Stedinger, J. R., and Tasker, G. D. (1985). "Regional hydrologic analysis. 1: Ordinary, weighted, and generalized least squares compared." *Water Resour. Res.*, 21(9), 1421–1432.

Thomas, W. O. (1985). "A uniform technique for flood frequency analysis." *J. Water Resour. Planning and Mgmt.*, ASCE, 3(3), 321–337.

Thomas, B. E., and Lindskov, K. L. (1983). "Methods for estimating peak discharge and flood boundaries of streams in Utah." *Water-Resources Investigations Report 83-4129*, U.S. Geological Survey, Washington, D.C.

Waananen, A. O., and Crippen, J. R. (1977). "Magnitude and frequency of floods in California." *Water-Resources Investigations 77-21*, U.S. Geological Survey, Washington, D.C.

Wahl, K. L. (1977). "Simulation of regional flood-frequency curves based on peaks of records." *Proc., Conf. on Alternative Strategies for Desert Development and Mgmt.*, United Nations Institute for Training and Research (UNITAR), Sacramento, Calif.

Wallis, J. R., and Wood, E. F. (1985). "Relative accuracy of log-Pearson III procedures." *J. Hydr. Engrg.*, ASCE, 3(7), 1043–1056.

ASSESSING TIME-VARIANT BRIDGE RELIABILITY DUE TO PIER SCOUR

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ABSTRACT: To prevent bridge failure due to pier scour, engineers are being asked to determine the reliability of new and existing bridges. There is currently little guidance for decision making regarding the control of scour at existing piers and the design of piers for avoiding failure due to pier scour. The purpose of this paper is to present a method to assess the risk of bridge failure due to pier scour during the life of the bridge. The analysis involves simulating pier scour for a period of time and determining the probability that the bridge will fail at various points in time during that period. A linear performance function describes the probability of bridge failure due to scour. The distributions and coefficients of variation of the random variables in the performance function are then determined by simulation, and the probability of failure is computed directly. The risk-based failure analysis will enable the engineer to base bridge pier designs and scour control on an estimated probability of failure, thus providing for the safer design of bridges.

INTRODUCTION

On April 5, 1987, the New York State Thruway bridge across the Schoharie Creek collapsed, killing 10 people. The cause of failure was attributed to scour around the piers. This event brought national attention to the pier-scour problem. As a result, engineers are being asked to improve scour depth prediction methods for the purpose of pier foundation depth design and to determine the safety of existing bridges where a scour hole has already formed.

Many prediction models are available in the literature for determining the maximum depth of scour. A summary of these equations is provided by Jones (1983). These equations are deterministic, because they do not account for uncertainties in hydrology, models, their parameters, and hydraulic variables. In addition, the independent variables that affect pier scour, such as the flow velocity and depth, are stochastic; therefore, the dependent variable, scour depth, is also stochastic. Deterministic equations cannot account for this stochastic behavior; therefore, a probabilistic approach is more logical than a deterministic one. The probabilistic approach may be used to estimate bridge reliability or probability of bridge failure.

Laursen (1970) eloquently stated the problem of bridge failure due to pier scour as:

> The common rule in bridge design is the accommodation of the 50-yr flood or the flood of record, whichever is larger. This sounds safe, yet in almost any major flood some bridge collapses because scour undermines a bridge pier or abutment. Therefore one may ask: "Was the designer unlucky or unwise?" If the added cost of deeper foun-

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dations would have been greater than the chance of failure times the basic cost of the bridge, then the designer was wise but unlucky. But, if the added cost would have been less, then the designer was unwise as well as unlucky. If the added cost was greater than justified, the designer was wasteful, but who would know. And, of course, it would be difficult to fault the wasteful designer if the rare, rare flood did occur, but on should.

Few studies have been conducted on the risk of failure due to pier scour. Laursen (1970) used a return period method to determine the economic risk. He concluded that all bridge pier foundations should be designed for the probable maximum flood, since the likelihood of the probable maximum flood is sufficient to justify the relatively small additional cost for a deeper pier foundation.

Using a return period or exceedence probability to estimate risk involves accounting for the statistical characteristics of floods by using a frequency analysis. The probability that a flood of a given magnitude is exceeded in any year is equal to the reciprocal of the return period. Yen (1970) summarized the assumptions of this method as: (1) The occurrences of the underlying random variables of the hydrologic system are independent; and (2) the hydrologic system is time-invariant. Yen also points out that both seasonal and long-term variations in the hydrologic system exist.

The return period method is simple, but the assignment of the design return period is without scientific basis and does not account for uncertainties or economic alternatives (Tung and Mays 1980). In addition, since the return period, and, thus, the exceedence probability, is based on one year as the time unit, it is not sufficient as a design criteria for hydraulic structures designed for a specified life (Plate and Duckstein 1988).

Another common method of accounting for risk is by using safety factors. Safety factors are commonly used in engineering designs to protect against uncertainties in the design parameters and models. The safety factor is typically set by policy prior to use. Most commonly, the safety factor may be defined as (Tung and Mays 1980; Lee and Mays 1983; Yen 1987)

$$SF = \frac{\bar{R}}{\bar{L}} \quad \text{(1)}$$

where $\bar{R}$ and $\bar{L}$ = the mean values of the resistance and load, respectively. Like the return period, the safety factor does not specifically account for uncertainties and economic alternatives (Tung and Mays 1980). Further, it is not a direct measure of risk, so it cannot be directly incorporated into a quantitative decision analysis (Hart 1982; Yen 1987). Tung and Mays (1980) and Lee and Mays (1983) developed risk-safety relationships for various return periods that could be used in design.

Objective

The objective of this study was to develop a new method for quantifying the probability of failure of a time-variant process, in this case bridge pier-scour. The method can aid engineers in assessing whether an existing bridge pier or a design is likely to fail at various stages during the design life. A pier-scour model that accounts for the cumulative scour process during a bridge's life was used with the methodology developed here to determine probabilities of failure.

Pier-Scour Model

The pier-scour model used in this application is a conceptual, time-variant model and is described only briefly. The model is described in detail in Johnson (1990) and Johnson and McCuen (1991). The model includes three main components that estimate: (1) The downflow velocity; (2) the termination velocity (analogous to a velocity of incipient motion for a channel bed); and (3) the erosion rate and scour depth. The streamflow characteristics are obtained through a storm-generation component. A brief description of each of these components follows.

Downward Velocity

As the streamflow approaches a 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 (Chiew 1984). According to Chiew, 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. While the vortex significantly contributes to erosion, the vortex flow pattern is highly correlated with the downflow; therefore, to estimate the depth of scour, a significant correlation between downflow scour and vortex scour is assumed in the model. Since the downflow can be described in terms of a measurable quantity, velocity, this was chosen as the best alternative to modeling the vortex.

The maximum downward velocity is a function of the approach velocity, pier size, and flow depth. An increase in either the velocity, pier size, or flow depth would result in an increase in the maximum downward velocity, $V_{d\max}$. Based on experimental observation (Shen et al. 1969), $V_{d\max}$ will probably not exceed one to two times the approach velocity; therefore, $V_{d\max}$ may be expressed as an exponential function

$$V_{d\max} = C_1 V_o[1 - \exp(-C_2 bY)] \quad \text{(2)}$$

where C_1 and C_2 = numerical constants; V_o = the approach velocity; b = the pier width; and Y = the approach flow depth. The form of (2) was selected because it defines a curve exhibiting the trend suggested by observations in laboratory studies [for example, see Melville (1975)].

$V_{d\max}$ 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 $V_{d\max}$. The downward velocity at any depth below the location of $V_{d\max}$ within the hole is then some portion of $V_{d\max}$

$$V_d = KV_{d\max} \quad \text{(3)}$$

where V_d = the downward velocity at some depth; and K = a function of the scour depth. Using Melville's (1975) data, it was found that 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 effect the relationship between V_d and $V_{d\max}$:

$$K = 1 - \frac{1}{1 + \exp(-C_3 D - C_4 b)} \quad \text{(4)}$$

where C_3 = a shape parameter; C_4 = a location parameter; and D = the 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. Eq. (4) assumes that the maximum 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 (4) into (3).

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 called 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

$$F_r = F_e \quad (5)$$

where F_r = the resisting force due to the weight of the sediment; and F_e = the erosive force due to the impinging jet of water in the downward flow just upstream of the pier. The shear stress caused by the impinging jet is modeled with the momentum equation.

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 both uniform and nonuniform sediments:

$$V_t = C_6\left(\frac{2\rho_s g d G \sin\phi}{3\rho\sqrt{2}(1 - \sin\phi)^{1/2}}\right)^{1/2} + C_7 G \exp(-C_5 V_o) \quad (6)$$

where ρ_s = sediment density; d = the mean grain size; and ϕ = the angle of repose. C_6 reflects other factors involved in initiating particle motion, such as the circulation in the hole, and C_7 and C_5 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 (Henderson 1966; Ariathurai and Arulanandan 1978; Chen and Anderson 1987). Therefore, the following functional form was selected to estimate erosion rates:

$$\frac{\Delta D}{\Delta t} = C_8(V_d - V_t) \quad (7)$$

where D = the scour depth (m); t = the time required to scour depth D (sec); and C_8 = a numerical coefficient. The downflow velocity, V_d, decreases according to (3) 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. Eq. (7) 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.

Assumptions

In developing a time-dependent model, some assumptions were made. One of the most important is that loose refill material that is 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 frequently been observed during laboratory experiments using sands and gravels that the scour hole forms in a very short 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 while others will erode the hole to a new depth. These varying flow-duration conditions have not been tested in a laboratory setting.

The third assumption of the model, inherent in the time-dependency assumption described previously, is that the model accounts only for cohesionless sediments such as 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 various components of the model presented here may be easily modified for cohesive channel materials. The model will then provide a time-dependent scour estimate for a variety of materials.

The conceptual model developed in this paper 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. During the course of a single storm and certainly during 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 has two parts (Johnson 1990). 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

$$q = -\theta \ln(U) + q_b \quad (8)$$

where θ = the exponential parameter; U = a random uniform number between 0 and 1; and q_b = the 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 about simulating a storm series may be found in Todorovic (1978), Cunnane (1979), North (1980), Cruise and Arora (1990), and Johnson (1990).

Simulation Procedure

During the design life of a bridge, scour holes can develop around piers placed in streams. The rate of scour depends on the magnitude of floods and on the type of material in the channel bottom. The scour hole progressively deepens as flood waters pass through the bridge site area. The margin of safety at the pier may be represented as

$$M = D_p - D \quad (9)$$

where D_p = pier depth; and D = scour depth. As the hole deepens, the margin of safety decreases, and the probability of bridge failure increases. The probability of bridge failure due to scour around the pier is then given by

$$P_f = P(M < 0) = P(D_p - D < 0) \quad (10)$$

Where all of the distribution types of a linear performance function [e.g., (9)] are normal, the probability of failure may be computed directly as follows:

$$\mu_M = \mu_{D_p} - \mu_D \quad (11a)$$

$$\sigma_M = (\sigma_{D_p}^2 + \sigma_D^2)^{1/2} \quad (11b)$$

where μ = mean; and σ = standard deviation. Eq. (10) may then be evaluated as

$$P_f = F_M(0) = \phi\left(\frac{0 - \mu_M}{\sigma_M}\right) \quad (12a)$$

$$P_f = 1 - \phi\left(\frac{\mu_M}{\sigma_M}\right) \quad (12b)$$

$$P_f = 1 - \phi(\beta) \quad (12c)$$

where F_M = cumulative distribution function of M; ϕ = standard normal variate; and β = safety index.

This process of pier scour was simulated using the time-variant pier-scour model and storm generation described previously. All storms were assumed to have a triangular hydrograph. The resulting mean values and standard deviations of scour depths were then used to compute the probability of bridge failure at regular time intervals during the life of the bridge.

Random Variables in Scour Simulation

The variables in the scour program were treated as either deterministic or random variables. Table 1 summarizes the random variables, their coefficients of variation (COVs), and their distribution types. Five of the eight numerical coefficients within the scour model were considered to be random variables. The distribution types were assumed to be normal for the five coefficients. The COV values are given in Table 2. The remaining three coefficients are deterministic, since they affect only the equilibrium timing of the scour process.

Random Variable Generation

To simulate pier scour, values for the random variables must be generated for each simulation cycle. Computer algorithms are readily available to generate random numbers for a uniform distribution. To transform uniform variates to another probability density function, the inverse transform method was used (Ang and Tang 1984).

Direct Simulation

The development of the scour hole during a bridge's design-life may be simulated by generating the underlying random variables, with the scour depths for selected intervals of time (e.g., 5 years or 10 years, etc.) computed for each simulation cycle. The process is then performed during many simulation cycles, such as N cycles. The scour depth for each time interval is given as the mean of the scour depths computed for the N simulation cycles. The mean and standard deviation, $S(D)$, at the end of each time interval are then used in computing the probability of failure for the ends of corresponding time intervals.

TABLE 1. Coefficients of Variation and Distribution Types of Variables Used in Scour Program

Variable (1)	Units (2)	Coefficient of variation (3)	Distribution type (4)	Reference (5)
Particle size (d_{50})	m	0.05	uniform	assumed
Angle of flow to pier (α)	degrees	0.2	normal	assumed
Sediment gradation (G)	—	0.1	lognormal	assumed

TABLE 2. Coefficients of Variation and Distribution Types for Numerical Coefficients (McCuen 1985; Brubaker and McCuen 1990)

Coefficient (1)	Coefficient of variation (2)	Distribution type (3)
V_{dmax} coefficient	0.1	normal
V_{dmax} exponential coefficient	0.1	normal
K shape coefficient	0.15	normal
K location coefficient	0.1	normal
sediment graduation coefficient	0.1	normal

The accuracy of the simulation process can be assessed by calculating the statistical (sampling) error as follows:

$$E(D) = \frac{S(D)}{\sqrt{N}} \quad \text{(13)}$$

This estimate of the standard deviation of the error can be used to establish confidence intervals about the estimated scour depths.

Number of Simulation Cycles

To compute the probability of bridge failure due to scour around the pier, it is necessary to have an estimate of the mean scour depth for the specified time interval, the COV or standard deviation of the scour depth, and the distribution type. Before these quantities can be obtained, an estimate of the number of simulation cycles that is required for sufficient accuracy of estimated scour depths must be determined for each desired time interval. This was accomplished by simulating the scour process for 5, 10, and 50 years using various numbers of simulation cycles. The mean scour depth was then plotted as a function of the number of simulation cycles, as shown in Fig. 1. The periods of 5, 10, and 50 years were chosen because the bridge simulations typically cover about 75 years of life. These three time intervals approximately cover this range.

For small numbers of simulation cycles, the value of the mean scour depth fluctuates, i.e., has a large standard error. As the number of cycles increases, variation in the mean scour depth stabilizes. The required number of cycles was defined by the point at which this stable prediction occurs.

The COV of the mean scour depth for the various numbers of cycles is plotted in Fig. 2. As the number of cycles increases, the COV decreases. For the 50-year simulation, the COV is quite low for all numbers of cycles. For the 5- and 10-year simulations, the COV quickly decreases as the number of cycles increases. It is desirable for the COV to be less than about 0.1. This is certainly the case for all simulation cycles greater than 100.

Based on Figs. 1 and 2, the minimum number of cycles for a 5-year simulation is 1,000. For the 10-year scour depth, 500 cycles are required, and for the 50-year scour depth, 200 cycles are required. A plot of the number of simulation cycles versus the simulation interval was then developed as a guide to the minimum number of cycles necessary for any desired time period. This plot is shown in Fig. 3. Since the greatest number of years tested was 50, it is assumed that 200 simulation cycles will be sufficient for time periods greater than 50 years.

Determination of Scour Depth Distributions

The distribution types for the 5-, 10-, and 50-year scour depths were determined by first plotting the scour depths and their frequencies of occurrence in histograms. Fig. 4 illustrates the histogram for the 10-year scour depth, based on 500 simulation cycles. Since the distributions for 5-, 10-, and 50-year scour depths had the familiar bell shape, they were tested for normality using the Kolmogorov-Smirnov (K-S) nonparametric test. The results of the three K-S tests confirmed that the hypothesis that all three distributions were normal could not be rejected at any of the levels of significance checked. It was then assumed that the scour depth for all other years and selected number of simulation cycles were also normal.

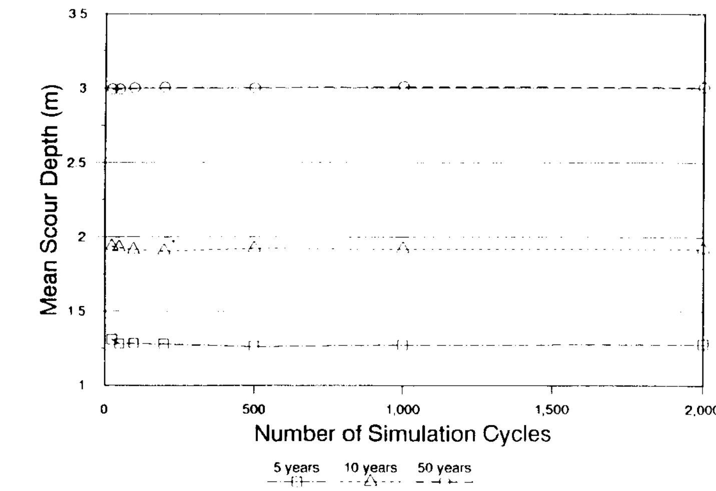

FIG. 1. Mean Scour Depth as Function of Number of Simulation Cycles

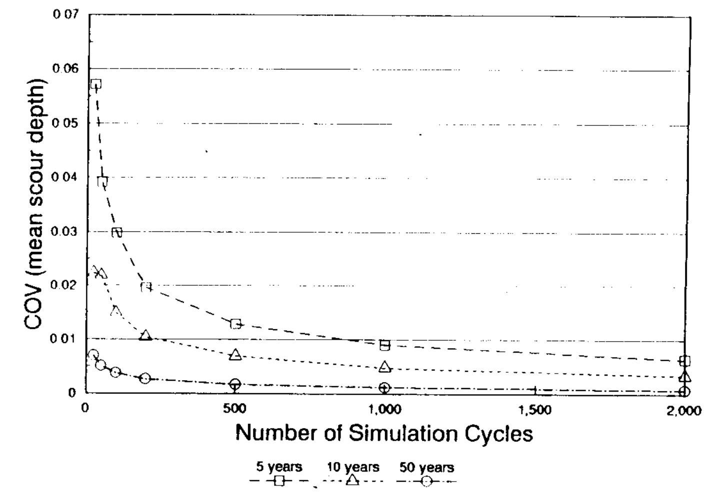

FIG. 2. Coefficient of Variation as Function of Number of Simulation Cycles

Probability of Failure

Having determined the distribution type, mean value, and standard deviation of the scour depth, the probability of bridge failure due to scour may now be computed. The probability of failure at the end of any specified time period was computed using the mean value and standard deviation of the scour depth [(11) and (12)] determined with the appropriate number of simulation cycles from Fig. 3. If the probability of failure is computed at intervals during the life of the bridge, the cumulative distribution function (CDF) of life is obtained by plotting the number of years of simulation versus the associated probabilities of failure.

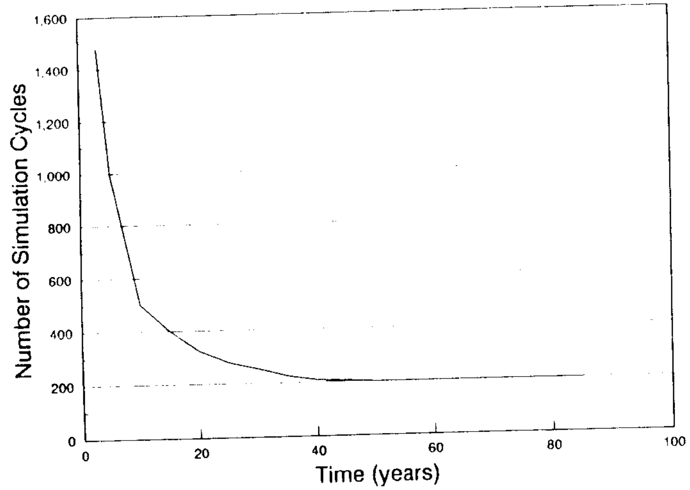

FIG. 3. Number of Simulation Cycles as Function of Time

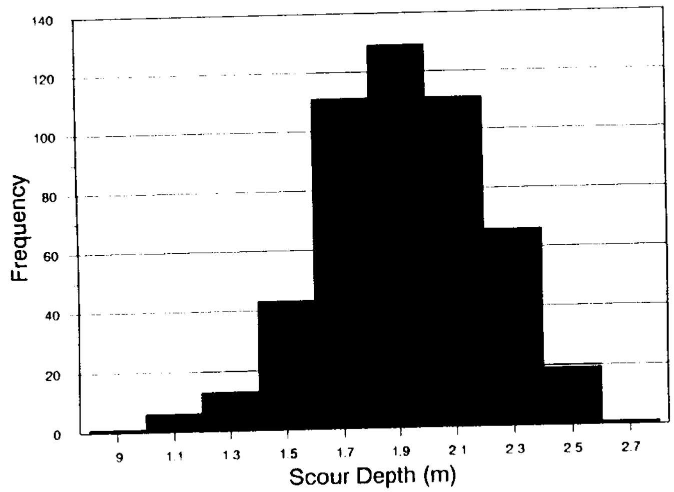

FIG. 4. Frequency Histogram for 10-Year Scour Depths Based on 500 Simulation Cycles

Cumulative Distribution Function of Life

To demonstrate the method of determining the CDF of life, the probability of failure was computed every five years for a sample bridge site that has a 75-year design life, and for the following characteristics:

- Pier characteristics:
  - Width = 2.5 m.
  - Length = 6 m.
  - Nose length = 1.5 m.

  - Depth = 4 m.
- Sediment characteristics:
  - Mean particle size = 10 mm.
  - Sediment gradation = 18.
- Hydrologic characteristics:
  - Drainage area = 275 km^2.
  - Angle of attack = 5°.
- Storm simulation characteristics:
  - Partial duration series threshold = 33 cm.
  - Exponential parameter = 25.
  - Poisson parameter = 3.

These characteristics were chosen as representative of typical bridge sites in the eastern United States.

The number of simulation cycles, N, needed for each time interval was obtained from Fig. 3. For example, the 5-year interval required 1,000 cycles, the 10-year period required 500 cycles, and the 20-year period required 425 cycles. For all time periods greater than 50 years, 200 simulation cycles were used.

The storm sequences and resulting scour were then simulated for each of the time periods. After the scour depth was computed N_k times for each time period k, the mean and standard deviation of the scour depth at the end of five-year intervals were computed. The probabilities of failure were then computed using (11) and (12).

Fig. 5 shows the increase in the mean scour depth with time. Fig. 6 shows the corresponding CDF of life for the bridge site. The probability of failure increased rapidly during the first 15 years as storm waters removed the initial volume of material. The probability of failure, however, remained low during this time, less than about 10^{-5}. At 20 years, the probability of failure had increased to a value greater than 10^{-5}, but the rate of increase in the probability of failure had decreased. By year 75, the probability of failure had nearly reached a limiting value greater than 0.006.

Parametric Analysis of Probability of Failure

To gain a better understanding of how the various parameters affect the probability of bridge failure and the CDF of life, three of the parameters were varied—pier width, mean sediment size, and pier depth. These parameters were chosen because they can be selected by bridge designers. The sediment size, of course, can only be varied in an excavation site where riprap is used. For each of the three parameters, the scour process around a cylindrical pier was simulated during a 35-year period and the CDF of life computed and plotted. The 35-year time period was chosen since it saved considerable computer time to run the simulations over a smaller period of time (compared to the 75-year period used in previous examples) and a 35-year life is sufficiently long to represent the sensitivity of the failure probabilities to the changes in the parameters.

Sediment Size

The effect of changes in sediment size on the probability of failure was examined to demonstrate the effect of placing riprap around the pier. Fig. 7 shows the 35-year CDF of life for mean sediment sizes of 1, 10, 100, and 200 mm. For the 1-mm and 10-mm sizes, the probability of failure increased

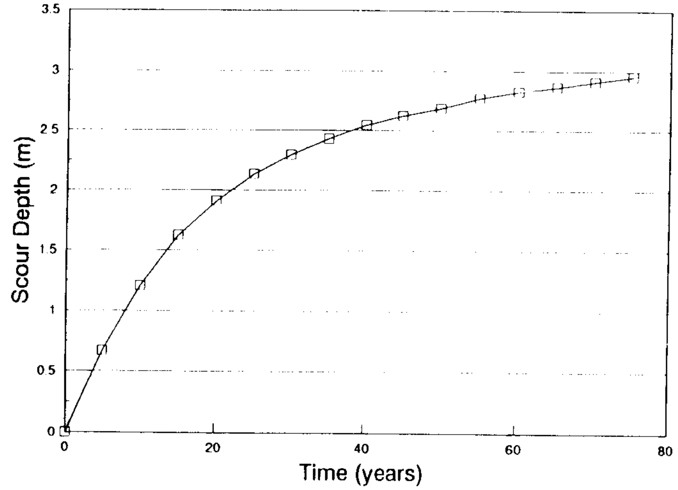

FIG. 5. Scour Depth as Function of Time

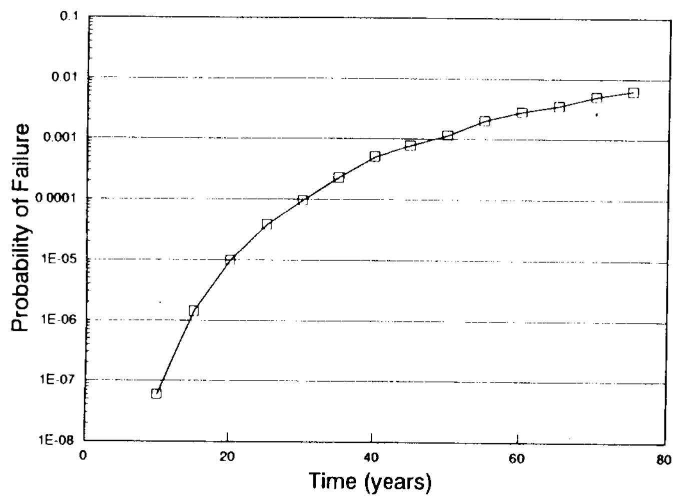

FIG. 6. Cumulative Distribution Function of Life for Bridge Site

quickly, then reached a limiting value. These very high probabilities of failure are not unlike the actual situation at some bridges in the United States. Many of these bridges are being classified as scour-critical, meaning that they require immediate attention. The CDF curves for the 100-mm and 200-mm sizes show that using larger sediment sizes (in the riprap range) significantly decreases the probability of bridge failure during a 35-year time period. However, at the end of this period, the probability of failure is still increasing; therefore, when the probability exceeds some specified value, it may be necessary for the riprap to be inspected and possibly replaced at this time or in the near future. The probability of failure after 35 years is not the final probability of failure; most bridges are meant to last consid-

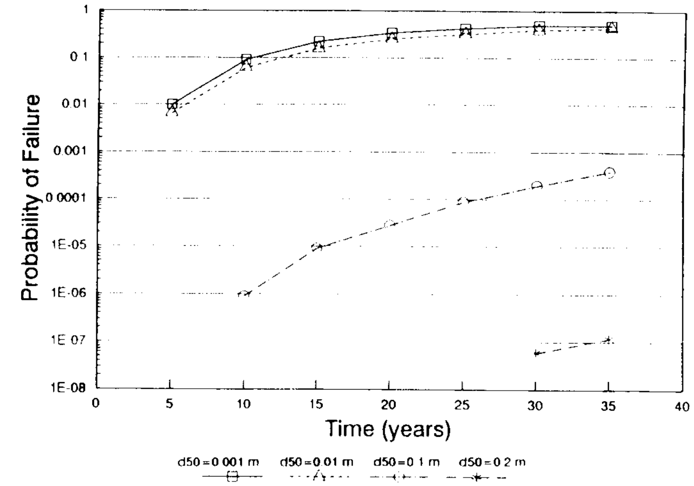

FIG. 7. Effect of Sediment Size on Probability of Failure

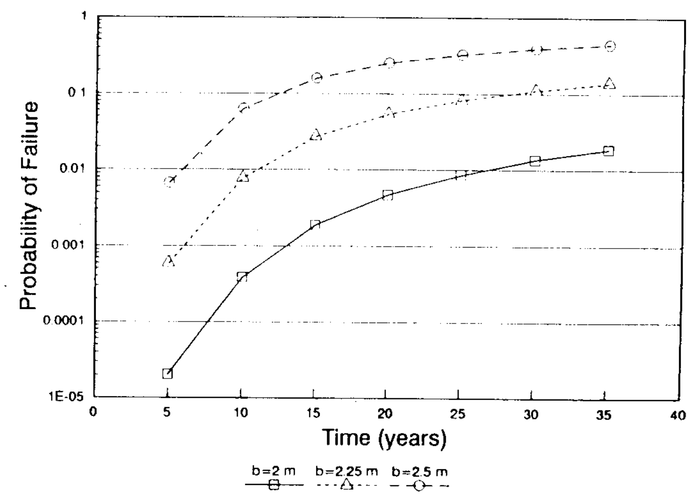

FIG. 8. Effect of Pier Width on Probability of Failure

erably longer than 35 years. A longer time period would need to be analyzed for an actual bridge.

Pier Width

The effect of pier width on the probability of failure during the life of the bridge was determined by varying the diameter of a cylindrical pier. Fig. 8 shows the CDF of life for 2.0-, 2.25-, and 2.5-m piers. A cylindrical pier was chosen so that the effects of length and angle of flow could be ignored.

As Fig. 8 shows, as the pier width increases, the probability of failure increases. This relationship appears to be nearly linear, i.e., a unit increase

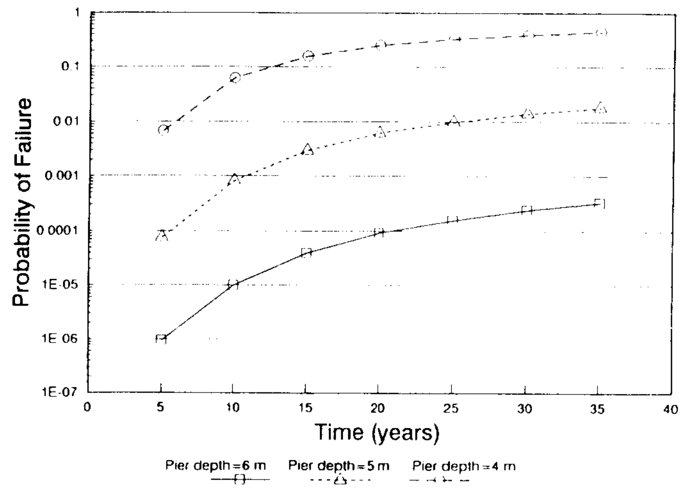

FIG. 9. Effect of Pier Depth on Probability of Failure

in the pier width causes a unit increase in the logarithm of the probability of failure. Fig. 8 clearly demonstrates that a small change in the pier width can result in a significant change in the probability of failure during the life of the bridge. As the pier width is increased, additional design changes should be made to compensate for the large change in the probability of failure due to the larger diameter.

Pier Foundation Depth

The depth of the pier foundation should have an obvious effect on the probability of failure, it must decrease as the depth increases. Fig. 9 demonstrates this relationship for pier depths of 4–6 m. The probability of failure increases equally for each incremental change in the pier depth. It should also be noted that the probability of failure decreases significantly with increasing pier depth.

Changing the magnitude of these three parameters, mean sediment size, pier width, and pier depth, can significantly impact the probability of failure due to scour during the life of the bridge. An increase in either the sediment size or the pier depth or a decrease in the pier width cause a decrease in the probability of failure. With this knowledge, the engineer can change one or more of these parameters to make the safest possible bridge.

Summary and Conclusions

A method was presented for determining the probability of a time-variant pier-scour process. The steps involved in this method may be summarized as follows:

1. Determine the number of simulation cycles.
   - Compute and plot the mean depth of scour for various numbers of simulation cycles.
   - Plot the corresponding COVs for the mean scour depths.
   - Locate the point on each of the two plots where the mean has stabilized and the COV is an acceptably small value. This is the number of simulation cycles that should be used.
2. Determine the distribution of the independent variable (in this case, the scour depth) for the number of cycles determined in step 1.
   - Simulate the scour depth over N simulation cycles.
   - Plot the resulting frequencies of occurrence in a histogram, determine the distribution, and test for goodness of fit.
3. Compute the probability of failure.
   - Assuming a normal distribution for both the scour depth and the pier depth, compute the mean and standard deviation of the margin of safety [(11)]. [If the distribution of the independent variable found in step 2 is not normal, the method outlined in step 3 cannot be used. Other methods are available in the literature for nonnormal distributions (Ayyub and Haldar 1984; Ayyub and White 1990)].
   - Compute the probability that the margin of safety is less than zero [(12)]. This is the probability of failure.

Although this method of determining the probability of failure has been applied to bridge pier-scour, the method may be used with any time-variant process (Ayyub and White 1990). This procedure significantly reduces the amount of computer time needed to obtain a certain level of accuracy in the probability of failure by not requiring the probability of failure to be determined as the number of failures divided by the number of simulation cycles (i.e., $P_f = N_f/N$). For small probabilities of failure, the computation of the probability of failure by failure counting can require a great deal of computer time.

Pier-scour equations currently being used do not account for the risk of failure. The current state of the art can be improved by developing a model that provides a risk-based approach for designing both new piers and scour control at existing piers. Risk-based methods may guide the engineer in answering such questions as: To what depth should the pier foundation be placed? What size and depth of riprap should be used? How will changing the size and depth of the riprap effect the risk of failure?

When designing a bridge pier, the engineer can use this method 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 should enable the engineer to design safer piers. To compare the various designs, multiple curves of time versus the probability of failure could be constructed using various design configurations (i.e., pier depths, pier sizes, or riprap sizes). The curves would be compared for the long-term and short-term scour depths and related failure probabilities. The designs may have different failure probabilities. In this case, the engineer can choose the best design in terms of a trade-off between an acceptable probability of failure and cost.

The probability of failure may also become a basis for policy in both the design of new bridges and control of scour around existing bridges. A particular level of risk during the life of the structure could be set by policy. Future designs would then have to meet this criteria either by providing an adequate pier depth or by providing scour control (e.g., riprap). If the computed probability of failure due to scour at an existing bridge is high or approaching an unacceptably high level, engineers would be alerted to a need for inspection and possible control of the scour hole around the piers. If the computed probability of failure is excessively high, local authorities may choose to close the bridge during major storms until the potential for scour failure can be reduced, through rehabilitation, to an acceptable level. Without an estimate of the probability of failure, these decisions would be based only on the engineer's judgment. The computed failure probabilities, however, do not eliminate the need for judgment, but provide valuable supplemental information for decision making.

Finally, in the event of a bridge failure, the design engineer is required to defend the original design. A design based on the engineer's experience is more difficult to defend in court than one based on a specified reliability (Plate and Duckstein 1988) or a risk of failure that is established as part of public policy.

Appendix I. References

Ang, A. H., and Tang, W. H. (1984). *Probability concepts in engineering planning and design, Volume II—Decision, risk, and reliability*. John Wiley & Sons, Inc., New York, N.Y.

Ariathurai, R., and Arulanandan, K. (1978). "Erosion rates of cohesive soils." *J. Hydr. Div.*, ASCE, 104(2), 279–283.

Ayyub, B. M., and Haldar, A. (1984). "Practical structural reliability techniques." *J. Struct. Engrg.*, ASCE, 110(8), 1707–1724.

Ayyub, B. M., and White, G. J. (1990). "Life expectancy assessment of marine structures." *Mar. Struct.*, 3(4), 301–317.

Brubaker, K. L., and McCuen, R. H. (1990). "Level of significance selection in engineering analysis." *J. Profl. Issues Engrg.*, ASCE, 116(4), 375–387.

Chen, Y. H., and Anderson, B. A. (1987). "Development of a methodology for estimating embankment damage due to flood overtopping." *Report No. FHWA/RD-86/126*, Federal Highway Administration, Washington, D.C.

Chiew, Y. M. (1984). "Local scour at bridge piers." *Report No. 355*, Univ. of Auckland, Auckland, New Zealand.

Cruise, J. F., and Arora, K. (1990). "A hydroclimatic application strategy for the Poisson partial duration model." *Water Res. Bull.*, 26(3), 431–442.

Cunnane, C. (1979). "A note on the Poisson assumption in partial duration series models." *Water Resour. Res.*, 15(2), 489–494.

Henderson, F. M. (1966). *Open channel flow*. Macmillan Publishers, Inc., New York, N.Y., 476–481.

Johnson, P. A. (1990). "The probability of bridge failure due to pier scour." PhD thesis, Univ. of Maryland, College Park, Md.

Johnson, P. A., and McCuen, R. H. (1991). "A temporal, spatial pier scour model." *Record 1319*, Transportation Research Board, Washington, D.C.

Jones, J. S. (1983). "Comparison of prediction equations for bridge pier and abutment scour." *Record 950 2nd Bridge Engrg. Conf.*, Vol. 2, Transportation Research Board, Washington, D.C.

Laursen, E. M. (1970). "Bridge design considering scour and risk." *J. Transp. Engrg.*, ASCE, 96(2), 149–164.

Lee, H.-L., and Mays, L. W. (1983). "Improved risk and reliability model for hydraulic structures." *Water Resour. Res.*, 19(6), 1415–1422.

McCuen, R. H. (1985). *Statistical methods for engineers*. Prentice-Hall, Inc., Englewood Cliffs, N.J.

Melville, B. W. (1975). "Local scour at bridge sites." *Report No. 117*, Univ. of Auckland, Auckland, New Zealand.

North, M. (1980). "Time-dependent stochastic model of floods." *J. Hydr. Div.*, ASCE, 106(5), 649–665.

Plate, E. J., and Duckstein, L. (1988). "Reliability-based design concepts in hydraulic engineering." *Water Res. Bull.*, 24(2), 235–245.

Shen, H. W., Schneider, V. R., and Karake, S. (1969). "Local scour around bridge piers." *J. Hydr. Div.*, ASCE, 95(6), 1919–1939.

Todorovic, P. (1978). "Stochastic models of floods." *Water Resour. Res.*, 14(2), 345–356.

Tung, Y.-K., and Mays, A. M. (1980). "Risk analysis for hydraulic design." *J. Hydr. Div.*, ASCE, 106(5), 893–913.

Yen, B. C. (1970). "Risk in hydrologic design of engineering projects." *J. Hydr. Div.*, ASCE, 96(4), 959–966.

Yen, B. C. (1987). "Reliability of hydraulic structures possessing random loading and resistance." *Engineering reliability and risk in water resources*, L. Duckstein and E. J. Plate, eds., Martinus Nijoff Publishers, Boston, Mass., 95–113.

Appendix II. Notation

The following symbols are used in this paper:

b = pier width;
c_i = numerical coefficient;
D = scour depth;
D_p = pier foundation depth;
d_{50} = mean sediment size;
$F(\ )$ = cumulative distribution function;
G = sediment gradation;
M = margin of safety;
P_f = probability of failure;
R = resistance;
L = load;
t = time;
U = uniform random number;
V_d = downward velocity;
V_o = approach velocity;
V_t = termination velocity;
V^* = friction velocity;
Y = approach flow depth;
β = reliability index;
θ = exponential distribution parameter;
μ = mean;
ρ = density;
σ = standard deviation;
τ = shear stress;
ϕ = angle of repose; and
$\phi(\ )$ = cumulative pdf of standard normal variate.