Comparison of the HEC-18, Melville and Sheppard Pier Scour Equations

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ABSTRACT

Three frequently cited pier scour equations are the HEC-18 (also known as the CSU equation), Melville and Sheppard equations. Direct comparisons of these three equations were conducted for a wide range of realistic hydraulic, pier size and sediment size conditions. Each equation was applied following the procedure prescribed in the applicable manuals. The range of conditions was intended to cover the vast majority of pier scour calculations that would be encountered during scour evaluations. More than 2500 scour calculations were performed for each equation. This exercise was not meant to determine which equation is "right," "wrong," "better," or "worse." It was meant to give insight into the similarities and differences between the results of the equations and to address the topic of the perceived degree of conservativeness in pier scour calculations.

Each of the equations predicts much greater or less scour than the other two depending on the specific input data. The Melville equation tends to produce the greatest scour and the Sheppard equation tends to produce the least. On average, the Melville equation computes scour over 30 percent more than the Sheppard equation and the HEC-18 equation computes scour approximately 15 percent more than the Sheppard equation. The majority of results were within +/- 30 percent for HEC-18 compared with Sheppard.

One difference between the equations is that Sheppard includes a threshold velocity condition for pier scour, so it can predict zero scour for some conditions. Neither the HEC-18 nor Melville equations include this threshold velocity, so some amount of pier scour is always computed for these equations.

INTRODUCTION

The three pier scour equations that were compared are the HEC-18 (Richardson and Davis 2001), Melville (Melville and Coleman, 2000) and Sheppard (Florida DOT, 2005) equations. The three equations were applied to the data contained in Table 1. The data are intended to represent the majority of conditions for pier scour experienced in practice. All combinations of six flow velocities, eight flow depths, eight pier widths and seven bed material (D50) sizes were used for total of 2688 pier scour calculations for each equation. No result was discarded. Pier shape was considered to be circular for each of the calculations because this shape is the basis for the vast majority of pier scour research and because adjustments for computed scour depth for various shapes tend to be similar between the equations.

The data in Table 1 result in the following range of hydraulic parameters and length scales:

Froude Number 0.03-1.53
Pier width/D50: 15.24-45,720
Pier width/Flow Depth: 0.025-10.0
Vc (ft/s) (HEC-18): 0.36- 2.54 m/s (1.17-8.33 ft/s)
Vc(ft/s) (Melville): 0.32-2.76 m/s (1.06-9.06 ft/s)
Vc(ft/s) (Sheppard): 0.32-2.69 m/s (1.04-8.84 ft/s)
V/Vc (HEC-18) 0.12-12.87
V/Vc (Melville) 0.11-14.09
V/Vc (Sheppard) 0.11-14.49

Note that each method includes a different equation for computing the critical velocity (incipient motion velocity) for particle movement.

Table 1. Data used for Pier Scour Comparisons.

Flow Velocity		Flow Depth		Pier Width		D50
ft/s	m/s	ft	m	ft	m	mm
1	0.3	3	0.9	1	0.3	0.2
2	0.6	6	1.8	2.5	0.8	0.5
5	1.5	9	2.7	5	1.5	1
7	2.1	12	3.7	7.5	2.3	2
11	3.4	16	4.9	10	3.0	5
15	4.6	20	6.1	15	4.6	10
		30	9.1	20	6.1	20
		40	12.2	30	9.1	

Each of the three pier scour equations was applied as recommended in the appropriate scour reference. The HEC-18 equation (also known as the CSU equation) was applied based on the information contained in HEC-18 with the following 5 assumptions. (1) The bed coefficient, K3, was set to 1.1 for all calculations, (2) wide pier adjustment, Kw, was used when applicable, (3) pier scour was limited to 2.4 times the pier width for Froude number <= 0.8, (4) pier scour was limited to 3.0 times the pier width for Froude numbers >0.8, (6) the armoring coefficient, K4, was not applied (therefore set to 1.0) because a uniform bed material size is used and because the equation for this factor has proven to be unreliable.

In general, each of the equations is empirical to the extent they are based on laboratory data. The HEC-18 equation is considered to be the most empirical because it is a regression fit of lab data with several "correction" factors that have been added over time. The Melville and Sheppard equations are similar in concept because they are based on a more complete set of factors from dimensional analysis (velocity/critical velocity, flow depth/pier width, pier width/sediment size). In comparison, the HEC-18 equation does not include critical velocity or sediment size, but does include Froude number, a variable that is difficult to justify.

The following sections compare the scour estimates produced by the three equations. For the range of data included in Table 1, the HEC-18 and Sheppard equations produce similar scour estimates with the HEC0-18 equation tending to produce more scour than Sheppard. The Melville and Sheppard results are the most dissimilar, with Melville consistently producing more scour than Sheppard. Each of the equations can produce more or less scour than the other two, depending on the particular situation.

COMPARISON OF MELVILLE AND HEC-18 EQUATIONS

Figure 1 shows the results of the 2688 scour calculation for the Melville and HEC-18 equations for the data included in Table 1. The three red lines in the figure are a line of perfect agreement (1:1) and lines depicting plus and minus 30 percent. A linear regression line (thin black line) indicates that Melville is, on average, 28 percent greater than HEC-18. Approximately 1400 of the points (52 percent) are within plus or minus 30 percent, leaving a significant number of points that are well outside this range. There are instances when either equation produces over five times the scour of the other equation. The Melville equation tends to predict much greater scour when the scour amounts are large and less scour when the scour amounts are small. For this exercise, the HEC-18 equation never computes less than one foot of scour though the Melville equation does.

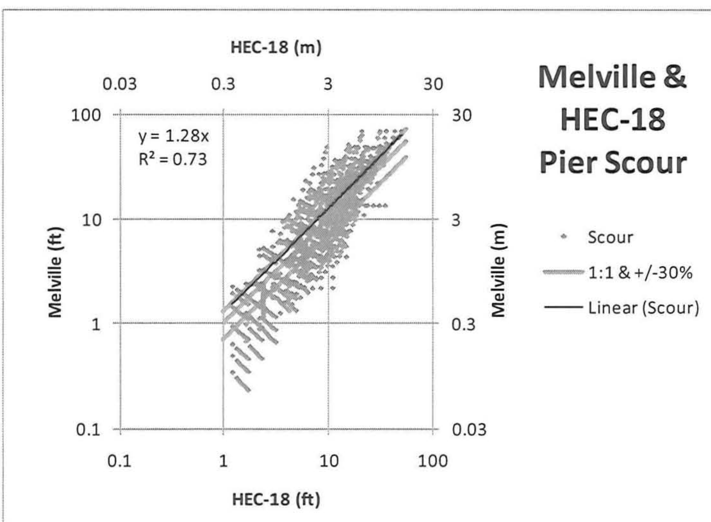

Figure 1. Pier Scour Comparison – Melville and HEC-18.

COMPARISON OF SHEPPARD AND HEC-18 EQUATIONS

As shown in Figure 2, the Sheppard and HEC-18 equations produce much more consistent results. Nearly 1900 of the data points (70%) are within plus or minus 30 percent. On average the Sheppard equation results in 15 percent less scour than the HEC-18 equation. There are two other significant features of this plot. First, there are several instances when the Sheppard equation computes no scour when the HEC-18 equation computes significant scour. This is because the Sheppard equation computes no scour when the velocity is less than 0.47Vc. Note that for purposes of graphical representation, the Sheppard equation is assigned a scour of 0.03 m (0.1 feet) when it computes no scour. The other significant feature is similar. There are "stringers" of data where the Sheppard equation produces scour well below the minus 30 percent line. These data are for velocities very close to 0.47Vc. As noted, the HEC-18 equation does not include a V/Vc factor, nor does it include a "no scour" condition.

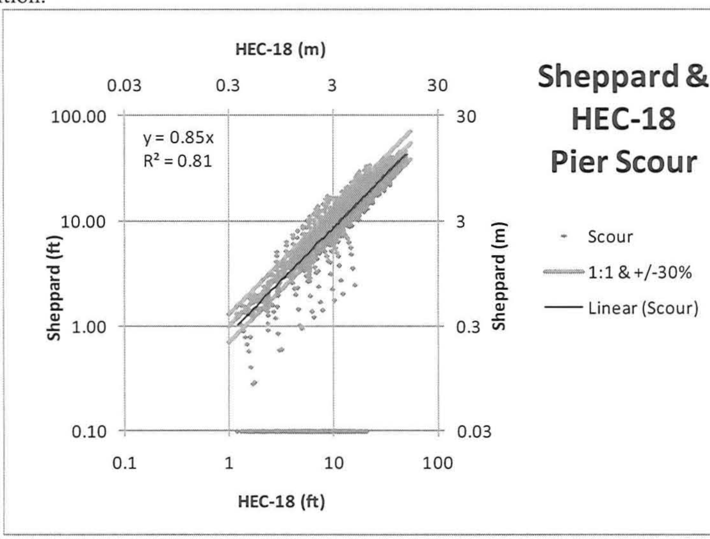

Figure 2. Pier Scour Comparison – Sheppard and HEC-18.

COMPARISON OF MELVILLE AND SHEPPARD EQUATIONS

Considering the results of the previous comparisons it is not surprising that, on average, the Melville equation computes significantly greater scour than the Sheppard equation. As shown in Figure 3, only about 900 of the Melville equations points (37 percent) are within plus or minus 30 percent of the Sheppard equation points, and very few are within the zero to minus 30 percent area. This plot demonstrates that the Melville equation also does not include a zero scour condition. Therefore, like HEC-18, it computes significant scour for conditions when the Sheppard equation results in no scour. Also similar to HEC-18, there are "stringers" of data for conditions close to 0.47Vc when the Melville equations computes significant scour and the Sheppard equation produces much less. This plot also shows that the Sheppard and Melville equations are correlated, at least much more so than the Melville and HEC-18 equations, but the Melville equation consistently produces more scour.

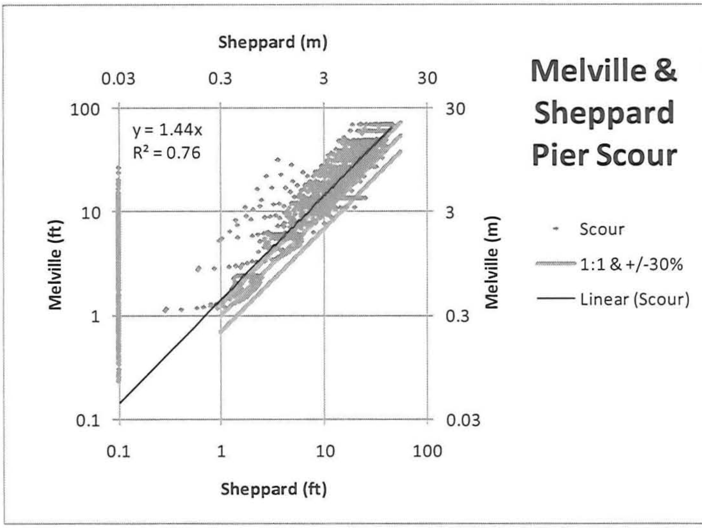

Figure 3. Pier Scour Comparison – Melville and Sheppard.

CRITICAL VELOCITY COMPARIONS

Each of the three approaches includes a method for computing critical velocity (incipient motion velocity). The Melville and Sheppard methods are very similar, with the only differences being the values of coefficients. The Melville equation results in slightly greater values of Vc than the Sheppard equation. The HEC-18 method is much simpler. Each method depends on particle size and flow depth. Figure 4 shows that the HEC-18 approach computes greater values of Vc for smaller particle sizes and smaller values of Vc for larger particles. The computations were performed for particle sizes and flow depths shown in Table 1.

For the initiation of particle movement at a pier (Vcp), the Sheppard approach uses a value of 0.47Vc. For velocities less than Vcp, the Sheppard approach results in no pier scour. The Melville scour manual indicates that Vcp may be as low as 0.3Vc, but does not include this as a lower limit in practice. In the Melville method pier scour is initiated for any non-zero velocity. The HEC-18 manual does include an equation for Vcp, but does not use it as the beginning of pier scour. Rather, Vcp is used as part of the "out of favor" armoring factor, but it is not the Vcp calculation that is in question. One difference between the HEC-18 and the other two equations for Vcp is that the HEC-18 equation includes pier size, so a larger pier starts to scour at lower velocities than smaller piers.

Figure 5, 6, and 7 are repeats of Figures 1, 2, and 3, except that 0.3Vc is used as Vcp (and the pier scour threshold) for the Melville equation and the HEC-18 equation for Vcp is the used as a pier scour threshold. Figure 5, which compares Melville and HEC-18, now looks much more like the Sheppard-Melville comparison (Figure 3).

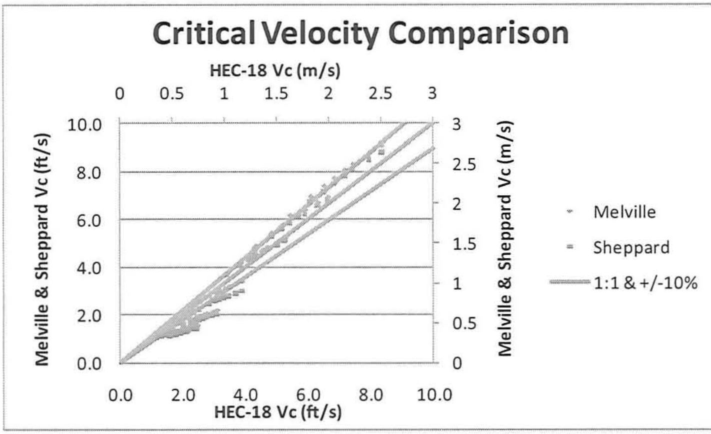

Figure 4. Critical Velocity Comparisons.

This is because many of the small scour results became no scour for both equations (shown as 0.03, 0.03 m) or no scour just for the HEC-18 equation. Figure 6, which compares Sheppard and HEC-18, has not change drastically from Figure 2, expect that the "stringers" are mostly gone. There are conditions when this approach results in either equation computing no scour and the other equation producing significant scour. There is very little change between Figure 3 and Figure 7, the Melville-Sheppard comparisons. This is because some of the points that Sheppard computed no scour also became no scour for the Melville equation.

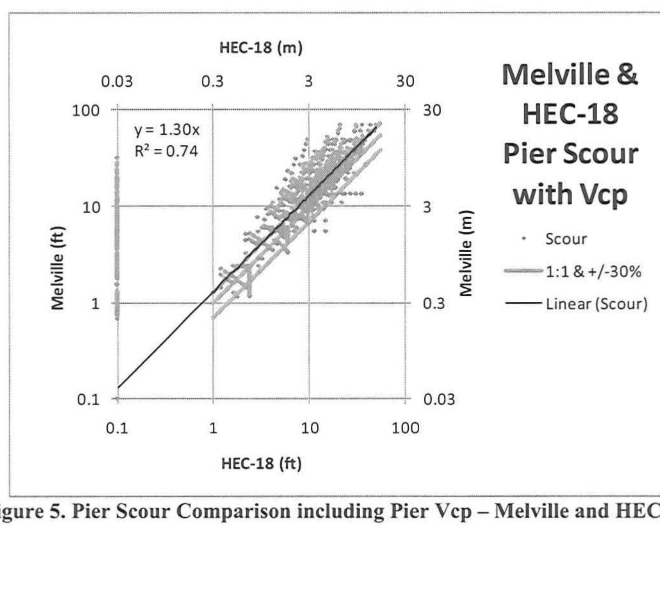

Figure 5. Pier Scour Comparison including Pier Vcp – Melville and HEC-18.

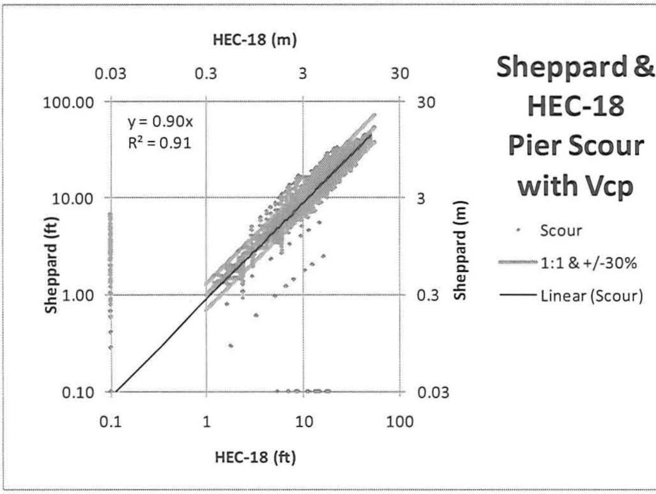

Figure 6. Pier Scour Comparison including Pier Vcp – Sheppard and HEC-18.

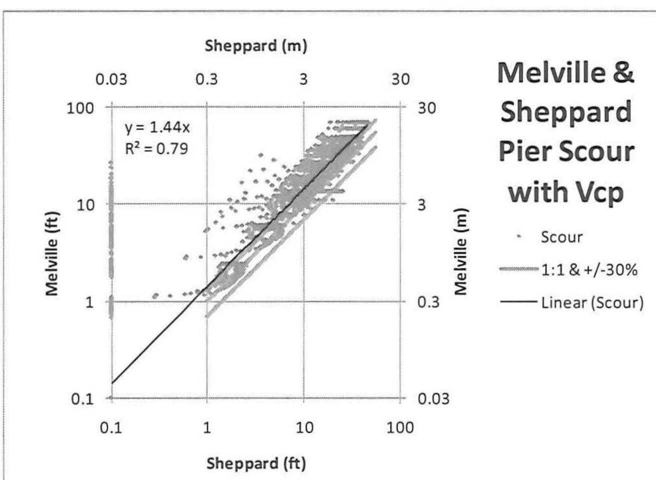

Figure 7. Pier Scour Comparison including Pier Vcp – Melville and Sheppard.

CONCLUSIONS

There is a strong perception that the HEC-18 equation consistently and considerably overestimates pier scour. Although this may be true, the results of the HEC-18 equations are consistently less than the Melville equation and, on average, only 15 percent more than the Sheppard equation. Therefore, if the HEC-18 equation is grossly over-conservative, then each of these equations must include a considerable degree of conservativeness. This is a good thing. If, on average, pier scour amounts were calculated as they would occur, then around have of the calculations would underestimate scour. For design purposes there needs to be a level of conservatism.

This paper does not argue that pier scour estimation should not or could not be improved or that one equation is better than the others. Many engineers believe that the Sheppard equation is more accurate and gives more realistic predictions of scour for design purposes. If it is, then there are cases when using the HEC-18 may under-predict scour. None of the scour equations is perfect and all probably over-predict frequently and under-predict occasionally. The biggest differences between the equations appear to be for flow conditions well below incipient motion (below around 0.6Vc). However, this is a relatively rare design condition. It may be appealing to improve, or at least gain some consistency between, the approaches for addressing the "no pier scour" condition, but it will not impact the vast majority of bridge design conditions. For flow velocities approaching and above live-bed conditions, the results are fairly consistent. The biggest difference between the equations appears to be a stronger inclination to use envelope curves by Melville.

This paper only addresses direct comparisons of the equations. Although it is also appealing to compare the equations to field data, the limitations of field data must also be recognized. These limitations include accuracy of the scour measurement, accuracy of the hydraulic variables, whether the scour has reached equilibrium conditions, bed material variability, and material erodibility (rock, clay, fines, etc.).

REFERENCES

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Richardson, E.V. and Davis, S.R. (2001). "Evaluating Scour at Bridges," Fourth Edition, Hydraulic Engineering Circular No. 18, FHWA-NHI-01-001, Washington, D.C.

River Engineering For Highway Encroachments FHWA HDS-6

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ABSTRACT

In the 1960's Frank Johnson, Chief of the Hydraulics Branch of the Federal Highway Administration, and Regional FHWA Hydraulic Engineers were concerned that State, Federal and Consulting Engineers were not cognizant of the interaction between highway crossings and encroachments and the river environment. As a result, FHWA funded the development of a manual and training course in 1975, which was revised in 1990. In 2001 the manual was revised and issued as FHWA HDS-6. HDS-6 has chapters giving the fundamentals of open channel flow; alluvial channel flow; sediment transport; geomorphology and river morphology; river stabilization and bank protection; scour at bridges; data needs and sources; design considerations; and examples of real design and evaluation problems. There are 646 pages, 231 figures, and 425 references in HDS-6. The course has been given 45 times to over 1,100 highway engineers. This paper summarizes the chapters in HDS-6 and the history of the manual and training course.

INTRODUCTION

In 1975 the U.S. Federal Highway Administration published a manual entitled "Highways in the River Enviorment (HIRE) - Environmental and Hydraulic Considerations" (Richardson et al, 1975) and established a one week training course for Federal and State Highway Engineers. The manual and training course resulted from the concerns of Carl Izzard, Les Herr, Frank Johnson and FHWA Hydraulic Engineers, that highway engineers did not have the knowledge and background to properly evaluate the threat to bridges and encroachments from stream instability and scour, to evaluate the environmental consequences that highway bridges and encroachments had on streams and rivers or the knowledge to alleviate or diminish these consequences. To alleviate this problem, Frank Johnson, Chief Hydraulic Engineer, FHWA, selected Colorado State University, with Dr. Everett Richardson as PI, to produce a manual and training course for the National Highway Institute. The selection was based on CSU's experience in river engineering, fluvial geomorphology, sediment transport and scour research and CSU's contribution of Drs. Everett V. Richardson and Daryl B. Simon's time.

The 1975 manual, written by E.V. Richardson, D.B. Simons, S. Karaki, M. A. Stevens, and K. Mahmood from 1973 to 1975, contains eight chapters. 1) Introduction, 2) Open Channel Flow, 3) Fundamentals of Alluvial Channel Flow, 4) Fluvial Geomorphology, 5) River Mechanics, 6) River Stabilization, 7) Needs and Sources of Data, and 8) Hydraulic and Environmental Considerations of Highway River Crossing and Encroachments.

Each author wrote a chapter, which was reviewed, added to and refined by the others. However, Richardson and Simons finalized the manual. FHWA Hydraulic Engineers Frank Johnson, Murray L. Cory, Dah-Cheng Woo, Milo Cress, Lawrence J. Harrison, Herbert Gregory and Gene Fiala and Mainard Wacker (Wyoming Highway Department) and Roger L. Dean (FHWA-NHI) served as a steering committee. They meet with Richardson and Simons quarterly to review, add to and critic the manual and course. Steering Committee members took one of the first two courses along with State Bridge Engineers to finalize the course. Graduate Students Peter F. Lagasse, V. M. Ponce, Larry Runquest and Tony Melone assisted the authors in writing the manual and instructing the first two courses, which were two weeks long, including a field trip to observe grade control structures on Sand Creek in Denver, Colorado.

In 1990 Richardson and Pierre Julian (Richardson et al, 1990) revised the manual adding new material. In 2001 at the request of Philip L. Thompson, Chief Hydraulic Engineer, FHWA, the manual was extensively revised by Lagasse and Richardson (Richardson et al, 2001) and published as FHWA Hydraulic Design Series Number 6 entitled "River Engineering for Highway Encroachments – Highways in the River Environment."

The HIRE training course was given by Drs. E.V. Richardson and D.B. Simons 45 times in the period 1974 to 1998. It was given in 23 States and Puerto Rico and to over 1,100 highway engineers and officials. The course is number FHWA-NHI-135010 in NHI (2010) catalog.

HDS-6 has eleven (11) chapters and three (3) appendixes, There are 646 pages, 231 figures, and 425 references. A glossary defines 277 words or terms that are used by river and highway engineering professionals. The manual uses dual units (SI and English). Example problems are solved and several case studies are given derived from actual river engineering for highway encroachments problems completed by the authors. The Publication No. is FHWA NHI 01-004. It is available from the NATIONAL TECHNICAL INFORMATION SERVICE, Springfield, VA 22161, (703) 487 4650, isddc.dot.gov.

RIVER ENGINEERING FOR HIGHWAY ENCROCHMENTS FHWA HDS-6

Chapter 1. Introduction

The chapter lays the foundation for the manual, by briefly classifying river crossings and encroachments. The dynamics of rivers, the affects of highway construction on river ***systems*** and affects of river development on highway crossings and encroachments are given. A brief review of the contents of each chapter is given.

Chapter 2. Open Channel Flow

The fundamental fluid mechanics of rigid boundary one-dimensional open channel flow are described. Rigid boundary open channel flow has no deformation of the bed or banks, whereas, in mobile boundary hydraulics the bed and banks can and do change (scour and fill). In text book style the three basic equations (conservation of mass, momentum and energy) are derived. Shear stress, velocity distribution, average velocity and the relation between shear stress and velocity for **steady uniform flow** are derived. The Manning and Chezy average velocity equations and the relation between them are given as are Tables for Manning's n.

Unsteady flow conditions are described and equations are derived or given to determine the relations between boundary conditions, velocity, discharge or depth. The unsteady flow conditions are gravity waves (deep or shallow water), surges, roll waves, steady rapidly varying flow (transitions, specific energy and discharge diagrams), super critical flow transitions, flow over drop structures, flow in bends, and gradually varied flow (water surface profiles).

Stream Gaging. The methods for measuring instantaneous, daily and annual flow of rivers and streams are described. Typical gaging stations, stage/time hydrographs and stage/discharge graphs are illustrated. Also, the USGS velocity/sub-area flow measurement method to determine discharge (Q) is described.

Chapter 3. Fundamentals of Alluvial Channel Flow

This chapter covers properties of alluvial material and methods of measuring these properties, sediment grade scale, flow in sand-bed channels, classification and prediction of bed forms, Manning's n for sandbed and other natural streams, how bed form changes affect highways in the river enviorment, beginning of motion, flow in coarse bed streams, critical velocity to move stones, Shields beginning of motion figures and physical measurement of sediment discharge in the field. The equation and figures to determine sediment particle fall velocity and a figure (Figure 1) for angle of repose of non-cohesive material is given. Four example problems are solved in dual units.

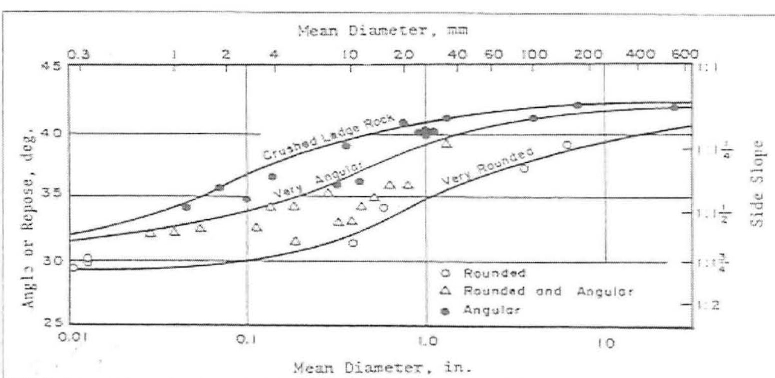

Figure 1. Angle of repose of non-cohesive materials (Richardson et al, 1975).

Based on the research of Simons and Richardson (1963, 1966), (Guy, et al, 1966), Richardson (1965), Richardson et al, (1962), (Richardson and Simons (1967), the **regimes of flow,** associated **bed forms**, sediment transport and resistance to flow (Manning's n) for alluvial sand bed streams are described in detail. These regimes of flow and associated bed forms are given in Figure 2. Resistance to flow in the **lower flow regime** is high (Manning's n 0.020 to 0.04) and bed material transport is low (less than 2,000 ppm). In the **upper flow regime** Manning's n ranges from 0.012 to 0.020 and sediment transport is large (greater than 2,000 ppm). Bed form in an alluvial sand channel depends on bed material fall velocity, which depends on particle size and water viscosity; depth of flow; flow velocity and slope of the energy grade line. With changes in depth (discharge), water viscosity (water temperature), or both the bed form can go from the lower flow regime to the upper flow regime or the reverse. This can result in a discontinuous stage/discharge relations (Beckman and Furness (1962), (Nordin, 1964), (Dawdy, 1961); changes in Manning's n and depth in navigation channels (U.S. Army Corps of Engineers (1968) and Manning's n and depth in major rivers (Figure 3).

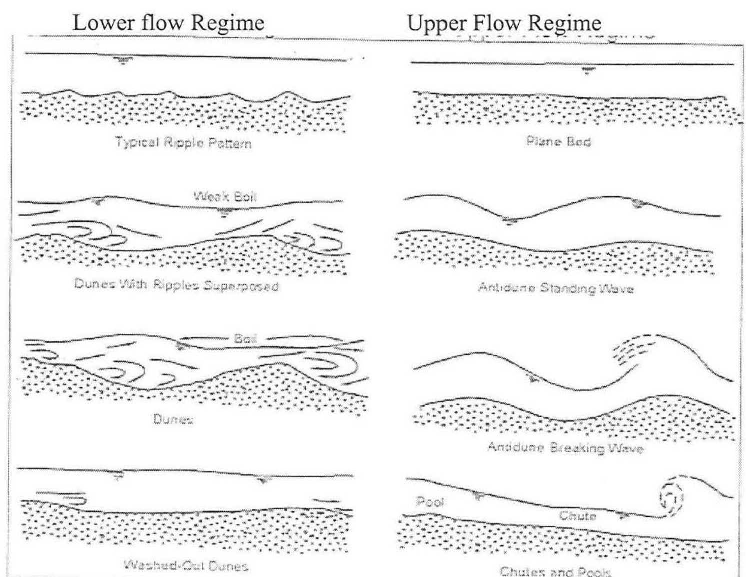

Figure 2. Bed forms in sand channels (Simons and Richardson 1963, 1966).

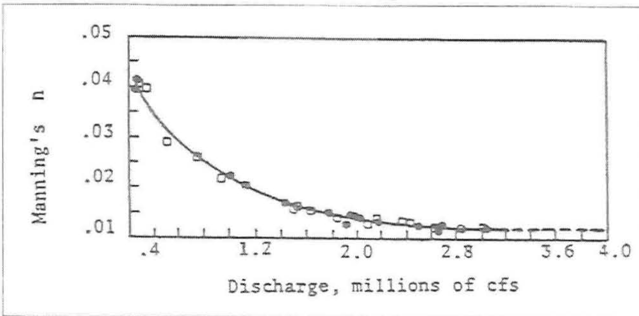

Figure 3. Change in Manning's n with discharge, Padma River in Bangladesh. (Richardson et al, 1975)

Chapter 4. Sediment Transport

Total sediment discharge at a cross section of a stream is the sum of the bed material discharge (commonly called bed load) and the fine sediment discharge (Einstein's (1950) wash load). Bed material discharge (bed load) is the discharge of the sediments that are in the bed and is transported at the capacity of the hydraulic conditions of the stream. Fine sediment discharge (wash load) is the sediment discharge of sediments not found in appreciable quantities in the bed. It comes from the watershed, washes through the section, and is usually not transported at the capacity of the stream. In the literature, using the term load to describe sediment discharge evolved because sediment discharge is expressed as tons per unit of time (tons per day for example).

HDS-6 defines total sediment transport by **type of movement** (contact load and suspended load); **by method of measurement** (measure load and unmeasured load): and by **source of the sediment** (wash load and bed material load). We prefer the use of sediment discharge for load but the literature is ingrained with load.

Chapter four defines the various terms used to describe and measure total sediment discharge. It defines suspended sediment discharge and develops Rouse's classic suspended sediment distribution equation. The Meyer-Peter and Muller (1948), Einstein's (1950), and Yang's (1996) equation and methods to compute bed material discharge (bed load) are included. Colby's (1964) method (using a figure giving the relation between sand bed discharge and mean velocity) for estimating bed material discharge and power functions for calculating bed material discharge are also given. Example problems are worked to illustrate the calculations.

Chapter 5. River Morphology and River Response

Streams (rivers, creeks, etc.) may be meandering, braided, or relatively straight. They may have flood plains. They may flow in mountains, on piedmonts, be tidal, on deltas or alluvial fans. They may have nickpoints or have headcutting. Their flow may be ephemeral or perennial. At a given location, streams can change significantly with time. For example, a stream on an alluvial fan may be entrenched, fairly straight and stable but in time may shift to a multiple channel, braided and unstable stream. All these morphology factors affect a highway crossing or encroachment. Also, a highway crossing or encroachment may affect or change the morphological factors.

HDS-6 describes river morphology for highway engineers so they can anticipate how a highway crossing or encroachment affects the river and how rivers affect his crossing or encroachment. Chapter 5 is based on the authors' field experience and knowledge of the literature on fluvial geomorphology. It describes how rivers affect highways and highways affect rivers. Papers by Davis (1899), Lane (1957), Leopold et al (1953, 60, 64), Schumm (1968, 72, 77, 81) Schumm et al, (1976, 98), Thorn (1997) and others are used and many figures and tables are included. For example, Brice and Blodgett's (1978) figure classifying rivers as to size, bed material, sinuosity and ten other characteristics and the Culberson et al (1967) classification are given together with descriptions of how these classification affect river crossings or encroachments. The Shen et al (1981) classification figure relating stream stability to river form and sediment transport is given and discussed. Meander characteristics of bends, crossings, point bars, alternate bars, concave and convex banks, radius of curvature and width and wave length are discussed together with recommendations of how to measure, alter, stabilize or design meander stream reaches.

Physical and mathematical hydraulic computer models available for the highway engineer are given. The chapter objective is to aid the highway engineer in understanding the role that fluvial geomorphology, river morphology, and river changes and response to change have in highway design, inspection and maintenance. Example problems are given.

Chapter 6. River Stabilization and Bank Protection

Many types and methods of river control and bank stabilization methods have evolved over the years to control the short and long term changes that were described in Chapter 5. Concrete, brick, willow, rock, and asphalt mattresses; sacked concrete; riprap; grouted riprap; sheet and timber piles; steel jack and brush jetties; angled and sloped rock-filled, earth-filled, riprapped and timber dikes or spurs; and concrete armor units are used for training, restoring and stabilizing rivers .

This chapter describes the causes of bank erosion and failure and river stabilization measures. It describes and gives design recommendations for flow control structures such as spurs for lateral control and drop structures for vertical control. The descriptions and design recommendations include riprap: bioengineering countermeasures; effects of canalization; channel restoration and rehabilitation; and overtopping of roadways. Problems to determine the design of riprap in SI and English units are solved.

To determine the size of rock for embankment slopes, the riprap stability factor design equation that was first developed by Stevens (Stevens and Simons,

1971) is developed. This is followed with the U.S. Army Corps of Engineers design equation (Maynord, 1988) along with the Corps charts and figures as design guides. Riprap gradation, thickness, filters and placement methods are described.

The use of and design of rock enclosed wire mattresses (gabions); soil cement, rock, sand, and earth filled sacks; and articulated concrete blocks to control bank erosion of erosion from roadway overtopping flows are described.

Chapter 7. Scour at Bridges

The four components that the design, maintenance and inspection of highway crossings and encroachments must address to ensure their safety from erosion (scour) are described. These are 1) long-term degradation or aggradation; 2) general scour (contraction scour); 3) local scour at piers and abutments; and 4) stream movement. Clear-water and live-bed scour conditions defined. Equations are given for contraction and local scour at piers and abutments. The original CSU equation for pier scour is discussed, as is the latest equation given in FHWA HEC-18 (Richardson and Davis, 2001). The reader is referred to latest FHWA manuals, HEC-18 (Richardson and Davis, 2001), HEC-20 (Lagasse et al, 2001) and HEC-23 (Lagasse et al, 2009), for a more comprehensive coverage of scour, stream stability and countermeasures for the design and inspection of highway crossings and encroachments. Also, to ASCE Compendium entitled "Stream Stability and Scour at Highway Bridges (Richardson and Lagasse, 1999).

Chapter 8. Data Needs and Data Sources

An extensive list of the data needed and the sources of data for the design and evaluation of a highway crossing or encroachment are given. Reference to computerized literature and data is given. For example, Transportation Research Board's TRIS computer program. The list is very extensive and all the items listed may not be needed for a given highway crossing or encroachment problem. Therefore, a check list is provided for the engineer to check what data is needed and to check off when it has been obtained.

Chapter 9. Design Considerations for Highway Encroachments and Crossings

This chapter presents the application of the fundamentals of hydraulics, hydrology, fluvial geomorphology, and river mechanics to the hydraulic and environmental design and evaluation of highway river crossings and encroachments. Both short-term and long-term design and evaluation of highways in the river environment are delineated. Conceptual and actual examples are described.

A three level analysis procedure for the design or evaluation of a highway crossing or encroachment is recommended: 1) simple geomorphic and qualitative analysis, 2) advanced geomorphic and quantitative engineering analysis, 3) mathematical or physical model studies. The method begins with broad considerations and proceeds through a series of steps of increasing complexity to narrow down to the final design or evaluation conclusion. Many projects are finished at Level Two, but a complex or costly project may need Level Three. These levels are described in detail in HDS-6 and FHWA'S HEC-18, HEC-20, and HEC - 23.

Local, upstream and downstream affects of the bridge or encroachment on a river or a river on the crossing or encroachment are illustrated using conceptual and actual field examples. Hydraulic, geomopholgical and environmental design and evaluation of sixteen river crossings or encroachments by highway engineers are given. In presenting the course, time is allotted to this chapter to allow students to present their hydraulic and fluvial geomorphology highway problems for discussion.

Chapter 10. Design Examples of Highways in the River Environment

Three examples are given in detail illustrating the application, methods and concepts of the previous chapters to the design or evaluation of highway encroachments and crossings. The designs or evaluation use the three level approach described in Chapter 9. They use the geomorphic, hydrologic, and hydraulic and river mechanics principles to design safe and economical crossing that protect, maintain and restore the river enviorment.

Chapter 11. References

The 425 references that were reviewed and used in the preparation of HDS-6 are given in this chapter.

Appendixes

There are three appendixes. A) Metric system, conversion factors and water properties, B) Analysis of selected sediment transport relationships and C) Index. The eleven sediment transport relationships that are evaluated in Appendix B are: 1) Ackers and White; 2) Bagnold; 3) Brownlie; 4) Einstein; 5) Karim; 6) Karim and Kennedy; 7) Laursen; 8) Shen and Hung; 9) Toffaletti; and Yang'73 and '84. They were evaluated as to their applicability for size of sediment (gravel, coarse sand, fine sand or silt) and size of river (small intermediate or large).

CONCLUSION

In the early 1970's the Federal Highway Administration produced a training manual and course titled "Highways in the River Environment –Hydraulic and Environmental Considerations." In 1990 the manual was revised to include new material. In 2001 it was extensively revised and published as SDS-6. The contents of HDS-6, which are summarized in this paper, are fundamentals of open channel flow, alluvial channel flow; sediment transport; geomorphology and river morphology, river stabilization and bank protection, scour at bridges; data needs and sources, design considerations, and examples of real design and evaluation problems. The training course has been given 45 times in 23 States and Puerto Rico and to over 1,100 highway engineers.

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Comprehensive Scour Analysis at Highway Bridges HEC-I8

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ABSTRACT

In 1988 The Federal Highway Administration issued FHWA Technical Advisory T5140.20 entitled "Scour at Bridges." It required the States to evaluate the scour risk at all bridges over water. Accompanying the Advisory was the publication "Interim Procedures for Evaluating Scour at Bridges." The "Interim Procedures" delineated the scour problem at highway encroachments and crossings as 1) stream instability and channel movement, 2) long term degradation or aggradation, 3) live-bed or clear-water contraction scour and 4) local scour at piers and abutments. The "Interim Procedures" provided guidance and equations for evaluating scour. This was the first time a manual was written that gave comprehensive methods and recommended equations for the hydraulic analysis to determine scour depths for design of foundations of new bridges or evaluation of existing bridges and to protect the river environment. Subsequently, the "Interim Procedures" were updated and issued as Hydraulic Engineering Circulars 18. The Fourth Edition of HEC-18 is summarized in this paper.

INTRODUCTION

In September 1988 The Federal Highway Administration issued FHWA Technical Advisory T5140.20 entitled "Scour at Bridges." It required the states to evaluate the scour risk at all bridges over water. Accompanying the Advisory was the publication "Interim Procedures for Evaluating Scour at Bridges." The "Interim Procedures" delineated the scour problem at highway encroachments and crossings as 1) stream instability and channel movement, 2) long term degradation or aggradation, 3) live-bed or clear-water contraction scour and 4) local scour at the piers and abutments. The "Interim Procedures" provided guidance for determining stream instability, channel movement, and long term elevation changes as well as methods to counteract them. It included equations to determine live-bed or clear-water contraction scour depths, based on the work of Emmett Laursen. To determine local pier scour depths, it recommended the "so called" Colorado State University Equation from FHWA's Publication "Highways in the River Environment." This pier scour equation was selected because a study of many pier scour equations by FHWA Research Engineer Sterling Jones (1983) showed this equation was the best fit. It enclosed all available scour depth research data and gave the smallest scour depths.

Recent studies indicate this is still the case (Mueller, 1996 and Mueller and Jones, 1999).

To determine local abutment scour depths, the "Interim Procedures" delineated seven abutment conditions (cases) such as abutment in the channel, at the bank, or set back and considering live-bed or clear-water scour. For each case it provided equations (Liu et al, 1961) or (Laursen, 1980) to determine scour depths and/or methods to protect the abutments.

The "Interim Procedures" (written by Everett V. Richardson and Stanley R. Davis) were the first comprehensive manual that gave detailed recommendations to determine stream instability, delineated the three components of scour at highway bridges (long term aggradations or degradation, contraction scour and local scour), and gave equations or methods to determine scour depths and/or countermeasures to protect highway bridges and encroachments from stream instability and scour.

The TA was written by Stanley Davis, Chief of FHWA's Hydraulics and Geotechnical Branch, with input by staff. Many drafts were prepared and reviewed by Stanley Gordon, Chief of the Bridge Division, FHWA legal staff and others before the TA was approved for dissemination. The TA was effective in implementing a national scour evaluation program that met the requirements of the Congress. While it presented policies and guidance for the program, it also permitted a degree of flexibility so that the states could carry out the program in a manner consistent with their existing organizations and procedures.

In 1991 FHWA updated and published the "Interim Procedures" as HEC-18, "Evaluating Scour at Bridges." (Richardson et al, 1991). The Fourth Edition was released May 2001 (Richardson and Davis, 2001) at which time FHWA also released two companion documents: HEC-20 entitled "Stream Stability at Highway Structures" (Lagasse et al, 2001) and HEC-23 entitled "Bridge Scour and Stream Instability Countermeasures" (Lagasse. et al, 2001, 2009). The three HECs provide guidance for bridge scour, stream stability analysis and the design of countermeasures. They contain the results of the latest research and form the basis of FHWA National Highway Institute's three short courses on scour (FHWA NHI, 2010). FHWA periodically updates these publications as new information becomes available. **The three HEC's are available from the National Technical Information Service, Springfield, VA 22161 (703) 487-4650.**

BACKGROUND

At 9:00 am on April 5, 1987 the Interstate (I-90) Highway Bridge over Schoharie Creek in Upstate New York collapsed killing 10 people. Four passenger cars and one truck fell 60 feet into the Creek. The failure received national television and newspaper coverage.

The National Transportation Safety Board investigated the accident and issued their findings in a highway accident report entitled "Collapse of New York

Thruway (I-90) over the Schoharie Creek near Amsterdam New York, April 5, 1987" (NTSB, 1988). Drs. Richardson and Lagasse were Consulting Engineers for the Safety Board's investigation, which included a physical model study made at Colorado State University. The Safety Board's findings were that scour of pier 3 caused the failure. All 5 piers were founded on spread footings without piles.

The U.S. Congress held hearings on the failure, where people such as Ralph Nader testified that the Federal Government should take over the design and construction of all highway roads and structures. FHWA officials and all State Highway Engineers and State political officials such as Governors opposed such move. But Congress instructed FHWA to strengthen its oversight of the design, construction and inspection of all bridges. In particular, Congress instructed FHWA to evaluate and determine the vulnerability of failure from scour of all bridges over water in the Federal bridge inventory and to periodically report back to Congress on the progress of the evaluation and condition of all bridges in the inventory as to their vulnerability to failure by scour. The FHWA was charged with the task of strengthening the National Bridge inspection program. FHWA responded by issuing Technical Advisory T5140.20 entitled "Scour at Bridges" and the accompanying "Interim Procedures for evaluating Scour at Bridges" requiring the States to evaluate the scour risk at all bridges over water.

HEC-18 EVALUATING SCOUR AT BRIDGES (FOUTH EDITION)

Design Philosophy (Chapter 2)

Bridge foundations should be designed to withstand the effects of scour without failing for the worst conditions resulting from floods equal to the 100-year flood or a smaller flood if it would cause scour depths deeper than the 100-year flood. Bridge foundations should be checked to ensure that they will not fail due to scour resulting from the occurrence of a superflood in the order of magnitude of a 500-year flood. Chapter 2 amplifies on the design philosophy and gives a general design procedure, concepts and a step by step detailed design procedure. Also, some miscellaneous hydraulic factors, such as drag forces on superstructures, ice forces and the design of spread footings placed on tremie seals or soils are described.

Basic Concepts and Definitions of Scour (Chapter 3)

The four components of a comprehensive scour analysis are defined and illustrated. These are: 1) Long term aggradation and degradation of the river bed. 2) General scour at the bridge (contraction scour or other general lowering of the bridge cross section. 3) Local scour at piers and abutments. 4) Lateral shifting of the stream. How sediment transport affects bridge foundations (that is the difference between clear-water and live-bed scour) is discussed in detail. Also, equations and methods of analysis are for non-cohesive soils. But are recommended for cohesive and cemented soils because the ultimate depth of scour is the same. Only time is the factor.

Long-term Aggradation and Degradation (Chapter 4)

The factors affecting long-term stream bed elevation changes, methods for evaluating these changes and the use of computer models are discussed. The role of geology, river mechanics, sediment transport, geomorphology and fluvial geomorphology are presented.

General Scour (Contraction Scour) (Chapter 5)

General scour is the general decrease in the elevation of the stream bed across the bridge opening. It does not include the local scour or the long term bed elevation changes. It can be cyclic, That is, there can be cutting and filling of the stream bed during the passage of a flood. Contraction scour is a main cause of general scour but other factors may cause general scour as well.

Contraction Scour Equations

Contraction scour occurs when the bridge and its approaches encroaches either on the stream channel or the stream's flood plain. This increases the stream velocity and sediment transport capacity. HEC-18 describes, with sketches, five cases of contraction scour at bridge crossings with two conditions of erosion (live-bed or clear-water). The cases are:

1. Bridge abutments project into the stream channel with or without overbank flow.
2. Bridge abutments at edge of the channel with overbank flow.
3. Bridge abutments setback from the channel and overbank flow.
4. Bridge crosses the stream at a narrow section.
5. Bridge piers significantly obstruct the flow (with or without debris) in the previous cases.

The "Interim Procedures" and HEC-18 give equations to determine contraction scour depth for each erosion condition. These are given below:

Live-bed contraction scour occurs at a bridge when the bridge opening contracts the flow and there is transport of bed material in the upstream reach into the bridge section. With live-bed contraction scour the area of the contracted section increases until, in the limit, the transport of sediment out of the contracted section equals the sediment transport in.

The equation, a modified version of Laursen's 1960 equation for live-bed scour in a long contraction, is;

$$y_2/y_1 = (Q_2/Q_1)^{6/7}\,(W_1/W2)^k$$

Clear-water contraction scour occurs when (1) there is no bed material transport from the upstream reach into the bridge cross section, or (2) the material transported in the upstream reach is transported through the bridge section in suspension and at less than the capacity of the flow. With clear-water contraction scour the area of the contracted section increases until the velocity of the flow or the shear stress on the bed is equal to the critical velocity or critical shear stress of a representative particle size in the bed material.

The "Interim Procedures" and HEC-18 recommended equation, based on a development given by Laursen in 1963 is:

$$y_2 = ((K_u\,Q_2^2)/(D_m^{2/3}\,W^2))^{3/7}$$

$$y_s = y_2 - y_0 = \text{average contracted scour depth}$$

HEC-18 states that scour depths with live-bed contraction scour may be limited by coarse sediments in the bed material. Where coarse sediments are present HEC-18 recommends calculating contraction scour using both equations and taking the smaller scour depth.

Determination of Local Scour at Piers Chapter 6)

The "Interim Procedures," based on the study by Sterling Jones (1983) recommended the CSU equation for both live bed and clear-water conditions. The equation was developed for the FHWA Publication "Highways in the River Environment, Environmental and Hydraulic Considerations" (Richardson et al, 1975). The succeeding HECs recommended a modified CSU equation. The modifications were to add additional corrections factor (Ks) based of new research and field experience. The 4th HEC-18 Edition equation for local pier scour is:

$$y_s/a = 2.0\ K_1\ K_2\ K_3\ K_4\ K_w\ (y_1/a)^{0.35}\ Fr_1^{\ 0.43}$$

The variables are defined in notation and values are given for the Ks in HEC-18. Also, HEC-18 places a limit on the maximum value of y_s/a.

Scour Depth Determination for Complex Piers

The 4th Edition of HEC-18 based on the research and papers of Jones (1989), Salim and Jones (1996, and 1999), Jones and Sheppard (2000), delineated a method for determining local scour depths for piers with complex geometry. Recent research supports the method and suggest a slight modification (Ataie-Ashtiani et al, 2010). Figure 1 illustrates the components of a complex pier and the methodology used. The reader is referred to the 4th Edition of HEC-18 for the development, an example problem and guidance in using the method.

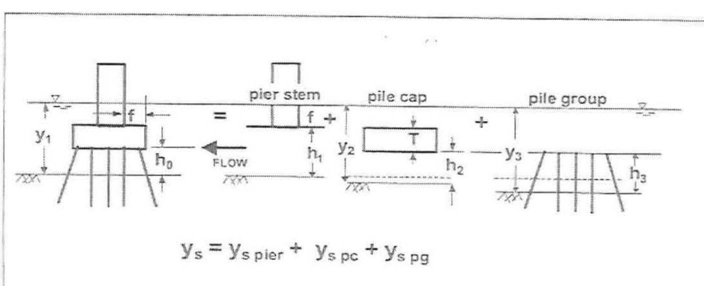

Figure 1. Definition sketch for scour at a complex pier (Jones and Sheppard, 2000).

Evaluating Local Scour at Abutments (Chapter 7)

The components of the local scour at abutments are illustrated in Figure 2. Note the horizontal vortex that produces scour depths at the upstream corner and side of the abutment. This is the scour depth determined by most abutment scour equations. But note also the wake vortex. This vortex erodes the downstream face of the abutment and approach embankment, causing abutment failure. Often this wake vortex causes a major scour problem. Erosion from the wake vortex can be easily controlled by recognizing the problem and placing riprap on the downstream face of the abutment and approach embankment.

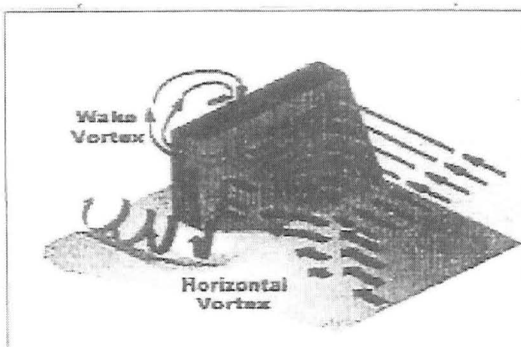

Figure 2. Schematic representation of abutment scour (HEC-18).

Equations for predicting local scour depths are mainly based on laboratory studies (Lieu et al (1961), Laursen (1980), Froehlich (1989) and Melville (1992). Little or no field data is available. The problem, as stated in HEC-18 is:

"The reason the equations in the literature predict excessive conservative abutment scour depths for the field situation is that, in the laboratory flume, the discharge intercepted by the abutment is directly related to abutment length; whereas, in the field, this is rarely the case."

The "Interim Procedures" and HEC-18 identified abutment site conditions, angle to the flow (skew), discharge intercepted by the abutment and approach embankment and abutment shape as scour depth factors. Researchers identified the same factors, but unfortunately used abutment and approach length as a substitute for discharge. Common abutment shapes are 1) vertical wall abutments, 2) vertical wall abutments with wing walls and spill-through abutments.

Abutments Local Scour depth Equations

HEC-18 recommends two equations for both live-bed and clear-water scour. They are Froehlich's (1989) and HIRE (Richardson et al, 2001). The latter is based of scour depths measure at the end of spurs in the Mississippi River and is applicable when the ratio of the projected abutment and embankment length to the flow depth is greater than 25.

Froehlich's (1989) live-bed abutments scour equation

$$\mathbf{y_s / y_1 = 2.27\, K_5\, K_6\, (L' / y_1)^{0.43}\, Fr^{0.61} + 1.0}$$

HIRE live-bed abutment scour equation

$$y_s / y_1 = 4\ Fr^{0.33}\ (K_5 / 0.55)\ K_6$$

Comprehensive Example Scour Problem (Chapter 8)

A comprehensive hydraulic analysis from a paper by Arneson et al (1991) of scour at a bridge crossing using the procedure and equations given in HEC-18 is presented. The analysis uses SI units in Chapter 8. But in Appendix H uses English units. The hydraulic variables were obtained using FHWA's WSPRO computer program. WSPRO's input and output is given in Appendix G.

Chapters 9 to 13 and Appendixes

CHAPTER	BRIEF DISCRIPTION
9	SCOUR ANALYSIS FOR TIDAL WATERWAYS
	The special condition of scour analysis in tidal unsteady flow given.
10	NATIONAL SCOUR EVALUATION PROGAM
	National evaluation program is described, with progress as of 2000.
11	INSPECTION OF BRIDGES FOR SCOUR
	A FHWA recommended inspection program with procedures is given.
12	SPECIAL CONSIDERATIONS FOR SCOUR AND STREAM INSTABLITY
	This chapter discusses the development of plan of actions for scour critical bridges, scour in cohesive or rock bed materials countermeasures etc.
13	LITERATURE CITED
	107 publications are cited.
APPENDIX	
A	METRIC SYSTEM, CONVERSION FACTORS, WATER PROPERTIES
B	EXTREME EVENTS
C	CONTRACTION SCOUR AND CRITICAL VELOCITY EQUATIONS
D	INTERIM PROCEDURES FOR ESTIMATING PIER SCOUR WITH DEBRIS
E	STURM ABUTMENT SCOUR EQUATIONS
F	MARYLAND ABUTMENT SCOUR EVALUATION METHOD
G	WSPRO INPUT AND OUTPUT FOR EXAMPLE PROBLEMS
H	COMPREHENSIVE SCOUR PROBLEM, ENGLISH UNITS
I	FHWA TECHNICAL ADVISORY T 5140.23
J	FHWA 1995 CODING GUIDE FOR NATIONAL BRIDGES
K	UNKNOWN FOUNDATIONS
L	SCOUR IN COHESIVE SOILS
M	SCOUR COMPETENCE OF ROCK

NATIONAL HIGHWAY INSTITUE

The FHWA's National Highway Institute established a short course titled "Stream Stability and Scour at Highway Bridges in 1991 using the "Interim Procedures" as the course text. Subsequent courses used the current edition of HEC-18 as the course text. At first, bridge inspectors attended the 3 day course. But FHWA and NHI established a 1-day course for inspectors titled "Stream Stability and Scour at Highway Bridges for Inspectors" (FHWA NHI, 2009). This course concentrates on visual keys to detecting scour and stream instability problems and emphasizes guidelines to complete the hydraulic and scour-related coding requirements. With the increase in knowledge of scour and stream instability countermeasures NHI and FHWA established a new course entitled "Countermeasure Design for Bridge Scour and Stream Instability." It uses HEC-23 (Lagasse et al, 2001, 2009) as the course text. In the period 1991 to 2005 Ayres Associates, Inc. presented the scour courses to more than 5,700 students in 45 States. However, engineers and highway officials in all 50 States have attended the course.

CONCLUSION

In 1988 the U.S. Department of Transportation, Federal Highway Administration as part of Technical Advisory T5140.20 "Scour at Bridges" released a manual titled "Interim Procedures for Evaluating Scour at Bridges. " The "Interim Procedures" delineated the scour problem at highway encroachments and crossings as 1) stream instability and channel movement, 2) long term degradation or aggradation, 3) live-bed or clear-water contraction scour and 4) local scour at the piers and abutments. The "Interim Procedures" provided guidance for determining stream instability, channel movement, and long term elevation changes as well as methods to counteract them. This was the first time that a manual was written that gave a comprehensive method with recommended equations for the hydraulic analysis to determine scour depths for the design of foundations of new bridges or evaluation of existing bridge foundations. In succeeding years the "Interim Procedures" were updated and issued as Hydraulic Engineering Circulars HEC-18. The Fourth Edition was issued in May 2001.

FHWA (1991) updated the advisory to T51140.23 titled "Evaluating Scour at Bridges." In 1992 the American Association of State Highway and Transportation Officials (AASHTO, 1992) addressing the problem of stream stability and scour stated "The probable depth of scour shall be determined by subsurface exploration and hydraulic analysis. Refer to Article 1.3.2 and FHWA Engineering Circular (HEC) 18 for general guidance regarding hydraulic studies and design."

NOTATION

a = Pier width, m (ft)
f = Upstream projection of a footer from pier stem, m (ft)
Dm = Diameter of the smallest nontransportable particle in the bed material in the contracted section (taken as 1.25 D_{50}) m (ft)
D_{50} = Median diameter of the bed material, m (ft)
y_1 = Average depth in the upstream main channel, or directly upstream of the pier or abutment, m (ft).

y_2 = Average depth in the contracted section, m (ft)
y_s = Scour depth in the contracted section, m (ft)
y_0 = Existing depth in the contracted section before scour, m (ft)
Q_1 = Discharge in upstream channel TRANSPORTING SEDIMENT. m^3/s (ft^3/s}
Q_2 = Discharge in the contracted channel or in the setback overbank area at the bridge. It is associated with the width W, m^3/s (ft^3/s)
W_1 = Bottom width of the upstream channel that is transporting bed material, m (ft)
W_2 = Bottom width of the contracted section less pier widths, m (ft)
k = Exponent determined below

V_*/w	k	Mode of Sediment Transport
<0.50	0.59	Mostly contact bed material transport.
0.50 to 2.0	0.64	Some suspended bed material transport.
>2.0	0.69	Mostly suspended bed material discharge

K_u = 0.025 SI units
K_u = 0.0077 English units
V_* = Shear velocity in the upstream section $(gy_1S_1)^{0.5}$ m/s (ft/s)
S_1 = Slope of the energy grade line in the upstream channel, m/m (ft/ft).
w = Fall velocity of the D_{50} of the upstream bed material, m (ft)
K_1 = Correction factor for pier shape, HEC-18
K_2 = Corection factor for angle of attack = $(\text{Cos. } 0 + L/a \text{ Sin. } 0)^{0.65}$
Maximum value of L/a is 12
K_3 = Correction factor for bed condition given in 4th Edition HEC-18
K_4 = Correction factor for armoring by bed material size 4th Edition HEC-18
K_5 = Coefficient for abutment shape = 1.0 for vertical wall abutment; 0.82 for vertical –wall with wing walls and 0.55 for spill-through.
K_6 = Coefficient for angle of embankment to flow. = $(0/90)^{0.13}$ (0<90 if embankment points downstream and 0>90 if embankment points upstream
K_w = Correction factor for pier width in shallow flows. HEC-18
L = Pier length, or abutment embankment length normal to the flow m (ft)
L' = Length of active flow obstructed by abutment and embankment m (ft)
A_e = Flow area obstructed by abutment and embankment m^2 (ft^2)
Q_e = Flow obstructed by abutment and embankment m^3/s (ft^3/s)
V_e= Q_e / A_e m/s (ft/s)
Fr = Froude Number directly upstream of the pier or abutment

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Revisiting the HEC-18 Scour Equation

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Abstract

Accurate pier scour predictions are essential to the safe and efficient design of bridge crossings. Current practice uses empirical formulas largely derived from laboratory experiments to predict local scour depth around single-bridge piers. These formulas have two problems. First, they are hindered by scaling effects; second, they do not consider detailed hydrodynamic forces at work in the scour process. These formula deficiencies can often produce excessive over prediction of scour depths that can lead to unnecessary construction costs.

In an effort to improve the predictive capabilities of the HEC-18 scour model, this work uses field-scale data and nonlinear regression to develop a family of equations optimized for various non-cohesive soil conditions. Improving the predictive capabilities of well-accepted equations will save scarce project dollars without sacrificing safety. To help improve acceptance of modified equations, the familiar form of the HEC-18 equation is maintained. When compared to the HEC-18 local pier scour equation, this process reduced the mean square error of a validation data set while maintaining over prediction.

Introduction

The Federal Highway Administration defines scour as the result of the erosive action of flowing water, excavating and carrying away material from the bed and banks of streams and from around piers and abutments of bridges. The United States has approximately 600,000 bridges; about 80 percent require some sort of scour mitigation (Nassif *et al.* 2002). However, during the 40-year period ending in 2005, more than 1,500 bridges in the United States failed; nearly 60 percent of these failures were hydraulic in nature (Kornel Kerenyi, personal communication, June

18,2009). The cost of bridge failure or bridge closure far exceeds the cost associated with repair. Therefore, accurately determining scour depth while sizing foundations and waterway openings will help reduce costs over time (Richardson and Davis 2001).

An accurate determination of the expected scour at a bridge crossing is important for an economic and safe bridge design. Several models are available to predict the ultimate scour depth near piers or abutments (see Johnson (1995) or Muller and Wagner (2005) for lists of the most commonly used scour equations). Many factors, including the amount of cohesion in the sediment, or clear-water or live-bed conditions, determine the appropriateness of a particular model. Over the last several decades, models were developed, adjusted and improved. For example, Molinas (2003) adjusted the Colorado State University pier-scour equation to account for the coarse material fraction which is known as the K_4 adjustment factor in HEC-18 pier scour equation.

Laboratory data is the primary source of information used in model development. However, many authors note scale as a source of error in models derived from laboratory data (Hopkins and Vance 1980). These laboratory investigations typically model straight, rectangular channels with uniform approach-flow velocities, approach-flow depths, and non-cohesive bed material (Wagner *et al.* 2006). These characteristics rarely represent field conditions.

Most scour equations in common use today are empirically based. Scour is a complex process and accurate predictions are not likely to come from empirical models. However, empirical models are necessary since budgetary restrictions prevent the implementation of more complex, physics-based modeling for every bridge design. According to Mueller and Wagner, none of the commonly used scour equations accurately and conservatively (over) predict the scour observed in the field (Mueller and Wagner 2005). Inaccuracies exist for several reasons including a lack of hydrodynamic variables, laboratory source data and inaccuracy in field data measurements.

The goal of this work is to improve the HEC-18 local pier scour equation, Equation 1, in two ways. First, there is an attempt to improve the fit between predicted and observed scour by re-deriving the HEC-18 equation with field measurements of scour. Second, stratifying data based on approach depth ratio and creating a family of equations. These modifications are expected to lead to improved prediction performance largely because similarly grouped derivation data is expected to reduce variance in predicted scour depth ratios. Data is stratified based on the approach depth ratio, which in the HEC-18 equation is the pier width divided by the approach depth ratio. Due to data limitations in this study, the data is stratified into two sets only resulting in two unique predictive equations. However, the family of equations could expand as field-data collection programs grow and more data becomes available.

$$\frac{y_s}{y_1} = 2K_1K_2K_3K_4\left(\frac{a}{y_1}\right)^{0.65} Fr^{0.43} \tag{1}$$

Data

The National Bridge Scour Database, last updated in 2004 and maintained by the U.S. Geologic Survey, provided the data for the present analysis and provides data from 20 sites in eight states. Records were chosen for this analysis based on completeness. A record must contain enough data to apply the current version of the HEC-18 scour equation for use in this study. The database produced 148 records. However, due to a limited amount of complete data from cohesive soil, all data used in this investigation are from non-cohesive sites. Most records that met the completeness condition as described above had approach-depth ratios of less than 0.75 and Froude numbers less than 0.46 as shown in Figure 1.

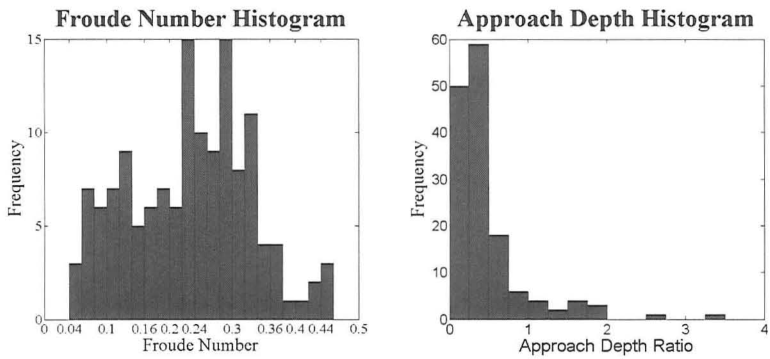

Figure 1: Approach depth ratios and Froude numbers of data for the combined derivation and validation data sets.

The first step in the data filtering process was to remove records with outlying relative scour depths, y_s/y_1. After removing the outliers, 145 records remained. The next step was to identify stratification points within the approach depth ratios. These break points are used to define the useful range of a particular equation in the family of equations being developed. Break points were determined by trial and error. These break points were selected as the largest group of data that would retain conservative prediction (i.e. predicted depths in excess of observed). Descriptive statistics used in the derivation of each equation in the family of curves are shown in Table 1 and Table 2.

The available field data was separated into a derivation data set and a validation data set. A single site may have many records, so validation data are chosen from two representative sites. No records from validation sites were also used for model derivation. This was done to ensure the new model could predict relative scour depths outside of the locations used to derive the equation (i.e., the equation was not relying on site-specific processes captured in the derivation process).

Table 1: Descriptive statistics used in deriving equation 2

Deriving Data Equation 2, lower range								
	flow depth (m)	velocity (m/s)	Froude number	scour depth (m)	relative scour depth	pier width (m)	approach depth ratio	median grain size (mm)
min	4.24	0.31	0.04	0.34	0.05	0.41	0.04	0.17
max	15.36	2.26	0.24	4.27	0.30	2.79	0.20	1
median	7.01	1.19	0.13	1.07	0.16	0.91	0.16	0.54

Table 2: Descriptive statistics used in deriving equation 2

Deriving Data Equation 2, upper range								
	flow depth (m)	velocity (m/s)	Froude number	scour depth (m)	relative scour depth	pier width (m)	approach depth ratio	median grain size (mm)
min	1.31	0.52	0.06	0.15	0.036	0.53	0.21	0.15
max	20.03	3.17	0.45	7.65	0.78	5.24	0.44	1.82
median	7.3	1.40	0.23	1.52	0.30	1.83	0.32	0.64

Regression

The HEC-18 equation was re-derived using nonlinear regression analysis. This process optimized parameters to a user-defined functional form. The resulting parameters minimize the error between predicted and observed values through an ordinary least-squares procedure. The functional form used in this analysis is the HEC-18 scour equation with an additive factor of safety, see Equation 2.

$$\frac{y_s}{y_1} = b_1 K \left(\frac{a}{y_1} \right)^{b_2} Fr^{\,b_3} + FOS \tag{2}$$

where y_s is the scour depth, "a" is the pier width, y_1 is the flow depth directly upstream of the pier and Fr is the Froude number, K is the correction factor (which embodies K_1 through K_4 of the HEC-18 equation) and was not modified, and each "b_i" is an optimized regression parameter. The independent variables are the approach depth ratio (a/y_1) and Froude number (Fr). Finally, FOS is the factor of safety.

The nonlinear regression described above yields a best-fit model that both under-and over predicts scour. A factor of safety is added to the best-fit equation in order to transform it into a design equation by ensuring all predictions exceed observations. The factor of safety is computed by examining the maximum under prediction from the deriving data set. This maximum under prediction was added to each predicted value in the validation set. Using an additive factor of safety as suggested in equation 1, an approach modeled after the Froehlich pier-scour design equation (Brunner 2008), increases the utility of the equation. A simple modification makes the equation appropriate for non-design applications.

The first equation developed in the family of equations uses a subset of the selected records where all approach depth ratios are less than 0.2. There are 21 data points in this deriving data set and 16 points in the validation data set. In the deriving data set, the Froude numbers ranged from 0.04 to 0.24, while in the validation data the range was 0.07 to 0.30.

The second equation in the family had approach depth ratios between 0.2 and less than 0.45. The deriving data set contained a total of 48 data points; the validation data set contained 18. The Froude number ranged from 0.06 to 0.45 in the deriving data set and from 0.04 to 0.44 in the validation data set as shown in Table 3. The approach depth ratios of the remaining 42 records are too sparse to produce a meaningful model and extend the family of equations beyond approach depth ratios of 0.45. However, with additional data the authors are optimistic the family of equations can continue to expand and cover a larger range of approach depth ratios.

Table 3: Froude numbers and regression parameters associated with each equation

	Froude Number		Regression Parameters			
	Deriving	Validation	b_1	b_2	b_3	FOS
Equation 2 lower	0.04 to 0.24	0.07 to 0.30	12.62	1.86	0.86	0.15
Equation 2 upper	0.06 to 0.45	0.04 to 0.44	1.23	2.90	-0.51	0.58

Results

The HEC-18 local pier-scour equation was derived across the entire range of available data with a one-size-fits-all approach. The error associated with the relative scour prediction increases linearly with the predicted relative scour depth (y_s/y_1) as shown in Figure 2. The one-size-fits-all approach leads to significant over prediction, especially at larger expected scour depths. This results from adjusting the model across the entire domain to ensure over prediction at a few hard-to-fit data points. The family of equations can accommodate hard-to-fit points as well, but does so without adjusting all values across the entire domain. This results in increasing or decreasing residual error depending on stratification points; however, all points remain over predicted, as shown in Figure 3.

Both lower and upper members of Equation 2 yield significant improvement in terms of mean square error when compared to predictions based on the original HEC-18 equation. Both equations still over-predict observed values of relative scour depth, but are significantly less than the original HEC-18 equation (Table 4 and Figure 3).

While any field-scale data is a welcome addition to the database, this work highlights the need for field-scale data with expected approach depth ratios between 0.45 and 1.25. Data with approach-depth ratios greater than 1.25, commonly referred to as wide-pier data, historically lacks representation in both laboratory and field-scale data sets. Should enough field-scale data become available to expand the family of equations to approach-depth ratios well beyond 1.25, wide piers will not require a special correction factor as is currently the case in Hydraulic Engineering Circular 18 (Richardson and Davis 2001).

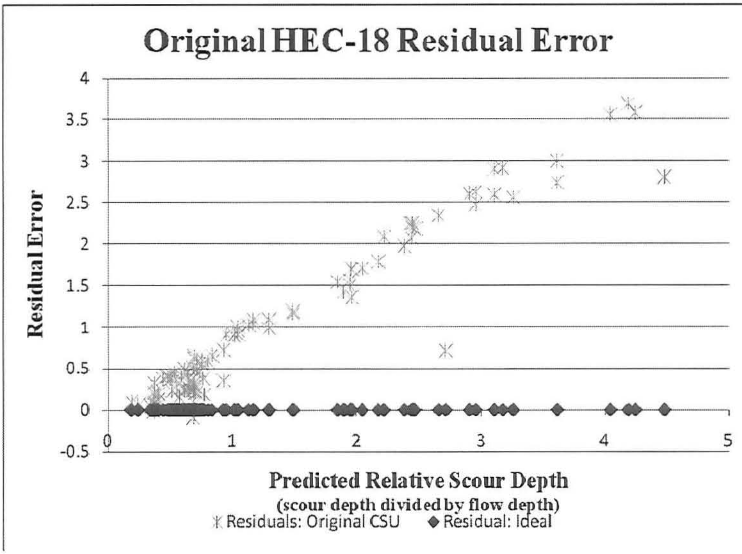

Figure 2: Residual error of the HEC-18 scour equation based on all available data.

A similar stratification analysis was also performed based on the Froude number. Initially, the same procedure as described above was implemented. Specifically, no restrictions were placed on the approach-depth ratios. However, due to the scarcity of data beyond an approach depth ratio of 0.75, favorable results were not obtained. Restricting data to approach depth ratios less than 0.75 yielded better results. With approach depth ratios restricted, the data were stratified based on Froude number. The first stratification point was a Froude number less than 0.25. All validation observations were over predicted but subsequent models could not over predict all of the observations in the validation data. The mean square error associated with the stratified Froude model is 0.07 on the deriving data set, while the mean square error associated with the original HEC-18 equation is 0.27.

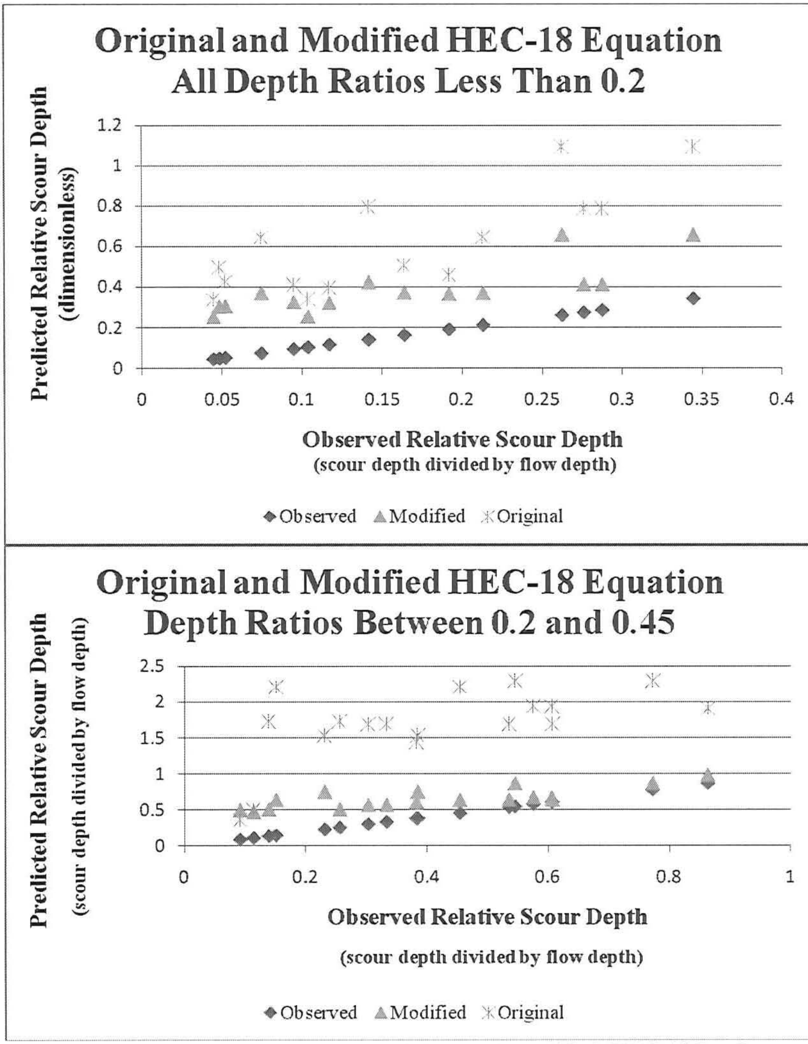

Figure 3: Comparisons between the original and modified HEC-18 local scour equation.

Table 4: Comparison of mean square error associated with the modified and original HEC-18 equation

	Mean Square Error from Validation Data Set	
	New Equation	Original HEC-18 Equation
Equation 2 lower	0.07	0.31
Equation 2 upper	0.10	2.18

Conclusions

Scour is a complex process that is difficult to describe with just a few easily obtained parameters. It is even more difficult to accurately describe scour with a single, one-size-fits-all equation. While this process showed stratifying the dataset and creating a family of equations can reduce error while maintaining safe design practices, the authors are mindful of the limited number of data points used in the construction of this model. For this reason, these authors recommend a continued effort to collect field-scale data especially across a wide range of expected conditions. With ample data, the family of equations can be expanded to cover the entire range of conditions currently covered by the HEC-18 local pier scour equation.

Acknowledgments

The authors would like to acknowledge the Michigan Department of Transportation who funded a component of this project including the field data collection under project number 85106.

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Review of Bridge Scour Practice in the U.S.

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ABSTRACT: For years bridge designers in the U.S. have used FHWA Publication Hydraulic Engineering Circular No. 18 (HEC 18) as a principal tool to determine scour depths. Increasingly, though, practitioners recognize that some of the circular's standard equations over predict scour depth for certain geologic and hydraulic conditions. In an effort to improve scour design and evaluation methods, the New Jersey Department of Transportation (NJDOT) recently conducted a survey of current scour practice of DOTs across the U.S. The ten-question survey queried agencies about their design standards, experiences with failures, monitoring programs, and countermeasure preferences, among other things. This paper presents the results of the nationwide scour survey. Highlighted are the creative and diverse approaches by some states to either modify HEC 18 procedures or develop alternative scour prediction methods. The paper also discusses critical geologic, hydraulic, and hydrologic parameters for rational evaluation of scour depth, gleaned from both the survey and local experience with New Jersey bridges.

INTRODUCTION

Prevention of bridge scour has now been a national priority for two full decades. Beginning in 1990 with the Federal Highway Administration's (FHWA's) issuance of Technical Advisory T5140.20, transportation agencies across the U.S. have been deliberately engaged in evaluating the scour susceptibility of bridges within their inventories. Those bridges found to be scour critical are now in various stages of remediation, ranging from monitoring to outright replacement. While progress is being made, many state and county DOTs are still in the process of implementing their action plans. The reason is the sheer number of bridges that detailed screening has determined to be scour susceptible, which number into the hundreds in some states.

Prudent action is warranted, since scour remains a leading cause of bridge failure in the U.S. Fortunately, the large majority of the failures are not sudden or catastrophic. More commonly, the responsible agency observes progressive erosion and scour, and then decides to repair the bridge or replace it preemptively.

For riverine flow the principal scour tool for U.S. bridge designers is Hydraulic Engineering Circular No. 18 (HEC 18) published by the FHWA. Increasingly, though, practitioners recognize that the standard equations in HEC 18 over predict scour depth for certain hydraulic and geologic conditions. This is not surprising, since most of the HEC 18 relationships are based on laboratory flume studies conducted with sand-sized sediments. It is fair to ask whether scale modeling can effectively represent a phenomenon as complex as scour, especially in view of the wide diversity of hydrologic, hydraulic, and geotechnical conditions that exists across the nation. Indeed, the scour behavior of a bridge spanning a mile-wide river with silty sediments in the Midwest is quite different from a bridge crossing a boulder-filled stream in the Mountain States, which differs yet again from another bridge spanning a modest-size river choked with coarse glacial outwash in the Northeast. Recognizing such regional differences, and driven by the funding limitations, it is prudent to re-examine predictive scour models.

BACKGROUND

HEC 18 has been a key companion resource for FHWA's national scour program. Now in its fourth edition, HEC 18 remains in wide use by transportation agencies and consultants. The scour design relationships contained in the publication are an amalgamation of work by various investigators. For abutment scour, the principal relationship is the Froehlich Equation, which is based on a regression analysis of 170 laboratory flume tests (Froehlich, 1989). The alternative HIRE equation is also provided, and it is originally based on field data for scour at the end of spurs on the Mississippi River (Richardson, Simons, and Lagasse, 2001). Other methods for estimating abutment scour are also cited, including Sturm (1999) and Melville (1992). These are mostly based on laboratory flume testing as well.

The principal design relationship in HEC 18 for estimating pier scour is the CSU equation, which was derived from laboratory data by researchers at Colorado State University (Richardson, Simons, and Lagasse, 2001). Other relationships developed from laboratory flume testing are also cited but not specifically recommended, e.g. Laursen (1983) and Jain and Fischer (1979).

Without question, HEC 18 has served a worthy function in the nation's scour safety program by providing agencies and consultants with access to a compendium of design relationships. However, HEC 18 was never meant to be a mandate, but rather a guidance document that describes the "state of knowledge and practice." It does not preclude a transportation agency from applying another method of scour prediction as long as it is rational and defensible.

A number of states have now opted to either modify the methods in HEC 18 or develop entirely new, alternate approaches for scour evaluation. Such efforts are typically backed by scientific studies that factor in the geologic and hydrologic conditions that exist within the respective state. An important motivation for these modified/alternate methods are the results of comparative field studies, which consistently show poor correlation between predicted scour using HEC 18 methods and actual scour observed in the field. And the disparity is becoming more apparent as the database of bridge inspection and monitoring data continues to expand.

COMPARATIVE SCOUR STUDIES

In recent years, several studies have compared the field scour observed at bridge sites with the scour values predicted by various equations. The studies reflect the ever increasing concern that current methods for estimating scour depth are principally based on laboratory experiments and do not necessarily correlate well with field conditions. These agencies are seeking more realistic procedures to estimate scour depth, since resources for construction and repair are chronically limited, and bridges need to be better prioritized so that funds are expended where they are truly needed.

Three recent comparative studies of bridge scour will be summarized in this section. All studies were rigorous, and in total they comprise more than 200 bridges located in five states.

(1) Lombard, P.J. and G.A. Hodgkins (2008)

This insightful study was recently completed by the U.S. Geological Survey (USGS) in cooperation with the Maine Department of Transportation. The investigators analyzed 50 bridges that were distributed geographically throughout the state. The median age of the bridges was 66 years, and all were single-span on non-tidal waterways. Field surveys were conducted to determine channel geometry and characteristics, as well as to measure observed abutment scour, which ranged from 0 to 6.8 ft. The average actual observed scour across all the sites was less than 1 ft. Skew angles of the abutments and embankments in relation to the channel showed wide variation, ranging from 0 to 50 degrees.

The four scour estimation methods applied to the bridges in the Maine study were the Froehlich/Hire method, the Sturm method, the Maryland Department of Transportation method, and the Melville method. A summary of the study results comparing predicted and observed scour are presented in **Table 1**. As indicated, no significant correlation was found between calculated scour and scour observed in the field for any of the four methods. In fact, predicted scour was frequently an order of magnitude greater than observed scour. Scour was also underpredicted by the equations 4% to 14% of the time. Given the lack of correlation between predicted and observed scour, the authors suggest it may be preferable to prescribe a single value of abutment scour and apply a suitable factor of safety.

TABLE 1: Summary of Predicted vs. Observed Abutment Scour for Maine Study (modified from Lombard and Hodgkins, 2008)

Method	*Overpredictions*			*Underpredictions*			*Correl. Coeff.*
	%	Avg (ft)	Max (ft)	%	Avg (ft)	Max (ft)	
Froehlich	96	10.8	33.2	4	2.2	3.9	0.00
Sturm	86	8.4	50.9	14	5.5	17.7	0.01
MD DOT	89	11.8	200.3	11	1.2	3.0	-0.09
Melville	86	4.3	21.3	14	1.4	3.2	0.08

(2) Benedict, S.T, N. Deshpande, N. M. Aziz and P.A. Conrads (2006)

In this study the USGS in cooperation with the FHWA analyzed 144 bridges in South Carolina. Scour depth predictions were based on hydraulic conditions associated with 100-year flow at all sites and the flood of record at 35 sites. Five published scour equations were used to analyze each substructure including the original Froehlich equation, the modified Froehlich equation, the Sturm equation, the Maryland Department of Transportation equation, and the HIRE equation. Comparisons of predicted and observed scour for all bridge sites led the investigators to conclude that all five of the equations frequently over predicted scour depth, and at times excessively so. The investigators also reported on the difficulty of obtaining representative samples of bed sediment. They cautioned against the use of surface "grab" samples to characterize sediment grain size, suggesting soil borings instead.

(3) Wagner, C.R., D.S Mueller, A.C. Parola, D.J. Hagerty, & S.T Benedict (2006)

This comparative scour study was conducted under the National Cooperative Highway Research Program (NCHRP), and its focus was 15 bridge sites located in the states of Minnesota, Montana, and South Dakota. The scour estimation equations applied to the studied bridges included the Sturm equation, the Froehlich equation, the modified Froehlich equation, and the HIRE equation. Upon comparing the predicted with the observed scour depths, the authors concluded that all methods were unreliable. Mostly, the scour equations over predicted scour depths, often by a factor of 2 to 40 times. However, under certain conditions predicted depths were less than observed depths. The authors cite the failure of laboratory research and one-dimensional models to capture the complexity of field conditions as the major reason for the unreliability of the predictive equations.

SCOUR PRACTICE SURVEY: DESIGN AND RESULTS

During summer 2009, a Scour Practice Survey was sponsored by the New Jersey Department of Transportation (NJDOT) in an effort to assess the varied scour design and evaluation methods used by transportation agencies. The survey objectives were threefold: (1) to compile an updated summary of scour practice as related to HEC 18; (2) to query about modified or alternative methods for estimating scour depth; and (3) to identify potential best practices that might be adopted in New Jersey.

The ten-question survey was designed and administered by the New Jersey Institute of Technology (NJIT). It queried agencies about scour design standards, experiences with failures, monitoring programs and countermeasure preferences, among other things. In an effort to maximize response rate, participants were given the choice of several response modes, including direct on-line (to a server), email attachment, mailed hard copy, or any combination of these. Respondents were also encouraged to forward files and links describing local scour practice.

NJDOT distributed the survey to all the State Bridge Engineers via the AASHTO Bridge Committee network in late July 2009. Reponses began to accumulate on the NJIT server immediately. Over the next 60 days, response to the Practice Survey was notably strong with a total of 35 responses received, representing a nearly 70% response rate. Some respondents also forwarded failure data, photos, and design standards and specifications. The authors believe that the favorable response rate reflects, in part, a growing desire by states to seek alternatives for the analysis tools in HEC 18.

The results of the Scour Practice Survey are summarized in **Table 2**. The first question serves to confirm the breadth of the scour problem nationally, with 68% of agencies responding that they have had bridges fail due to scour, either by outright failure or by preemptive replacement. The most common type of scour erosion reported in the survey was local (23 responses), followed by meandering (17), contraction (16), debris (15), and degradation (14). Overtopping was reported by only six agencies as a problem. About 40% of the respondents indicate that they have installed fixed instrumentation to measure scour at abutments or piers, while only 17% have actually generated any summaries that compare predicted scour with field measurements. A similar number of agencies report that they have undertaken either field or laboratory measurement of erosion rates for soil or rock materials.

Among the most interesting result of the survey was the response to the Question 6, which asked whether there was a need to modify current HEC 18 design procedures. An overwhelming 79% of the agencies responded in the affirmative. Consistent with this response, nine agencies indicate that they are now using modified or alternative scour analysis methods for new bridges, while 11 states indicate that they employ modified/alternative analysis methods for existing bridges.

The final two questions provide insight about natural and artificial scour protection means currently in use. Slightly over half (54%) of the agencies consider the effects of natural armoring in their scour computations. Natural

TABLE 2: Summary of Scour Practice Survey Results

<table>
<tr><td>Q1: Have you had any bridges that have failed due to scour, including outright failures or preemptive replacements?</td><td colspan="3">Yes = 69%</td><td colspan="4">No = 31%</td></tr>
<tr><td>Q2: In your experience, what are the most prevalent types of erosion that have caused failure and/or created potential danger of failure? (may check more than one; no. of responses is shown)</td><td>Local = 23</td><td>Meandering = 17</td><td>Contraction = 16</td><td>Debris = 15</td><td>Degradation = 14</td><td>Overtopping = 6</td><td>Other = 5</td></tr>
<tr><td>Q3: Have you generated any summaries that compare field measurements with predicted scour (either published or unpublished)?</td><td colspan="3">Yes = 17%</td><td colspan="4">No = 83%</td></tr>
<tr><td>Q4: Have you installed fixed instrumentation to measure scour at abutments or piers (either automated or semi-automated)?</td><td colspan="3">Yes = 43%</td><td colspan="4">No = 57%</td></tr>
<tr><td>Q5: Have you made any field or laboratory measurements of erosion rates of soil or rock materials for the purpose of scour evaluation?</td><td colspan="3">Yes = 17%</td><td colspan="4">No = 83%</td></tr>
<tr><td>Q6: In the light of your experience of different types of bridge scour, is there a need to modify current HEC-18 design procedures?</td><td colspan="3">Yes = 79%</td><td colspan="4">No = 21%</td></tr>
<tr><td rowspan="2">Q7: What scour procedures and equations do you use in the design of new bridges?</td><td colspan="3">Standard FHWA Version</td><td colspan="4">Modified/Alternate Vers.</td></tr>
<tr><td colspan="3">75%</td><td colspan="4">25%</td></tr>
<tr><td rowspan="2">Q8: What scour procedures and equations do you use in the evaluation of existing bridges?</td><td colspan="3">Standard FHWA Version</td><td colspan="4">Modified/Alternate Vers.</td></tr>
<tr><td colspan="3">69%</td><td colspan="4">31%</td></tr>
<tr><td>Q9: Do you consider the effects of natural armoring in your scour computations?</td><td colspan="3">Yes = 54%</td><td colspan="4">No = 46%</td></tr>
<tr><td>Q10: Does your agency have a preference for particular kinds of scour counter-measures? (may check more than one; no. of responses is shown)</td><td>Riprap = 32</td><td>Gabion = 15</td><td>Debris Deflec./ Removal = 11</td><td>Foundation Strengthen. = 9</td><td>Articulated Conc. Blocks = 6</td><td>Other = 5</td><td>Concrete Pavement = 3</td></tr>
</table>

armoring occurs when a residual layer of coarse particles is exposed on the stream bed due to erosion and removal of fines. With regard to scour countermeasures, riprap remains the preferred choice by more than a 2:1 ratio. Gabions, debris deflection/removal, and foundation strengthening were the next most applied countermeasure methods. A small minority of the agencies report use of articulated concrete blocks, concrete pavement, or "other" methods.

Those states that are currently using modified or alternate scour analysis methods also generously furnished supporting documentation. In some cases, the method changes were for internal agency use only. These included: (1) use of 100-year flows for existing bridges as a maximum; (2) reduction of the factor of safety of the Froehlich equation; and (3) guided application of engineering judgment.

Other states have published formal design standards and/or rigorous scientific studies supporting their deviations from the standard methods in HEC 18. Selected examples of such modified or alternate scour analysis methods are listed and briefly described in **Table 3**. Reference links are also provided.

NEW JERSEY'S SCOUR PROGRAM

In 1990, NJDOT launched a robust statewide Scour Evaluation Program to assess the nearly 2,400 existing state and county highway bridges over waterways. Based primarily upon underwater inspection reports, the Stage I screening studies initially identified 313 state-owned bridges as potentially scour susceptible. In-depth Stage II scour evaluations were then carried out in four phases following the analysis procedures described in HEC 18. Upon completion of the Stage II evaluations, a total of 165 state bridges were determined to be scour critical.

In 2006, the Department launched a Plan of Action for the state's scour critical bridges. The Plan addressed corrective work for all scour critical structures, which is currently underway. The Plan also prescribed a new real-time flood monitoring program for bridges on the State Watch List to help safeguard the traveling public until corrective work was completed. The real-time monitoring program is Internet-based and is currently in operation. It is triggered by flood warnings and stream gauges located in the major watersheds around the state. Field crews are automatically dispatched to potentially affected bridges, and they are authorized to take preventive and/or corrective actions, as required. The real-time program is a cooperative effort between NJDOT's Structural Evaluation Group, Operations Group, and Regional Maintenance Engineers.

The Department has also recently engaged the USGS West Trenton Office to conduct erosion monitoring at selected scour critical bridges. These bridges, located along watercourses with high environmental sensitivity, have no history of observed scour and were placed on the critical list solely based on HEC 18 analysis methods. Assuming that no significant erosion is recorded over a several year period, consideration will be given to removal from the scour critical list.

A research study is also currently underway by NJIT to review and revise NJDOT's scour evaluation. Although the study is only about half-completed, the

TABLE 3: Selected Examples of Modified or Alternative Scour Evaluation Methods across the U.S.

State	Method Description
Alabama	This USGS Scientific Investigations Report published in 2008 provides an alternate method to assess scour depth in the Black Prairie Belt soil, a consolidated, highly cohesive, organic clay within Alabama's Coastal Plain. Envelope curves are presented based on observations of clear-water contraction scour at 25 bridge sites. Related link: http://pubs.usgs.gov/sir/2007/5260/
Illinois	Illinois DOT permits reduction in scour depth computed by HEC 18 methods for bridges founded in cohesive soil or rock. Such reductions are graduated from 0 to 100%, depending on soil strength or degree of lithification of rock. Related link: http://www.dot.state.il.us/bridges/brmanuals.html
Maine	This USGS Water Resources Investigations Report collected and analyzed pier-scour data for eight bridges across Maine over a four year period. Observed maximum scour depths were compared with predictions using the CSU equation in HEC-18. The relation performed well for rivers in Maine, and MaineDOT currently uses it for evaluation of existing and new bridges. Related link: http://me.water.usgs.gov/reports/wrir02-4229.pdf
Pennsyl-vania	PennDOT scour design method recognizes the variable erosion behavior of geologic materials in scour design. It establishes three classifications: sound bedrock, erodible bedrock and coarse soil (gravel, cobbles and boulders), and specific embedment depths and footing details are prescribed for each. Related link: ftp://ftp.dot.state.pa.us/public/PubsForms/Publications/PUB%2015M.pdf
South Carolina	A recently published USGS Report of Investigation extends the earlier 2006 USGS study described above in "Comparative Scour Studies." It recommends use of envelope design curves to estimate scour depth. The curves are rigorously justified with field observations and laboratory data. SCDOT has incorporated the method into their latest scour design standards. Related links: http://pubs.usgs.gov/sir/2009/5099/ http://www.scdot.org/doing/pdfs/requirements2009.pdf
Texas	This comprehensive study performed by Texas Transportation Institute summarizes a new method to assess a bridge for scour. It uses three levels of bridge scour assessment (BSA 1, 2, & 3) and erosion classification charts. Scour vulnerability is determined by comparing the predicted scour depth with allowable scour depth of the foundation. The method is relatively simple to apply, and it overcomes some of the over-conservatism in current methods. Related link: http://tti.tamu.edu/documents/0-5505-1.pdf

NJIT Team has already reached several preliminary conclusions:

(1) The HEC 18 equations have led to excessively conservative design values for some state bridges. Revised computational procedures are needed to permit designers to adjust safety factors according to field performance and risk level.

(2) The bed materials in New Jersey's rivers are geologically diverse, and they often contain scour-resistant materials such as boulder trains, stiff clay, and shale.

Revised analysis procedures are being developed to reflect New Jersey geology, which will reduce predicted scour depths for some sites.
(3) A review of the Stage II studies indicates that better standardization is needed in sampling and analysis of the stream bed materials. It appears that D_{50} values used for analysis are biased towards finer grain sizes on account of wide use of surface grab sampling and lack of consideration for cobbles and boulders when present. In view of these findings, an important focus of the current study will be to develop a viable and uniform geotechnical sampling protocol so that scour analyses are based on a deeper vertical profile of actual stream bed sediments.
(4) The stream discharges used in the Stage II studies were developed using different methodologies (e.g. extreme value, regression analysis) and data from different agencies (e.g. FEMA, USSCS, USGS). Recently, the USGS has published a report (Watson and Schopp, 2009) providing an updated methodology for estimating flood magnitude and frequency for New Jersey streams, which reflects changes in factors such as impervious cover and population density. Differences between the flow data generated using the new USGS model and the original Stage II data are being investigated and assessed.

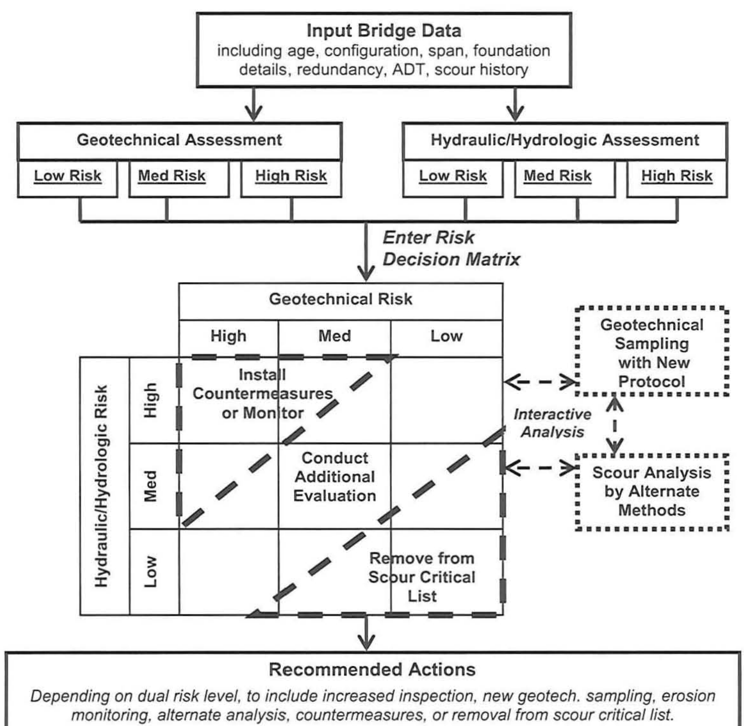

FIGURE 1: NJDOT Decision Matrix Model for Scour Evaluation

A "Decision Matrix Model" is also being developed as part of NJDOT's scour research project. The tiered, risk-based model will allow the Department to reassess the bridges on the state's Scour Critical List. It is expected its application will better prioritize bridges and permit selection of more appropriate corrective actions. An abbreviated flowchart of the model is shown in **Figure 1**. The first step is to input relevant data including age, configuration, span, ADT and scour history. The next step is to perform geotechnical and hydraulic/hydrologic risk assessments using existing and new data. This information is the entered into the Risk Decision Matrix to determine overall scour risk. In the final step, one or more recommended actions are taken depending on risk level, which may include modification of inspection frequency, additional analyses, erosion monitoring, installation of countermeasures, or removal from the list.

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Hydraulic Variables for Scour using HEC-RAS

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ABSTRACT

The Maryland State Highway Administration, Office of Structures has adopted a proactive approach with respect to the determination of hydraulic variables for computing scour at structures, most notably through the development of the ABSCOUR program (*Reference 2*). Scour analyses are very sensitive to hydraulic variables and the Office of Structures places great emphasis on the hydraulic model development, both in its ABSCOUR training workshops and in its design reviews. The following lists several areas of concern:

- Careful consideration of potential tailwater conditions and their effect on scour.
- Development of reasonable hydraulic water surface profiles through the structure.
- Review of design/check flood flow distributions from HEC-RAS upstream, downstream and at the structure.

The third bullet, which addresses flow distribution, represents the main focus of this paper. Flow distribution has been identified as a key component of the effort to compute realistic scour depths. The Office of Structures asked KCI to develop a procedure within HEC-RAS (*Reference 4*) involving geometry file adjustments to provide a more reasonable progression of flow from upstream of the approach section, to downstream of the structure. The flow progression is viewed in the context of the left overbank, main channel and right overbank. For instance, percentage change in flow is viewed in the main channel in each successive section such that significant changes are avoided (say 20% or less change) from one section to the next downstream section. Three (3) typical cases are defined to demonstrate the flow distribution adjustment process. One case (Case 3) is included to explain the process of balancing flow through the bridge versus flow overtopping the roadway. A comparison is made of the flow distribution in a non-adjusted channel reach versus an adjusted channel reach. The significance of these flow distribution adjustments is illustrated by applying Laursen's live-bed scour equation for estimating contraction scour at a bridge. The contraction scour estimate was reduced significantly by making reasonable adjustments to the hydraulic model.

INTRODUCTION

The focus of this paper is on the development of a reasonable flow distribution for evaluating scour at a bridge. However, three conditions are necessary in the use of the approach discussed herein: The first is that *one-dimensional flow modeling is appropriate for modeling the structure.* (In Maryland, It has been our experience that the great majority of hydraulic models for determining variables for scour are performed using the one-dimensional HEC-RAS model. This is not to say that certain complex flow conditions do not require a two-dimensional model; however, these cases have been relatively rare.) The second is that *potential tailwater effects on the structure have been thoroughly investigated.* Inaccurate tailwater elevations can have a significant effect on scour results. Often, we have found that tailwater investigations do not extend far enough downstream, specifically on low-gradient streams. Normal depth assumptions for downstream boundary conditions should include a tailwater sensitivity analysis. Downstream control structures such as bridges, culverts and dams should be assessed for their effect on tailwater. Complex hydraulic conditions such as a downstream confluence or tidal flow may necessitate investigating multiple tailwater scenarios. The third condition is that *reasonable hydraulic profiles through the structure have been computed.* The flow distribution adjustments depend heavily on the hydraulic profiles through the structure as initially computed by HEC-RAS. The discharges in the channel and overbanks through the structure provide the target flow distribution values for the upstream adjustments.

It should be noted that the HEC-RAS flow distribution option does not perform any adjustments to the flow; rather it simply divides the initial flow distribution (based on conveyance) into the number of flow tubes specified by the user. Therefore, flow adjustments as described in this paper are necessary to provide for a reasonable progression of flow. It is emphasized that the adjustment process should be carried out by experienced HEC-RAS users who understand the significance and validity of such adjustments.

Selection of Approach Section

There are a number of desirable attributes to look for in selecting the location of the approach section: located at a station about one bridge length upstream; located upstream of the contracted flow pattern created by the bridge; representative of the channel and flood plain characteristics of the upstream cross-sections; and selection of a cross-section where the channel flow is essentially parallel to the flood plain (valley) flow. For many stream crossings, and especially for smaller channels, there may not be one section that satisfies all of the above criteria. In such cases, judgment is needed to select the most appropriate section. If there is no desirable section available, it may be helpful to perform a sensitivity analysis by comparing the scour results from two candidate approach sections. This can be accomplished efficiently using the ABSCOUR program.

FLOW DISTRIBUTION ADJUSTMENTS

The goal of the flow distribution adjustments is to provide a reasonable progression of channel and overbank flows from upstream of the approach section to downstream of the structure. Due to the nature of the flow distribution adjustments, a specific scour plan should be created in HEC-RAS to separate scour hydraulics from other hydraulic evaluations such as those intended for permitting purposes. There are three (3) typical flow distribution cases: Case 1 - Bridge abutments located at or near the channel banks, no overtopping of structure; Case 2 - Abutments set back from channel banks, no overtopping of structure and Case 3 - Abutments set back from channel banks, with overtopping of the structure. The following discussion outlines the general flow dsitribution adjustment approach:

1) Target flows, as described in this paper, are the flows in the left overbank, channel and right overbank sections at the bridge as computed by the initial HEC-RAS run. Determine target flow distribution values through the structure using the *Flow Distribution Locations* option under the steady flow simulation button in HEC-RAS (for abutments that are set back from channel). If abutments are at or near the channel banks, assume 100% of the flow is in the channel. For overtopping flow, the target values should be adjusted to account for any weir flow that is on the left overbank, channel and right overbank at the structure, dividing the total weir discharge provided by HEC-RAS based on proportions of the weir length. The HEC-RAS prercentage flows in the left overbank, channel and right overbank for Case 3 (with overtopping) are for flow through the bridge only and they must be recomputed based on total discharge (see Case 3 example).
2) Look for trends in the flow distribution that HEC-RAS computes prior to any adjustments by reviewing Q percent left, Q percent channel and Q percent right in a user-defined HEC-RAS table. Look for (1) reasonably consistent flow in the overbanks and the channel for sections upstream of the influence of the structure or (2) a consistent flow contraction that shows flow moving into the channel as it approaches the structure. The latter scenario may require only minor adjustments in the flow distribution.
3) Start flow distribution adjustments several sections above the approach section selected for the scour evaluation. Beginning on overbanks areas, adjust Manning's roughness up or down and/or make the edges of the floodplain ineffective to redistribute flow. Flow prior to the contraction should stay fairly consistent, with percent flow changes between successive sections within an overbank or in the channel that does not exceed 15%. For larger streams and rivers, a maximum 20% change may be more appropriate.
4) For a typical flow contraction (Cases 1 and 2), the main channel discharge should steadily increase in the direction of flow as flow is pushed into the channel from the overbanks. Changes to roughness and/or ineffective area limits can be used to achieve this pattern.
5) Overtopping conditions (Case 3) need to be carefully considered in terms of the downstream flow distribution since tailwater elevation and the hydraulics of the bridge can be affected. Immediately downstream of the bridge, overbank flow should be limited to the flow overtopping the road and/or bridge. In typical situations, the flow through the bridge cannot expand quickly enough to be effective on the overbanks just below the structure. A blocked obstruction may be used to reflect this condition; that is, reduce the amount of flow in the section immediately downstream of the bridge. To add flow to an overbank area, the

elevation of the floodplain can be lowered. This may be necessary in a situation where HEC-RAS places all the flow in an incised channel, but overtopping flow on a roadway approach is known to exist.

6) If the bridge hydraulics changes due to the downstream flow distribution adjustments (revised tailwater elevation or flow through the bridge, etc.), a second iteration in the adjustments may be needed to establish new target values (See Example Case 3). If the percent of the total flow that overtops the road is 15% or less, there probably will not be much of a change in the target values and no changes to the flow distribution would likely be required.

Changes to the HEC-RAS geometry to adjust the flow distribution must be reasonable. For instance, adjustments to Manning's roughness values in the channel or overbank areas must be within the bounds of what could reasonably be expected based on site conditions and engineering judgement. The adjustments should result in relatively minor changes in water-surface elevations as compared with the initial condition.

Sample Case 3 Flow Distribution Adjustments (Abutments set back from channel banks, with overtopping of the structure)

For illustrative purposes, the following provides a synopsis of the flow distribution approach for Case 3. The HEC-RAS Flow Distribution Output table for the bridge shows flow percentages of 23%, 56% and 21% respectively for the left overbank, channel and right overbank for the 87% of the total flow that passes through the bridge (13% overtops road from left overbank). Since the HEC-RAS ouput (23%, 56%, 21%) is for flow through bridge only, the percent of *total* flow at the bridge (including weir flow) must be computed. Percentages based on total flow (203.9 cms or 7200 cfs) are used as target values to adjust the approach flow distribution. Since the overtopping flow is entirely on the left overbank in this example, this overtopping flow percentage of flow is added to the QLOB percentage of flow. If the overtopping flow was distributed over the approaches and bridge deck, the percentage overtopping flow could be divided between the LOB, channel and ROB based on proportions of total weir length to estimate percent flows. Figure 1 illustrates the bridge target values:

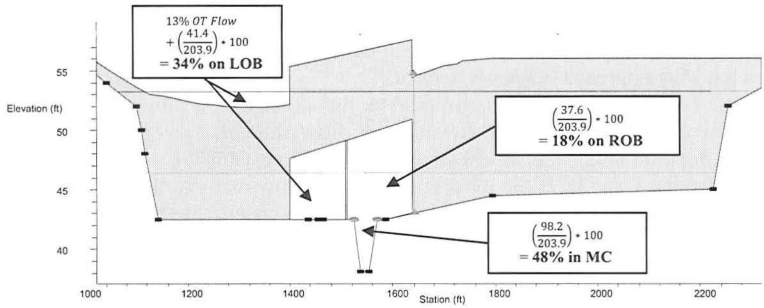

Figure 1 - Bridge Section with Target Values

The flow distribution at the river stations for the initial HEC-RAS run is presented below. Target values of 48% of the flow in the main channel (MC), 34% on the left overbank (LOB) and 18% on the right overbank (ROB) at the bridge was selected as the basis for the flow adjustments in the upstream river stations. Note that this example assumes that the flow distribution upstream of RS 6000 as computed initally by HEC-RAS is reasonable. A comparison of the target values to those determined initally by HEC-RAS upstream of the bridge indicates that some adjustments should be made to provide for a more reasonable progression of flow, as illustrated in Table 1:

Table 1 – Initial Flow Distribution from HEC-RAS

River Station (RS)	Percent LOB	Percent MC	Percent ROB	Comments
7000	14	50	36	Reasonable distribution
6000	14	50	36	Begin adjustments
5000	16	31	53	Too little flow in MC, too much flow on ROB
4500 Approach XS	**25**	**25**	**50**	Too little flow in MC, too much flow on ROB
3500	47	25	28	Too little flow in MC, too much flow on ROB and LOB
2000	57	32	11	Too little flow in MC, too much flow on LOB
1500 Bridge XS	***34***	***48***	***18***	***Bridge Target Values***
1000	50	36	14	Too little flow in MC, too much flow on LOB
100	17	79	5	Too much flow in MC, too litte flow on LOB

Table 2 provides the initial flow distribution for comparison to the adjusted flow distribution:

Table 2 – Initial Flow Distribution and Adjusted (Adj.) Flow Distribution

River Station	Percent LOB		Percent MC		Percent ROB	
(RS)	Initial	Adj.	Initial	Adj.	Initial	Adj.
7000	14	-	50	-	36	-
6000	14	17	50	39	36	44
5000	16	20	31	42	53	38
4500 Approach XS	25	23	25	43	50	35
3500	47	27	25	49	28	24
2000	57	30	32	50	11	20
1500 BR (Targets)[1]	***30***		***51***		***19***	
1000	50	29	36	55	14	16
100	17	28	79	64	5	8

[1] Note that the target values changed slightly due to decreased bridge tailwater.

The following discusses how the adjustments were made. The simplest approach to redistributing the flow is to make adjustments to Manning's roughness values within HEC-RAS using the Manning's roughness table under Geometric Data. The initial roughness values in the channel or overbank can either be raised to reduce the flow or lowered to increase the flow, resulting in flow being shifted from one portion of the cross section to another. The adjusments were initiated at RS 6000, working in the downstream direction. Notice that Table 1 shows too little flow in the channel from RS 5000 to RS 2000. Therefore, channel roughness values were decreased for these river stations to shift flow to the channel, as shown in Table 3. There is too much flow is on the right overbank from RS 5000 to RS 3500 and roughnesses were raised to shift flow. The end result is that flow was shifted from the right overbank to the channel in order to produce the pattern of the contraction of the flow that is expected to occur. Table 3 highlights the roughness changes that were made to redistribute the flow in this example:

Table 3 – Manning's Roughness Adjustments

River Station	ROB		MC		LOB	
(RS)	Initial n	Adj. n	Initial n	Adj. n	Initial n	Adj. n
6000	0.1	0.08	0.04	0.05	0.1	0.08
5000	0.1	0.09	0.04	0.033	0.1	0.15
4500 Approach XS	0.1	0.18	0.04	0.03	0.1	0.16
3500	0.1	0.18	0.04	0.03	0.1	0.16
2000	0.1	0.18	0.04	0.031	0.1	0.08
1500 BR	-	-	-	-	-	-
1000	0.12	0.12	0.04	0.035	0.12	0.14
100	0.12	0.08	0.04	0.055	0.12	0.08

Ineffective Flow and Blocked Obstructions

Additional adjustment techniques include moving ineffective flow limits, the placement of blocked obstructions and the lowering of overbank elevations. In this example, an ineffective flow limit was added to RS 3500 to reduce left overbank flow and the right overbank ineffective limit was moved out at RS 2000 to increase flow here. This technique can be used to shift the overbank flow when the desired flow redistribution cannot be achieved solely by changing the Manning "n" roughness. A blocked obstruction was added on the left overbank at the bridge upstream bounding section (RS 2000) to reduce flow. In addition, a blocked obstruction was added on the left overbank at the bridge downstream bounding section (RS 1000) to reduce the flow such that it approximately matched the weir flow over the approach roadway. This situation often occurs when HEC-RAS models approach roadway overtopping. The 1-D HEC-RAS model is unable to recognize the fact that the flow cannot expand quickly enough to make the entire left overbank effective at RS 1000. Ineffective flow area blocks could also be used.

In some situations where there is overtopping flow, adjustments to the flow distribution downstream of the crossing can change the tailwater on the bridge, which in turn, can change the flow through the bridge. Therefore, the adjusted HEC-RAS flow distribution through the bridge should be checked to see if the target values require revision. For instance, a lower tailwater could increase bridge flow and reduce overtopping flow, thereby altering the target values. This is the main reason for extending the flow distribution adjustments downstream of the crossing, especially in overtopping situations. Finally, lowering floodplain elevations may serve to increase overbank flow. This approach may be helpful at a bridge with overtopping flow where the channel is incised. The HEC-RAS model may indicate that there is no overbank flow, but it is known that overbank flow occurs. Changing ground point elevations represents the least preferred adjustment method due to the potential for water-surface elevation changes that may exceed the minor changes that typically would be seen with the previous techniques.

Case 3 Summary

The distribution based on the revised (lower) tailwater elevation is still appropriate, since the target values changed only slightly. This is due to the fact that the amount of overtopping flow is fairly low (less than 15%). Notice that the channel portion of the flow distribution at the approach section has changed dramatically from the initial condition to the adjusted condition. Table 2 indicates that at the approach section (RS 4500), the channel flow increased significantly from 25% to 43% (from 50.9 cms to 86.8 cms or 1798 cfs to 3065 cfs). The higher approach channel discharge results in a lower scour depth in the channel at the bridge as compared to the scour depth without flow distribution adjustments.

Considering the live-bed contraction scour equation as presented in "Hydraulic Engineering Circular 18" (*Reference 3*) and assuming k1=0.64 (some suspended bed material discharge):

$\frac{Y2}{Y1} = \left(\frac{Q2}{Q1}\right)^{\frac{6}{7}} \left(\frac{W1}{W2}\right)^{k1}$ $\quad Ys = Y2 - Yo$

Initial Flow Distribution: $\frac{Y2}{4.0} = \left(\frac{98.2}{50.9}\right)^{\frac{6}{7}} \left(\frac{16.8}{13.7}\right)^{0.64}$

$$Y2 = 8.0\,m$$

$$Ys = 8.0 - 4.6$$

$$Ys = 3.4\,m\ (11.2\,ft).$$

Adjusted: $\frac{Y2}{4.0} = \left(\frac{102.7}{86.8}\right)^{\frac{6}{7}} \left(\frac{16.8}{13.7}\right)^{0.64}$

$$Y2 = 5.3\,m$$

$$Ys = 5.3 - 4.5$$

$$Ys = 0.8\,m\ (2.5\,ft)$$

The primary reason for this change is the decrease in the ratio of the main channel flow ($\frac{Q2}{Q1}$) from the initial flow distribution condition to the adjusted condition. For this case, the decrease in the contraction scour depth is very significant. There would be an even greater change in the ABSCOUR computations for abutment scour, since contraction scour is used in the computations for abutment scour.

ABSCOUR VERSION 9

ABSCOUR 9 is a computer program developed by the Maryland SHA, Office of Structures for evaluating scour at bridges and bottomless arch culverts. The program serves as an analytical tool to assist the user in identifying and utilizing the appropriate bridge geometry, hydraulic factors, stream morphology and soil/rock characteristics to evaluate scour at structure foundations. The program estimates scour for both live-bed and clear-water conditions. It evaluates pressure and contraction scour as well as local pier and abutment scour. The user can also input information regarding lateral channel movement and aggradation/degradation to incorporate these factors into the scour evaluation. For the most part, the equations used in ABSCOUR are based on the methodology developed by the FHWA as presented in HEC-18. A Users Manual for ABSCOUR 9 is included in the Office of Structures "Manual for Hydrologic and Hydraulic Design" (*Reference 1*). ABSCOUR 9 also provides guidance and help for each cell used in the input menus.

Verification and calibration efforts of the ABSCOUR methodology have been on-going for the last 10 years. These include:

- Cooperative studies with FHWA, utilizing the J. Sterling Jones Hydraulic Laboratory in McLean, Virginia,
- Cooperative studies with the US Geological Survey using a database of measurements of clear water abutment scour collected at South Carolina Bridges.
- Continuing evaluation of the method within the Office of Structures on a bridge by bridge basis to determine ways and means of improving the accuracy of the results and to facilitate its use by others. The Office of Structures presents periodic workshops on the use of the program.

The accuracy of the answers obtained (scour depths) depends on the accuracy of the input information, the selection of the most appropriate analytical methods available in the program and the user's judgment. The latest version, ABSCOUR 9, along with the "Manual for Hydrologic and Hydraulic Design" is available at no cost at the web site: www.gishydro.umd.edu.

Input information

As discussed earlier in this paper, the most important information for the scour evaluation is a reasonable water surface profile to determine water surface elevations and flow distributions in the approach, bridge and downstream cross-sections of the study reach. The Maryland SHA uses the HEC-RAS program for this purpose. The stream morphology report serves to investigate the characteristics of surface soils and the probable types of scour (live-bed or clear-water) for various flood discharges under consideration. It also provides information on the potential for aggradation/degradation and lateral stream movement. The preliminary plans describe the proposed bridge geometry. Borings are taken at each proposed foundation element along with at least one channel boring for information on subsurface conditions. ABSCOUR can consider the effect of up to three layers of soil/rock in evaluating clear-water scour.

Output information

The program prints a detailed scour report for determining contraction and abutment scour. A separate module serves to estimate pier scour, taking into consideration the extent of contraction scour. The program also prints a complete scour cross-section for the channel and flood plain sections under the bridge. A Utilities module is available for various other items of interest, such as sizing riprap for abutment installations.

Sensitivity Analysis

A powerful attribute of ABSCOUR is the ability to conduct sensitivity analyses of the input parameters. The user can test the effect of various factors (such as soil particle size) on scour depths and can print out a complete report for each factor in a matter of a few minutes. Over-ride features serve to allow the user to select procedures and parameters for computing scour other than the ones selected by ABSCOUR. The Office of Structures recommends caution in the use of over-rides. This approach is best left to engineers with a practical understanding of the inter-relationships of the various factors affecting the computation of scour. Design considerations for scour should include all factors affecting the bridge foundations as discussed in the Manual for Hydrologic and Hydraulic Design.

CONCLUSIONS

Some bridge owners are concerned that the HEC-18/ABSCOUR 9 methodologies may over-estimate scour depths. Since these methodologies have been developed to evaluate worst-case scour conditions, they can be expected to produce conservative but reasonable results. To assure the results are reasonable, the engineer needs to verify that the appropriate analytical methods are used and that the input parameters are representative of the field conditions. The foregoing discussion relating to developing a HEC-RAS model with a reasonably consistent flow distribution pattern is a good example of what can be done to improve the accuracy of scour estimates. Experienced HEC-RAS users should be able to make flow distribution adjustments in a relatively short time frame, say two to three hours. Other reasons for high estimates of scour may include:

- Over-estimating the design discharge. This may occur in the use of hydrologic models, such as TR-20, if the models are not constructed properly,
- Selection of overly-conservative calibration factors for scour computations,
- Inaccurate measurements/estimates of soil properties,
- Addition of all the various elements of scour (contraction scour, pressure scour, pier scour, channel movement, bend scour, degradation, etc.) to compute total scour when it may not be reasonable to assume that all possible types of scour will occur at the same time. These combinations should be evaluated on a case by case basis.

The Maryland State Highway Administration, Office of Structures has spent considerable time and effort in working with other agencies to evaluate and calibrate the ABSCOUR 9 Program. Careful attention to obtaining accurate input information, and following the guidance in the user's manual should result in reasonable estimates of scour.

REFERENCES

1) Maryland State Highway Administration, "Manual for Hydrologic and Hydraulic Design", March 2010.
2) Maryland State Highway Administration, ABSCOUR Version 9, Build 2.1, January 2010.
3) U. S. Department of Transportation, Federal Highway Administration, (2001) "Evaluating Scour at Bridges, Fourth Edition" FHWA-IP-90-017, Hydraulic Engineering Circular No. 18.
4) U.S. Army Corps of Engineers, Hydrologic Engineering Center, HEC-RAS River Analysis System Version 4.0.0, March 2008.