Assessing Vulnerability of Bridges to Floods

EMMETT M. LAURSEN

ABSTRACT

The capacity of both new and old bridges to withstand scour at their foundations and any other flow phenomena that could lead to failure needs to be examined. The problem is somewhat different in the two cases because a new bridge should be designed for the maximum flood to be expected, and there is ample opportunity in the design process to suggest foundations--or even bridge configurations--that may lead to safer, less costly bridges. Making existing bridges less vulnerable is likely to be difficult, awkward, and costly. However, even old bridges can be valuable--as can be discovered after they are lost--but the cost of remedial measures for the maximum expected flood may be more than can be justified. The prediction of scour at bridge foundations is a three-step procedure: (a) the establishment of the flood magnitude-frequency relationship, (b) the conceptualization and analysis of the flow characteristics of floods that might occur during the life of the bridge, and (c) the prediction of scour. The first step needs evidence of the maximum flood that should be expected; the second step is the most difficult as a general rule; the third step is likely to raise questions about scour that have not yet been answered adequately. As a result of the Silver Bridge failure, visual examination of bridges for structural integrity has become routine. Despite occasional spectacular failures like the Interstate bridge in Connecticut, there are probably more bridges lost in floods than from structural inadequacy. The assessment of the vulnerability of existing bridges to floods is also needed and would pay dividends. Recent research, sponsored by the Arizona Department of Transportation and the FHWA, has resulted in relationships for predicting the scour at the toe of a vertical wall and at the toe of a sloping sill. On the basis of the depth of scour, the structural form, and the ease of adding to the sill structure if need be in the future, the sloping sill is the preferred solution. Recent unsponsored student research indicates that the previous solution for sizing riprap was too conservative. Both of these studies are aids to the engineer seeking ways to make existing bridges less vulnerable.

Bridges are expensive. Moreover, bridges are vital links in our transportation system, and our transportation system is a vital part of our economic machine. It would seem obvious, therefore, that we can spend as much to ensure against the possible failure of a bridge as the cost associated with the failure times the probability of the failure. Usually it will be found that the bridge is so expensive, and the added cost of being sure it will not fail is so little, that just saving the bridge is sufficient justification for designing for the maximum flood truly to be expected (1). The magnitude of the maximum, limiting flood used for design purposes depends on the situation. The spillway design flood for a dam located just above a densely populated area will probably be as large as there is any evidence it can be. For a bridge, the maximum can be reduced to be comparable to the largest flood that has happened in the region and that can be expected to happen again.

To say that a bridge should not fail (2) is not to say that traffic will be maintained or that the approach embankments will not fail. These are separate questions. If the economic (or political) cost of traffic delay is not much, perhaps the bridge should be designed as an overflow structure. Highway embankment is comparatively cheap and can be replaced quickly. The acceptable risk of failure for the approach embankments might well be greater than that for the bridge itself.

All too often the risk of failure is evaluated as the probability of an event happening one or more times during the life of the structure; this probability turns out to be equal to unity minus the probability of nonoccurrence (because the sum of the probabilities of nonoccurrence, once-and-only-once, twice-and-only-twice, and so forth is unity). This is correct for a catastrophy after which the bridge is not to be rebuilt. But it is incorrect for recurring loss if the bridge is to be rebuilt. For recurring loss, the risk of failure should be evaluated as the product of the life of the structure times the probability of occurrence of the event in any 1 year. The difference in the two methods is small if that product is 0.1 or less, but becomes quite extreme if one considers designing a culvert, for example, for a 10-year flood if the culvert is supposed to function for 50 years.

This raises the questions of the value of the bridge over its life and the probability of occurrence of a flood of a given magnitude. The answers to both can be off by a factor of two. The optimum of the total expected cost curve (the sum of initial cost and probable loss) is so flat that one solution is not much better or worse than another as shown in Figure 1. Designing more or less conservatively can be justified on the basis of other losses on the one hand, or other societal needs on the other.

Both new bridges and old bridges need to have their vulnerability to floods assessed. The difference between the two cases is found in what can be done about the situation if there is vulnerability. Engineering design is, as a rule, an iterative procedure of assumption, analysis, appraisal, reassumption, reanalysis, and so forth until an acceptable solution is obtained. The hydraulic engineer should

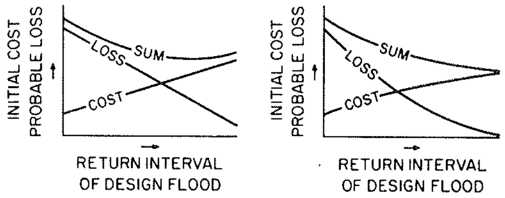

FIGURE 1 Expected cost curves.

be a member of the design team from the beginning to the end. He or she should have a voice in determining the location of the bridge; some locations can be much better than others from a hydraulic viewpoint. The hydraulic engineer should approve the final design as a final assessment of the vulnerability of the bridge. The final solution will not necessarily be the best solution from a hydraulic standpoint, but the hydraulics of the final solution can be important. During the design process, there should be an interplay between the various requirements of function, structure, aesthetics, and hydraulics. For example, if a safety factor is required in the design of a pile foundaton, it would be much better from a scour standpoint to make the piles longer than to require more piles. Or, if debris is anticipated (and it should be), widely spaced columns for a pier would be preferable to closely spaced columns; indeed, a solid pier might be preferable to closely spaced columns.

New bridges can probably always be made safe by making the foundations a little bit deeper. A little bit (comparatively) is enough because the magnitude of the flood that has twice the return interval is not double, the flow depth of a flood of twice the discharge is not double, and the scour depth for twice the depth of flow is not double. A foundation is required for a bridge even if it is not over a river. The added cost of making the bridge safe during the maximum flood to be expected is a small fraction of the cost of the bridge. This is why the best hydraulic solution is not necessarily the best overall solution in the design of a particular bridge and why vulnerable bridges can seldom be justified.

Old bridges present a different problem than do new bridges for several reasons. First of all, there are many more of them; second, their remaining life is limited and their value may be also; and last, decreasing their vulnerability is likely to be more costly if it is feasible at all.

In the aftermath of the failure of the Silver Bridge over the Ohio River, the inspection of bridges for structural integrity and traffic safety and adequacy has become routine. Much can be seen in a visual inspection and, if deemed prudent, measurements can be made and analyses can be performed to evaluate the safety, integrity, and adequacy of the structure. In respect to the vulnerability of the bridge to floods, seeing is only a small part of the assessment procedure required. A hydraulic engineer must predict what scour and what lateral forces could occur and a foundation engineer must predict whether the foundations are adequate for these conditions. If all old bridges over rivers are to be included in an inspection program, more than a few people will be needed. Do all bridges need to be included? An example of a little, old bridge in Arizona is illustrative. This 77-ft bridge was built in 1929 on the highway between Tucson and Nogales. Eventually it was used to carry a frontage road of the Interstate because it, like several other similar bridges, was still serviceable for this purpose. Nothing happened to the bridge until 1968 when one abutment settled about a foot during a 100-year flood. For repair, the bridge was jacked up and concrete was poured for a new bridge seat. Then in 1978 another 100-year flood occurred, and the three piers and four spans went down. One evaluation of the value of this almost-50-year-old bridge would be the cost of the bridge that was built to replace it (several hundred thousand dollars). Old bridges should not necessarily be excluded from a program of re-evaluation.

Certainly not all old bridges, or even all bridges, need to be evaluated. Bridges scheduled for demolition for whatever reason, or which are virtually worthless and are left standing for minor convenience, and bridges whose foundations are on rock or out of the reach of the river and whose superstructures are above floodwaters and debris are not part of the problem. There are still many existing bridges that need to have their vulnerability to floods assessed. As late as 1970 in response to a question about scour predictions, 67 of 97 engineering organizations said that they used engineering judgment, limited the flow velocity, or made no predictions or they did not reply (3). That more bridges do not fall is probably due to the conservatism of foundation engineers in their evaluation of soil properties and soil-structure interaction. However, they are probably as prone to use more piles than they think are really necessary as they are to use longer piles.

Because there are so many bridges, the evaluation of existing bridges needs to be a two-part effort. First, a quick check should be made of all bridges (starting with the most important, the most used, the most expensive) to divide them into three groups: (a) those that might well be vulnerable, (b) those that might not be vulnerable, and (c) those that cannot be categorized because they are too different or special. Then, those bridges that might be vulnerable should be examined carefully, and, if they are vulnerable, some means to make them invulnerable or less vulnerable should be devised. This last is easier said than done. It is easy in the design of a new bridge to specify foundations 5 or 10 feet deeper, and it does not cost very much. Whether one can actually add to the depth of existing foundations is arguable (but to no good purpose); however, a measure such as encircling existing piers and abutments with sheet piling is going to be costly. Any measures to reduce the vulnerability of a bridge are likely to be costly, and the remaining life and value of the bridge may well be less than when built (in constant dollars, of course). Therefore, it may not be economical to build in resistance to the maximum expected flood. However, protection against some level of flooding can probably be justified in most cases--after all, a bridge failure will always represent a sizable monetary loss, not only that of the bridge itself, but also that associated with delay of the traffic that cannot use the bridge.

Eventually all bridges should be examined, even those that were initially included in the group that might not be vulnerable. It will be the third group--those bridges that are special or different--that will give the most trouble. Either the examiner will need a lot of good imagination or there will need to be more research in the laboratory. All geometry and all situations have not been studied adequately, so the examiner who sets out to predict what can happen to existing bridges is going to encounter things unknown and unanswered questions that point to the need for further research.

RESEARCH

There are three things that need to be predicted in assessing the vulnerability of a bridge to floods: (a) the scour at the bridge foundations, (b) the lateral force of the flow on the bridge, and (c) the backwater due to the bridge. Then, of course, it is necessary to go on to predict how the bridge will react to the scour and lateral force and what additional flooding will occur because of the backwater.

The prediction of scour is a three-step process: (a) the establishment of a flood magnitude-frequency relationship including the magnitude of the maximum expected flood, (b) the quantitative description of the hydraulics of the flood flows including the divination of the future course and degradation of the river, and (c) the prediction of the scour at the bridge foundations.

Hydrology

There are those who believe in the infinite tails of the probability distribution functions used in hydrologic studies; they, as a consequence, also believe in infinite flood magnitudes. In this finite world there are no infinite floods; it is not possible to design for an infinite flood; and therefore those believing in the infinite flood will almost inevitably advise building for the wrong flood--one that is either too big or too small. Because bridges are usually expensive and because of the societal and economic need for the bridge, the flood of primary interest is the maximum expected flood. This is not the maximum flood anybody can conceive of happening, but merely a flood comparable to the largest floods that have happened in the region, or maybe slightly larger if the records are short. For the question of the maximum expected flood, recent work by geomorphologists is very important. Through the examination of slack-water deposits, Kochel et al. (4), have shown that the 1954 flood on the Pecos River was the largest in the past 2,000, perhaps 10,000, years. That, certainly, should qualify the 1954 flood as the maximum expected flood.

Because design for the maximum expected flood is the ultimate, the design process should start with it and then ask how much can be saved by building for a lesser flood and taking a chance on a loss. Will it be worth the risk? Seldom will the answer be "yes," and when it is, it will probably be in a case where traffic can be interrupted by a frequent (2- to 10-year) flood as long as the bridge is not substantially damaged. Presuming that one designs for the optimum (however one operationally finds the optimum), precise return intervals or probabilities for various magnitude floods are not necessary; one design is about as good or bad as another because the optimum is flat, not a cusp. In this writer's opinion, the 100-year flood is a political flood we could better do without, and the money being spent on determining the 100-year flood and its extent could better be spent investigating prehistoric floods to establish maximum expected floods.

Hydraulics

Deciding on the depth of flow, the direction of flow, and the distribution of flow across the full width of flow to predict scour is not easy, even for a river channel and floodplain as they exist. This is not enough. One must describe the character of the flow for the channel and floodplain as they might exist at some time during the life of the bridge. This is the step where divine inspiration would be very helpful but where imagination and some knowledge and experience in the behavior of rivers must suffice.

The description of the flow must be more than the one-dimensional analysis that is commonly employed to describe the flow in the river. Some notion of the two-dimensional pattern of flow in the plan view is absolutely required, and overtones of the third dimension are more than just desirable. The sediment-transport pattern of supply and capacity is needed for the determination of scour and deposition. The depth and direction of flow are needed to predict the scour at piers. The depth of flow and the quantity of flow obstructed by the approach embankment and abutment are needed to predict the scour at the abutment.

For existing bridges, the routine inspection program should be expanded to include observations of the river upstream and downstream (with pictures for the record and for comparison over the years). If the channel is shifting or the streambed is degrading or aggrading, the original design calculations should be referred to in order to check whether what is happening is within the conditions for which the foundations were designed. If not, there should be a determination of whether something should be done. There is always the possibility that in the "big flood," the river will change drastically and suddenly. Insofar as possible, however, this drastic, sudden change should have been anticipated during design. It is known that rivers widen during major hurricane floods (5,6), and the regime equations from India (7) imply widening. A recent dissertation by Silverston (8) gives an analytic basis for this tendency to widen and indicates that for rivers in regions like Arizona the widening tendency is more extreme than would be predicted by the regime equations.

Scour

This writer feels that the prediction of local scour at the piers and abutments is the easier part of the game as long as the geometry of the situation bears some resemblance to the geometries that have been studied in the laboratory. For others, this may not be quite the case. There are a number of relationships that have been proposed for predicting the depth of scour; for most, therefore, the first difficulty is to decide which relationship to use. To use all of them and take an average is to plead an inability to make a crucial decision. This paper is not the place to go into a comprehensive critique of the various scour relationships, but Figure 2 shows the primary difference between some of the scour relationships. The experimental data are from a thesis by Alawi (9). The Laursen curve includes the clear-water scour case, the good general movement case, and the transition between (10-12). The deepest scour is that predicted by the Colorado State University version of scour, which can be found in a training and design manual prepared for the Federal Highway Administration (13). The Jain and Fischer relationship is from a study, conducted at the Iowa Institute of Hydraulic Research, that achieved supercritical flow (14). The difference in opinion about the effect of velocity (or Froude number) on the depth of scour is shown again but from different points of view in Figures 3 and 4.

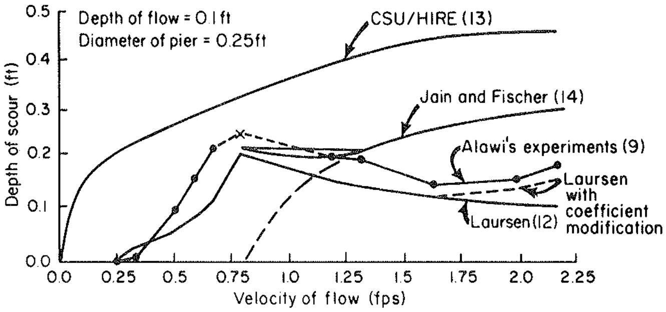

FIGURE 2 Comparison of various scour formulas with experimental data.

How does one "prove" which scour relationship (if any) is correct? By means of field measurements, of course. A few have been made. Most if not all are somewhat flawed and are not entirely satisfying.

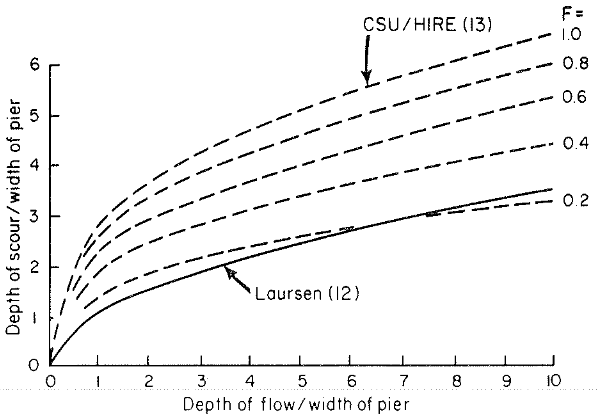

FIGURE 3 Predicted pier scour comparison.

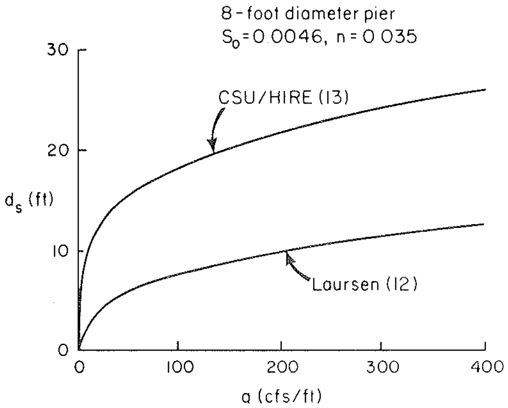

FIGURE 4 Scour predictions for typical Arizona conditions.

Figure 5 is a comparison of some measurements and the Laursen scour predictions, done as honestly and well as possible. Finally, it can be stated with little fear of contradiction that if the CSU/HIRE (13) scour relationship predicted the true state of affairs, there would be very few bridges still standing in Arizona.

This observation is illustrated by the same little, old bridge between Tucson and Nogales mentioned before (15) that had footings only 6 ft below streambed. A reasonable estimate of the flow pattern of the flood of 1968 resulted in a prediction using the Laursen relationships that the one abutment would settle as it did. Another "reasonable" estimate of the flow pattern of the flood of 1978 resulted in the prediction that the piers would fail (some bank-protection work had been done and a borrow pit had been dug during the construction of the Interstate). If a more correct scour prediction had been much less, there was no good reason for the damage and failure that occurred. If a more correct scour prediction had been much more, the bridge should have failed years before in some lesser flood. Therefore, the scour predictions must have been better than just "in the ball park," and predictions twice as large because of a velocity effect could not have been more correct.

Backwater

Highway embankments across the floodplains and a bridge across a river channel are without a doubt obstructions to the flood flow. Therefore, the question of how much backwater is caused by the obstruction is valid and should be asked. Unfortunately, the usual answer is based on some variation of the backwater to be expected for a boundary constriction in a comparatively narrow fixed bed and bank flume. If the valley is wide, and especially if it is heavily vegetated, the backwater may be much different and much greater (16). On the other hand, if there is scour, the backwater may be much less (11). Experiments at Colorado State University (CSU) found that if there was scour the backwater was so small it was sometimes measured as negative (17). It is possible the negative backwater was real and not measurement error because, if the jet issuing from the bridge opening digs itself a long contraction, the overall energy loss can be less than that of the natural flood flow. The backwater with scour found in a small laboratory flume is shown in Figure 6 (18). In Figure 7 that backwater is compared with the backwater predicted by the FHWA procedures without and with scour. The scour used was the measured scour, because the FHWA procedure gave no hint of how to predict scour.

Lateral Force

A solution for the lateral force on a bridge superstructure was published recently by Naudascher (19).

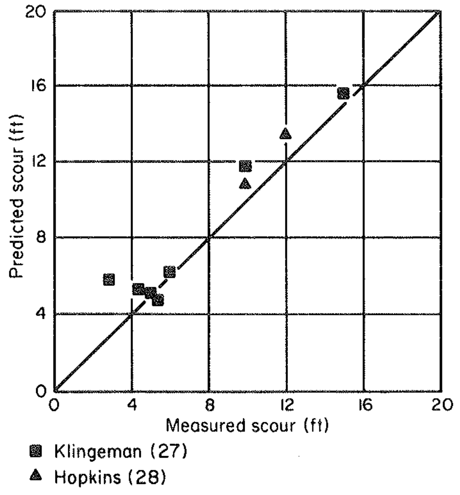

FIGURE 5 Predicted (11) and measured pier scour.

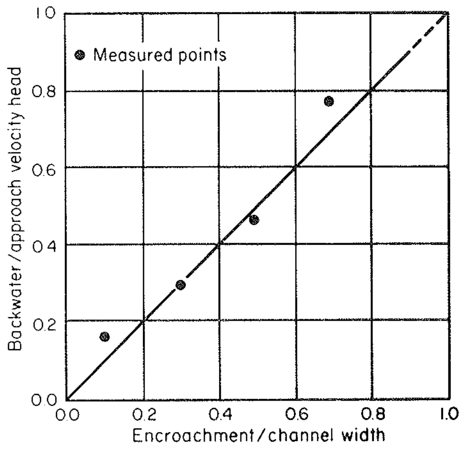

FIGURE 6 Backwater versus encroachment with scour (18).

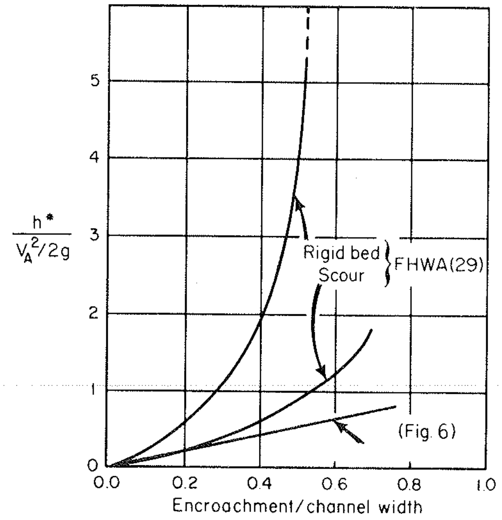

FIGURE 7 Backwater comparisons with scour (18).

This solution does not include the lateral force on the piers, which can be important even if the water level does not get up to the bridge superstructure. More important, this solution does not include the effect of debris. Of course, one must know how much of what kind of debris, and this is a great unknowable. Today one starts to think in terms of mobile homes when one thinks of debris. These could be even worse than the timber and lumber that were plastered against the bridges in northern California in the floods of about 15 years ago.

Naudascher's solution does not consider the effect of scour either. The sluice-gate flow pattern at the water surface would probably result in scour of an erodible bed. The flow, as a whole, would then experience very little contraction. It is difficult to say whether the flow around the superstructure and the pressure distribution around the structual members would be noticeably different with and without scour, but the backwater would be less and the water should not be as high on the superstructure. Experiments need to be repeated with an erodible bed.

INADEQUATELY ANSWERED QUESTIONS

The instant one tries to assess the vulnerability of bridges to floods, one begins to ask questions and finds the anwers are not completely satisfying. The first questions are about hydrology, and quickly one finds that information about the maximum flood to be expected is lacking. Carmody (20), looking at the largest floods in Arizona, found none of the largest floods adequately documented and, upon critical examination, many appeared to be estimated ultraconservatively high. The next questions are about the hydraulics of the flow, and one finds that there is not a satisfactory and convincing method of solving flood flow as a two-dimensional problem.

These questions relate to the preliminary steps in predicting scour. When one comes to predicting the scour itself, more questions will arise. The questions of which scour prediction equation is best, what is the effect of velocity or Froude number, and what is the backwater have been raised already and answers suggested. One is almost bound to run into geometry that has not been tested: Do very short elliptical piers lose all their shape effect? What about very long, thin piers at an angle to the flow and spaced so closely that the piers overlap? Exploratory research indicated that most but not all of the shape effect is lost, even for 2 to 1 and 1.5 to 1 ellipses, and that the overlapping length of the long, thin piers can be ignored (21,22)

REMEDIAL MEASURES

When an existing bridge has been determined to be vulnerable to scour, there are several remedial measures that can be considered and that are not changes to the bridge itself: (a) spur dikes to guide the flow and thereby lessen the scour, (b) riprap to limit the depth of scour, and (c) a sill structure to raise the streambed.

Spur Dikes

Spur dikes can guide the flow, but how long must they be to correct the flow in the center of the channel? A good answer to this question is still lacking. Spur dikes also shift the deepest scour to the end of the spur dike, away from the abutment. This is good, but it means that the spur dike must be long enough so that the tail of the scour hole has about reached bed elevation, as was demonstrated in another exploratory study (23). However, another caution in the use of spur dikes is that they must stay there during the big flood or they will not do their job when most needed. Again, there is really no available information on how the spur dike should be protected or how long it will last during the big flood.

Riprap

The available means of sizing riprap is the result of an analysis of the long contraction of the clear-water case (10) adapted to the pier or abutment in the same manner as the sediment-transporting case. Approximate evaluations for the boundary shear and the critical tractive force, which were reasonable and perhaps slightly conservative for channel flow, were employed in this analysis. A little bit of fragmentary evidence from the CSU tests seemed to be confirmatory. The solution seemed to work fairly well in predicting field measurements from Alaska; however, the size of the self-sorted riprap was not measured there and had to be assumed. Another exploratory investigation at the University of Arizona (24) suggests that the critical tractive force can be taken as 7d instead for 4d (d = riprap size in feet), giving riprap sizes of about half those that would be given by the original solution. Especially in Arizona, that is welcome news because of the high flow velocities of 15 or 20 fps, requiring riprap of 6-foot diameter or more using the 4d criterion. Much more testing needs to be done to be more sure of this preliminary finding.

Sill Structures

A sill downstream from the bridge is often the preferred solution in Arizona for reducing the vulnerability of an existing bridge. One reason is that degradation is a contributing factor; stream slopes are so great that a small percentage flattening of the slope or shortening of the stream can result in many feet of degradation. Combining several parallel washes in one drainage structure is, in effect, a long contraction case with lesser slope. Another reason is that, in much of Arizona, one is not worried about backwater and flooding. In addition, a construction style of H-piles (or old railroad rail), heavy wire fabric, and loose rock (of which there is plenty in Arizona) used by contractors experienced in this construction keeps prices down.

The key question in designing a sill structure is the depth of scour on the downstream side, or toe, of the structure. If the scour can be predicted, various sill geometries for various flows can be designed, and a decision can be made about what to build. Note that most of the possible loss is the bridge. Because no one was able to find the needed scour-predicting formulas, this became the subject of a research project sponsored by the Arizona Department of Transportation. The two geometries studied were the vertical wall and a 1 vertical (V) to 4 horizontal (H) sloping sill.

Experiments with a vertical-wall sill structure resulted in the following equation to predict the scour at the toe (25):

$$D_s/y_c = 8(V_c/w_0)^{3/4} - \{[6 + (V_c/w_0)]/[1 + (2\Delta WS/y_c)]^{1/2}\} \tag{1}$$

where

D_s = the scour measured from the downstream water surface,
y_c = the critical depth of flow,
V_c = the critical velocity,
w_0 = the fall velocity of a quartz sphere of the median sieve diameter, and
ΔWS = the drop in water surface upstream to downstream of the sill.

Figure 8 shows that, except for a few inexplicable runs, the equation predicts the depth of scour correctly or conservatively. It should be noted that although the flow was supposed to be two dimensional, the resulting scour was not two dimensional. Variation in the scour across the stream is to be expected in real life.

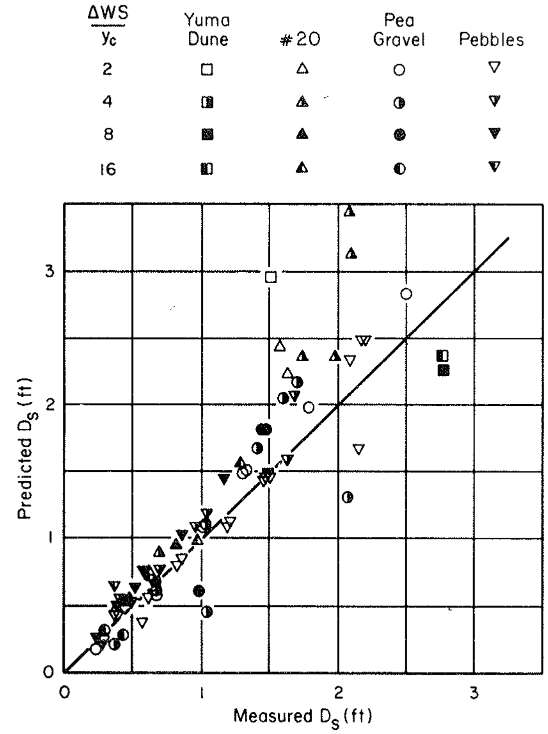

FIGURE 8 Comparison of measured and predicted scour at the toe of a vertical wall (25).

The sediment left the scour hole in suspension in the case of the vertical wall. The next experiments were with a sloping sill with a slope of 1V to 4H (26). In this case, the sediment left the scour as bed load except during the early stages of the development of the scour hole. Another difference was that the drop in water surface was not an important parameter, but that the size of the riprap covering the sill structure was. The scour prediction equation was

$$D_s/y_c = 4(y_c/d)^{0.2} - 3(d_{rr}/y_c)^{0.1} \tag{2}$$

where

D_s = the scour measured from the downstream water surface,
y_c = the critical depth of flow,
d = the diameter of the sediment being scoured out, and
d_{rr} = the diameter of the riprap.

The comparison of measured and predicted scour is shown in Figure 9.

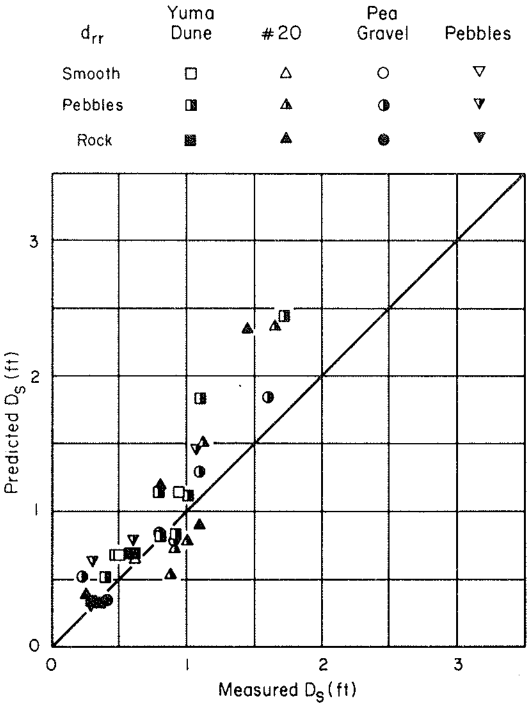

FIGURE 9 Comparison of measured and predicted scour at the toe of a sloping sill (26).

For both sills the scour is less when the flow is transporting sediment, but how much less depends on how much sediment is being transported. If the critical flow at the brink of the sill is transporting the maximum load it can, the scour is about 75 percent of the clear-water scour. A great many runs would be necessary to evaluate the sediment-transporting case adequately, but because the reduction is probably less than 25 percent, the effort does not seem warranted at this time.

The scour can also be less if the scour hole is riprapped, whether by self-sorting or artificially. In the case of the vertical sill, the depth of scour of the riprapped hole needs to be 50 percent more than it would be if the riprap were the sediment being scoured. This is because, at the natural limit, there is still action in the bottom of the scour hole; the surface material is moved about and the underlying material is exposed. Even if new material is not brought in for riprapping, it is suggested that the best practice would be to excavate a preformed scour hole and cover the bottom of the scour hole with the coarsest fraction of the excavated material. This assumes, of course, that there is coarse material in the native material; otherwise, it will have to be imported. This technique will ensure that all the coarse material will be available for riprap; otherwise, in the initial stages of scour-hole development, much coarse material can be lost.

The sloping sill would seem to be the preferable form for several reasons: The scour depth is less; the basic structure is just an earthen dike (although it needs to be protected by anchored riprap); and it can be added to easily if the tailwater continues to drop because of continued degradation.

SUMMARY

The thesis of this paper is that it is in the public interest that the vulnerability of bridges to floods be lessened. Bridges are expensive; they are vital links in our transportation system; and many of them are vulnerable to an unjustifiable (probably also unknown) degree.

In the design phase of new bridges, hydraulics and scour should be duly considered in planning, site selection, and final design. This is not to say that hydraulics should dictate design, merely that it should be properly considered and any vulnerability should be assessed and justified.

The matter of existing bridges, new and old, is much more difficult to work out. A program is needed to assess the vulnerability of every existing bridge and to contrive remedial measures to lessen the vulnerability of those bridges worth saving. This would be a program comparable to the ongoing bridge inspection program in size, in scope, and in importance. It would differ in that when a bridge has once been assessed for vulnerability to floods, only cursory inspection should be required to be sure hydraulic conditions have not changed.

There would be difficulty in carrying out such a program--partly because of the sheer number of bridges that would be involved; partly because there are questions regarding hydrology, hydraulics, and scour that remain unanswered or not answered adequately and convincingly.

In hydrology the most important and difficult question is the magnitude of the maximum flood to be considered in evaluating designs. In hydraulics the most important and difficult question is the two-dimensional flow pattern in the vicinity of the bridge, including the backwater due to the bridge. For scour the most difficult question is probably which scour-prediction relationship to use.

Further investigations are needed to better answer all these questions. In contriving remedial measures, other studies will be needed. To illustrate this contention, several exploratory studies have been referred to; the most promising of these is the one that indicated that the riprap size needed to arrest scour might be half that previously indicated.

Finally, the results of a laboratory investigation of the scour at the toe of sill structures are given in the form of scour-predicting equations for a vertical wall and for a sloping sill. The need to have this question answered was apparent to the Arizona Department of Transportation as they embarked on a program to re-evaluate the most vulnerable of their bridges and to contrive remedial measures for those too vulnerable.

ACKNOWLEDGMENT

This paper is based, in part, on two research projects. The first was sponsored solely by the Arizona Department of Transportation and was entitled "A Study to Advance the Methodology of Assessing the Vulnerability of Bridges to Floods." The second on scour at sills was sponsored by the Arizona Department of Transportation in cooperation with the FHWA as Research Project HPR-1-19(184).

REFERENCES

1. E.M. Laursen. Bridge Design Considering Scour and Risk. Journal of the Transportation Engineering Division, ASCE, Vol. 96, No. TE2, May 1970.
2. M.L. Corry, J.S. Jones, and P.L. Thompson. The Design of Encroachments on Flood Plains Using Risk Analysis. Hydraulic Engineering Circular 17. FHWA, U.S. Department of Transportation, Oct. 1980.
3. Scour at Bridge Waterways. NCHRP Synthesis of Highway Practice 5. TRB, National Research Council, Washington, D.C., 1970.
4. R.C. Kochel, V.R. Baker, and P.C. Patton. Paleohydrology of Southwestern Texas. Water Resources Research, Vol. 18, No. 4, Aug. 1982.
5. J.E. Costa. Response and Recovery of a Piedmont Watershed From Tropical Storm Agnes, June 1972. Water Resources Research, Vol. 10, No. 1, Feb. 1974.
6. A. Gupta and H. Fox. Effects of High Magnitude Floods on Channel Form: A Case Study in Maryland Piedmont. Water Resources Research, Vol. 10, No. 3, June 1974.
7. T. Blench. Regime Theory for Self-Formed Sediment-Bearing Channels. Transactions, ASCE, Vol. 78, 1952.
8. E. Silverston. The Stable Channel as Shaped to Flow and Sediment. Ph.D. dissertation. University of Arizona, Tucson, 1981.
9. A.J. Alawi. Effect of Velocity on Scour. M.S. thesis. University of Arizona, Tucson, 1981.
10. E.M. Laursen. An Analysis of Relief Bridge Scour. Journal of the Hydraulics Division, ASCE, Vol. 89, No. HY3, May 1963.
11. E.M. Laursen. Scour at Bridge Crossings. Journal of the Hydraulics Division, ASCE, Vol. 89, No. HY2, Feb. 1960.
12. E.M. Laursen. Predicting Scour at Bridge Piers and Abutments. General Report No. 3. A Study to Advance the Methodology of Assessing the Vulnerability of Bridges to Floods, Engineering Experiment Station, University of Arizona, Tucson, Feb. 1980.
13. E.V. Richardson, D. Simons, S. Karaki, K. Mahmood, and M. Stevens. Highways in the River Environment: Hydraulic and Environmental Design Considerations. FHWA, U.S. Department of Transportation, May 1957.
14. S.C. Jain and E.E. Fischer. Scour Around Circular Bridge Piers at High Froude Numbers. Report FHWA-RD-79-104. Iowa Institute of Hydraulic Research, University of Iowa, Iowa City, April 1979.
15. E.M. Laursen and D. Duffy. I-19 West Frontage Road Bridge (Old US 89) at Agua Fria Canyon Wash (North of Nogales). Site Report 1. Engineering Experiment Station, University of Arizona, Tucson, Feb. 1980.
16. E.M. Laursen. Bridge Backwater in Wide Valleys. Journal of the Hydraulics Division, ASCE, Vol. 96, No. HY4, April 1970.
17. H.K. Lui, J.N. Bradley, and E.J. Plate. Backwater Effects of Piers and Abutments. Report

CER57HKL10. Civil Engineering Department, Colorado State University, Fort Collins, 1957.

18. E.M. Laursen and N.J. Antonas. Scour and Backwater and Progressively Encroaching Embankments. Research Report 2. Engineering Experiment Station, University of Arizona, Tucson, Feb. 1980.
19. E. Naudascher and H.J. Medlarz. Hydrodynamic Loading and Backwater Effect of Partially Submerged Bridges. Journal of Hydraulic Research, Vol. 21, No. 3, 1983.
20. T. Carmody. A Critical Examination of the "Largest" Floods in Arizona. General Report No. 1. A Study to Advance the Methodology of Assessing the Vulnerability of Bridges to Floods, Engineering Experiment Station, University of Arizona, Tucson, Feb. 1980.
21. A. Elhasan. The Effect of Pier Shape and Angle of Attack on the Relative Depth of Scour. CE 300 Report. Department of Civil Engineering and Engineering Mechanics, University of Arizona, Tucson, July 1983.
22. E.M. Laursen and N.J. Antonas. Scour at Long, Thin Piers Which Are Closely Spaced. Research Report 1. Engineering Experiment Station, University of Arizona, Tucson, Feb. 1980.
23. E.M. Laursen and N.J. Antonas. Scour As Affected by a Spur Dike. Research Report 3. Engineering Experiment Station, University of Arizona, Tucson, Feb. 1980.
24. K.B. Marcus. Determination of the Size and Depth of Gravel Used in a Riprap Layer Around a Circular Pier. CE 900 Report. Department of Civil Engineering and Engineering Mechanics, University of Arizona, Tucson, Aug. 1983.
25. E.M. Laursen and M.W. Flick. Predicting Scour at Bridges: Questions Not Fully Answered--Scour at the Toe of a Vertical Wall. Report FHWA-AZ/83/184-2. Engineering Experiment Station, University of Arizona, Tucson, Aug. 1983.
26. E.M. Laursen. Predicting Scour at Bridges: Questions Not Fully Answered--Scour at the Toe of a Sloping Sill. Report FHWA-AZ/83/184-2. Engineering Experiment Station, University of Arizona, Tucson, Sept. 1983.
27. P.C. Klingeman. Prediction of Pier Scour in Western Oregon. FHWA, U.S. Department of Transportation, 1971.
28. G.R. Hopkins, R.W. Vance, and B. Kasraie. Scour Around Bridge Piers. Interim Report FHWA-RD-75-56. FHWA, U.S. Department of Transportation, March 1975.
29. J.N. Bradley. Hydraulics of Bridge Waterways. Hydraulic Design Ser. 1. FHWA, U.S. Department of Transportation, 1970.

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

Computer-Based Prediction of Alluvial Riverbed Changes

F.M. HOLLY, JR., T. NAKATO, and J.F. KENNEDY

ABSTRACT

Recent investigations and research programs at the Iowa Institute of Hydraulic Research have involved both the analysis and development of computer-based simulation techniques for alluvial riverbed evolution. The primary use of such techniques is in the prediction of riverbed aggradation and degradation caused by perturbations in the river's equilibrium geometry and sediment inflow supply over extended reaches. In this paper the mathematical basis of the problem is reviewed and several general numerical approaches and associated difficulties are described. Seven published programs are then described, and their performance when applied to three actual field situations is compared. The conclusions point out a critical dependence on field data and identify the need for further research in understanding physical mechanisms such as sediment sorting, armoring, scour, and deposition.

Mother Nature, in providing the Earth with a system of drainage channels to return surface waters to the sea, has endowed man in general, and river engineers in particular, with both a blessing and a curse. The blessing is that rivers whose channels are formed of loose, noncohesive alluvium are able to adjust their geometry to carry widely varying discharges with only moderate changes in water-surface elevation. The curse is that river engineers have found this self-regulating mechanism extremely difficult to understand and accommodate in their projects.

The sheer complexity of alluvial river response, which involves dozens of relevant variables and even ambiguity as to which are the dependent and independent ones, has defied attempts to formulate a coherent, reliable, "desktop" methodology for alluvial river design. Although field experience and laboratory tests have led to the establishment of fairly reliable procedures for the prediction of local scour around bridge piers, bank stability, and other such local phenomena, no such procedures exist for the analysis of alluvial riverbed and bank changes over long river reaches and extended periods of time.

The design engineer's interest in alluvial river response is generally focused on anticipating how the riverbed and water-surface elevations will change if an existing stable or equilibrium situation is perturbed. This perturbation may be the