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The Observational Method for Scour and the Schoharie Creek Bridge Failure

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ABSTRACT

The observational method for estimating the future scour depth at existing bridges was introduced by Briaud et al. (2009) and Govindasamy (2009). The method utilizes measured scour data and observed or estimated flow parameters at a bridge to evaluate the future scour depth at an existing bridge. It provides more realistic scour risk estimates due to the fact that it utilizes measured data and accounts for time dependent scour depth in clays. Other important features of the method are its ability to recognize and efficiently filter scour depths exceeding foundation allowable values and also account for scour in multilayered soil deposits. The Schoharie Creek bridge failure of 1987 was selected as a case history to illustrate the how the observational method would have identified the bridge as requiring immediate attention if it was used to evaluate the bridge prior to its collapse, hence preventing the serious consequences of the disaster.

Introduction

The observational method for estimating the future scour depth at existing bridges was introduced by Briaud et al. (2009) and Govindasamy (2009). The method utilizes measured scour data and observed or estimated flow parameters to evaluate the future scour depth at an existing bridge. The observational method provides more realistic scour risk estimates due to the fact that it utilizes measured data and accounts for time dependent scour depth in clays. The method also does not require site specific erosion testing and therefore reduces the effort and cost associated with evaluating a bridge for scour. The features of the observational method that form the crux of this study are its ability to recognize and efficiently filter scour depths exceeding allowable (threshold) values for foundations (more specifically in this case, footings) and also account for multilayered deposits, namely the presence of a strong layer overlying a weak layer. A case history was selected to highlight the importance of these features in bridge scour predictions. The case history is the Schoharie Creek bridge failure of 1987 (Figure 1).

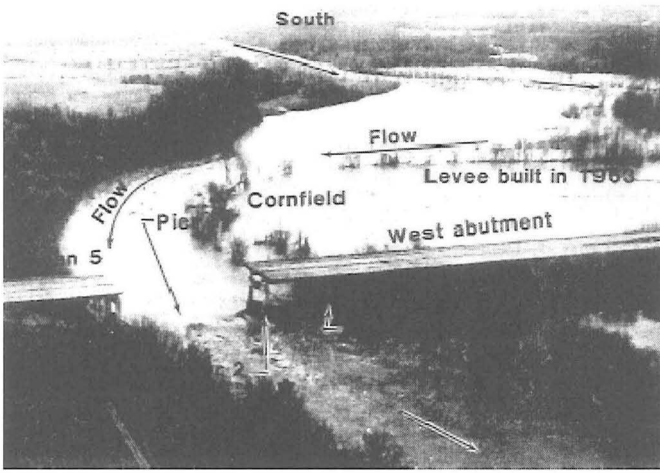


Figure 1. The 1987 Schoharie Creek Bridge Failure (NTSB 1987).

The Observational Method for Scour

The observational method is the first phase of a three-phase bridge scour assessment procedure. The main idea behind the method is to obtain the scour depth corresponding to a specified future flood event using scour depth observations at the site and from charts that relate the scour depth ratio (Z_{fut}/Z_{mo}) to the velocity ratio (V_{fut}/V_{mo}). Z_{fut} is the scour depth corresponding to a specified future flood, Z_{mo} is the maximum observed scour at the bridge, V_{fut} is the velocity corresponding to the specified future flood, and V_{mo} is the maximum velocity observed at the bridge until the time Z_{mo} is measured. These charts are termed the Z-Future Charts (Briaud et al. 2009 and Govindasamy 2009). The velocity ratio, V_{fut}/V_{mo} is obtained through a simplified hydrologic analysis. The general steps in the observational method are outlined in Table 1.

Table 1. General Steps in the Observational Method for Scour

Step 1	Observe the maximum scour at the bridge Z_{mo}
Step 2	Determine the velocity ratio V_{fut}/V_{mo}
Step 3	Extrapolate/interpolate field measurements to predict future scour depth using the Z-Future Charts. This is represented by $Z_{fut}/Z_{mo} = f\,(V_{fut}/V_{mo})$.
Step 4	Compare the future scour depth Z_{fut} to the allowable (threshold) scour depth of the foundation Z_{thresh}

As mentioned previously, the observational method can account for scour in a uniform soil deposit as well as in a multilayered soil deposit. For the latter, the procedure for obtaining hydraulic information is extended to obtain explicit values of V_{fut} and V_{mo}. The reader is referred to Briaud et al. (2009) for a detailed description of this procedure.

Overview of the Schoharie Creek Bridge Failure

The bridge was a five-span, 540-ft long highway bridge over the Schoharie Creek in Montgomery County near Amsterdam, New York (National Transportation Safety Board 1987). The bridge was built in 1954 and was founded on spread footings that were approximately 19 ft wide and 5 ft thick. On April 5, 1987, one of the piers of the bridge (Pier 3) collapsed, causing two spans of the bridge to plunge into the creek (Figure 2). This was followed by the collapse of an adjacent pier (Pier 2). The failure of this bridge caused the deaths of 10 people. The cause of the failure was attributed to scour (National Transportation Safety Board 1987; Resource Consultants, Inc., and Colorado State University 1987; Wiss, Janney, Elstner Associates, Inc., and Mueser Rutledge Consulting Engineers 1987).

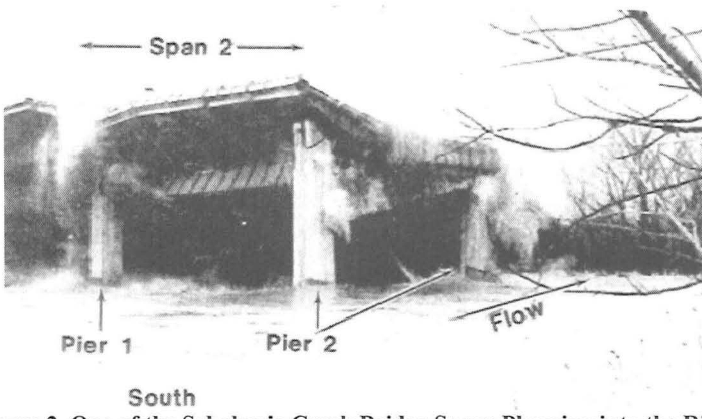


Figure 2. One of the Schoharie Creek Bridge Spans Plunging into the River (NTSB 1987).

Flow History at the Schoharie Creek Bridge

The bridge experienced its largest flood in 1955. The second largest flood was the flood that took place in 1987 during the failure of the bridge. According to the National Transportation Safety Board (NTSB) (1987), the magnitudes of both floods (peak) were $Q_{peak,1955} = 2084\ m^3/s$ (73,600 cfs) and $Q_{peak,1987} = 1758\ m^3/s$ (62,100 cfs), respectively. The flow velocities at Pier 3 were obtained from the one-dimensional flow computer model, Water-Surface Profile Computations or WSPRO (FHWA 1986). The computer simulations were carried out by Resource Consultants, Inc., and presented by NTSB (1987) (Table 2).

Table 2. Peak Discharge versus WSPRO Mean Velocity at Schoharie Creek Pier 3 (after NTSB 1987).

Peak Discharge (cfs)	WSPRO Mean Velocity (ft/s)
10,000	3.6
20,000	5.5
30,000	7.0
40,000	8.2
50,000	9.4
60,000	10.3

The flow-velocity data shown in Table 2 were plotted and shown in Figure 3. A regression was performed on the data to obtain the flow-velocity relationship. The regression produced an R^2 value of 0.99. Using the relationship shown in Figure 3, the flow values $Q_{peak,1955} = 2084$ m^3/s (73,600 cfs) and $Q_{peak,1987} = 1758$ m^3/s (62,100 cfs) translate into velocities $V_{peak,1955} = 3.6$ m/s (11.8 ft/s) and $V_{peak,1987} = 3.2$ m/s (10.5 ft/s), respectively.

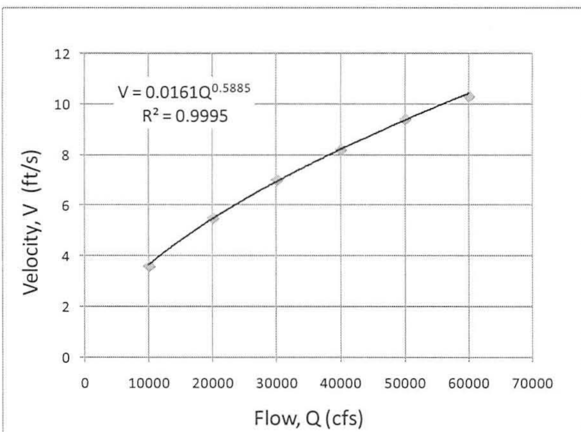


Figure 3. Flow-Velocity Relationship for Schoharie Creek Pier 3.

Previous and Current Investigations into the Failure

Prior investigations into the failure revealed that riprap was placed at the bridge piers prior to 1955 as protection against scour. NTSB (1987) states, "At Piers 2 and 3, riprap was installed from bottom of footing (elevation 270 ft) sloping to elevation 279.5 ft prior to the 1955 flood. Therefore, at Pier 3 the thickness of the riprap was approximately 9.5 ft (Figure 4). Photos taken on October 30th 1956 showed riprap movement at Piers 2 and 3. Various photographs taken from 1954 to 1977 during low water showed that some of the rocks had moved northward (downstream) during that time. Photographic analysis of Pier 2 (aided by computers) confirms the downstream movement of rock at Pier 2 from 1954 to 1977." Figure 5 shows Pier 3 in 1956. Figure 6 shows Pier 2 in 1977.

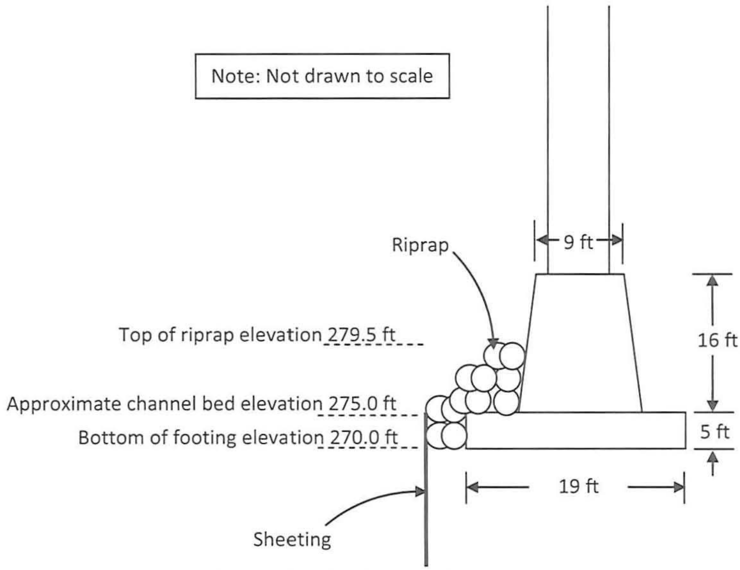


Figure 4. Schoharie Creek Pier 3 (after NTSB 1987).

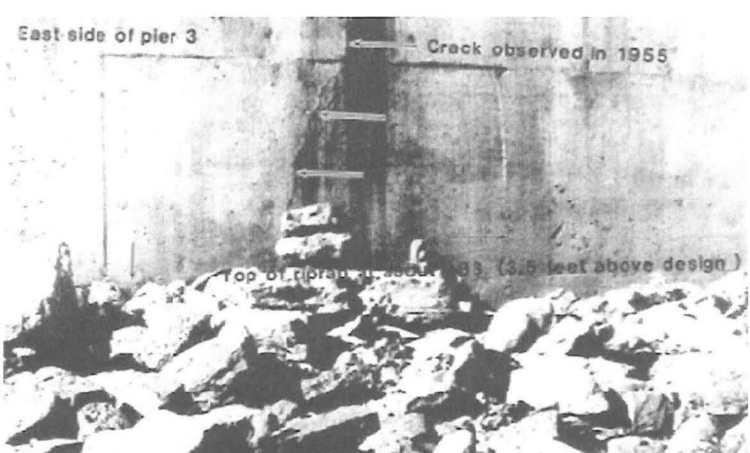


Figure 5. Photo of Pier 3 Taken in 1956 (NTSB 1987).

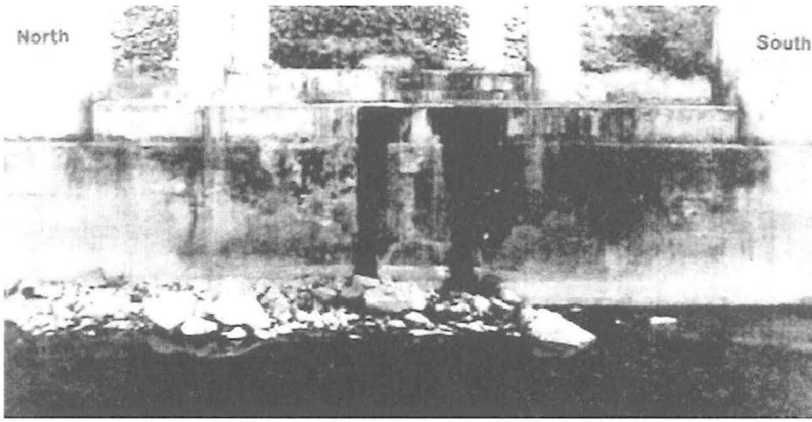


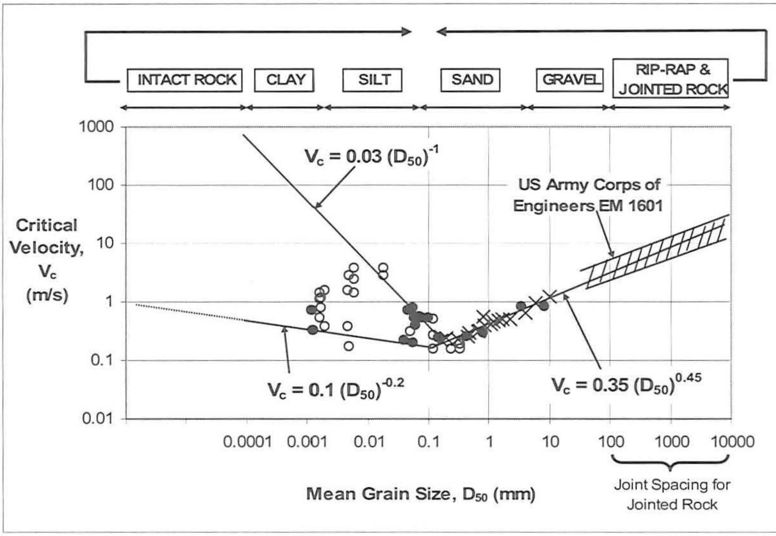
Figure 6. Photo of Pier 2 Taken in 1977 (NTSB 1987).

With reference to the riprap placed at the bridge prior to the 1955 flood, NTSB (1987) states, "The only riprap dimensions specified in the bridge plans should be a minimum thickness of 8 inches and a maximum thickness of 15 inches. The plans also call for the riprap to be an Item 80 riprap according to the New York Department of Public Works (DPW) specifications. An Item 80 riprap should have at least 50% of the stones weighing in excess of 300 lbs each."

In order to obtain the critical velocity of the riprap, the Erosion Threshold Chart (Briaud et al. 2009, Govindasamy 2009) is used (Figure 7) This chart relates the mean particle diameter D_{50} to the critical velocity V_c. For D_{50} = 8 inches = 203 mm:

$$V_c\,(m/s) = 0.35[D_{50}(mm)]^{0.45}$$

$$V_c = 0.35(203)^{0.45} = 3.8\ m/s\ (12.5\ ft/s)$$



Legend:

- ● TAMU Data as reported by Briaud, J.-L. et. al. (2001). "Erosion Function Apparatus for Scour Rate Predictions." *J. Geotech. and Geoenvir. Engrg.,* ASCE, 127(2), 105-113.
- o TAMU Data as reported by Briaud, J.-L. (2006). "Erosion Tests on New Orleans Levee Samples." *Texas A&M University Internal Report.*
- x Data from Shields, Casey, US.WES, Gilbert, White as reported by Vanoni, V.A., ed. (1975). "Sedimentation Engineering." *ASCE manuals and reports on engineering practice,* ASCE, New York.

Figure 7. The Z-Threshold Chart (Govindasamy 2009)

For a DPW Item 80 riprap, assuming a spherical piece of riprap weighing 136 kg (300 lb) and with a specific gravity S_g=2.65, results in a diameter of 460 mm (1.5 ft).

Again from the Figure 7:

$$V_c\ (\text{m/s}) = 0.35[D_{50}(\text{mm})]^{0.45}$$

$$V_c = 0.35(460)^{0.45} = 5.5\ \text{m/s}\ (18.1\ \text{ft/s})$$

It should be noted that the critical velocity of 5.5 m/s (18.1 ft/s) is for a non-porous spherical boulder.

However, NTSB (1987) states, "field observations and photographs indeed showed movement of riprap between 1954 and 1977, the critical velocity, V_c of the riprap should be less than 3.6 m/s, which is the largest flood velocity experienced at the Schoharie Creek bridge." It goes on to state, "it is evident that there was riprap movement between 1956 and 1977." The maximum flow between 1956 and 1977 was 1144 m^3/s (40,400 cfs) (National Transportation Safety Board 1987), which corresponds to an approach velocity of 2.5 m/s (8.3 ft/s). Therefore, it is reasonable to assume that the critical velocity of the riprap should be below 1.5 times the approach velocity, 3.75 m/s. This is the local velocity at the pier. In order to illustrate the erodibility of the riprap, the Erosion Function Chart (Briaud et al. 2009, Govindasamy 2009), which relates the erosion rate to velocity is used (Figure 8). Approximating V_c of the riprap as 3.5 m/s (below 3.75 m/s), the upper boundary of a Category V material (very low erodibility) in Figure 8 can be taken as the approximate erosion function of the riprap. According to Resource Consultants, Inc., and Colorado State University (1987), V_c of the glacial till = 1.5 m/s (4.9 f/s). The upper boundary of a Category IV material (low erodibility) is translated to the right so that the critical velocity corresponds to the critical velocity of the glacial till (Figure 8).

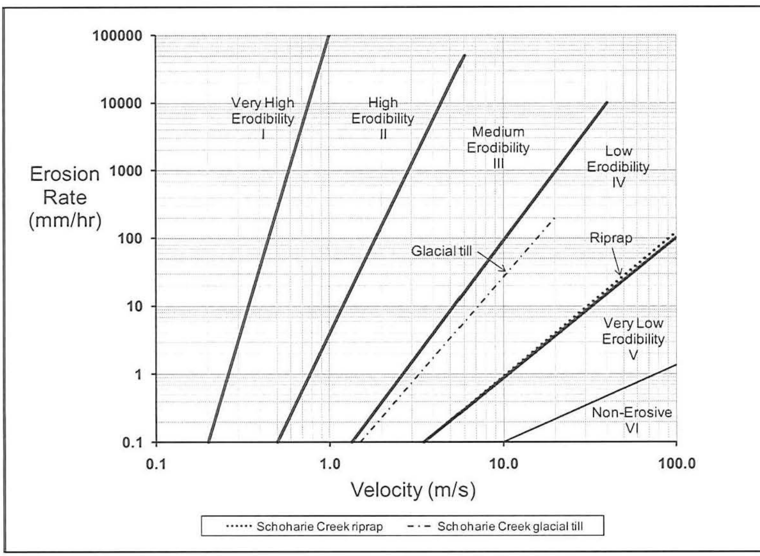


Figure 8. Estimated Erosion Functions for the Schoharie Creek Riprap and Glacial Till.

Through prior investigations into the Schoharie Creek bridge failure, it was found that the 1955 flood and following smaller floods caused the riprap to move between 1955 and prior to the 1987 collapse. Since the riprap was placed down to the bottom level of the footing, it is believed that there was still some remaining riprap just prior to the 1987 flood. Otherwise, the erosion would have undermined the footing before the 1987 flood. Since the velocity of the 1987 flood is believed to have been greater than V_c of the riprap (although the previous sections of this paper approximate $V_{peak,1987}$ to be slightly smaller than V_c based on D_{50}), it is highly likely that the 1987 flood moved the remaining riprap, thus exposing the more erodible glacial till beneath. As shown in Figure 8, the till was more erodible than the riprap.

Once the till was exposed, the footing was undermined, very rapidly causing the bridge to fail.

Therefore, the reason for the Schoharie Creek Bridge failure under a lesser flood in 1987 than the flood of 1955 is a multilayer deposit response and not a uniform deposit response. Indeed, during the 1955 event, the scour hole remained in the riprap, while in 1987 it eroded what was left of the riprap (strong layer) and rapidly advanced in the glacial till below (weak layer).

Conclusion

If the observational method for scour presented by Briaud et al. (2009) and Govindasamy (2009) was used to evaluate the Schoharie Creek bridge prior to its collapse, it would have identified the bridge as requiring immediate attention. This is because Z_{thresh} would have been exceeded (for footings, Z_{thresh} is normally taken as the length between the original as-built channel level and the top of the footing). In the case of the Schoharie Creek bridge, the riprap below the top of footing level had moved prior to the 1987 collapse. Moreover, the method also has provisions to account for multilayer deposit response and would have accounted for the rapid erosion in the more erodible glacial till underlying the riprap.

Acknowledgements

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References

1. Briaud, J.-L., F.C.K. Ting, H.-C. Chen, Y. Cao, S.W. Han, and K.W. Kwak (2001a). "Erosion Function Apparatus for scour rate predictions." Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Vol. 127, No. 2, pp. 105-113.

2. Briaud, J.-L. (2006). "Erosion Tests on New Orleans Levee Samples," Internal Report, Zachry Department of Civil Engineering, Texas A&M University, College Station, Texas.

3. Briaud J.-L., A.V. Govindasamy, D. Kim, P. Gardoni, F. Olivera, H. C. Chen, C. Mathewson, and K. Elsbury (2009). "Simplified Method for Estimating Scour at Bridges," Report 0 5505-1, Texas Department of Transportation, Austin, Texas.

4. Federal Highway Administration (1986). "Bridge Waterways Analysis Model (WSPRO) User's Manual," Report No. FHWA-IP-87-3 Draft Report.

5. Govindasamy, A.V. (2009). "Simplified Method for Estimating Future Scour Depth at Existing Bridges," Ph.D. Dissertation, Texas A&M University, College Station, Texas.

6. National Transportation Safety Board (1987). "Collapse of the New York Thruway (I-90) Bridge over the Schoharie Creek, near Amsterdam, New York, April 5, 1987." Highway Accident Reports, NTSB/HAR-88/02, United States Government, Washington, D.C.

7. Resource Consultants, Inc., and Colorado State University (1987). "Hydraulic, erosion, and channel stability analysis of the Schoharie Creek Bridge failure, New York." Report for the National Transportation Safety Board, Washington, D.C., and the New York State Thruway Authority, Albany, New York.

8. Vanoni, V. (1975). "Sedimentation Engineering," ASCE Manual and Report on Engineering Practice, American Society of Civil Engineers, New York.

9. Wiss, Janney, Elstner Associates, Inc., and Mueser Rutledge Consulting Engineers (1987). "Collapse of the Thruway Bridge at Schoharie Creek for New York State Thruway Authority, Albany, NY."