A Temporal, Spatial Pier Scour Model

PEGGY A. JOHNSON AND RICHARD H. MCCUEN

Civil engineers are faced with problems involving all aspects of aging infrastructure. One area in critical need of attention is the design, monitoring, and maintenance of bridge piers where local pier scour is a concern. The scour depth at a particular time during the design life of a bridge is a function of the flow-duration characteristics of storms that occurred before the time of interest. Current scour models and equations provide a single-valued estimate of the ultimate scour depth for a design discharge; the engineer does not have a time-dependent estimate of the scour depth. The objective of this study was to develop a conceptual, time-dependent pier scour model and a methodology to assess the risk of bridge failure because of pier scour over the life of the bridge. The time-dependent pier scour model was formulated and calibrated with laboratory data and observations as well as field observations. The time dependency enables the engineer to estimate the scour depth at any time during the life of the bridge.

Civil engineers are faced with problems involving all aspects of aging infrastructure—from monitoring and maintenance to replacement and design. One area in critical need of attention is the design and protection of bridge piers where local scour is a concern.

Current literature contains many equations and models for predicting the depth of scour that can be expected around a bridge pier. Most of these models provide a single maximum value for the scour depth rather than the scour depth as a function of time. There are two significant problems with this. First, these models are time invariant in that they cannot estimate the number of years that will be required to scour to the computed depth; thus, the time-invariant models do not suggest the safe design life associated with the predicted scour depth. Second, at a bridge where the current scour depth has been measured, the existing models do not enable the engineer to estimate the future progress of scour.

The goals of this study were (a) to formulate a model that could account for the temporal and spatial variation of scour at a bridge pier, and (b) to use the model to determine the effect of pier geometry, channel bed materials, and hydrologic characteristics on pier scour.

BRIDGE PIER SCOUR

Scour is the erosive action of water in excavating and carrying away material from the channel bed (*1*). An obstruction, such as a bridge, interferes with the flow of the stream, which changes the flow patterns at the obstruction. This results in a deepening of the scour hole around the bridge pier beyond the level that would naturally occur from degradation and general scour. This is commonly referred to as "local scour."

Department of Civil Engineering, University of Maryland, College Park, Md. 20742.

The flow pattern around the bridge pier is complex. As the streamflow approaches the pier, an adverse pressure gradient caused by the pier drives a portion of the approach flow downward just ahead of the pier. This downflow is thought by some researchers to be the main cause of local scour around bridge piers (*2*). According to Chiew (*2*), the rate of erosion of the scour hole is directly associated with the magnitude of the downflow, which is directly related to the velocity of the approaching river flow. The jet of water in the downflow can be used to represent the strength of the vortex. The downflow is an integral part of the vortex that forms in the scour hole. Although the vortex significantly contributes to the erosion, the vortex flow pattern is highly correlated with the downflow; therefore, for the purpose of estimating the depth of scour, a significant correlation between downflow scour and vortex scour is assumed in the scour model. Because the downflow can be described in terms of a measurable quantity, velocity, this description was chosen as the best alternative to modeling the vortex.

The many parameters that affect bridge pier scour may be classified as streamflow characteristics, such as velocity and flow depth; bed material characteristics, such as the sediment size and distribution; and bridge pier characteristics, such as the width, shape, and length of the pier. Although the model uses these parameters as input, these parameters are defined elsewhere (*3–5*).

TEMPORAL PIER SCOUR MODEL

A conceptual, time-dependent pier scour model was developed to assess the effect of various hydrologic conditions and other parameters on pier scour. The model determines the temporal development of a scour hole just upstream of a bridge pier. The functional forms used to represent the processes were selected so that the model would reflect relationships suggested by laboratory observations and measurements in the literature (*2,3,6–9*).

The purpose of developing a time-dependent model was to provide a method of assessing the effects of changes in parameters—such as pier width, flow velocity, and riprap size—on the scouring process as storms of various flow velocities and durations move past a bridge pier. Such a conceptually based model can be used for either project evaluation (analysis) or design (synthesis).

Model Components

The model consists of three main components, which estimate (a) the downflow velocity, (b) the termination velocity (anal-

ogous to a velocity of incipient motion for a channel bed), and (c) the erosion rate and scour depth. A brief description of each component follows. Further detail is provided by Johnson (*10*).

Downward Velocity

The maximum downward velocity is a function of the approach velocity, pier size, and flow depth. An increase in the velocity, pier size, or flow depth would result in an increase in the maximum downward velocity (V_{dmax}). However, on the basis of experimental observation, V_{dmax} probably does not exceed one to two times the approach velocity. Therefore, V_{dmax} may be expressed as an exponential function

$$V_{dmax} = C_1 V_0[1 - \exp(-C_2 bY)] \quad (1)$$

where

C_1 and C_2 = numerical constants,
V_0 = approach velocity,
b = pier width, and
Y = approach flow depth.

The form of Equation 1 was selected because it defines a curve exhibiting the trend suggested by results from laboratory studies.

The maximum downward velocity occurs at some depth beneath the level of the original bed elevation. From this depth to the bottom of the scour hole, the downward velocity gradually decreases from V_{dmax}. The downward velocity at any depth below the location of V_{dmax} within the hole is then some portion of V_{dmax}

$$V_d = KV_{dmax} \quad (2)$$

where V_d is the downward velocity at some depth and K is a function of the scour depth. Using Melville's (*7*) data, the decrease in velocity toward the bottom of the hole can be approximated with a logistic function. The following expression for K provides a rational structure and incorporates variables that are expected to affect the relationship between V_d and V_{dmax}:

$$K = 1 - \frac{1}{1 + \exp(-C_3 D - C_4 b)} \quad (3)$$

where

C_3 = shape parameter,
C_4 = location parameter, and
D = depth of scour.

Using D equal to the current depth of the scour hole enables the value of K at the base of the hole to be computed. Equation 3 assumes that the downward velocity occurs at the level of the original bed elevation, then decreases with depth in the hole. The velocity at the base of the hole for any depth D may then be found by substituting Equation 3 into Equation 2.

Termination Velocity

The downward velocity eventually decreases to a value that is no longer capable of causing erosion in the hole. The velocity at which this occurs is termed the "termination velocity" and is derived using a summation of the primary resisting and erosive forces. At incipient motion of the particles at the bottom of the scour hole,

$$F_r = F_e \quad (4)$$

where F_r is the resisting force caused by the weight of the sediment and F_e is the erosive force caused by the impinging jet of the water in the downward flow just upstream of the pier. The shear stress caused by the impinging jet, as shown in Figure 1, is modeled with the momentum equation

$$\tau = \{(\rho V_d^2 \cos\phi)^2 + [\rho V_d^2(1 - \sin\phi)]^2\}^{0.5}$$
$$\tau = \rho V_d^2 (2)^{1/2}(1 - \sin\phi)^{0.5} \quad (5)$$

where

V_d = downward velocity at the base of the hole,
ρ = density of water, and
ϕ = angle of repose (as shown in Figure 1).

At incipient motion, the termination velocity, V_t (here, V_t is actually the initiation velocity) may be substituted for V_d in Equation 5.

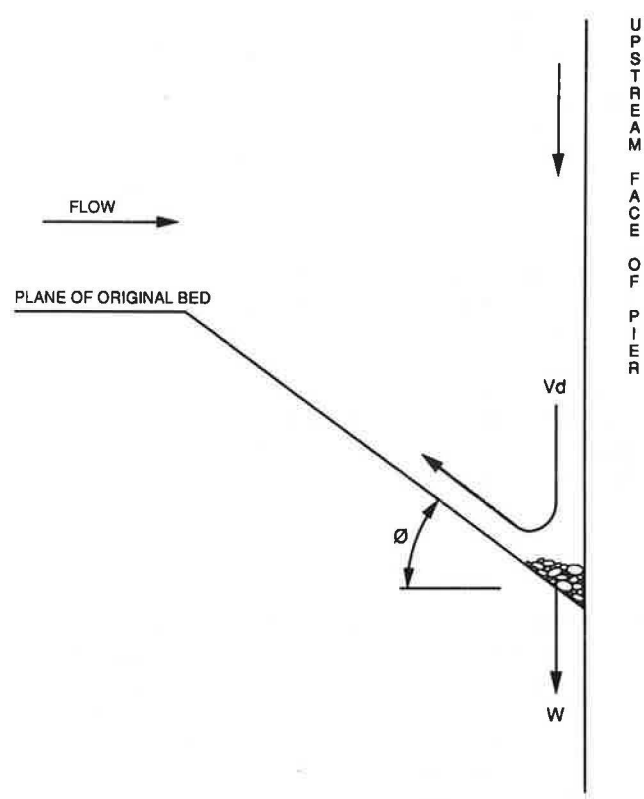

FIGURE 1 Impinging jet at the base of a pier in the scour hole.

Assuming that the particles are spherical with a cross-sectional or projected area of $\pi d^2/4$ (d is the sediment diameter), the weight component along the slope is

$$W = \rho_s g\left[\frac{4}{3}\pi\left(\frac{d}{2}\right)^3\right]\sin\phi \tag{6}$$

where ρ_s is the density of the sediment. Equations 5 and 6 may be substituted into Equation 4 to obtain

$$\frac{1}{6}\rho_s g\pi d^3(\sin\phi) = \frac{1}{4}\pi d^2\rho V_t^2(2)^{1/2}(1-\sin\phi)^{0.5} \tag{7}$$

Solving for the termination velocity in the hole yields

$$V_{tu}^2 = \frac{2\rho_s g d\sin\phi}{3\rho(2)^{1/2}(1-\sin\phi)^{1/2}} \tag{8}$$

where V_{tu} is the unadjusted termination velocity. Equation 8 is valid only for uniform sediments and is similar to White's (*11*) derivation for the threshold condition on a channel bed. Equation 8 must be modified to account for turbulence and circulation factors through the use of a numerical coefficient.

For nonuniform sediments, the termination velocity is also a function of the sediment gradation ($G = d_{84}/d_{50}$), particularly at low velocities. As G increases, the effective termination velocity increases, the erosion rate decreases, and the scour depth decreases. The following equation was developed to represent the termination velocity for uniform and nonuniform sediments:

$$V_t = C_6\left[\frac{2\rho_s g d G\sin\phi}{3\rho(2)^{1/2}(1-\sin\phi)^{1/2}}\right]^{1/2} + C_7 G\exp(-C_5 V_0) \tag{9}$$

where C_6 reflects other factors involved in initiating particle motion, such as the circulation in the hole; and C_5 and C_7 are fitting coefficients.

Erosion Rate

The erosion rate is directly associated with the downward velocity. Erosion rate equations commonly involve the difference between a value of shear stress or velocity and a critical shear stress or critical velocity (*12–14*). Therefore, the following functional form was selected for estimating erosion rates:

$$\frac{\Delta D}{\Delta t} = C_8(V_d - V_t) \tag{10}$$

where

D = scour depth (m),
t = time required to scour depth D (sec), and
C_8 = numerical coefficient.

The downflow velocity (V_d) decreases according to Equation 2 until it reaches the termination velocity (V_t). For clear-water conditions, scour ceases when the erosion rate becomes zero. For live bed scour, scour ceases when the erosion rate equals the rate of sediment transport into the scour hole. Equation 10 may be used to determine the scour depth for a given time increment. The final scour depth is taken as the equilibrium or maximum depth.

Model Calibration

Data obtained from laboratory studies by Melville (*7*) and Chiew (*2*) were used to calibrate the model. The coefficients for Equations 1, 3, 4, and 10 were fit simultaneously using a nonlinear least squares method. Initial estimates of the coefficients, which are required for operating the nonlinear least squares program, were obtained from the literature where possible. An angle of repose of 30 degrees was assumed for the calibration, because this value is typical for the rounded sands found in a river setting. The specific gravity of the sediment was assumed to be 2.65. The optimization procedure resulted in the following coefficients, which are presented in Table 1: from Equation 1, $C_1 = 0.8$ and $C_2 = 134$; from Equation 3, $C_3 = 63$ and $C_4 = 1.05$; from Equation 9, $C_5 = 31.2$, $C_6 = 2$ for live-bed conditions and 0.65 for clear-water conditions, and $C_7 = 1.0$; and from Equation 10, $C_8 = 0.000015$.

For the overall data set, the resulting set of coefficients yielded a correlation coefficient of approximately 0.80 and a standard error–to–standard deviation ratio of about 0.62.

Calibration for Field Scale

On the basis of the model calibrated for laboratory conditions, a model appropriate for field use was fit. The numerical coefficients were adjusted to represent larger piers, flow depths, and flow velocities at actual bridge sites. Those coefficients associated with sediment size were not adjusted because the sediments used in the laboratory experiments were not unlike those that are expected in a river environment.

The coefficients were adjusted by assuming average pier sizes, flow depths, and velocities for field conditions, then scaling the coefficients accordingly. Many pier widths are in the range of 1 to 3 m; thus an average pier width of 2 m was assumed. Dividing this average width by the average width used in the laboratory (0.04 m) results in a laboratory-to-prototype ratio of 1:62. Then, assuming that pier Froude

TABLE 1 NUMERICAL COEFFICIENTS FOR LABORATORY AND FIELD SCOUR MODELS

C_i	Units	Laboratory	Field
1 (Eq. 1)		0.8	0.8
2 (Eq. 1)	m_2^{-1}	134.0	0.2
3 (Eq. 3)	m^{-1}	63.0	1.02
4 (Eq. 3)	m^{-1}	1.05	1.05
5 (Eq. 9)	$(m/s)^{-1}$	31.2	3.95
6 (Eq. 9)		2.0	2.0
7 (Eq. 9)	m/s	1.0	1.0
8 (Eq. 10)		0.000015	0.000015

numbers for laboratory and field conditions are similar results in a velocity scaling ratio of $1:(62)^{1/2}$. The scaling ratio for flow depth was obtained in a similar manner. An average flow depth of 2 m was assumed. Dividing the average laboratory flow depth of 0.2 m by 2 m results in a scaling ratio of 1:10. The resulting numerical coefficients for the field model are presented in Table 1.

The set of coefficients determined by the method described here may be used as default values. A more accurate set of coefficients may be determined by scaling the values for the particular bridge site, rather than scaling for a typical or average bridge; however, the default values will provide an estimate of the progression of the scour hole around the bridge pier.

After the field model was tested and satisfactory results obtained, components were added to the model to determine the effects of the shape of the pier nose and the angle of attack of the approach flow to the pier.

Pier Geometry

The geometry of the pier is important in determining the depth of scour. The wider the pier, the greater the resulting scour depth. Other aspects of the pier geometry, such as the shape of the nose of the pier and the angle at which the pier is situated with respect to the direction of the approach flow, also significantly affect the depth of scour. The pier width may be multiplied by two factors, K_1 and K_2, reflecting these factors, to obtain an effective pier width

$$b_e = K_1 K_2 b \tag{11}$$

The angle of attack and nose shape factors were added to the computer program using information from tables and figures provided by Tison (*15*), Laursen and Toch (*16*), Chabert and Engeldinger (*17*), and Laursen (*18*).

ASSUMPTIONS AND LIMITATIONS OF THE MODEL

In developing a time-dependent model, a number of assumptions were made. One of the most important assumptions is that loose refill material deposited in the scour hole as the flood recedes is instantaneously blown out of the hole when the next flood flow occurs.

The model described herein is classified as being time dependent. It has been observed frequently during laboratory experiments using sands and gravels that the scour hole forms in a short period of time. This observation may lead one to believe that pier scour is relatively independent of time. In the field, however, the scouring process may be time dependent for several reasons. The first is that the peak flood discharge of a single storm event may not be maintained long enough to scour to a maximum depth; therefore, it would take more than one such storm to scour to the maximum depth. Second, the sediments around the bridge piers will be subject to many different flood discharges; some will have an insignificant effect on the scouring process, and others will erode the hole to a new depth.

These varying flow-duration conditions have not been tested in a laboratory setting. The time dependency, even in cohesionless sediments, can be seen in bridge failures like that at the Schoharie Creek bridge. Although the piers were placed on glacial till (a nearly nonerodible geologic formation), the base of the scour hole was only as wide as the excavation. The scour hole developed in the cohesionless riprap and backfill placed in the excavation. Flood records show that the largest storm in the history of the bridge (76,500 ft^3/sec, a 100-year storm) occurred in 1955. The bridge failed 32 years later when a 50-year flood (64,900 ft^3/sec) occurred. Because the bridge did not fail during the larger storm, it must be concluded that the peak discharge was not sustained long enough to permit the hole to scour to the depth required for failure and therefore that there exists a time dependency.

The third assumption of the model, inherent in the time-dependency assumption, is that the model accounts only for cohesionless sediments, that is, sand and gravel. These are the sediments typically found in the excavation pits around the piers as backfill and riprap. The model does not account for situations where piles are driven into cohesive soils where there is no excavation.

The conceptual model considers only local scour around the piers. Perhaps a greater concern in some cases, and certainly an integral part of the scour process, is channel degradation. Over the course of a single storm and certainly over the life of a bridge, the bed elevation of the channel may change: either degradation or aggradation may occur. In some cases, degradation may be a more critical problem than local scour, exposing the pier to failure conditions. The assumption made here, however, is either that degradation does not occur or that the process of degradation can be treated separately from local scour.

STORM GENERATION

To assess the long-term progression of scour at a bridge pier, time-dependent hydrologic input must be used with the scour model. For this purpose, a storm-generation model was developed to simulate realistic flow sequences. The model can then be simulated for the design life of a bridge (e.g., 50 years) to evaluate the effect of individual design parameters (e.g., riprap size).

The model to generate flood flows consists of two parts (*10*). First, the number of storms above a threshold discharge (q_b) that will occur in a year (i.e., a partial duration series) is determined. Second, the storm magnitude for each of the storms in the partial duration series is generated.

The number of storms above the threshold discharge may be generated using a Poisson process. The Poisson parameter is estimated using the mean number of storms above q_b. The magnitudes of the storms (q) in the partial-duration series may be represented by the exponential distribution

$$q = -\beta \ln(U) + q_b \tag{12}$$

where

β = exponential parameter,
U = random uniform number between 0 and 1, and
q_b = base level or threshold discharge.

Although this single-parameter distribution is not easily fitted to a given set of data, it is a relatively simple distribution with a closed-form solution, making it easily adaptable in a simulation model.

The parameter of the exponential distribution is estimated by the mean of the discharge data. However, the fit of the distribution may be improved by optimizing the parameter. More information regarding the simulation of storm series was provided by Todorovic (*19*), Cunnane (*20*), North (*21*), Cruise and Arora (*22*), and Johnson (*10*).

EXAMPLE APPLICATION

As an example of simulating bridge pier scour over a period of years, the scour depth was computed and accumulated over a 75-year period. The 75-year simulations were performed under the assumptions of a 275-km^2 watershed, a pier 2.5 m wide and 6 m long, and a sediment size of 10 mm with a gradation of 18 (i.e., d_{84}/d_{50} = 18). The angle of attack of the approach flow to the pier was 5 degrees. For the storm simulator, the partial duration series threshold was 33 m^3/sec, the exponential parameter was 25, and the Poisson parameter was 3. The individual storms were assumed to be 24 hr in duration, distributed in a triangular hydrograph.

Figure 2 shows the increase in the mean scour depth with time. The scour depth increased rapidly during the first 15 years as storm waters removed the initial volume of material; however, the scour depth during this time remained low, less than about 1.7 m. At 20 years, the scour depth had increased to a value greater than 1.9 m, but the rate of increase in the scour depth had decreased. By 75 years, the scour depth had nearly reached a limiting value greater than 2.9 m.

RECOMMENDATIONS

To improve the model presented here, or any pier scour model, further research into the scour process is necessary. The vortex mechanism at the base of the pier must be examined. It is a complex process; thus, research must be undertaken that will focus on characterizing the vortex strength rather than attempting to derive a theoretical model. Characterizing the strength will provide a usable expression for the vortex strength in terms of shear stress or velocity that is a function of both the pier and flow characteristics.

The understanding and modeling of scour would also be enhanced by field data. Although some field data exist, they have been collected at various times during flooding (some during the flood recession, some several days later, etc.), using different equipment different criteria for determining where the bottom of the hole is. Research must be aimed at developing methods of monitoring scour around piers, including equipment that can withstand the turbulence in the vicinity of the pier and determine the depth to the actual bottom of the scour hole.

Most scour research deals only with sand and gravel as the sediment at the base of the pier. This is reasonable for many bridge pier situations, particularly those where the footing excavation was filled with alluvial backfill and riprap. At other bridges, the piers may rest directly on pilings that have been driven into the channel bottom. In these cases, the sediment is usually not cohesionless sand and gravel. Instead, the channel sediment may consist of cohesive soil, compacted sediments, cemented sediments, or a combination of these. The scour process will occur at a much slower rate in these sedi-

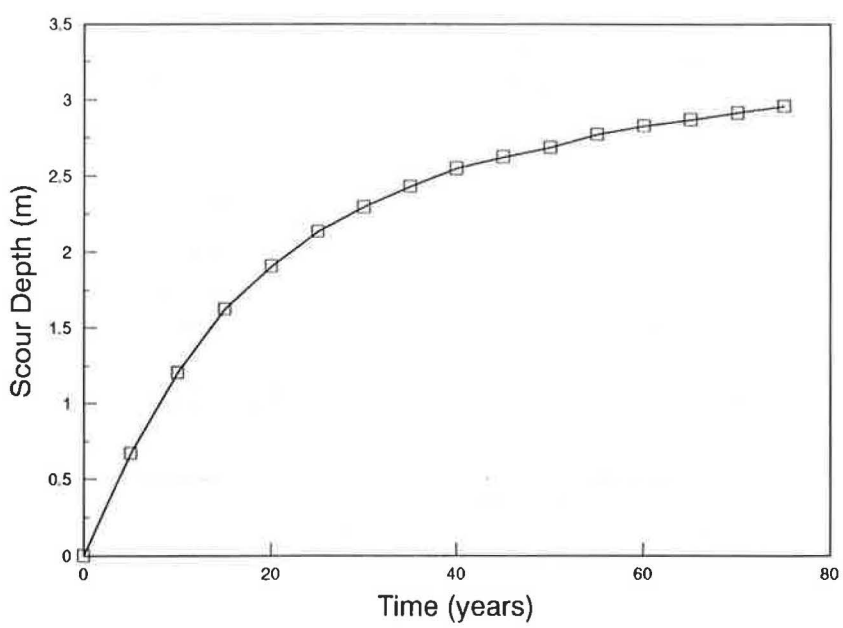

FIGURE 2 Scour depth as a function of time.

ments than in sand and gravel. Therefore, these models may provide a magnitude for the ultimate scour depth that can only be reached given an unreasonably long period of time. The time rate of scour must be studied for these cases.

DISCUSSION AND CONCLUSIONS

The purpose of this study was to develop a conceptual, time-dependent pier scour model. A time-dependent pier scour model was considered necessary for several reasons. First, bridge failures, such as the Schoharie Creek bridge, indicate that the magnitude, duration, and frequency of the flood flows determine the depth to which the scour hole develops during any one flood event. The peak flow rate may not be sustained for an adequate period of time to develop the scour hole to an ultimate depth; therefore, a number of storms may be required to scour the hole to its maximum depth. Second, the development of the time-dependent pier scour model provides a basis from which the probability of failure can be estimated over the design life of the bridge.

Input Requirements of the Model

The pier scour and risk-of-failure model developed here requires information that is generally available or that is routinely obtained for most bridge sites. The model requires the pier characteristics (pier width, shape, and length) and sediment characteristics (mean sediment size, sediment gradation, and friction velocity).

In addition to the pier and sediment characteristics, hydrologic data are required. If data from a gauged station nearby are available, a partial-duration series may be developed. If the design site is at an ungauged location, it is possible to develop a partial-duration series by estimating the appropriate parameter. Assuming an exponential distribution for the partial-duration series, only the exponential parameter and the mean number of storms, which is required for simulating the number of storms using a Poisson process, are required to simulate the hydrology.

Implications

The model developed here yields an estimate of the depth of scour for a period of time, such as the design life. Besides being used to estimate scour depth, the model may be used to determine the effects of various scour control improvements. In particular, riprap is commonly placed around the pier and within the excavation area of the pier to control scour. The model can be used to determine whether the riprap is of adequate size to remain within the excavation area over an extended period of time. This would simply entail entering the desired riprap size as the mean particle size (d_{50}) and determining the scour depth over the desired number of years on the basis of expected hydrologic conditions. If the scour depth is excessive or greater than some allowable value, then the particle size should be increased and the test rerun until an acceptable solution is found.

When designing a bridge pier, the engineer can use the model to evaluate various design alternatives as a function of parameters such as the size, shape, and depth of the pier. Having a better understanding of the effects of the various parameters on scour will enable the engineer to design safer piers.

The effect of changes in the upstream watershed on the scour depth may also be determined using the pier scour model. Upstream changes often affect the stream discharge; reflecting these changes in the hydrologic simulation will enable the engineer to determine the effect of the changes on the scour depth.

If a measured discharge record is used at a site, the history of the scour progression at a bridge site may also be simulated using this model. Estimates of the temporal variation of scour history may be desired for different reasons, including model calibration when measurements of the scour depth are made or legal cases involving bridge failure. Rather than a flood series simulation, the measured flood discharge record can be used. In this way, the temporal variation of scour for the historic record of discharges would be estimated. This may be important if measurements of scour have not been taken in the past, but the current scour depth estimation is desired for reasons of safety or recordkeeping.

The various components of the model may be easily modified as new developments are made through further research. As information is collected on erosion rates of compacted and cohesive channel materials, the erosion rate component can be modified to reflect these findings. The model will then provide a time-dependent scour estimate for a variety of materials.

REFERENCES

1. G. R. Hopkins, R. W. Vance, and B. Kasraie. *Scour Around Bridge Piers.* Report FHWA-RD-79-103. FHWA, U.S. Department of Transportation, 1980, 131 pp.
2. Y. M. Chiew. Local Scour at Bridge Piers. Report 355. School of Engineering, University of Auckland, New Zealand, 1984.
3. H. N. Breusers, G. Nicollet, and H. W. Shen. Local Scour Around Cylindrical Piers. Journal of Hydraulic Research, Vol. 15, No. 3, 1977, pp. 211–252.
4. H. H. Chang. *Fluvial Processes in River Engineering.* Wiley, New York, 1988, 432 pp.
5. E. V. Richardson, D. B. Simons, and P. Y. Julien. *Highways in the River Environment*, 2nd ed., FHWA, U.S. Department of Transportation, 1988.
6. H. W. Shen, V. R. Schneider, and S. Karake. Local Scour Around Bridge Piers. *Journal of the Hydraulics Division, ASCE*, Vol. 95, No. HY6, 1969, pp. 1919–1939.
7. B. W. Melville. Local Scour at Bridge Sites. Report 117. School of Engineering, University of Auckland, New Zealand, 1975.
8. Baker. Local Scour at Bridge Piers in Nonuniform Sediment. Report 402. School of Engineering, University of Auckland, New Zealand, 1986.
9. A. J. Raudkivi. Functional Trends of Scour at Bridge Piers. *Journal of Hydraulic Engineering, ASCE*, Vol. 112, No. 1, 1986, pp. 1–13.
10. P. A. Johnson. *The Probability of Bridge Failure Due to Pier Scour.* Ph.D. dissertation, University of Maryland, College Park, 1990, 231 pp.
11. C. M. White. Equilibrium of Grains on the Bed of a Stream. *Proc., Royal Society* (London), Series A, Vol. 174, 1940, p. 322.
12. F. M. Henderson. *Open Channel Flow.* Macmillan, New York, 1966, pp. 476–481.

13. R. Ariathurai and K. Arulanandan. Erosion Rates of Cohesive Soils. *Journal of the Hydraulics Division, ASCE*, Vol. 104, No. HY2, 1978, pp. 279–283.
14. Y. H. Chen and B. A. Anderson. *Development of a Methodology for Estimating Embankment Damage Due to Flood Overtopping.* Report FHWA-RD-86-126. FHWA, U.S. Department of Transportation, 1987.
15. L. J. Tison. Erosion Autour de Piles de Pont en Riviere. Annales des Travaux Publics de Belgique, Vol. 41, No. 6, pp. 813–871.
16. E. M. Laursen and A. Toch. Model Studies of Scour Around Bridge Piers and Abutments—Second Progress Report. *HRB, Proc.*, Vol. 31, 1956, pp. 82–87.
17. J. Chabert and P. Engeldinger. Etude des Affouillements Autour des Piles de Ponts. Lab. Nat. d'Hydr. Chatou, France, Oct. 1956.
18. E. M. Laursen. The Total Sediment Load of Streams. *Journal of the Hydraulics Division, ASCE*, No. HY1, 1958, pp. 1530–1536.
19. P. Todorovic. Stochastic Models of Floods. Water Resources Research, Vol. 14, No. 2, 1978, pp. 345–356.
20. C. Cunnane. A Note on the Poisson Assumption in Partial Duration Series Models. *Water Resources Research*, Vol. 15, No. 2, 1979, pp. 489–494.
21. M. North. Time-Dependent Stochastic Model of Floods. *Journal of the Hydraulics Division, ASCE*, Vol. 106, No. HY5, 1980, pp. 649–665.
22. J. F. Cruise and K. Arora. A Hydroclimatic Application Strategy for the Poisson Partial Duration Model. *Water Resources* Bulletin, Vol. 26, No. 3, 1990, pp. 431–442.

Publication of this paper sponsored by Committee on Hydrology, Hydraulics, and Water Quality.