Shear Stress at Base of Bridge Pier

PEGGY A. JOHNSON AND J. STERLING JONES

The vortex motion around an obstruction in a movable bed is extremely complex. To model pier scour as a function of the size and strength of the vortex at the base of the pier, it may be desirable to characterize the strength of the vortex under various conditions in terms of a measurable quantity. The vortex strength is characterized in terms of stress at the base of a bridge pier as a function of pier width and scour depth. A flume experiment was conducted to determine indirectly the relative magnitudes of shear stress for various pier diameters and scour depths using a sediment that is uniform in size and shape. One advantage of the experimental method is that no instrumentation was required in the scour hole; therefore, there was no interruption of the flow pattern around the pier or within the scour hole. From the data, a relationship between the shear stress and the equilibrium scour depth may be developed.

Bridge pier scour can be modeled using a variety of methods. One approach is to derive a theoretical equation or set of equations to describe the scour process. Because the scour process around a bridge pier is extremely complex, many simplifications and assumptions are required to obtain an analytical model. The maximum scour depth, rather than the scour process, is more commonly modeled empirically as a function of various scour parameters such as pier width and approach flow characteristics. This method shows the effect of individual parameters on the maximum depth of scour.

Another approach to modeling pier scour is to analyze the vortex at the base of the pier. The vortex is believed to be directly responsible for the occurrence of scour holes at the base of bridge piers (*1*). After making assumptions and necessary simplifications, the size and strength of the vortex is modeled to determine the amount of erosion that will be caused by the vortex. To model pier scour as a function of the size and strength of the vortex, it is necessary to understand the effect of pier width, hydrologic conditions, and scour depth on the magnitude of the vortex strength. There are many difficulties in modeling pier scour by this approach; the vortex motion around an obstruction in a movable bed is extremely complex. It may be desirable, therefore, to characterize the strength of the vortex under various conditions in terms of a measurable quantity.

The objective of this study is to characterize the vortex strength in terms of shear stress at the base of a bridge pier as a function of pier width and scour depth. This study should provide information about the magnitude of the shear stress for the purpose of predicting both scour depths and rates.

P. A. Johnson, Department of Civil Engineering, University of Maryland, College Park, Md. 20742. J. S. Jones, Turner-Fairbank Highway Research Center, Federal Highway Administration, McLean, Va. 22101.

PREVIOUS STUDIES

Although a number of studies reported in the literature have been aimed at modeling the vortex at the base of an obstruction, few studies exist in which measurements of velocity or shear stress within the vortex or at the upstream face of the pier are reported. Melville (*2*) indirectly measured velocities in the diving current along the upstream face of a model pier by measuring the velocities at various points around the pier and using trigonometric relationships to obtain the downflow velocities. He found that the magnitude of the vertical velocity in the diving current is a maximum near the surface of the scour hole. As the diving current approaches the base of the scour hole, the vertical velocity decreases. Melville also found that the shear stress within the hole decreases as the hole deepens, indicating that the vortex strength diminishes with depth. He concluded that the downflow velocity is a function of the approach flow velocity and the pier width; however, Melville's experiment was conducted using one pier size, so there were no data with which to correlate the effect of pier size on either the shear stress or the downflow velocity.

Shen et al. (*3*) used potential flow theory to determine the vertical velocity in the diving current near the pier. He found that the maximum vertical velocity is equal to the approach flow velocity and that the shear stress at the bottom of the scour hole is approximately equal to the shear stress of the approach flow at maximum scour condition.

A method of determining the approach velocity at which riprap around a bridge pier will fail was developed by Parola (*4*). In his experiment, Parola set a 4-in. model bridge pier in sand, scoured a hole to a predetermined depth, stabilized both the scour hole and bed surface, then lined the hole with ¼-in. gravel. He then introduced a flow to the flume, gradually lowered the tailgate, and watched for failure (i.e., movement) of the gravel within the hole. When the gravel failed, he measured the upstream flow depths and velocities at various points along a cross section. He repeated the experiment for various scour depths and two pier configurations. He assumed that the effective velocity at the pier at the time of failure of the gravel was equal to the velocity of incipient motion for that particle size. Parola found that the effective velocity at the pier was approximately 1.5 times the approach velocity required to cause failure of the riprap for a circular pier and 1.7 times the approach velocity for a rectangular pier. Shear stress is a function of velocity squared; hence the effective shear stress at the pier is on the order of 2.25 to 2.90 times the shear stress of the approach flow. This indirect approach to "measuring" velocity and shear stress at a pier was the basis for the design of the experiment in this study.

EXPERIMENTAL APPROACH

The experiment to determine indirectly the shear stress within a scour hole at the base of a pier was conducted in the FHWA Hydraulics Laboratory at the Turner-Fairbanks Highway Research Center in McLean, Virginia. The experiment was conducted in a rectangular flume 6 ft wide and 70 ft long in the center of which was a recessed section 8 ft long, 6 ft wide, and 1½ ft deep. Marbles were used instead of gravel (as in Parola's experiment) in an attempt to reduce scatter in the data. Gravel varies significantly in shape and size, thus the particles tend to interlock, causing large variations in the initial movements of the sediment. Marbles were chosen because they are uniform in size and shape.

The experiment was divided into two phases: (a) an unobstructed flow experiment to determine the threshold of movement for the marbles, and (b) an experiment to determine shear stresses within a scour hole for different pier sizes.

Unobstructed Flow

The purpose of this portion of the study was to determine the threshold shear stress that would just cause the marbles to move. To prepare the bed of the flume, one layer of marbles was glued to the bed from the headbox to a distance downstream of the observation area in order to establish the proper flow resistance. At the observation area, a layer of red marbles was glued to a level board placed in the recessed area. On top of the red marbles, a single layer of yellow marbles was placed; these marbles were free to move.

After the flume was prepared, a flow rate of about 6 ft^3/sec was supplied to the flume with the tailgate in an upright position such that the flow depth was great enough and the velocity low enough that the marbles would not move in the observation area. The tailgate was then lowered very slowly, by small increments, so that steady, uniform flow could be assumed. The marbles in the observation area were observed closely so that movement could be detected. When a discernible patch of red appeared, the tailgate was held at its position and upstream velocity and flow depth measurements were recorded.

This process was repeated four times. The shear stress necessary to cause movement of the marbles was then computed as a function of the average flow velocity and depth by using the integrated form of the assumed logarithmic velocity distribution:

$$\tau_o = \frac{\rho V^2}{\left[5.75 \log\left(12.27 \frac{y_o}{k_s}\right)\right]^2} \qquad (1)$$

where

τ_o = average shear stress on the channel bottom upstream of the pier,
V = average approach velocity,
y_o = flow depth, and
k_s = particle diameter.

For the second portion of the study, the value of τ_o was assumed to be the effective shear stress at the base of a pier, τ_p, at the time of failure of the marbles. The value of τ_p is thus equal to the average of τ_o for the four runs.

Shear Stress Within the Scour Hole

The flume was next prepared to develop a scour hole so that the shear stress within the hole could be determined. A model bridge pier, constructed from 4-in.-diameter PVC pipe, was attached to a square Plexiglas plate and bolted to the floor of the flume in the recessed section. The recessed section was then filled with a medium-grained sand, saturated with water, and leveled across the surface. A flow rate of about 6 ft^3/sec was supplied to the flume with the tailgate sufficiently raised so that scour would not occur. The scour hole was then developed by lowering the tailgate and increasing the velocity until the sand just began to move (approximately incipient motion). The flow was maintained at this velocity and depth for about 2 hr to develop a scour hole of maximum depth around the pier. After the 2-hr, the flume and scour hole were drained. The hole was stabilized with an epoxy spray and allowed to dry for about 24 hr. After drying, the sand surface was leveled, and care was taken not to disturb the scour hole. The surface was then cemented so that the bed would remain in place at higher velocities. The bed was again allowed to dry for 24 hr.

To determine indirectly the shear stress at the base of the scour hole, the hole was lined with 9⁄16-in. marbles. First it was lined with yellow marbles, then with white. When the white marbles began to move, the event was clearly visible as the yellow appeared. Failure of the marbles was then defined as the appearance of a discernible path of yellow. A flow rate of about 6.5 ft^3/sec was supplied to the flume with the tailgate raised sufficiently high to prevent movement of the marbles in the scour hole. The tailgate was lowered slowly in intervals such that the assumptions of steady, uniform flow were not violated significantly. When the white marbles began to move and a discernible patch of yellow was detected, the tailgate was stopped and flow measurements were taken. Measurements of the flow depth and velocity were taken in four locations across each of two cross sections upstream of the model pier in the undisturbed flow. Velocity measurements were taken at 20, 40, 60, and 80 percent of the flow depth and at ¼ in. from the channel bottom using a Nixon propeller meter.

After the flow measurements were completed, the experiment was repeated in the same scour hole (which had been relined with marbles), for flow rates of about 7.5 and 9.0 ft^3/sec. Flow measurements were again recorded after a yellow patch appeared.

The entire process was repeated for other predetermined scour depths. The scour hole was refilled with sand and a new hole scoured, this time to a shallower depth. The scour hole and bed were fixed as described above, the hole lined with marbles, and the three flow rates supplied to the flume. This process was repeated until the depth of the scour hole was zero, that is, level with the flume bed. After flow measurements were made for the scour depth of zero, the "pier" was removed and the experiment repeated for 6- and 10-in. model piers.

RESULTS

The unobstructed flow experiment resulted in four values of τ_o computed from Equation 1. The average shear stress for the four runs was 0.052 lb/ft^2 with a coefficient of variation of 0.077.

The scour hole experiments resulted in flow measurements for 45 experimental runs. The data, including the pier diameter, flow depth, flow velocity, scour depth, flow rate, and Froude number, are listed in Table 1. The flow velocities and depths represent the values that caused failure (i.e., a discernible patch of yellow) for a particular pier diameter, flow rate, and scour depth.

TABLE 1 EXPERIMENTAL RESULTS

Pier Width (ft)	Flow Depth (ft)	Velocity (ft/s)	Scour Depth (in)	Fr #	Q (cfs)	τ (lb/ft^2)	τ_p/τ_a
0.375	0.854	1.833	0	0.350	9.38	0.036	2.16
0.375	0.708	1.796	0	0.376	7.63	0.037	2.09
0.375	0.638	1.690	0	0.373	6.46	0.034	2.27
0.375	0.850	1.829	0.625	0.350	9.32	0.036	2.16
0.375	0.768	1.701	0.625	0.342	7.83	0.032	2.40
0.375	0.578	1.768	0.625	0.410	6.12	0.039	2.00
0.375	0.818	1.914	1.5	0.373	9.39	0.040	1.95
0.375	0.655	1.961	1.5	0.427	7.70	0.045	1.70
0.375	0.614	1.851	1.5	0.417	6.81	0.041	1.86
0.375	0.713	2.128	2.375	0.444	9.10	0.052	1.49
0.375	0.563	2.275	2.375	0.534	7.68	0.065	1.19
0.375	0.494	2.264	2.375	0.568	6.71	0.067	1.14
0.375	0.714	2.131	3.25	0.445	9.12	0.052	1.49
0.375	0.596	2.141	3.25	0.489	7.66	0.056	1.38
0.375	0.523	2.130	3.25	0.519	6.68	0.058	1.32
0.549	0.926	1.615	0	0.296	8.97	0.027	2.86
0.549	0.808	1.574	0	0.309	7.62	0.027	2.86
0.549	0.722	1.576	0	0.327	6.82	0.028	2.74
0.549	0.904	1.634	1	0.303	8.86	0.028	2.77
0.549	0.682	1.779	1	0.380	7.27	0.037	2.10
0.549	0.620	1.686	1	0.378	6.27	0.034	2.25
0.549	0.790	1.895	2	0.376	8.98	0.039	1.96
0.549	0.674	1.845	2	0.396	7.46	0.040	1.95
0.549	0.546	1.939	2	0.463	6.35	0.048	1.62
0.549	0.854	1.811	3	0.346	9.28	0.035	2.21
0.549	0.686	1.866	3	0.397	7.68	0.040	1.91
0.549	0.618	1.799	3	0.403	6.66	0.039	1.98
0.549	0.812	1.875	4	0.367	9.13	0.038	2.02
0.549	0.676	1.911	4	0.410	7.75	0.042	1.81
0.549	0.592	1.980	4	0.454	7.03	0.048	1.61
0.828	0.934	1.629	4.5	0.297	9.12	0.027	2.82
0.828	0.737	1.773	4.5	0.364	7.84	0.035	2.18
0.828	0.630	1.739	4.5	0.386	6.57	0.036	2.13
0.828	0.814	1.891	3.1	0.370	9.23	0.039	1.99
0.828	0.722	1.794	3.1	0.372	7.77	0.036	2.11
0.828	0.646	1.689	3.1	0.370	6.54	0.034	2.28
0.828	0.817	1.908	2	0.372	9.35	0.039	1.96
0.828	0.786	1.669	2	0.332	7.86	0.031	2.52
0.828	0.648	1.775	2	0.389	6.90	0.037	2.07
0.828	0.944	1.634	1.5	0.296	9.25	0.027	2.81
0.828	0.852	1.515	1.5	0.289	7.74	0.024	3.15
0.828	0.716	1.518	1.5	0.316	6.51	0.026	2.94
0.828	1.058	1.441	0	0.247	9.14	0.020	3.77
0.828	0.914	1.431	0	0.264	7.84	0.021	3.62
0.828	0.726	1.530	0	0.317	6.66	0.026	2.91

Scour Depth

Clearly, for a given pier diameter and flow depth, the flow velocity must increase to cause failure of the marbles as the scour hole deepens. For the 6-in.-diameter pier and a flow depth of approximately 0.7 ft, the approach velocity increases from 1.58 ft/sec, when the scour depth is 0, to 1.77 ft/sec, when the scour depth is 4.5 in.

Assuming that the critical shear stress of 0.052 lb/ft^2 is the shear stress at the pier when the marbles fail, ratios of τ_p (shear stress at the pier) to τ_a (bed shear stress of the approach flow causing the marbles to fail) may be computed for various scour depths. These values are also given in Table 1. Equation 1 was used to compute values of τ_a. Figure 1 shows the decrease in τ_p/τ_a with increasing scour depth for pier diameters of 4 and 6 in. The scour depth in Figure 1 is normalized by the maximum (equilibrium) scour depth obtained for each of the pier diameters. For the 6-in. pier,

$$\tau_p/\tau_a = 2.74 \text{ (at surface)}$$

$$\tau_p/\tau_a = 1.81 \text{ (at } d_s = 4.0 \text{ in.)}$$

where d_s is the scour depth. The depth of two layers of marbles within the hole is 1.1 in.; therefore, the shear stress ratios could not be evaluated at the absolute base of the scour hole. If the scour process stops when the shear stress at the base of the hole is equal to the shear stress on the channel bottom, then τ_p/τ_a must approach 1 as the bottom of the scour hole is approached. Knowledge of this effect is useful in modeling a time-dependent function of the scour process, particularly when cohesive materials are a concern.

Pier Width

It is well recognized that bridge pier width has a significant effect on the depth of scour (*5–8*); in general, the wider the pier, the greater the scour depth. To determine the effect of pier width on the shear stress at the pier, three pier diameters were tested. As shown in Figure 2, for a plane bed (d_s = 0),

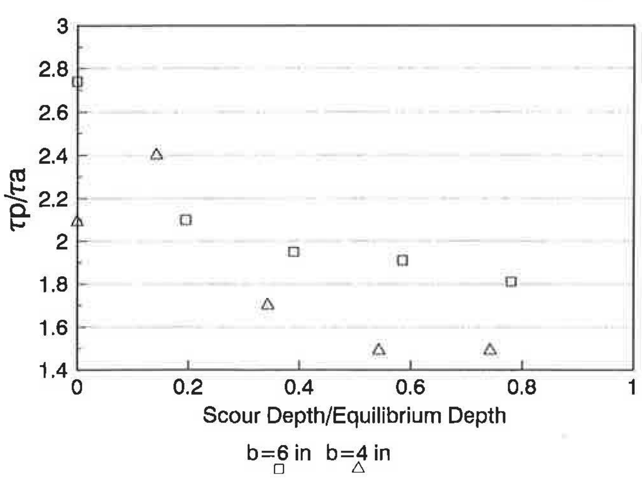

FIGURE 1 Shear stress as function of scour depth (flow depth = 0.7 ft).

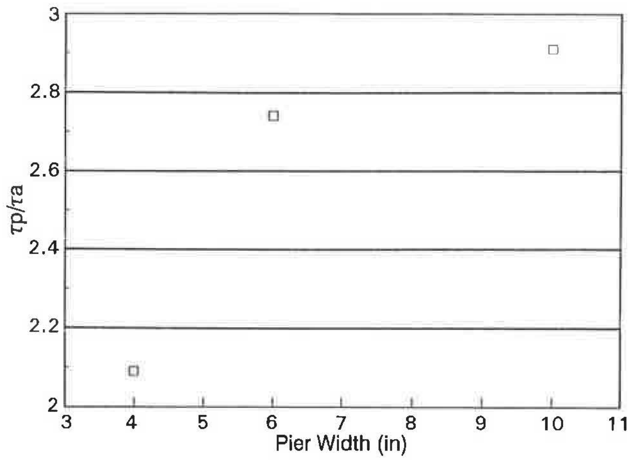

FIGURE 2 Shear stress as function of pier width (flow depth = 0.7 ft, scour depth = 0).

τ_p/τ_a increased nonlinearly as the pier diameter was increased. Figure 2 shows that the change in shear stress is greater between the smaller pier widths. The shear stress ratios varied from 2.09 to 2.91; therefore, for an approximate twofold increase in pier diameter, the shear stress ratio increased about 1.4 times. This result is in agreement with other studies, showing the increase in scour depth for larger pier diameters (*8*). Clearly, the increase in pier diameter is responsible for an increase in shear stress at the pier, which, in turn, is responsible for the increase in scour depth. A comparison of the results reported here to those obtained by Parola (*4*) show the results of this study to be within the range of ratios computed from Parola's data.

Although the data reported here are in agreement with data from previous studies, caution should be exercised in using them. For a flow depth of 0.7 ft, pier diameters of 4, 6, and 10 in. correspond to flow-depth–to–pier-width ratios, y/b, of 2.1, 1.4, and 0.8, respectively; therefore, these results have most likely included shallow water effects. Melville and Sutherland (*9*) found that the shallow water effect could be overcome or accounted for by the use of a factor (K) ranging from 0 for y/b = 0 to 1 for y/b greater than about 3.5, that is, when y/b is greater than about 3.5, shallow water effects are negligible. For y/b less than about 3.5, scour depth increases with flow depth.

Sources of Error

There are several possible sources of error for the shear stress ratios reported here. Determining the critical velocity for marbles was difficult: they tended to roll and bunch up rather than be lifted and moved across the bed surface. Also, the specific gravity (2.445, rather than 2.65) and the shape of the marbles differ from those of 9/16-in. gravel (spherical as opposed to irregular), so established curves of incipient motion, such as Shields' curve, will yield incorrect results. In addition, it was assumed that a single value of the average critical shear stress (τ_c = 0.052 lb/ft^2) could be used as the shear stress at the pier at the time of failure. There are two possible sources of error due to this assumption. First, τ_c was based on an average of four values. The coefficient of variation was quite low, so the average is most likely a reasonable value; however, four data points constitute a rather small sample size, so the actual value of τ_c could vary somewhat from the computed value. Second, τ_c is presented as a deterministic (constant) value. In actuality, there is a random component associated with the critical shear stress, however this was ignored in the analysis of the data.

Another source of error may be due to the subjectivity of the experiment. It was critical that one observer perform all experiments so that failure of the marbles be determined in the same way each time. The observer was required to record flow measurements when a "discernible patch of yellow" was observed. Each observer will interpret this definition of failure differently. By using the same observer each time, this element of subjectivity was greatly reduced. Because relative rather than absolute magnitudes of shear stress were of interest, this source of error was reduced even further.

CONCLUSIONS

A laboratory experiment was conducted to determine the relative magnitudes of shear stress at the base of a bridge pier as a function of pier diameter and scour depth. Marbles, rather than sand or gravel, were used as the bed sediment in the experiment in order to reduce scatter in the data by using a particle that was uniform in size and shape.

The results of this study were based on an experimental method in which the shear stress at the base of the scour hole was measured indirectly. There are advantages and disadvantages in using such a method. One distinct advantage is that no instrumentation was required in the scour hole; therefore, there was no interruption of the flow pattern around the pier or within the scour hole. Another advantage was the low cost of this method compared with that of using expensive instrumentation such as lasers. It is also quite possible that the accuracy obtained from such instrumentation is not required in light of the many other uncertainties in modeling bridge pier scour. For example, the uncertainties in extrapolating laboratory data to a real-world situation and uncertainties in the hydrologic conditions may be great enough to overshadow uncertainties caused by using a less accurate method of determining shear stresses in the laboratory flume.

The main disadvantage of using an indirect method of measuring shear stresses is the high degree of subjectivity in determining at what point the marbles failed; however, the error due to this problem was reduced by using a single observer and by depending on relative magnitudes rather than absolute magnitudes. In addition, the size of the marbles relative to the pier diameter was rather large in this experiment. For the smallest pier used, the pier diameter was only about seven times the diameter of the marbles. This relative size corresponds to that of riprap around a bridge pier. Although this size was adequate for this purpose, caution should be used in extrapolating information about the movement of sand around a bridge pier. The results of the experiment showed that the approach bed shear stress required to move marbles at a 10-in. model pier was about 1.4 times greater than at the 4-in. pier. This increased shear stress is consistent with the greater scour depth at larger piers often noted in scour literature.

The results also showed that the shear stress at the base of the pier decreases as scour depth increases. As the scour depth continues to increase, the shear stress approaches the bed shear stress upstream of the scour hole. The relative magnitude of the shear stress at the base of the scour hole and the bed shear stress is important information for modeling scour as a time-dependent function. This is particularly important for cohesive materials for which the erosion of the material depends highly on the amount of time that it is exposed to a particular shear stress.

On the basis of these results, it can be concluded that the shear stress at the base of a pier increases with increasing bridge pier diameter; however, the increase is not a linear one. The shear stress increases nonlinearly with increasing pier diameter, as does the depth of scour. Once a relationship between the shear stress ratio and the equilibrium depth of scour is established, then the indirect shear stress measurements could become a laboratory expedient for conducting pier scour experiments and could help explain some of the effects of various bed materials.

ACKNOWLEDGMENT

This study was sponsored by FHWA and conducted at the Turner-Fairbank Highway Research Laboratory.

REFERENCES

1. H. W. Shen, Y. Ogawa, and S. S. Karaki. Time Variation of Bed Deformation near Bridge Piers. *Proc., 11th Congress of the International Association for Hydraulic Research*, Leningrad, USSR, 1967, Paper 3.14.
2. B. W. Melville. Local Scour at Bridge Sites. Report 117. School of Engineering, University of Auckland, Australia, 1975.
3. H. W. Shen, V. R. Schneider, and S. Karaki. Local Scour Around Bridge Piers. *Journal of the Hydraulics Division*, ASCE, Vol. 95, No. HY6, 1969, pp. 1919–1939.
4. A. C. Parola. *The Stability of Riprap Used To Protect Bridge Piers*. Ph.D. dissertation. Pennsylvania State University, State College, 1990.
5. H. N. Breusers, G. Nicollet, and H. W. Shen. Local Scour Around Cylindrical Piers. *Journal of Hydraulics Research*, Vol. 15, No. 3, 1977, pp. 211–252.
6. G. R. Hopkins, R. W. Vance, and B. Kasraie. *Scour Around Bridge Piers*. Report FHWA-RD-79-103. FHWA, Washington, D.C., 1980.
7. B. P. Jain and P. N. Modi. Comparative Study of Various Formulae on Scour Around Bridge Piers. *Journal of the Institution of Engineers* (India), Vol. 67, Part Cl 3, 1986, pp. 149–159.
8. *Scour at Bridges*. Hydraulic Engineering Circular 18. FHWA, U.S. Department of Transportation, Washington, D.C., 1990.
9. B. W. Melville and A. J. Sutherland. Design Method for Local Scour at Bridge Piers. *Journal of Hydraulics Engineering*, Vol. 114, No. 10, 1988, pp. 1210–1226.