ method in Missouri:

- 1. In the office, the location of the site is established, the area and slope of the contributing drainage are determined, and the physiographic province in which it lies is determined. From this information, the 100-year and 500-year peak discharges, Q_{100} and Q_{500} , are calculated using the regression equations in Alexander and Wilson (1995).
- 2. The site is visited, and the presence of potential road overflow sections or additional bridges is established. If present, the peak discharges are modified, and the discharges through the bridge, Q_{2a} and Q_{2b} , are determined.

- 3. The flow angle of attack, or skew of the bridge to flow, θ , is determined.
- 4. The clear span of the bridge (length of the bridge from abutment face to abutment face), *b*, is adjusted by the skew angle, as

$$b_{\rm adi} = b * \cos \theta. \tag{15}$$

- 5. The average bed is calculated at the location where the area below the average bed in the channel approximately equals the area above the average bed on the setback areas of the bridge (fig. 4). The base of a survey rod is set at this location.
- 6. The preliminary unit discharge, q_2 , is determined by dividing the discharge through the bridge by the adjusted span length as the top width of flow,

$$q_2 = Q_2 / b_{adi.}$$
 (16)

- 7. The average velocity is determined from equation 1 or 2, and the average depth of flow above the average bed is determined from equation 7.
- 8. Using the survey rod set at the average bed, the depth of flow is measured and the top width of flow is adjusted to account for spill-through abutments (if necessary), and then adjusted for skew, b^*_{adj} .
- 9. Repeat steps 6 through 8 until the depth of flow, y_2 , from one iteration to the next changes by less than 0.5 feet.
- 10. Determine the difference in water-surface elevation from the approach to the downstream bridge face, Δh , by squaring the average velocity from step 7 and using equation 3.
- 11. Determine the average main channel depth of flow at the approach as:

$$y_1 = y_2 + \Delta h. \tag{17}$$

- 12. Repeat steps 6 through 11 for the second discharge, either the impendent road overtopping discharge, Q_{imp} , or the 500-year peak discharge, Q_{500} .
- 13. Determine the width of flow, *W*, an average Manning's roughness coefficient, *n*, and the elevation relative to the average bed of the left and right setbacks at the bridge, the left and right flood plains at the approach, and the channel at the bridge and at the approach.
- 14. Approximate the median particle size of the material in the channel, D_{50mc} , and on the overbank immediately upstream from the bridge, D_{50ob} , using visual identification of the material as cobbles, gravel, sand (coarse, medium, or fine), or silt/clay.
- 15. Determine the average width, total length, and flow angle of attack (if different than the bridge skew) for the bents or piers. If there is a difference between bents and/or

piers on the overbank and those in the main channel, obtain information for each type.

- 16. Determine the abutment shape coefficient (Richardson and others, 1993).
- 17. Take the parameters collected at each site and process them using a spreadsheet that calculates the estimated depth of scour using the envelope curves and scour predictor equations developed by Holnbeck and Parrett (1997).

Scour Predictor Equations

The scour predictor equations as applied in Missouri are reproduced below. A full explanation of the development, implementation, and limitations of these equations, as well as the envelope curves associated with the scour predictor equations, can be found in Holnbeck and Parrett (1997).

In the following equations for contraction scour, the depth, y, width of flow, W, and an average Manning's roughness coefficient, n, collected during step 13 in the previous section are used. These three variables were collected for the left and right setbacks at the bridge, the left and right flood plains at the approach, and the main channel at the bridge and approach. For simplicity in the following equations, the three variables are:

- y_x is the average depth in the section, in feet;
- W_x is the width of the section, in feet; and

 n_x is the average Manning's roughness coefficient of the section (Chow, 1959; Barnes, 1967; Arcement and Schneider, 1989).

The subscript *x* of the variables follows the convention:

- 1 is the main channel at the approach;
- 2 is the main channel at the bridge;
- *lob* is the left overbank or flood plain at the approach;
- rob is the right overbank or flood plain at the approach;
- *lsb* is the left setback at the bridge; and
- *rsb* is the right setback at the bridge.

Additionally, discharge for each section was apportioned using the conveyance ratio of each section, γ_x , determined by

$$\gamma_x = K_x / K_{\text{tot}} \tag{18}$$

where

- γ_x is the conveyance ratio of the section;
- K_x is the conveyance of the section, in cubic feet per second (ft³/s); and
- K_{tot} is the total conveyance of the whole section, in cubic feet per second (ft³/s).

Equation 18 is the same as equation 14, except that the conveyance ratio is for a portion of the flow area at the bridge or approach rather than for one bridge of a multiple bridge site. Then, using equation 18 and the form of equation 13, the conveyance ratio for the main channel at the approach, γ_1 , is

$$\gamma_{1} = \frac{K_{1}}{K_{\text{tot}}} = \frac{K_{1}}{(K_{1} + K_{lob} + K_{rob})} = \frac{\left(\frac{1.49}{n_{1}}W_{1}y_{1}^{5/3}\right)}{\left(\frac{1.49}{n_{1}}W_{1}y_{1}^{5/3}\right) + \left(\frac{1.49}{n_{lob}}W_{lob}y_{lob}^{5/3}\right) + \left(\frac{1.49}{n_{rob}}W_{rob}y_{rob}^{5/3}\right)}$$
(19)

where the width of the section, W_x , is used in place of the bridge length, b, from equation 13. Simplifying and rearranging yields

$$\gamma_{1} = \frac{1}{1 + \left(\frac{n_{1}}{W_{1}y_{1}^{5/3}}\right) \left(\frac{W_{lob}y_{lob}^{5/3}}{n_{lob}} + \frac{W_{rob}y_{rob}^{5/3}}{n_{rob}}\right)}{= \left[1 + \left(\frac{n_{1}}{W_{1}y_{1}^{5/3}}\right) \left(\frac{W_{lob}y_{lob}^{5/3}}{n_{lob}} + \frac{W_{rob}y_{rob}^{5/3}}{n_{rob}}\right)\right]^{-1}.$$
(20)

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Similarly, the conveyance ratios for the main channel at the bridge is

$$\gamma_2 = \left[1 + \left(\frac{n_2}{W_2 y_2^{5/3}} \right) \left(\frac{W_{lsb} y_{lsb}^{5/3}}{n_{lsb}} + \frac{W_{rsb} y_{rsb}^{5/3}}{n_{rsb}} \right) \right]^{-1}, \quad (21)$$

for the left setback at the bridge is

$$\gamma_{lsb} = \left[1 + \left(\frac{n_{lsb}}{W_{lsb}y_{lsb}^{5/3}}\right) \left(\frac{W_2 y_2^{5/3}}{n_2} + \frac{W_{rsb}y_{rsb}^{5/3}}{n_{rsb}}\right)\right]^{-1}, \quad (22)$$

and for the right setback at the bridge is

$$\gamma_{rsb} = \left[1 + \left(\frac{n_{rsb}}{W_{rsb}y_{rsb}^{5/3}}\right) \left(\frac{W_2 y_2^{5/3}}{n_2} + \frac{W_{lsb}y_{lsb}^{5/3}}{n_{lsb}}\right)\right]^{-1}.$$
 (23)

These are the conveyance ratios by which the discharge in the approach section, Q_1 , or through the bridge, Q_2 , can be weighted to represent flow in the appropriate section in the following equations.

For main channel contraction scour, it was necessary to determine which scour scenario (whether live bed or clear water) would occur. In Missouri, the main channel typically was assumed to experience live-bed contraction scour, except when the average velocity in the channel, V_2 , was less than the critical velocity of incipient motion, calculated as

$$V_c = 11.17 \ y_1^{1/6} \ D^{1/3} \tag{24}$$

where

- V_c is the critical velocity above which bed material of size *D* and smaller will be transported, in feet per second (ft/s);
- y_1 is the average depth of flow in the upstream channel, in feet; and
- D is the particle size for V_c , in feet, typically assumed to be the approximate median particle size of material in the main channel, D_{50mc} .

The presence of vegetation on the flood plains upstream from relief bridges or the overbank areas upstream from the main bridge would prevent transport of material to the contracted section; therefore, these areas were assumed to experience clear-water scour.

For live-bed conditions, the contraction scour predictor variable for main channel contraction scour is

$$\chi_{\rm mc,lb} = y_1 \left(\frac{\gamma_2 Q_2}{\gamma_1 Q_1} \right) \left(\frac{W_1}{W_2} \right) - y_1 \,.$$
 (25)

where

- Q_2 is the total discharge through the bridge, in cubic feet per second (ft³/s); and
- Q_1 is the total discharge in the approach, in cubic feet per

second (ft³/s); all other variables are defined previously.

When there is no flow over adjacent road embankments or through additional bridges such that $Q_1 = Q_2$, equation 25 is the same as equation 22 in Holnbeck and Parrett (1997) when the appropriate conveyance terms (eqs. 20 and 21) are used. The discharge terms are included in equation 25 because road overflow and the presence of additional bridges occur regularly in Missouri. For clear-water conditions, the contraction scour predictor variable for main channel contraction scour is

$$\chi_{\rm mc,cw} = 0.122 y_1 \left[\frac{\gamma_2 Q_2}{D_{50mc}^{\frac{1}{3}} y_1^{\frac{7}{6}} W_1} \right]^{\frac{7}{7}} - y_1 \,.$$
(26)

Equations 25 and 26 can be used at a bridge without setbacks. The conveyance ratio term, γ_2 , in the numerator of both equations weights the total discharge through the bridge, Q_2 , for flow on the setbacks. This term becomes 1 in equations 25 and 26 when no setbacks are present, and the total discharge through the bridge is used to determine the scour predictor variable.

For contraction scour in the left and right setback areas at the bridge, -6

$$\chi_{lsb} = 0.122 y_{lsb} \left[\frac{\gamma_{lsb} Q_2}{D_{500b}^{\frac{1}{3}} y_{lsb}^{\frac{7}{6}} W_{lsb}} \right]^{\frac{7}{7}} - y_{lsb}$$
(27)

and

$$\chi_{rsb} = 0.122 y_{rsb} \left[\frac{\gamma_{rsb} Q_2}{D_{50ob}^{\frac{1}{3}} y_{rsb}^{\frac{6}{5}} W_{rsb}} \right]^{\frac{6}{7}} - y_{rsb}$$
(28)

where

- χ_{lsb} is the contraction scour predictor variable for clearwater contraction scour on the left setback, in feet;
- χ_{rsb} is the contraction scour predictor variable for clearwater contraction scour on the right setback, in feet; and
- D_{50ob} is the approximate median particle size of the material on the overbanks immediately upstream from the bridge, in feet; all other variables are defined previously.

The contraction scour predictor variable, χ , determined from one of the above equations is entered on the abscissa of the appropriate envelope curve found in Holnbeck and Parrett (1997) to determine the estimated main channel or overbank contraction scour depth.

For pier scour, the average pier width, a, is entered on the abscissa of the pier scour envelope curve found in Holnbeck and Parrett (1997) to obtain the value for the pier scour function, ξ , defined as

$$\xi = \frac{y_{ps}}{K_{p2} \mathrm{Fr}_{2}^{0.15}}$$
(29)

where

- ξ is the pier scour function, in feet;
- y_{ps} is the pier scour depth determined by the rapidestimation method, in feet;
- K_{p2} is the correction factor for flow angle of attack on the pier, computed as $[\cos \theta_p + (L_p/a) \sin \theta_p]^{0.65}$;
- L_p is the length of the pier, in feet;
- *a* is the average pier width, in feet;
- θ_p is the flow angle of attack on the pier (which may or may not be the same as the skew of the bridge), in degrees;
- Fr₂ is the Froude number of the average flow at the downstream face of the bridge, computed as $V_2/(g y_2)^{1/2}$;
- V_2 is the average velocity in the main channel at the bridge determined by equation 1 or 2, in feet per second (ft/s);
- g is the acceleration of gravity, 32.2 feet per second squared (ft/s^2); and
- y_2 is the average depth of flow at the downstream face of the bridge, in feet.

The average pier width was used to determine a value of the pier scour function, ξ , for a typical pier or bent in the main channel and a typical pier or bent on the setback, and equation 29 was solved for the pier scour depth, y_{ps} .

For abutment scour, the flow depth at the abutment, y_a , is entered on the abscissa of the abutment scour envelope curve found in Holnbeck and Parrett (1997) to obtain the value for the abutment scour function, Ψ , defined as

$$\Psi = \frac{0.55}{K_{a1}} y_{as} \tag{30}$$

where

 Ψ is the abutment scour function, in feet;

- K_{a1} is the coefficient for abutment shape (Richardson and others, 1993); and
- y_{as} is the abutment scour depth determined by the rapid estimation method, in feet.

The average flow depth at the abutment was taken as y_{lob} for the left abutment and y_{rob} for the right abutment. These depths were used to determine the value of the abutment scour function, Ψ , and equation 30 was solved for the abutment scour depth, y_{as} .

Final Qualitative Assessment

The scour depths estimated at each bridge were compared to limiting depth criteria set by MoDOT; sites with one or more scour depths larger than the set limits became candidates for a detailed hydraulic assessment (Level 2). Limiting criteria were set at 5.0 feet of contraction or abutment scour and 10.0 feet of pier scour; bridges with depths greater than one or more of these values were examined using the Level 1 site assessment forms, photos, and bridge plans. The qualitative criteria used in the examination, based on MoDOT recommendations, were as follows:

- 1. If only main channel contraction scour existed and the main channel piers were set on bedrock, the site was no longer a Level 2 candidate.
- If only abutment scour existed, and the abutments were protected, the site was no longer a Level 2 candidate. Abutment scour depths were viewed with skepticism—particularly when any sort of protection existed—with the realization that abutment scour depths were overestimated by the HEC-18 equations.
- 3. If the contributing drainage area was 25 square miles or less, such that the flood wave would be short and the substructural elements would be visible upon flood recession, the bridge was no longer a Level 2 candidate. In the Mississippi Alluvial Plain physiographic province, if the impendent discharge was less than 5,000 cubic feet per second or the bridge was shorter than 100 feet, it was considered a "small drainage area" site (≤ 25 square miles).
- 4. If road overflow sections existed on both flood plains, such that the public would not be able to access the bridge during floods of a magnitude greater than the impendent road overtopping discharge, the site was no longer a candidate (provided criteria 5 was not met for the scour values at the impendent road overtopping discharge).
- 5. If the estimated scour depths were 5 feet greater than the initial limiting criteria, especially in the case of piling-type substructures, items 1 through 4 were ignored and the site remained a Level 2 candidate.
- 6. In the Mississippi Alluvial Plain physiographic region, the presence of levees on both upstream banks or flood plains would cause little to no constriction of flow at the bridge, and the site was no longer a Level 2 candidate. The primary cause of channel change at these bridge sites will be channel degradation because of dredging. This criterion was used to remove the candidacy of bridges that had large clear-water overbank scour values; however, it is intuitive that if there is little to no constriction at the bridge because of levees on the upstream banks, there will be no substantial contraction scour.

Estimated scour depths for all of the bridges assessed using the Level 1+ method are provided in table 3, on the compact disk at the back of this report. A summary of the Level 1+ assessment by MoDOT district in Missouri is shown in table 4.

MoDOT district	Bridges that received Level 1+ assessment	Level 1+ bridges deemed not susceptible to scour by limiting criteria	Level 1+ bridges with depths greater than limiting criteria (preliminary Level 2 candidates)	Bridges removed from preliminary Level 2 candidate list by qualitative assessment	Level 2 candidates from Level 1+ assessment ¹
01	170	28	142	71	71
02	171	22	149	74	75
03	95	25	70	34	36
04	83	23	60	35	25
05	114	28	86	54	32
06	96	61	35	22	13
07	93	70	23	19	4
08	52	28	24	19	5
09	78	48	30	19	11
10	444	256	188	136	52
Grand Total	1,396	589	807	483	324

 Table 4. Summary of rapid estimation assessments (Level 1+) and resultant detailed hydraulic assessment (Level 2) candidates by

 Missouri Department of Transportation (MoDOT) district.

¹Values in this column do not equal MoDOT district totals in table 1 or table 6 because Level 2 assessments were performed at bridges in addition to those from the Level 1+ assessment.

Detailed Hydraulic Assessments (Level 2)

Detailed hydraulic assessments (Level 2), were conducted at 398 bridges at 350 unique sites throughout Missouri (fig. 5). Not all of these bridges were Level 2 candidates based on the results of the Level 1 or Level 1+ assessments. Twenty-two bridges that were not Level 2 candidates based on the Level 1 and Level 1+ assessments were assessed because they were in proximity (or hydraulically connected) to a Level 2 candidate, such as a bridge on the flood plain designed to carry overflow, or a pair of bridges upstream from a confluence. Twenty-nine bridges over major rivers and two bridges on smaller streams were evaluated without a Level 1 or Level 1+ assessment. However, eight Level 2 candidates from the Level 1+ assessment were dropped because of bridge replacements, improvements, or other changes at the bridge that mitigated scour.

The procedure of a Level 2 assessment is fully described in the four editions of the FHWA Hydraulic Engineering Circular No. 18 (HEC-18) (Richardson and others, 1991, 1993; and Richardson and Davis, 1995, 2001); the analysis procedure and scour-depth equations were applied in Missouri with only slight modification. However, HEC-18 underwent changes with each edition; the general form of the various equations for contraction, pier, and abutment scour did not change substantially, but coefficients and terms were added or removed, and the methods used to determine certain variables evolved with time. A discussion of the hydraulic analysis and each scour depth equation follows, along with variations in each edition of HEC-18 and any modifications when used in Missouri.

Hydraulic Analysis

Application of the HEC-18 scour depth equations required estimates of flood discharges, hydraulic properties, and watersurface profiles. Flood discharge estimates for the 100-year and 500-year peak discharges (Q_{100} and Q_{500}) were determined for each site based on the regression equations described in Alexander and Wilson (1995). The Water-Surface PROfile Computation (WSPRO) step-backwater model (Shearman, 1990; Shearman and others, 1986) was used to determine hydraulic properties and water-surface profiles at each site, using the field data collected at the site such as cross sections, bridge geometry, and Manning's roughness coefficient estimates.

In addition to the 100-year and 500-year peak discharges, scour computations often were made for the discharge that caused either impending road overtopping or impending levee overtopping (impendent discharge, or Q_{imp}). If the site experienced road overtopping by either the 100-year or 500-year discharge, the impendent discharge typically was included as a potential worst-case scour scenario (for example, maximum flow pressure at the bridge, without relief from road overflow). Similarly, if levees were present on the upstream or downstream flood plains that were overtopped by either the 100-year or 500-year discharge, the discharge that caused impendent levee overtopping usually was included as a check to see if it created a worst-case scour scenario. The impendent discharge (whether road or levee) was determined by trial and error in the WSPRO model of the site.

The results of the hydraulic analysis were used as input to a series of spreadsheets and FORTRAN programs that calculated the contraction, pier, and abutment scour depths using the HEC-18 equations for each discharge. The spreadsheets and programs were modified and updated with the release of each edition of HEC-18.



Figure 5. Distribution of detailed hydraulic assessment (Level 2) sites in Missouri.

Contraction Scour

Contraction scour is evaluated for two basic scenarios: live-bed scour, which occurs when the approach flow is transporting bed material; and clear-water scour, which occurs when the approach flow is not transporting bed material (Richardson and Davis, 2001). In Missouri, live-bed scour typically occurs in the main channel, and clear-water scour occurs at the main bridge setbacks or at relief bridges.

The fundamental equation for both types of contraction scour is

$$y_{cs} = y_{c2} - y_{c0} \tag{31}$$

where

- y_{cs} is the average contraction scour depth, in feet;
- y_{c2} is the average depth in the contracted section after scour (for example, at the bridge), in feet; and
- y_{c0} is the existing depth in the contracted section before scour, in feet.

The existing depth in the contracted section before scour, y_{c0} , was introduced with the third edition of HEC-18 (Richardson and Davis, 1995). Before this, equation 31 was

$$y_{cs} = y_{c2} - y_{c1}$$

where

 y_{c1} is the average depth in the upstream main channel, in feet.

The third and fourth editions of HEC-18 indicated that y_{c1} could be used to estimate y_{c0} , particularly if the contraction scour hole fills in during the falling stage. For this reason, y_{c1} consistently was used in place of y_{c0} for assessments in Missouri.

For live-bed contraction scour, y_{c2} is determined as

$$y_{c2} = \left(\frac{Q_2}{Q_1}\right)^{6/7} \left(\frac{W_1}{W_2}\right)^{k_{c1}} y_{c1}$$
(32)

where

- Q_1 is the discharge in the upstream channel transporting sediment, in cubic feet per second (ft³/s);
- Q_2 is the discharge in the contracted channel, in cubic feet per second (ft³/s);
- W_1 is the width of the upstream main channel that is transporting bed material, in feet;
- W_2 is the width of the main channel in the contracted section less pier width(s), in feet; and
- k_{c1} is a coefficient that depends on whether the material transported is mostly contact bed material ($k_{c1} = 0.59$), contains some suspended material ($k_{c1} = 0.64$), or is mostly suspended bed material ($k_{c1} = 0.69$).

An additional term came and went from equation 32 with the various editions of HEC-18, which modified the contraction scour based upon a ratio of the Manning's roughness coefficients between the approach and the contracted section. However, when the roughness essentially was the same between the sections, the term dropped out of the equation. Therefore, this term was ignored in all of the assessments in Missouri. Nevertheless, the Manning's roughness coefficient is implicitly included in the equation, because it is included in the determination of the two discharge terms, Q_1 and Q_2 .

For clear-water contraction scour, y_{c2} is determined as

$$y_{c2} = \left[\frac{K_{u}Q_{o}^{2}}{D_{m}^{2/3}W_{o}^{2}}\right]^{3/7}$$
(33)

where

- K_u is a coefficient equal to 0.0077 (English units);
- Q_o is the discharge through the bridge or on the setback area at the bridge associated with the width, W_o , in cubic feet per second (ft³/s);
- *W_o* is the width of the contracted section less pier width(s), in feet;
- D_m is the diameter of the smallest non-transportable particle in the bed material in the contracted section, typically assumed to be 1.25 D_{50} , in feet; and
- D_{50} is the median diameter of bed material, or the particle size in a mixture of which 50 percent are smaller, in feet.

For clear-water scour applications of equations 31 and 33 in Missouri, y_{c0} was assumed to be the average depth of flow through the relief bridge or on the setback area at the main bridge, which essentially equalled the depth of flow of the unscoured bed.

Equation 33 has undergone only minor changes since the first edition of HEC-18. In the first edition (Richardson and others, 1991), K_u was equal to 1/120, and D_m was assumed to be D_{50} , rather than 1.25 D_{50} . The second edition (Richardson and others, 1993), introduced D_m as 1.25 D_{50} , and the third edition changed K_u to approximately 1/130 (which comes from $K_u = 1/40$ in SI units).

As in the Level 1+ assessments, the main channel typically was assumed to experience live-bed contraction scour, except when the average velocity in the approach channel, V_2 , was less than the critical velocity of incipient motion, calculated by equation 24 and shown here for clarity;

$$V_c = 11.17 y_{c1}^{1/6} D^{1/3}$$

where

- V_c is the critical velocity above which bed material of size *D* and smaller will be transported, in feet per second (ft/s);
- y_{c1} is the average depth of flow in the upstream channel, in feet; and
- D is the particle size for V_c , in feet, typically assumed to be D_{50} .

When the average approach velocity was nearly equal to the critical velocity, both live-bed and clear-water scour were computed for the main channel, and the values were compared. The presence of vegetation on the flood plains upstream from relief bridges or the overbank areas immediately upstream from the

main bridge would prevent transport of material to the contracted section; therefore, these areas were assumed to experience clear-water scour.

Pier Scour

The equation for pier scour is

$$y_{ps} = y_p \left[2.0 K_{p1} K_{p2} K_{p3} K_{p4} \left(\frac{a}{y_p}\right)^{0.65} \mathrm{Fr}_p^{0.43} \right]$$
(34)

where

- y_{ps} is the pier scour depth, in feet;
- \hat{y}_p is the depth of flow just upstream from the pier, in feet;
- K_{p1} is the correction factor for pier nose shape, ranging from 0.9 to 1.1;
- K_{p2} is the correction factor for flow angle of attack, computed as $[\cos \theta_p + (L_p/a) \sin \theta_p]^{0.65}$;
- L_p is the length of the pier, in feet;
- a is the pier width, in feet;
- θ_p is the flow angle of attack on the pier (which may or may not be the same as the skew of the bridge), in degrees;
- K_{p3} is the correction factor for bed condition, ranging from 1.1 to 1.3, assumed to be 1.1 in Missouri;
- K_{p4} is the correction factor for armoring by bed material, assumed to be 1.0 in Missouri;
- Fr_p is the Froude number just upstream from the pier, computed as $V_p/(g y_p)^{1/2}$;
 - g is the acceleration of gravity, 32.2 feet per second squared (ft/s^2); and
- V_p is the velocity of flow upstream from the pier, in feet per second (ft/s).

For round-nosed piers aligned with the flow, HEC-18 puts a limit on the total depth of scour that can be anticipated. For Fr_p < 0.8, the maximum scour depth is 2.4 times the pier width, *a*; for $Fr_p > 0.8$, the maximum scour depth is 3.0 times the pier width, *a*.

Equation 34 has undergone only minor changes since the first edition of HEC-18. In the first edition (Richardson and others, 1991), K_{p3} and K_{p4} were not in the equation; K_{p3} was added in the second edition (Richardson and others, 1993), and K_{p4} was added in the third edition (Richardson and Davis, 1995). However, neither of these additional factors substantially changed the results of pier scour in Missouri. The bed condition factor, K_{p3} , varies from 1.1 to 1.3 depending on the dune heights in the channel, but is 1.1 for dunes as much as 10 feet high (typically not seen in channels in Missouri, with the exception of the large rivers). Furthermore, the dune height was not measured at sites in Missouri. Similarly, the armoring factor, K_{p4} , is 1.0 for situations when $D_{50} < 2$ mm (millimeters) or $D_{95} < 20$ mm, which typically is the case in Missouri.

When equation 34 was applied to Level 2 assessments in Missouri for piers or bents located in the channel or on the channel banks, the depth of flow upstream from the pier, y_p , and the

velocity upstream from the pier, V_p , were assumed to be the maximum values in the channel area. This was to account for possible shifts in the channel thalweg during a flood event. However, for piers or bents located on the setback areas, the depth of flow upstream from the pier or bent was used for y_p , and the maximum velocity on the setback area was used for V_p for all piers or bents on a particular setback.

With the fourth edition of HEC-18 (Richardson and Davis, 2001), a comprehensive procedure for determining pier scour at complex pier foundations was introduced. Complex pier foundations are present extensively in Missouri, and consist of a column with a pile group, pile cap, or spread footer. Most pier scour research has focused on solid piers, with little research on pile groups, pile caps, and solid piers in any combination being exposed to flow (Richardson and Davis, 2001). Until the fourth edition of HEC-18, pier scour caused by exposed footings, pile caps, or pile groups were estimated using equation 34 with a modified approach velocity and depth. However, the fourth edition provided equations for pier scour at complex foundations that separated the foundation into its various components, and computed scour for each component relative to its position to the channel bed. The total complex foundation pier scour depth was a sum of the scour components. The complex foundations procedure was used in Missouri for Level 2 assessments performed after 2001. Before this time, pier scour depths for exposed footings were calculated using equation 34 with and without footing exposure; the deeper of the two computed depths was reported as a worst case.

Abutment Scour

Two equations are used to determine abutment scour depth; Froehlich's live-bed scour equation, and the HIRE equation. The Froehlich equation is

$$y_{as} = y_a \left[2.27 K_{a1} K_{a2} \left(\frac{L_e}{y_a} \right)^{0.43} \operatorname{Fr}_{a1}^{0.61} + 1 \right]$$
 (35)

where

- y_{as} is the abutment scour depth, in feet;
- y_a is the average depth of flow on the flood plain (computed as A_e/L_e in HEC-18, but assumed to be the depth of flow at the abutment toe in Missouri), in feet;
- A_e is the flow area of the approach section obstructed by the embankment, in square feet (ft²);
- L_e is the length of the embankment projected normal to flow (computed as $L_e adj = A_e/y_a$ in Missouri), in feet;
- K_{a1} is the coefficient for abutment shape, ranging from 0.55 to 1.0;
- K_{a2} is the coefficient for angle of the embankment to flow, computed as $\left(\frac{\theta_e}{90^\circ}\right)^{0.13}$;
 - θ_e is the angle of the embankment to the flow, in degrees; with θ_e less than 90 degrees if embankment points downstream and θ_e greater than 90 degrees if

embankment points upstream;

- L_e' is the length of active flow obstructed by the embankment (determined by engineering judgement), in feet;
- Fr_{a1} is the average Froude number of approach flow upstream from the abutment toe, computed as $V_e/(g y_a)^{1/2}$;
 - V_e is the average velocity of approach flow, computed as Q_e/A_e , in feet per second (ft/s); and
- Q_e is the approach flow obstructed by the abutment and approach embankment, in cubic feet per second (ft³/s).

The HIRE equation is

$$y_{as} = y_a \left[4 \operatorname{Fr}_{a2}^{0.33} \frac{K_{a1}}{0.55} K_{a2} \right]$$
(36)

where

- Fr_{a2} is the Froude number based on the velocity and depth adjacent to and upstream from the abutment toe, computed as $V_a/(g y_a)^{1/2}$; and
- V_a is the flow velocity at the abutment toe, in feet per second (ft/s), and all other variables are defined previously.

The HIRE equation did not change in the four editions of HEC-18. In Missouri, the Froehlich equation primarily was used, and the HIRE equation was used when the embankment length was greater than 25 times the depth of flow at the abutment toe.

The Froehlich equation had a tendency to overestimate abutment scour-noted in every edition, but especially in the fourth edition (Richardson and Davis, 2001)-and a modification to address this issue occurred in the fourth edition. The length of the embankment projected normal to flow, L_{e} , was changed to the length of active approach flow obstructed by the embankment, L_e' , which removed the effect of areas on the flood plain that convey shallow or low-velocity flow. However, before the fourth edition of HEC-18 was released, it was recognized in Missouri that the abutment scour equation likely overestimated abutment scour, so the abutment scour equation variables were modified. Rather than determining the average depth of flow on the flood plain using the area of blocked flow and the embankment length normal to flow (as designated in HEC-18), the depth of flow at the abutment toe was used as y_a and the effective length of the embankment, $L_{e adj}$, was determined by dividing the area of blocked flow by this depth. This resulted in less extreme values being calculated for abutment scour by shortening the embankment length normal to flow. When the fourth edition of HEC-18 was released, the new active flow length term, L_e' , was used in place of $L_{e \text{ adj}}$.

Reports

After the various scour depth values were determined using the HEC-18 equations, the scour depths were plotted on a profile of the bridge to create a scour prism (fig. 6). Plots for each discharge used in the WSPRO and scour analyses were created (fig. 6), as well as a plot that shows the scour prisms for all of the various discharges (fig. 7).

A report was created that provided the details of the hydraulic analysis and the scour depth computations for each site. The report contains a brief discussion of the site, which included characteristics of the site, a description of the bridge(s), trends of channel change with time, and the amount of exposure the bridge foundations experience as a result of the computed scour depths. An aerial photograph of the site from USGS Digital Orthophoto Quadrangles (DOQs) with the location of sections used in the WSPRO model was included with sites completed after 1998; before this time, a topographic map of the site with the location of the sections used in the WSPRO model was included. Photographs of the site taken during the site visit, tables of computed scour depths, and the various scour prisms also were included. An appendix to each report contained a detailed description of the site and associated drainage basin, Manning's roughness coefficients assigned to the channel and flood plains, the assumptions used in the WSPRO analysis and in the scour computations, the input and output files from WSPRO, and the output files from the scour depth computation program.

The reports include a rough classification of each bridge as having substantial, moderate, or minimal exposure. Substantial exposure was defined as when more than two-thirds of the total foundation depth was exposed or the foundation was undermined; moderate exposure was defined as when one-quarter to two-thirds of the total foundation depth was exposed; minimal exposure was defined as when less than one-quarter of the total foundation depth was exposed. Computed scour depths for all of the bridges assessed using the Level 2 method are given in table 5, on the compact disk at the back of this report. A summary of the Level 2 assessment by MoDOT district in Missouri are shown in table 6.

Comparison of Level 1+ Results to Level 2 Results

The Level 1+ method was designed such that estimates of scour depths should nearly equal or exceed the values computed by the Level 2 method to provide conservative estimates of scour depth (Holnbeck and Parrett, 1997). Level 2's were completed at 82 bridges that were not assessed as Level 1+'s; however, Level 2's were completed at 316 bridges for which Level 1+'s had been done, which provides a wide data base with which to compare the results of the two assessment procedures.

To provide a reasonable comparison between the results of the two assessments, it was necessary to identify a variable that was common between the two assessments for a given bridge. As mentioned in the previous sections, computed scour depths are associated with the discharge through the bridge opening (flow in the main channel and setbacks under the bridge, as opposed to the total discharge, which might include road overflow or flow through an adjacent bridge). For a majority of the bridges, scour depths were computed for two discharges in the







Table 6. Summary of detailed hydraulic assessments (Level 2) by Missouri Department of Transportation (MoDOT) district.

 [--, no bridges]

MoDOT district	Bridges that received Level 2 assessment	Level 2 bridges with substantial exposure	Level 2 bridges with either substantial or moderate exposure ¹	Level 2 bridges with moderate exposure	Level 2 bridges with either moderate or minimal exposure ¹	Level 2 bridges with minimal exposure	Level 2 bridges with foundations set on bedrock ²
01	87	61	4	17		5	5
02	96	62	2	23	2	7	5
03	42	27		11	2	2	1
04	31	18		5	1	7	5
05	44	32		8		4	10
06	20	12		8			3
07	4	2		1		1	1
08	5	5					3
09	11	4		6		1	4
10	58	35		11	1	11	4
Grand Total	398	258	6	90	6	38	41

¹Occasionally, a range of possible pile penetration lengths is given for a bridge pier or bent; these bridges could be classified as one or the other, depending on the true pile penetration in the field.

²Sites with foundations set on bedrock are a non-exclusive subset of the total number.

Level 1+'s (Q_{100} and either Q_{500} or Q_{imp}), and for as many as three discharges in the Level 2's (at least Q_{100} and Q_{500} , as well as Q_{imp} if road overflow occurred in the Q_{100} or Q_{500} discharges). The two discharges for a given bridge in the Level 1+, Q_{2a} and Q_{2b} , were compared to the discharges used in the Level 2 for the same bridge to find a match. For example, the discharge through the bridge opening for the 100-year peak discharge of the Level 1+ was compared to the discharge through the bridge opening for the 100-year peak discharge of the Level 2 for the same bridge; the same was done with the 500-year peak discharge or the impendent discharge. If the discharge through the bridge for the Level 1+ matched the Level 2 discharge within 10 percent, it was considered a good match and the scour depths were considered comparable.

Because of the coarse nature of the methods to determine the impendent discharge or modify the discharge for an adjacent bridge in the Level 1+'s, it often was difficult to find a match between the impendent discharge of the Level 1+ and the impendent discharge of the Level 1+ and the impendent discharge in the Level 1+ was substantially different from the Level 2 impendent discharge (for example, the Level 1+ Q_{imp} was *greater* than the Q_{100} , whereas the Level 2 Q_{imp} was *less* than the Q_{100}), finding a match between any of the Level 1+ and Level 2 discharges for a given bridge also was compromised. If a match could not be made between the discharges at a specific recurrence interval (for example, the 100-year peak discharge), the two Level 1+ discharges through the bridge were compared to any and all of the Level 2 to find a match.

Consequently, of the 316 bridges common to both assessment methods (with 595 individual discharges by Level 1+, and 1,019 individual discharges by Level 2), 327 discharges at 212 bridges matched within 10 percent (fig. 8). Five bridges had no scour depths computed in the Level 1+ assessment; three were relief structures with no observable scour, and two were well protected bridges over a large river with no observable scour. An additional 22 bridges had no match between the various discharge types of the Level 1+ and the Level 2 (for example, the Level 2 had a Q_{100} and Q_{500} , but the Level 1+ had Q_{imp} only, as frequently occurred in the Mississippi Alluvial Plain region). The remaining bridges had one or more discharges through the bridge opening that did not match within 10 percent (fig. 8).

The 327 discharges that matched within 10 percent were examined for comparable values of main channel contraction scour, overbank contraction scour (both left and right), pier scour (main channel and overbank), and abutment scour (left and right). Occasionally, a bridge had a scour depth determined by the Level 1+ but not the Level 2, or vice versa; these values are not included in the comparisons of Level 1+ and Level 2 depths. For each type of scour, the number of comparable values are stated as "matches." For example, there were main channel contraction values determined by both the Level 1+ and the Level 2 assessments at all 212 bridges for the 327 matching discharges, so there are 327 "matches" of main channel contraction scour. However, there were only 137 of the matching discharges that had values for the left overbank in both the Level 1+ and Level 2 assessments; these were combined with 133 such "matches" for the right overbank, for a total of 270 "matches" of overbank contraction scour. Similarly, there were 313 main channel pier scour "matches," and 83 overbank pier scour "matches," for a total of 396 "matches" of pier scour. There were 296 left abutment scour "matches," and 297 right abutment scour "matches," for a total of 593 "matches" of abutment scour.

In all of the comparisons, the scour depth computed in the Level 1+ is compared to the value obtained in the Level 2. An



Figure 8. Comparison of discharge through bridge used in rapid estimation assessment (Level 1+) with that used in detailed hydraulic assessment (Level 2).

equal-value line is shown in all comparison plots. When a point plots above the equal-value line, the Level 1+ value is a conservative (larger) estimate of the Level 2 scour depth; the closer to the equal-value line, the more closely the Level 1+ estimates the Level 2 value.

Contraction Scour

Contraction scour was separated into its two types, live bed and clear water. In the Level 1+ assessments, contraction scour in the main channel generally was considered to be live bed, because the velocity in the approach channel, V_1 , was assumed to be equal to the average velocity in the channel, V_2 . However, in the Level 2 assessments, 41 of the 327 comparisons were computed as clear-water scour, typically because the computed velocity in the approach channel, V_1 , was insufficient to transport material in the Level 2 assessment. The main channel contraction scour depths computed by both the live bed and the clear-water equations in the Level 2's are shown in figure 9. The contraction scour on the overbanks (fig. 10) was considered to be clear water without exception in both assessments.

Main Channel Contraction Scour

Generally, the Level 1+ method did not provide either a markedly conservative or a close estimate of the Level 2 main channel contraction scour depth in Missouri (fig. 9A). There is a substantial amount of variability in the Level 1+ depths, and the Level 1+ depths appear to underestimate the Level 2 scour as much as overestimate. When the 41 Level 1+ estimates that were computed as clear-water scour in the Level 2 are removed (fig. 9A), the range of variability is not improved, and more than one-half of the matches (154 of 286 live-bed matches) plot below the equal-value line.

The variability does not appear to be dependent upon the physiographic region (figs. 9B, 9C, and 9D). When the main channel contraction scour values are examined by physiographic region, the Level 1+ method does not provide any better estimate of the Level 2 main channel scour depth in the Central Lowlands (region I, fig. 9B) than in the Ozark Plateaus (region II, fig. 9C). A majority of the Central Lowlands data plot below the equal-value line (104 out of 222 live-bed matches were above, fig. 9B), as do a majority of the Ozark Plateaus data (28 out of 63 live-bed matches were above, fig. 9C). There were only two sites in the Mississippi Alluvial Plain for which the discharge through the bridge provided a match (region III, fig. 9D), and the Level 1+ estimate underestimated the Level 2 depth (the single live-bed match was below the equal-value line, fig. 9D).

The range of variability below the equal-value line is more substantial than above the equal-value line in all three regions. A best-fit trend line from the origin through the data in each region falls well below the equal-value line (figs. 9A, 9B, 9C, and 9D), which confirms that the magnitude of the Level 1+ underestimations is larger than the magnitude of the overestimations. However, it should be noted that no matter which region is examined, there were several cases where the extremes of variability were seen (that is, the Level 1+ estimated a zero depth of scour when the Level 2 resulted in some finite depth, and vice versa).

Overbank Contraction Scour

Generally, the Level 1+ method provides a more conservative estimate of the Level 2 overbank channel contraction scour depth than for main channel contraction scour in Missouri (161 of 270 matches fall above the equal-value line, fig. 10A). However, as is the case for the main channel contraction scour, there is a substantial amount of variability in the estimates. There appears to be a slight difference in the variability when the overbank contraction scour values are examined by physiographic region (figs. 10B, 10C, and 10D). A larger majority of the Central Lowlands data plot above the equal-value line (105 of 169 matches, fig. 10B), and the estimates are clustered more closely together around the equal-value line, which implies a better estimate of the Level 2 depths. A slight majority also plot above the equal-value line for the Ozark Plateaus data (55 of 100 matches, fig. 10C), but the data exhibit more variability than the Central Lowlands data. The single Mississippi Alluvial Plain data point plots above the equal-value line (fig. 10D). A best-fit trend line from the origin through the data in the Central Lowlands and the Ozark Plateaus regions falls below the equal-value line (figs. 10B and 10C), as does the best-fit trend of all of the data (fig. 10A), which implies that the magnitude of the Level 1+ underestimations is larger than the magnitude of the overestimations.

Discussion of Contraction Scour Depth Estimate Comparisons

As stated earlier, the Level 1+ method was developed to provide conservatively high estimates of scour depth relative to more-detailed procedures like the Level 2 method. However, discrepancies between scour depths determined by the Level 1+ method and scour depths determined by the Level 2 method for a number of sites in Missouri lead to the conclusion that the Level 1+ method may have underestimated contraction scour in some instances. Likely factors that account for many of the discrepancies in estimating scour depth by the two methods are, therefore, offered and discussed further here.

First, the Level 1+ method uses surrogate variables for flow depth and width based on simplified hydraulic procedures. These variables are then used to derive other hydraulic variables, and all variables are then used in the scour depth calculations. Application of these methods to the complex bridge openings and channels typically encountered in the Missouri study resulted in flow depths and widths that differed somewhat from those determined by the Level 2 method, and these differences account for some of the discrepancy in calculated contraction scour depths. Flow depths and widths determined by the Level 2 method, however, are equally subject to interpretation and can vary based on an individual's judgment and the data collected in the field, thus scour depths produced by a particular method can vary.



Figure 9. Comparison of average main channel contraction scour depths determined by rapid estimation assessment (Level 1+) with those determined by detailed hydraulic assessment (Level 2).



--- BEST-FIT LINEAR REGRESSION FORCED THROUGH ORIGIN FOR MATCHED DISCHARGE CLEAR-WATER CONTRACTION SCOUR

Figure 10. Comparison of average overbank contraction scour depths determined by rapid estimation assessment (Level 1+) with those determined by detailed hydraulic assessment (Level 2).

Secondly, in addition to being sensitive to flow-depth and width variables, calculated contraction scour depths also are sensitive to certain equation variables that are common to both methods. For live-bed scour calculations, the important common variable is the Manning's roughness coefficient and for clear-water scour calculations, the important common variable is the median particle size, or D_{50} . Because these equation-sensitive variables are common to both methods and should ideally be determined based on similar levels of effort, critical comparisons between the two methods should, strictly speaking, be based on use of identical values of roughness coefficient and median particle size. For many bridge sites in this study, values of roughness coefficient used in the Level 1+ method did not equal values used in the Level 2 method, mainly because many Level 2 analyses were conducted by a different individual than conducted the corresponding Level 1+ assessment. Furthermore, the median particle size often was visually estimated in the Level 1+ assessment, whereas more specific values based upon lab analysis or more thorough evaluation were used in the Level 2 assessments. Differences in calculated scour depth between methods are thus an outcome to be expected for sites where variables common to both methods differed in the values used between methods.

In an attempt to address these considerations, a subset of the bridges for which the Level 1+ underestimated the Level 2 contraction scour depths was randomly chosen, and the Level 1+ contraction scour depths were recomputed using values for width, Manning's roughness, and median particle size from the Level 2 assessment (flow depths were not altered, because depths in the Level 1+ were interdependent and based on the location of the "average bed" and not a true bed elevation). Contraction scour depths for 24 bridges in the Central Lowlands and 13 bridges in the Ozark Plateaus were recomputed using the values from the Level 2 assessments. The 24 bridges in the Central Lowlands represented 39 of 118 underestimated values of livebed main channel contraction scour and 47 of 64 underestimated values of overbank contraction scour, and the 13 bridges in the Ozark Plateaus represented 19 of 35 underestimated values of live-bed main channel contraction scour and 27 of 45 underestimated values of overbank contraction scour. A substantial decrease in the number of underestimations was observed when the values of width, roughness, and median particle size from the Level 2 assessments were used in the Level 1+ equations. Proportionally extending the results from the random subset to the entire set of underestimations in the Central Lowlands, a substantial majority of the main channel contraction scour results (approximately 163 out of 222 live bed) and the overbank contraction scour results (approximately 152 out of 169 matches) presumably would overestimate the Level 2 results. Similarly in the Ozark Plateaus, a substantial majority of the main channel contraction scour results (approximately 51 out of 63 live-bed matches) and the overbank contraction scour results (approximately 87 out of 100 matches) presumably would overestimate the Level 2 results. This would imply that approximately 18.4 percent of the Level 2 contraction scour depths would be underestimated by Level the 1+ method. By

way of comparison, Holnbeck and Parrett (1997) experienced approximately 15.2 percent underestimation of Level 2 contraction scour depths in their study.

A majority of the remaining underestimates of contraction scour likely can be attributed to the location of the average bed in the Level 1+, or a difference in the depth of flow in the channel used in the Level 1+ compared to the Level 2. As described in the Determination of Average Bed Location section earlier in this report, the average bed of the bridge was chosen so that the average flow area would approximate a rectangle in the Level 1+ assessments. Therefore, the "average bed" actually was the average bed of the whole bridge opening-including any setback areas of the bridge-and not the average bed of the channel alone. The water-surface elevation at the bridge was taken as the "average bed" plus the average depth computed by equation 5. In both the Level 1+ and the Level 2 assessments, the depths of flow on the setbacks and on the approach flood plains were measured relative to the water-surface elevation. However, the depth of flow in the channel was taken as the average depth in the Level 1+ assessments, whereas in the Level 2 assessments, the depth of flow in the channel was determined utilizing the average ground elevation of the channel bed relative to the water-surface elevation. For many bridges in Missouri, this difference in the channel depth of flow did not create a substantial difference in the contraction scour, because the setback areas were small relative to the overall bridge opening, and therefore, the average depth nearly equalled the channel depth of flow used in the Level 2 assessment. However, bridges with substantial setback areas often had a depth of flow in the channel in the Level 2 assessment that was much greater than the average depth used in the Level 1+, which would lead to substantially different results for main channel contraction scour. Furthermore, if the average bed was located incorrectly at a bridge with substantial setback areas, the resultant water-surface elevation used to determine flow depths on the setbacks and flood plains in the Level 1+ also would be incorrect, which subsequently could lead to incorrect values of width on the flood plains.

Pier Scour

The pier scour depth computed in the Level 1+ assessments compared to the Level 2 for all bridges with matching discharges is shown in figure 11A. In general, the points form a cluster around the equal-value line, which implies that the Level 1+ method provides a reasonable close estimate of the Level 2 pier scour depth in Missouri (but typically not a conservative estimate). However, computations were made for piers with exposed footings or complex foundations exposed to flow in the Level 2 assessments, but not in the Level 1+ assessments. When the results for piers without exposed foundations are considered alone (fig. 11A), the Level 1+ method generally provides a conservative estimate of the Level 2 scour depth, as a majority of the values plot above the equal-value line in figure 11A (150 of 248 matches). There are no substantial differences between





- ADJUSTED MATCHED DISCHARGE PIER SCOUR WITH COMPLEX FOUNDATION EXPOSURE, USING FOOTING WIDTH AS PIER WIDTH
- MATCHED DISCHARGE PIER SCOUR WITHOUT COMPLEX FOUNDATION EXPOSURE
- MATCHED DISCHARGE PIER SCOUR WITH COMPLEX FOUNDATION EXPOSURE
- ALL DISCHARGE PIER SCOUR WITH AND WITHOUT COMPLEX FOUNDATION EXPOSURE
- EQUAL-VALUE LINE
- BEST-FIT LINEAR REGRESSION FORCED THROUGH ORIGIN FOR MATCHED DISCHARGE PIER SCOUR (WITHOUT COMPLEX FOUNDATION EXPOSURE AND ADJUSTED)
- LEVEL 1+ VALUES WITHIN 2.5 FEET OF LEVEL 2 VALUE

Figure 11. Comparison of average pier scour depths determined by rapid estimation assessment (Level 1+) with those determined by detailed hydraulic assessment (Level 2).

regions when the pier scour data are examined by physiographic region (figs. 11B, 11C, and 11D).

However, if the piers with footing exposure are reassessed using the *footing* width instead of the pier width in the Level 1+ pier scour estimation equation, a large majority of the piers with footing exposure plot above the equal-value line (131 of 148 matches with footing exposure). In the field, however, it is difficult to predict if footing exposure will occur without a copy of the bridge plans. For this reason, it would be helpful to have some sort of indication of the depth of the pier footings in future assessments, and use the footing width as the pier width in the Level 1+ assessment if the potential for footing exposure exists.

A majority of the Level 1+ values that plot below the equal-value line likely arise from the difference between the velocities used in the Level 1+ and Level 2 assessments. The Level 1+ uses a Froude number based on the average velocity calculated by equation 1 or 2, whereas the Level 2 uses a Froude number based on the maximum velocity in the main channel and the maximum overbank velocity on the overbanks. A substantial difference existed between the average velocity used in the Level 1+ assessment and the maximum channel velocity for most of the bridges that were underestimated by the Level 1+ assessment. This difference was most noticeable at bridges that had substantial setbacks, which would reduce the average velocity compared to a bridge with smaller setbacks. The other variable in the Level 1+ pier scour estimation equation is the flow angle of attack, or skew, and differences in this variable between the Level 1+ and Level 2 assessments accounts for several underestimations, as well as a few substantial overestimations. However, 99 of 114 Level 1+ underestimations (for piers with and without footing exposure) are less than 2.5 feet different from the Level 2 pier scour depth, and 112 of 114 are less than 5 feet different. Holnbeck and Parrett (1997) also observed occasional underestimations, with 7 out of 24 Level 2 pier scour depths (approximately 29.2 percent) being underestimated by the Level 1+. A best-fit trend line from the origin through the pier data (with and without footing exposure) falls above the equal-value line in each region (figs. 11A, 11B, 11C, and 11D).

The distinct horizontal linear bands of values in the Level 1+ pier scour depths for piers without footing exposure in figure 11 are a function of the simplification of the Level 1+ pier scour estimation equation (eq. 29) as compared to the full pier scour equation used in Level 2 assessments (eq. 34). The principal variables in the Level 1+ equation are pier width, alignment to flow, and average Froude number. MoDOT typically uses standardized pier shapes throughout the state, and regularly aligns the piers to flow; furthermore, the average Froude number is determined by the average depth and average velocity, which are related to each other by equations 1, 2, and 3. This results in a narrow band of possible scour depths for a given pier geometry for piers aligned with the flow. The lowest distinct band (with Level 1+ pier scour depths approximately 3.2 feet) corresponds to bents with 10-inch wide steel H-piles. The second band (with Level 1+ depth approximately 3.9 feet) corresponds to bents with 1-foot diameter timber piles. The next distinct band (with Level 1+ values approximately 7.7 feet) corresponds

to piers or bents with 2.5-feet wide columns. Similar bands are seen in the results modified for footing exposure, and correspond to standard footing widths of 6 feet and 9 feet. Level 1+ pier scour depth values between these distinct linear bands typically are those with some other column or footing geometry, or with a skewed flow angle of attack (measured in 5 degree increments, which creates other less distinct bands on the plot). In contrast, the full pier scour equation used in Level 2 assessments (eq. 34) is affected by several variables, and yields a wider range of pier scour depths for a given pier geometry.

Abutment Scour

The abutment scour depth computed in the Level 1+ assessments compared to the Level 2 for all bridges with matching discharges is shown in figure 12. The Level 1+ method generally provides a conservative estimate of the Level 2 scour depth in Missouri, as a majority of the values (502 of 593 matches) plot above the equal-value line in figure 12A. However, like the contraction scour comparison, the Level 1+ method does not provide a close estimate, as there is a substantial amount of scatter around the equal-value line.

Also like the overbank contraction scour comparison, the Central Lowlands data cluster together above the equal-value line (fig. 12B), which implies a conservative estimate of the Level 2 depths. There is more variability in the Ozark Plateaus data (fig. 12C). Only the Mississippi Alluvial Plain data plot entirely above the equal-value line (fig. 12D), but the amount of available data is sparse. A best-fit trend line from the origin through the data in each region falls above the equal-value line (figs. 12A, 12B, 12C, and 12D).

For 63 of the 91 abutment scour depths underestimated by the Level 1+, the difference between the Level 1+ and the Level 2 abutment scour depth was less than 5 feet. Holnbeck and Parrett (1997) also observed occasional underestimations, with 11 out of 66 Level 2 abutment scour depths (approximately 16.6 percent) being underestimated by the Level 1+. However, for Missouri, 6 of the Level 1+ depth underestimates were 10 feet or more different from the Level 2 abutment scour depth.

The only variables in the Level 1+ abutment scour estimation equation (eq. 30) are the depth of flow on the approach flood plain and the abutment type (vertical, vertical with wing walls, or spill through). Therefore, for a given abutment type, the abutment scour depth is dependent entirely upon the depth of flow on the approach flood plain. If this depth is underestimated, the abutment scour also would be underestimated. As was noted in the contraction scour depth estimate comparison discussion, an incorrectly located average bed at the bridge could lead to an incorrect flood-plain flow depth, and it was observed that 49 of the 91 underestimations of abutment scour depth also had an underestimation of main channel contraction scour. The larger underestimations of abutment scour are almost entirely the result of an underestimation of the depth of flow on the flood plain in the Level 1+ assessment.



Figure 12. Comparison of average abutment scour depths determined by rapid estimation assessment (Level 1+) with those determined by detailed hydraulic assessment (Level 2).

Validation of Level 1+ Procedures Using Level 2 Results

The two region-specific variables described in the Rapid Estimation Method (Level 1+) section are the average main channel velocity at the bridge, V_2 , and the difference in water-surface elevation from the approach section to the downstream face of the bridge, Δh , and are defined by equations 1 through 3. Holnbeck and Parrett (1997) developed these relations for Montana using Level 2 data from Montana and Colorado. However, Level 2 data for all three physiographic regions of Missouri were not available before the Level 1+, so gage information was used to develop the relation for V_2 , and Level 2 data for the Central Lowlands region was used to develop the relation for Δh throughout Missouri. Now that Level 2 data exists for all three regions, a validation of the data used to develop the relations can be made.

Average Main Channel Velocity

The average main channel velocity at the bridge, V_2 , was determined from a relation with the unit discharge through the bridge, q_2 . Unit discharge and average channel velocity data from the original 35 sites in the Central Lowlands are shown with the gage data used to develop equation 1 in figure 13A. Similarly, unit discharge and velocity data for bridges in the Ozark Plateaus are shown with the gage data used to develop equation 2 in figure 13B. The lines represented by equations 1 and 2 also are shown. It appears that equations 1 and 2 are both somewhat low compared to the Level 2 data for these regions, and that a more appropriate best fit might be represented by the dashed lines in figure 13A and 13B.

Using the dashed lines instead of equations 1 and 2 would result in a higher average velocity in the channel for a given unit discharge. Based on the discussion in the comparison of pier scour results earlier in this report, an increase in velocity would yield higher estimations of pier scour depth for a given pier geometry and skew. On the other hand, equation 7 shows that an increase in velocity would result in a corresponding decrease in flow depth in the main channel. Based on the discussion in the comparison of abutment scour results earlier in this report, a decrease in depth would yield lower estimations of abutment scour depth for a given abutment type. Presumably, a similar decrease in main channel and overbank contraction scour depths also would occur. However, because equations 1 and 2 are logarithmic, the depth decreases exponentially with increases in velocity. For this reason, the abutment scour depth estimates (and presumably the main channel and overbank contraction scour depth estimates) are more sensitive to a decrease in depth than the pier scour depth estimates are to the corresponding increase in velocity. Therefore, the relations defined by equations 1 and 2 yield more conservative results than would relations defined by the Level 2 data.

Difference in Water-Surface Elevation

The difference in water-surface elevation from the approach section to the downstream face of the bridge, Δh , was determined from a relation with the square of the average main channel velocity, V_2 . The Level 2 data for the original 35 sites in the Central Lowlands used to develop equation 3 are shown in figure 14, as is the line represented by equation 3. Only the Central Lowlands data were available before the Level 1+, and it was assumed that this relation could be applied throughout Missouri. Upon completion of the Level 2's, similar data for 72 bridges in the Ozark Plateaus were obtained (fig. 14). A best-fit regression equation for the Ozark Plateaus data yields the following relation:

$$\Delta h_{\rm OP} = 0.030 \ V_{\rm 2OP}^2 + 0.05 \tag{37}$$

where

 $\Delta h_{\rm OP}$ is the change in water-surface elevation from the approach section to the downstream face of the bridge in the Ozark Plateaus, in feet; and

 $V_{2\text{OP}}$ is the average main channel velocity for bridges in the Ozark Plateaus, in feet per second (ft/s).

In comparison, equation 3 (using only Central Lowlands data) is:

$$\Delta h = 0.030 V_2^2 + 0.03$$

which means for a given V_2 , equation 37 and equation 3 yield a Δh that differs by only 0.02 feet. Furthermore, when Level 2 data for 22 bridges in the Mississippi Alluvial Plain region are plotted, the trend of the data also is along the line defined by equation 3 (fig. 14). Therefore, using equation 3 throughout Missouri was a valid assumption.

Determination of Road Overflow

In the Level 1+ assessments, when the impendent road overtopping discharge, Q_{imp} , was less than the 100-year peak discharge, Q_{100} , the flow through the bridge opening, Q_{2a} , had to be adjusted for the amount of flow over the road. However, at the time of the Level 1+ assessment, there were insufficient data to quantitatively determine the amount of flow that would pass through the bridge opening when flow overtopped an adjacent low road embankment. When Q_{imp} was less than Q_{100} , it was assumed that one-half of the difference between Q_{100} and Q_{imp} would pass over the road, and the flow through the bridge opening was assumed to be

$$Q_{2a} = Q_{100} - \frac{1}{2}(Q_{100} - Q_{imp})$$

as defined earlier in equation 11, or

$$Q_{2a} = Q_{\rm imp} + \frac{1}{2}(Q_{100} - Q_{\rm imp}).$$
 (38)



EXPLANATION

- GAGE DATA
- DETAILED HYDRAULIC ASSESSMENT (LEVEL 2) DATA
- BEST-FIT LINEAR REGRESSION LINE FOR CENTRAL LOWLANDS GAGE DATA
- BEST-FIT LINEAR REGRESSION LINE FOR OZARK PLATEAUS GAGE DATA
- -- POSSIBLE BEST-FIT LINE FOR LEVEL 2 DATA

Figure 13. Relation between unit discharge $({\bf q}_2)$ and average main channel velocity at bridge (V_2) for selected sites in Missouri.

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- BEST-FIT LINEAR REGRESSION FOR CENTRAL LOWLANDS DATA
- BEST-FIT LINEAR REGRESSION FOR OZARK PLATEAUS DATA

Figure 14. Relation between average main channel velocity at bridge squared (V_2^2) and difference in water-surface elevation between bridge and approach sections (Δh) for selected sites in Missouri.

It was presumed that equation 11 or equation 38 overestimated the part of the total flow that would pass through the bridge opening by underestimating the flow over the road embankment, particularly for bridges with wide flood plains and low road embankments, but no quantitative data existed to prove otherwise.

Using the data from the Level 2 assessments, the flow through the bridge opening as estimated by equation 38 was compared to the flow through the bridge opening as determined by the hydraulic analysis using WSPRO in the Level 2 assessments for sites where Q_{imp} was less than Q_{100} . Generally, equation 38 resulted in a flow through the bridge opening, Q_{2a} , that was within a range of 85 percent to 150 percent of the value determined by WSPRO analysis, except for sites with very low road embankments and wide flood plains (fig. 15). The range was considered acceptable for most bridges with road overflow because it overestimates flow through the bridge opening as compared to the WSPRO analyses, resulting in reasonable, conservative scour estimates.

However, for sites with very low road embankments and wide flood plains, where Q_{imp} is less than approximately 25 percent of Q_{100} , equation 38 resulted in a Q_{2a} that was substantially greater than (as much as 470 percent) the value determined by WSPRO analysis, which would result in unreasonable overestimations of scour depth. For these sites, it was determined that limiting the flow through the bridge to 150 percent of the impendent discharge, or

$$Q_{2a} = 1.5 Q_{\rm imp}$$
 (39)

brought the value of Q_{2a} back into the range of 85 percent to 150 percent of the value determined by WSPRO analysis (fig. 15). In future work with the Level 1+ method at sites where $Q_{imp} < 0.25 Q_{100}$, therefore, the use of equation 39 is encouraged to determine Q_{2a} to maintain a reasonable estimate of flow through the bridge opening as scour depth.

General Limitations of Scour Depth Estimation Methods

Estimates of scour depths using either the rapid estimation method (Level 1+) or the detailed hydraulic assessment (Level 2) are designed to be conservative in terms of risk. In the latest edition of HEC-18, Richardson and Davis (2001) state:

"The equations and methodologies presented in [the HEC-18] manual, which predict the maximum scour depth in non-cohesive soil, may, in some circumstances be too conservative. The pier scour equation represents an envelope curve of the deepest scour observed during the various laboratory studies and field data."

The conservative approach usually is reasonable and cost-effective from the standpoint of bridge design. When evaluating existing structures, however, the conservative approach could indicate that a structure will be undermined by a peak discharge that historically had been observed at a bridge without the bridge failing. Factors determining the vulnerability of an existing structure are complex, but several considerations that apply to both scour depth estimation methods, as applied in Missouri, are discussed below.

Verification of Scour Depth Estimates Using Field Data

The equations and methodologies provided in HEC-18 generally were formulated using laboratory research with noncohesive soils with limited verification using field data. In the latest edition of HEC-18, Richardson and Davis (2001) state:

"The current equations and methods for estimating scour at bridges are based primarily on laboratory research. Very little field data have been collected to verify the applicability and accuracy of the various design procedures for the range of soil conditions, stream flow conditions, and bridge designs encountered throughout the United States. In particular, DOTs are encouraged to initiate studies for the purpose of obtaining field measurements of scour and related hydraulic conditions at bridges for evaluating, verifying, and improving existing scour prediction methods."

This limitation of the equations and methods has always been recognized, as a similar statement is found in each edition of HEC-18 (Richardson and others, 1991, 1993; Richardson and Davis, 1995). Given that the HEC-18 equations and methods represent an "envelope curve approach," it is reasonable to assume that the scour depths estimated by the Level 1+ and Level 2 assessments for sites in Missouri may overestimate the actual scour that would occur.

For pier scour, HEC-18 uses a modified form of an equation developed at Colorado State University (CSU) because it was good for design as it rarely under-predicted measured scour depth when compared to measured scour depths (Richardson and Davis, 2001). However, it is acknowledged that the HEC-18 pier scour equation also frequently over-predicts observed scour. In a study of scour in Missouri by Becker (1994), pier scour depths observed at several sites throughout Missouri were compared to depths computed using seven pier scour estimation equations. Becker's study included the original CSU equation with a 10-percent increase, which makes it equivalent to the HEC-18 pier scour equation used in the Level 2 assessments in Missouri (eq. 34) with coefficients K_{p3} and K_{p4} set to 1.1 and 1.0, respectively. Becker indicates that the modified CSU equation overestimated the observed scour depth for all the sites examined in Missouri (fig. 16). An approximate best-fit trend line through the data indicates that as the estimated scour depth increases, the magnitude of the overestimation increases.

The scour depths estimated by the Level 1+ and Level 2 assessments at sites in Missouri for this report generally were not verified with field measurements, and Becker's (1994) study predominantly includes floods with recurrence intervals less than 5 years, which are substantially less than those used in the Level 2 assessments. However, three of the sites in Becker's study subsequently received a Level 2 assessment and had scour depth measurements obtained during the 1993 floods that had recurrence intervals estimated to be between 50 and 500 years (Parrett and others, 1993), which is comparable to those used in the Level 2 assessments. A comparison of the pier scour depths measured and estimated by Becker are shown in table 7.

The contraction scour and abutment scour equations in HEC-18 primarily were developed through laboratory experimentation, also with limited field verification (Richardson and Davis, 2001). The only exception is the HIRE abutment scour equation (eq. 36), which was based on field measurements of scour at the end of spurs in the Mississippi River obtained by the U.S. Army Corps of Engineers, a field situation that closely resembles the laboratory experiments for abutment scour. Nevertheless, both abutment scour equations are acknowledged to have a tendency to overpredict the magnitude of scour that may develop. Only the contraction scour equations specifically are not acknowledged to be conservative in HEC-18. As was stated earlier in the Comparison of Level 2 Results to Level 1+ Results section, the Level 1+ method was designed such that estimates of scour depths should nearly equal or exceed the values computed by the Level 2 method to provide a conservative estimate. Therefore, if the Level 2 results tend to overestimate observed scour depths, the Level 1+ also will tend to overestimate observed scour depths.



Figure 15. Comparison of 100-year peak discharge through bridge estimated using rapid estimation assessment (Level 1+) equation with that determined in detailed hydraulic assessment (Level 2) for bridges with impendent road-overtopping discharge less than 100-year peak discharge.



Figure 16. Relation between scour depth estimated using the Colorado State University equation and the residual (measured scour minus the estimated scour depth) at selected sites in Missouri (modified from Becker, 1994).

Scour Depth as a Function of Flood Duration in Cohesive Soils

Scour depths estimated by the rapid estimation method (Level 1+) and the detailed hydraulic assessment (Level 2) shown in tables 3 and 5 are the maximum scour depths for a non-cohesive soil. The maximum scour depth often is assumed to occur at a given bridge during a single flood event. However, this may not be the case for cohesive soils (silts and clays) such as are found in various locations throughout Missouri. In the latest edition of HEC-18, Richardson and Davis (2001) state:

"The maximum depth of local scour at piers in cohesive soils is the same as in non-cohesive soils. *Time is the difference*. Maximum scour depth is reached in hours or one runoff event in non-cohesive sand, but may take days and many runoff events in cohesive clays. Local pier scour in cohesive clays may be 1,000 times slower than non-cohesive sand. In addition, by inference, contraction scour and local scour at abutments in cohesive soils do not reach maximum depth as rapidly, but the ultimate scour depth will be the same as for non-cohesive soil." (emphasis added)

"Because cohesive soils can scour much slower than non-cohesive soils, it is reasonable to include the scour rate in the calculations. Indeed, while one flood may be sufficient to create the maximum scour depth in cohesionless soils, the scour depth after many years of flood history at a bridge in an erosion resistant cohesive soil may only be a fraction of [the maximum scour depth]." **Table 7.** Summary of discharge data, measured scour depths, and scour depths estimated using various methods at selected sites

 in Missouri (modified from Becker, 1994).

MoDOT structure number	USGS station number	Date of measurement	Time	Scour measurement discharge (ft ³ /s)	Flood peak discharge (ft ³ /s)	Recurrence interval range (years)	Measured scour depth (feet)	Estimated scour depth (feet) ¹				
Chariton River on State Route 6 in Adair County												
L 534	06904500	07/13/1993	0030		20,900	$10-50^2$						
			1445	17,400			1.9	7.0				
L 344	06905500	Chari 07/08/1993 07/13/1993	ton River 0915 1945	on State Route 1 27,200 	29 in Charitor 31,300	• County 	4.7	8.2				
Gasconade River on State Route 89 in Osage County												
A 1411	06934000	09/28/1993	1000		106,000	54						
			1328	101,000			4.1	11.1				

[MoDOT, Missouri Department of Transportation; USGS, U.S. Geological Survey; ft³/s, cubic feet per second; --, no data; >, greater than]

¹Calculated using modified Colorado State University equation (Becker, 1994), which essentially is the same equation as was used in detailed hydraulic assessment (Level 2). Becker's scour measurement discharge and the associated field parameters were used in the equation to provide a direct comparison with the measured scour depth.

²From Parrett and others (1993).

Little research has been done on the time rate of scour for cohesive soils. Therefore, HEC-18 provides no specific guidance on calculating scour depths in cohesive soils. From the standpoint of bridge design, using the conservative approach is reasonable, as it is assumed that the maximum scour will be reached sometime within the service life of a typical bridge subjected to multiple flood events. However, the time rate of scour can have a profound effect on the stability of an existing structure. Based on the comments in HEC-18, it appears to be possible that a structure in a cohesive soil could experience a single flood event of sufficient magnitude that would cause it to be undermined based on the results of the Level 2 assessment, and yet not be undermined because the flood event was not of sufficient duration to cause the maximum scour depth to be reached.

Assumption of Soil Homogeneity

The contraction scour estimation equations in both the Level 1+ and Level 2 methods are dependent upon the median particle size, D_{50} , of the soils present in the channel or on the overbanks. As described in the General Procedure of Level 1+ Assessments section earlier in this report, the median particle sizes of the main channel and overbank materials in the Level 1+ assessments were estimated by visually identifying the material as cobbles, gravel, sand (coarse, medium, or fine), or silt/clay. In the Level 2 assessments, the median particle sizes of the main channel and overbank materials were determined by laboratory analysis of a representative sample of the material obtained from the channel or overbank. Both assessment methods used a representative sample to account for surface variations in soil composition in the channel or on the overbank. The

difference in the level of effort used to obtain the median particle sizes was observed to lead to a difference in contraction scour results between the two assessment methods.

However, in addition to the level of effort difference between the assessment methods, the representative surface sample did not account for variations in soil composition with depth, such as stratified layers of gravel, sand, silt, clay, or rock. Both assessment methods assume that the surface sample also is representative of a homogeneous soil that does not change with depth, which will allow the maximum scour depth to be reached. This limitation was acknowledged in every Level 2 assessment. Changes in the subsurface strata will result in different scour depths at a given site because of the time rate of scour difference between non-cohesive to cohesive soils as discussed earlier, or because of the presence of scour-resistant rock, both of which might limit scour. Information from soil borings would be helpful to determine the presence of subsurface strata that would be resistant to scour and would, therefore, limit the maximum scour depth.

Accuracy and Consistency of Variables

As stated in the contraction scour depth estimate comparison, discrepancies in the calculated contraction scour depths arose from differences in the flow depths and widths between the Level 1+ and Level 2 methods. Furthermore, calculated contraction scour depths also were sensitive to certain equation variables that are common to both methods, most notably the Manning's roughness coefficient for live-bed contraction scour, and the median particle size for clear-water contraction scour. Therefore, equation-sensitive variables that are common to both methods should ideally be determined by similar methods, and critical comparisons between the two methods should be based on use of similar or identical values of flow depth, flow width, roughness coefficient, and median particle size. Obviously, this logic extends to the calculation of the pier scour and abutment scour depth. Differences in calculated scour depth between methods should be expected for sites where inconsistent values are assigned to variables common to both methods.

Finally, whether a scour analysis is conducted by the Level 1+ or the Level 2 method, unreliable scour-depth estimates can result for either method if important equation variables are determined based on an insufficient level of effort or where variables are not appropriate for the hydraulic conditions being analyzed. Furthermore, interpretations involved in the application of either method can produce results for estimated scour depth that vary within a particular method as well as between methods. Holnbeck and Parrett (1997) acknowledge these limitations in the Level 1+ assessment method, and state that field parameters should be obtained by individuals with "considerable field experience and sound judgment regarding bridge scour, flood hydrology, and hydraulics. Even for highly experienced individuals, the results of field testing of the method...indicated that some experience with the rapid estimation method itself is required to produce reliable and generally reproducible results."

Summary

A potential-scour assessment (Level 1) was used by the U.S. Geological Survey, in cooperation with the Missouri Department of Transportation, at 3,082 bridges in Missouri to identify bridges that might be susceptible to scour. Values assigned to characteristics of the site, such as the presence of existing scour and evidence of past scour, were combined into an observed scour index. Other site characteristics that might exacerbate or mitigate scour were combined into a potentialscour index. These indices were used to rank the 3,082 bridges for their susceptibility to scour. Nearly 1,300 bridges were identified as scour-susceptible, requiring additional analysis.

A rapid estimation method (Level 1+) was used to estimate contraction, pier, and abutment scour depths at 1,396 bridge sites throughout Missouri. The results were used in conjunction with a qualitative assessment recommended by the Missouri Department of Transportation (MoDOT) to determine which sites were potentially scour critical and would require additional analysis.

A detailed hydraulic assessment method (Level 2) was used to compute estimates of contraction, pier, and abutment scour depths at 398 bridges throughout Missouri. MoDOT uses the scour results to determine if the bridge sites are scour critical and will need further monitoring or application of scour countermeasures. Thirty-five Level 2 assessments were completed before implementation of the Level 1+ method. Twenty-nine additional assessments were completed at bridges over major rivers without either a Level 1 or Level 1+ assessment, and 22 assessments were completed at bridges that had been deemed not susceptible to scour but were in proximity and hydraulically connected to a Level 2 bridge.

A Level 2 assessment was performed at 316 bridges that had received a Level 1+ assessment, which provided a broad data base to compare the two scour assessment methods. The scour depths computed by each of the two methods were compared for bridges that had similar discharges through the bridge opening. For Missouri, the Level 1+ assessment was a reasonable conservative estimator of Level 2 pier scour depths if the pier width is used for piers without footing exposure and the footing width is used for piers with footing exposure. The Level 2 abutment scour depths were conservatively estimated by the Level 1+ assessment, but there was substantial variability in the estimates and several substantial underestimates. The Level 1+ assessment did not provide a reasonable conservative estimate of the Level 2 contraction scour depths in Missouri, but the discrepancy primarily was the result of using different values for variables that were common to both of the assessment methods and, therefore, should have been similar or identical in the two methods.

References

- Alexander, T.W., and Wilson, G.L., 1995, Technique for estimating the 2- to 500-year flood discharges on unregulated streams in rural Missouri: U.S. Geological Survey Water-Resources Investigations Report 95–4231, 33 p.
- Arcement, G.J., Jr., and Schneider, V.R., 1989, Guide for selecting Manning's roughness coefficients for natural channels and flood plains: U.S. Geological Survey Water-Supply Paper 2339, 38 p.
- Barnes, H.H., Jr., 1967, Roughness characteristics of natural channels: U.S. Geological Survey Water-Supply Paper 1849, 213 p.
- Becker, L.R., 1994, Investigation of bridge scour at selected sites on Missouri streams: U.S. Geological Survey Water-Resources Investigations Report 94–4200, 40 p.
- Chow, V.T., 1959, Open-channel hydraulics: New York, McGraw-Hill Book Company, 680 p.
- Fenneman, N.M., 1938, Physiography of Eastern United States: New York, McGraw-Hill Book Company, 723 p.
- Hauck, H.S., and Nagel, C.D., 2004, Water Resources Data for Missouri, Water Year 2003, U.S. Geological Survey Water-Data Report MO–03–1, 776 p.
- Holnbeck, S.R., and Parrett, C., 1997, Method for rapid estimation of scour at highway bridges based on limited site data:
 U.S. Geological Survey Water-Resources Investigations Report 96–4310, 79 p.
- Huizinga, R.J., and Rydlund, P.H., Jr., 2001, Simulation of flow and evaluation of bridge scour at Horse Island Chute bridge near Chester, Illinois: U.S. Geological Survey Water-Resources Investigations Report 01–4176, 28 p.

Huizinga, R.J., and Waite, L.A., 1994, Methodology for the assessment of scour at bridge sites in Missouri: U.S. Geological Survey Open-File Report 94–544, 23 p.

Lagasse, P.F., Shall, J.D., Johnson, F., Richardson, E.V., Richardson, J.R., and Chang, F., 1991, Stream stability at highway structures: U.S. Federal Highway Administration Publication FHWA-IP-90-014 Hydraulic Engineering Circular No. 20, 195 p.

Parrett, Charles, Melcher, N.B., and James, R.W., Jr., 1993, Flood discharges in the Upper Mississippi River Basin, 1993, *in* Floods in the Upper Mississippi River Basin, 1993: U.S. Geological Survey Circular 1120–A, 14 p.

Richardson, E.V., and Davis, S.R., 1995, Evaluating scour at bridges (3d ed.): U.S. Federal Highway Administration Publication FHWA-IP-90-017 Hydraulic Engineering Circular No. 18, 204 p.

Richardson, E.V., and Davis, S.R., 2001, Evaluating scour at bridges (4th ed.): U.S. Federal Highway Administration Publication FHWA NHI 01-001 Hydraulic Engineering Circular No. 18, 378 p.

Richardson, E.V., Harrison, L.J., and Davis, S.R., 1991, Evaluating scour at bridges: U.S. Federal Highway Administration Publication FHWA-IP-90-017 Hydraulic Engineering Circular No. 18, 105 p.

Richardson, E.V., Harrison, L.J., Richardson, J.R., and Davis, S.R., 1993, Evaluating scour at bridges (2d ed.): U.S. Federal Highway Administration Publication FHWA-IP-90-017 Hydraulic Engineering Circular No. 18, 132 p.

Shearman, J.O., 1990, User's manual for WSPRO—A computer model for water surface profile computations: U.S. Federal Highway Administration Publication FHWA-IP-89-027 Hydraulic Computer Program HY-7, 177 p.

Shearman, J.O., Kirby, W.H., Schneider, V.R., and Flippo, H.N., 1986, Bridge waterways analysis model-research report: U.S. Federal Highway Administration Research Report FHWA-RD-86-108, 122 p.

U.S. Federal Highway Administration, 1988, Scour at bridges: Washington, D.C., Federal Highway Administration Technical Advisory 5140.20, 4 p.

For additional information contact: U.S. Geological Survey Missouri Water Science Center 1400 Independence Road Mail Stop 100 Rolla, MO 65401 (573) 308-3664 or http://mo.water.usgs.gov



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