HENRY

Hydraulic Engineering Repository

Ein Service der Bundesanstalt für Wasserbau

Conference Paper, Published Version

Kwak, Kiseok; Briaud, Jean-Louis; Cao, Yiwen; Chung, Moonkyung; Hunt, Bea; Davis, Stan

Pier Scour at Woodrow Wilson Bridge and SRICOS Method

Pier Scour at Woodrow Wilson Bridge and SRICOS Method

By

Kiseok KWAK[1], Jean-Louis BRIAUD[2], Yiwen CAO[3], Moon-Kyung CHUNG[4],
Bea HUNT[5] and Stan DAVIS[6]

ABSTRACT

The Woodrow Wilson bridge across the Potomac River in Washington D.C. is a six lane bridge which is being replaced by a twelve lane bridge due to the rapid growth of traffic. In the design process the scour depth around the bridge piers must be predicted as it affects the depth of the foundations. In this study, the scour depth at selected piers are calculated for the existing bridge and for the new bridge by using the SRICOS method. First, the bridges are described including their foundation, the soil condition, and the river at the site. Second, the calculations and the results using the SRICOS method are presented. Third, the calculated scour depths are compared to the scour depths measured at the existing bridge. A discussion follows.

THE EXISTING WOODROW WILSON BRIDGE

The existing Woodrow Wilson bridge is located in Prince George County (Maryland), Alexandria (Virginia), and Washington (D.C.) and carries Interstate Routes 95 and 495 over the Potomac River. This bridge is an essential element of the I-495/95 beltway around Washington D.C. Due to the rapid growth of traffic, a replacement bridge is being designed to handle future demand safely and efficiently.

The existing Woodrow Wilson bridge is a draw bridge which has 58 spans and is approximately 1,800 m long. It was opened to traffic in 1961 with a design capacity of 75,000 vehicles per day. The design capacity was reached just 8 years after completion of the bridge (1969). In 1998, approximately 190,000 vehicles were using the bridge everyday. The projected 2020 average daily traffic volume is 300,000 vehicles per day. The main river piers of the existing bridge are massive and embedded in the river bed. The width of the piers which cross over the river

[1] Senior Researcher, Civil Engineering Division, Korea Institute of Construction Technology, 2311 Daehwa-dong, Ilsan-gu, Koyang City, Kyonggi-do, 411-712, Korea, (kskwak@kict.re.kr).
[2] Spencer J. Buchanan Professor, Department of Civil Engineering, Texas A&M University, College Station, TX 77843-3136.
[3] Graduate Student, Department of Civil Engineering, Texas A&M University, College Station, TX 77843-3136, USA.
[4] Research Fellow, Civil Engineering Division, Korea Institute of Construction Technology, 2311 Daehwa-dong, Ilsan-gu, Koyang City, Kyonggi-do, 411-712, Korea.
[5] Hydraulic Engineer, Hardesty & Hanover, LLP, New York, NY 10036, USA.
[6] Hydraulic Engineer, Maryland State Highway Administration, Office of Bridge Development, 2nd Floor, 707 North Calvert, Baltimore, MD 21202. USA.

channels and the shape of the front of the piers are listed in Table 1. The location of the piers can be found in Fig. 1. All piers are founded on piles.

At the bridge site, the Potomac River can be divided in three areas: the main channel, the secondary channel, and the median area between the two channels. The main channel is near the west shore and is approximately 305 m wide; the secondary channel is along the east shore and is approximately 275 m wide. Fig. 1 shows a best estimate of the soil profile along the existing Woodrow Wilson bridge and the replacement Woodrow Wilson bridge. Some soil properties are listed in Table 2.

THE REPLACEMENT WOODROW WILSON BRIDGE

The replacement bridge will be built immediately south of the existing Woodrow Wilson bridge. The proposed design has two parallel six-lane bridges to replace the existing single six-lane bridge and incorporates a drawbridge for ship traffic. The overall cost of the project including the approach embankments and associated interchanges is estimated at 2.2 billion dollars. The new bridge will have fewer but wider piers than the existing bridge. The piers are designed to have exposed pile foundations to be capped near the water surface. The two bascule piers that support the drawbridges will be protected from vessel impact by a fender system (Jones, 2000). Large dolphins were considered at one point.

The foundation of pier M1 which is one of the bascule piers is shown in Fig. 2. The dimensions of all the piers are shown with the scour result in Table 6. The foundation system for the replacement Woodrow Wilson bridge evolved continuously as design proceeded. The pier sizes, the dolphin diameter, and the deep foundation dimensions mentioned are the ones considered during this study but not necessarily those that will be finally retained.

THE SRICOS METHOD

The development of the scour depth in fine-grained soils is generally much slower than in coarse-grained soils. Applying the equations for coarse-grained soils to fine-grained soils regardless of time appears to be overly conservative. Therefore, a scour analysis method for fine-grained soils needs to consider the time effect as well as soil properties, hydraulic parameters, and pier size.

Because the soil at the site of the Woodrow Wilson bridge is fine-grained (cohesive), the SRICOS method (Briaud et al., 1999 (a), (b), Kwak, 2000 and Briaud et al. 2001 (a), (b)) was used. A brief summary of this method is described as a necessary background. The SRICOS method was developed to predict the scour depth z versus time t curve around a cylindrical bridge pier. This method has already been described in details in the references cited. The SRICOS method recognizes that the scour process is highly dependent on the shear stress τ imposed by the flowing water at the soil-water interface. Through tests performed on soil samples from the bridge site using the EFA (Erosion Function Apparatus, http://tti.tamu.edu/geotech/scour), the scour rate $\dot{z}$ versus the shear stress τ is obtained; this $\dot{z}$ vs. τ curve is the erosion function. Using this relationship and the maximum scour depth equation, a hyperbolic function describing the scour depth z versus time t curve can be developed. The SRICOS method was extended to include

a random velocity-time history and a multilayer soil stratigraphy; it is called the E-SRICOS method. The E-SRICOS method requires the use of a simple computer program and gives the scour depth versus time curve for a given hydrograph. The S-SRICOS which is a simplified version of the E-SRICOS method only requires simple hand calculations and gives the final scour depth at the end of the bridge life.

HYDROLOGIC DATA

The drainage basin at the Woodrow Wilson (WW) Bridge on the Potomac River has an area of 30,742 square kilometers. It is comprised of portions of Pennsylvania, West Virginia, Virginia, Maryland, and Washington, D.C. The nearest gaging station (Gage Station 01646500) was found at the USGS web site (www.usgs.gov). This gaging station is located on the Potomac River approximately 13 km upstream of the Woodrow Wilson Bridge near the Little Falls pump station and has a drainage area of 29,965 km^2. The discharge hydrograph was downloaded from the web site, multiplied by the drainage area ratio (30742/29965), and prepared as an input to the SRICOS program. The discharge hydrograph at the bridge site, calculated in such a way, is shown in Fig. 3. The maximum discharge occurred in 1972 and was 9850 m^3/s.

In this study, the computer program entitled Hydrologic Engineering Center's River Analysis System (HEC-RAS, 1997) developed by the United States Army Corps of Engineers was used for flood analysis. The input to this program is the average slope of the channel bed, the river bed cross-section profile, Manning's roughness coefficient and many selected discharges varying from 0 to the maximum discharge. The output of this program is the velocities and the water depths at the bridge pier location had the bridge not been there; the velocities and water depths correspond to the input discharges. The relationship between velocity and discharge and between water depth and discharge can then be obtained by regression. Using these relationships, the discharge hydrograph, which is the flow discharge versus time curve, is transformed into the water depth hydrograph and more importantly into the velocity hydrograph or velocity versus time curve used in the SRICOS program. The water depth history is needed for considering the water depth effect or wide pier effect. The water depth hydrographs for pier 1E and 27E of the existing Woodrow Wilson Bridge are shown in Fig. 4 over the length of the bridge from 1960 to 1999. Pier 1E is in the main channel and pier 27E in the secondary channel.

GEOTECHNICAL DATA

The Woodrow Wilson Bridge over the Potomac River in Washington D.C. is located within the Atlantic Coastal Plain Province which consists of a broad belt of flat-lying sediments over deep bedrock. Throughout the area, the ground surface has been altered in historic times by man-made fills especially in low lying areas and along rivers and streams. The soils below the main channel bed are mostly alluvial deposits, which consist of soft clay, silt, and silty sand, extending down to approximately 25 m over the layer of Pleistocene deposits which consist of dense sand, silt, and gravel.

For this study, soil samples were taken near the location of piers 1W, 2E, and 4E in the main channel, and piers 21E and 27E in the secondary channel by using thin-wall Shelby tubes with 76.2 mm outside diameter. The drilling locations are shown in Fig. 1 with the stratigraphy. The soil samples were tested using the EFA (http://tti.tamu.edu/geotech/scour). Before performing the EFA tests, basic soil properties were obtained by performing laboratory tests. All the soil property tests were conducted according to American Society for Testing and Materials (ASTM) standards. The undrained shear strength of the soil was measured at least twice using the vane test. The results of the soil property tests are shown in Table 2.

The purpose of the EFA test is to obtain the curve that relates the scour rate $\dot{z}$ to the shear stress τ induced by the flowing water. The water flows over the sample at a chosen velocity and the sample is advanced 1 mm as soon as it is eroded. These experiments are performed repeatedly for six or seven different velocities varying between 0.1 m/s and 5 m/s on each Shelby tube sample. The flowing water generates an average bed shear stress over the soil sample in the test section. The range of τ values imposed is approximately 0.1 N/m^2 to 100 N/m^2. The hydraulic shear stress imposed by the water on the soil is calculated by using Moody Chart (Moody, 1944). The critical shear stress is considered to be the shear stress when the scour rate is equal to 1 mm/hr. This number is used as a practical definition of the critical shear stress.

The erosion functions, scour rate $\dot{z}$ versus shear stress τ, were obtained for all the samples. Two examples are shown in Fig. 5 and 6. The soil at pier 27E (2.6 - 3.2 m depth) is a soft organic clay and the undrained shear strength is relatively low (22.0 kPa), however, the critical shear stress τ_c is relatively high (5.09 N/m^2, Fig. 5). The soil at pier 27E (11.2 - 11.7 m depth) is a hard mineral clay and the undrained shear strength is relatively high (130.0 kPa) however, the critical shear stress is relatively low (0.16 N/m^2, Fig. 6). In order to investigate the influence of cohesive soil properties on the erosion function, two erodibility parameters were defined: the critical shear stress τ_c and the initial erodibility S_i which is the initial slope of the erosion function. The two erodibility parameters τ_c and S_i were plotted against soil properties such as plasticity index, undrained shear strength, and percent passing sieve #200. All correlations were poor. In Fig. 7, the correlations between critical shear stress, initial erodibility, and undrained shear strength are shown with the data from a previous study (Briaud et al.,2001(a)). The poor correlations lead to think that obtaining these parameters by direct measurement in the EFA is more reliable than using correlations.

MEASURED SCOUR DEPTH

The existing Woodrow Wilson Bridge is approximately 1,800 m long and has 58 spans (57 piers). The piers are numbered beginning at the center of the bascule section in the main channel and increase as they approach each shore. Piers 1W through 26W are on the west side and piers 1E through 31E are on the east side. All the piers and abutments are made of reinforced concrete and are founded on piles. The junction between the pier and the piles is well below the current scour

depth. In other words, the width to be considered for scour analysis is the pier width not the piles width.

Some piers on the west side (4W to 26W) and some piers in the median area (6E to 22E) are not considered in the scour analysis because these piers are not over water. The parameters for the piers in water are shown in Table 1. The attack angle of the flow is 0° for all the piers.

The channel bed was monitored in 1998. The scour measurement results for each pier are shown in Table 1. The depth of local scour is defined here as the difference between the bed level at the pier and the bed level away from the pier. The bed level away from the pier is typically taken as the average of several points measured in the unscoured region around the obstruction. In this study, there was some ambiguity on the measured local scour depth because several interpretations of the scoured bed profile were possible. An example is shown in Fig. 8 for pier 5E. It was decided to use a range of possible values in all cases; Fig. 8 shows an example of minimum and maximum values. All values are listed in Table 1.

PREDICTED SCOUR DEPTH

The scour depth z versus time t curves were calculated for each pier of the existing bridge over the time period from 1960 to 1999. This period spans from the date the bridge was built to the date this study was performed. When soil samples were not taken from the exact pier location, the erosion function of the nearest soil samples was used as input to the SRICOS program. Examples for two representative piers of the existing bridge in the main and secondary channels (Pier 1E and 27E) are shown with the respective velocity hydrographs in Fig. 9 and 10.

In every case, the predicted final scour depth z does not reach the predicted maximum scour depth z_{max} even though the life of the existing bridge is about 39 years. The values of z and z_{max} are compared in Table 3. The ratio of the final scour depth over the maximum scour depth for all piers averages 66%.

The shape of the scour depth z versus time t curve depends on the scour rate of the soil as well as the shape and intensity of the hydrograph (Kwak, 2000). The scour depth at pier 1E increased gradually and the maximum velocity which occurred in 1972 did not greatly contribute to the scour depth because a certain amount of the scour had already developed when it occurred (Fig. 9). In the case of pier 27E, the maximum velocity in 1972 had a sudden influence on the scour depth because the low velocities prior to 1972 generated shear stresses below the critical shear stress of the soil and no scour developed before 1972 (Fig. 10).

The scour depth for each pier of the existing bridge is also calculated by using the S-SRICOS method. The length of the hydrograph t_{hydro}, the maximum velocity v_{max}, and the initial scour rate $\dot{z}_i$ of the soil are used to calculate the equivalent time t_e. The parameters and the results are shown in Table 4.

Scour analyses for the 100-year and the 500-year floods were also performed for the replacement bridge by using the S-SRICOS method because the S-SRICOS only requires the peak velocity. The peak discharges for the recurrence intervals (100 and 500 years) were obtained from the Maryland State Highway

Administration and are shown in Table 5. They were transformed into peak velocity by using HEC-RAS as was done for the existing bridge. The equivalent pier width was taken as the sum of the projected widths of the piles obstructing the flow. It was used for these calculations because the piers of the replacement bridge are designed to have exposed pile foundations with the pile cap near the water surface. The SRICOS predictions are shown together with the HEC-18 results in Table 6 and 7. It is clear that the HEC-18 predicted scour depths are much higher than the SRICOS predicted scour depths.

COMPARISON

The predicted scour depths using the E-SRICOS and the S-SRICOS method are compared with the measured scour depths for the existing bridge in Fig. 11 and 12. The piers in the main channel (Pier 2W to Pier 3E) are excluded from the comparison because riprap was placed in the main channel in 1980 to prevent further scour.

As shown in Fig. 11 and 12, the E-SRICOS and S-SRICOS methods give reasonable predictions. The scatter in the predictions may be due to the fact that the erosion function for the soil was not always from samples taken at the scour location. Indeed the samples were taken near piers 4E and 27E. For those piers the coefficient of determination (R^2) is 0.79. For other piers the coefficient of determination (R^2) is 0.41. The scatter on Fig. 11 and 12 gives an idea of the factor of safety necessary to minimize the number of cases where the measured scour depth is much larger than the predicted scour depth. It is also very important to note that the larger the scour depth is, the more precise the prediction is.

CONCLUSION

The Woodrow Wilson Bridge across the Potomac River in Washington D.C. is being replaced due to the rapid growth of traffic. The scour depths were calculated for the existing Woodrow Wilson Bridge by using the E-SRICOS method and the S-SRICOS method and compared with measured scour depths. A scour analysis for the replacement bridge was also performed by using the S-SRICOS method for the design floods.

1. As shown by the results of the EFA tests, the scour rate of the soil samples taken from the bridge site is relatively high, however the critical shear stress is also relatively high. The EFA results confirm that the correlations between soil erodibility and soil properties are very weak at best.

2. In all cases, the measured and the predicted final scour depths did not reach the maximum predicted scour depth z_{max} even though the bridge life is about 39 years. The average predicted final scour depth for all piers was 66% of the average predicted maximum scour depth. This is an indication of the margin of safety that existed for that bridge.

3. A high velocity flood does not greatly contribute to the scour depth in erosion resistant cohesive soils when a certain amount of scour depth has already been developed. The scour depth development in cohesive soils tends to be much

more gradual than in cohesionless soils and therefore allows more time for inspection and maintenance.
4. Both of the E-SRICOS and the S-SRICOS methods gave reasonable predictions for the existing Woodrow Wilson Bridge. The simple SRICOS (S-SRICOS) method correlates well with the extended SRICOS (E-SRICOS) method.
5. The HEC-18 equation gave predicted scour depths larger than the final scour depths predicted by the SRICOS method.
6. A factor of safety should be used on the predicted scour depth to minimize the risk of having an actual scour depth much larger than the predicted one.

ACKNOWLEDGEMENTS

REFERENCES

Briaud, J.-L , Ting, F., Chen, H. C., Gudavalli, S. R., Perugu, S., and Wei, G. (1999 a). "SRICOS: Prediction of scour rate in cohesive soils at bridge piers", *Journal of Geotechnical and Geoenvironmental Engineering*, Vol. 125, No. 4, ASCE, Reston, Virginia, USA, pp.237-246.

Briaud, J.-L., Ting, F., Chen, H.C., Gudavalli, R., Kwak, K., Philogene, B., Han, S.-W., Perugu, S., Wei, G., Nurtjahyo, P., Cao, Y., Li, Y. (1999 b), "SRICOS: Prediction of scour rate at bridge piers", *Report No. 2937-1*, to the Texas Department of Transportation, Texas Transportation Institute, The Texas A&M University System, College Station, Texas, USA.

Briaud J.-L., Ting F., Chen H.-C., Cao Y., S.-W., Kwak K., (2001a), "Erosion Function Apparatus for Scour Rate Predictions,", *Journal of Geotechnical and Geoenvironmental Engineering*, Vol. 127, No. 2, pp. 105-113, Feb. 2001, ASCE, Reston, Virginia.

Briaud J.-L., Chen H.-C., Kwak K., Han S., Ting F., (2001b), "Multiflood and Multilayer Method for Scour Rate Prediction at Bridge Piers", *Journal of Geotechnical and Geoenvironmental Engineering*, Vol. 127, No. 2, pp. 114-125, Feb. 2001, ASCE, Reston, Virginia.

HEC-RAS River Analysis System (1997). *User's manual, Version 2.0*, Hydrologic Engineering Center, U.S. Army Corps of Engineers, Davis, California, USA.

Jones, J. S. (2000), "Hydraulic testing of Wilson Bridge designs", *Public Roads, March/April*, U.S. Department of Transportation, FHWA, Washington, DC, USA, pp.40-44.

Kwak, K. (2000), "Prediction of scour depth versus time for bridge piers in cohesive soils in the case of multi-flood and multi-layer soil systems", Ph.D. Dissertation, Texas A&M University, College Station, Texas, USA.

Kwak, K., Briaud J.-L., Chen H.-V., (2001), "SRICOS: Computer Program for Bridge Pier Scour", Proceedings of the 15th International Conference on Soil Mechanics and Geotechnical Engineering, held in Istanbul, Turkey on August 27-31, 2001, A.A. Balkema Publishers, Rotterdam, the Netherlands.

Melville, B. W. and Coleman, S. E. (1999), Bridge scour, Water Resource Publications, LLC, Highlands Ranch, Colorado, USA, pp.196-199.

Moody, L.F. (1994), "Friction factors for pipe flow", Transaction of the American Society of Mechanical Engineers, Vol. 66.

Richardson, E. V. and Davis, S. M. (1995). "Evaluating scour at bridges", *Pub. No. FWHA-IP-90-017, HEC No.18*, U.S. Department of Transportation, Washington, DC, USA.

Table 1 - Pier Parameters and Measured Scour Depths for the Existing Woodrow Wilson Bridge

Pier	Shape	Width (m)	Measured Scour Depth (m)	
			Min.	Max.
3W	Square	2.51	1.31	2.72
2W	Square	2.51	0.97	1.46
1W	Square	9.75	0.92	2.14
1E	Square	9.75	1.22	1.79
2E	Square	2.51	0.76	3.13
3E	Square	2.51	1.53	2.80
4E	Square	2.51	1.98	3.28
5E	Circle	1.68	0.77	1.72
23E	Circle	1.22	0.37	0.64
24E	Circle	1.22	0.37	0.60
25E	Circle	1.22	1.01	1.50
26E	Circle	1.22	0.76	0.88
27E	Circle	1.22	0.73	1.15
28E	Circle	1.22	0.61	0.73
29E	Circle	1.22	0.31	0.52

Table 2 - Soil Properties of the Soil Samples from the Woodrow Wilson Bridge Site

Sample Location	Depth (m)	Soil Type	Undrained Shear Strength (kPa)	Bulk Density (kN/m³)	% Passing #200	Liquid Limit (%)	Plastic Limit (%)	Water Content (%)	Critical Shear Stress (N/m²)	Initial Erodibility S_i
Pier 1W	4.0-4.6	Clay	11.5	18.1	57	53	12	56	3.90	4.0
Pier 1W	10.1-10.6	Clay	19.0	15.6	71	51	18	35	10.20	1.9
Pier 2E	5.5-6.1	Clay	14.0	18.5	48	47	14	29	1.30	182.9
Pier 4E	5.5-6.1	Sandy Clay	14.1	16.3	64	37	14	35	0.43	9.0
Pier 21E	2.1-2.7	Clay	6.1	15.4	86	68	13	47	1.92	2.7
Pier 27E	2.6-3.2	Organic	22.0	15.2	40	-	-	82	5.09	11.2
Pier 27E	5.2-5.6	Silt	-	15.2	73	43	39	66	3.80	91.0
Pier 27E	11.2-11.7	Clay	130.0	21.3	78	86	14	24	0.16	3.2
Pier 27E	11.9-12.5	Sand	12.0	17.1	9	-	-	59	0.025	1665.2

Table 3 - Predicted Scour Depths at the Existing Woodrow Wilson Bridge Using E-SRICOS Method

Pier	Final Scour Depth z (m)	Max. Scour Depth z_{max} (m)	$\dfrac{z}{z_{max}}$ (%)
3W	1.64	2.85	57.5
2W	2.92	3.66	79.8
1W	5.72	9.21	62.1
1E	6.14	9.51	64.6
2E	3.69	3.97	92.9
3E	3.34	3.57	93.6
4E	2.61	3.28	79.6
5E	1.07	1.89	56.6
23E	0.47	1.22	38.5
24E	0.52	1.25	41.6
25E	0.17	1.29	13.2
26E	1.07	1.54	69.5
27E	1.41	1.74	81.0
28E	1.40	1.74	80.5
29E	1.36	1.71	79.5

Table 5 - Peak Discharges for the Potomac River at the Woodrow Wilson Bridge

Recurrence Interval (years)	Peak Discharge (CMS)
100	13592
500	19822

Table 4 - Predicted Scour Depths at the Existing Woodrow Wilson Bridge Using S-SRICOS Method.

Pier No.	Length of Hydrograph t_{hydro} (yrs)	Max. Discharge Q_{max} (CMS)	Max. Velocity v_{max} (m/s)	Pier Width B (m)	Max. Scour Depth z_{max} (mm)	Shear Stress τ_{max} (N/m²)	Initial Scour Rate $\dot{z}_i$ (mm/hr)	Equivalent Time t_e (hrs)	Final Scour Depth z (m)
Pier 3W	39	9850.5	1.41	2.51	2852.4	9.85	25.77	108.6	1.48
Pier 2W	39	9850.5	2.09	2.51	3662.2	20.04	115.73	157.4	3.10
Pier 1W	39	9850.5	2.30	9.75	9212.6	17.92	90.88	194.5	6.25
Pier 1E	39	9850.5	2.42	9.75	9514.9	19.62	110.47	204.0	6.88
Pier 2E	39	9850.5	2.37	2.51	3966.6	25.14	696.67	136.2	3.82
Pier 3E	39	9850.5	2.01	2.51	3572.6	18.68	696.67	102.8	3.42
Pier 4E	39	9850.5	1.76	2.51	3283.6	14.70	163.50	109.6	2.81
Pier 5E	39	9850.5	1.28	1.68	1889.8	8.92	15.03	102.6	0.85
Pier 23E	39	9850.5	0.88	1.22	1215.8	4.79	5.88	65.3	0.29
Pier 24E	39	9850.5	0.92	1.22	1250.6	5.19	5.88	70.4	0.31
Pier 25E	39	9850.5	0.97	1.22	1293.3	5.72	5.88	77.1	0.34
Pier 26E	39	9850.5	1.28	1.22	1542.4	9.47	50.59	80.5	1.12
Pier 27E	39	9850.5	1.55	1.22	1741.7	13.40	79.16	102.0	1.43
Pier 28E	39	9850.5	1.55	1.22	1741.7	13.40	79.16	102.0	1.43
Pier 29E	39	9850.5	1.50	1.22	1705.8	12.62	75.78	97.3	1.39

Table 6 - Predicted Scour Depths at Replacement Woodrow Wilson Bridge Using S-SRICOS Method and HEC-18 (100 Year Flood)

Pier No.		M10	M9	M8	M7	M6	M5	M4	M3	M2	M1	Dolphin	V1	V2
Equivalent Pier Width (m)		6.4	6.4	6.4	6.4	6.4	6.4	6.4	6.9	6.9	9.6	13.7	9.6	6.9
Velocity (m/s)		1.80	1.80	1.80	1.80	1.16	1.16	1.16	2.80	2.80	2.80	2.80	2.80	2.80
Water Depth (m)		6.86	6.86	6.86	6.86	3.60	3.60	3.60	13.62	13.62	13.62	13.62	13.62	13.62
Scour Depth (m)	S-SRICOS	3.18	3.18	3.18	1.09	0.37	0.25	0.25	7.47	7.47	7.83	7.50	7.83	6.73
	HEC-18	17.07	17.07	17.07	17.07	17.07	12.31	12.31	26.64	26.64	33.10	17.68	33.10	26.64

Table 7 - Predicted Scour Depths at Replacement Woodrow Wilson Bridge Using S-SRICOS Method and HEC-18 (500 Year Flood)

Pier No.		M10	M9	M8	M7	M6	M5	M4	M3	M2	M1	Dolphin	V1	V2
Equivalent Pier Width (m)		6.4	6.4	6.4	6.4	6.4	6.4	6.4	6.9	6.9	9.6	13.7	9.6	6.9
Velocity (m/s)		2.38	2.38	2.38	2.38	1.65	1.65	1.65	3.60	3.60	3.60	3.60	3.60	3.60
Water Depth (m)		7.62	7.62	7.62	7.62	4.36	4.36	4.36	14.42	14.42	14.42	14.42	14.42	14.42
Scour Depth (m)	S-SRICOS	4.90	4.90	4.90	4.73	2.37	1.99	1.99	8.86	8.86	10.36	10.66	10.36	8.55
	HEC-18	19.29	19.29	19.29	19.29	19.29	14.51	14.51	29.50	29.50	36.67	19.57	36.67	29.50
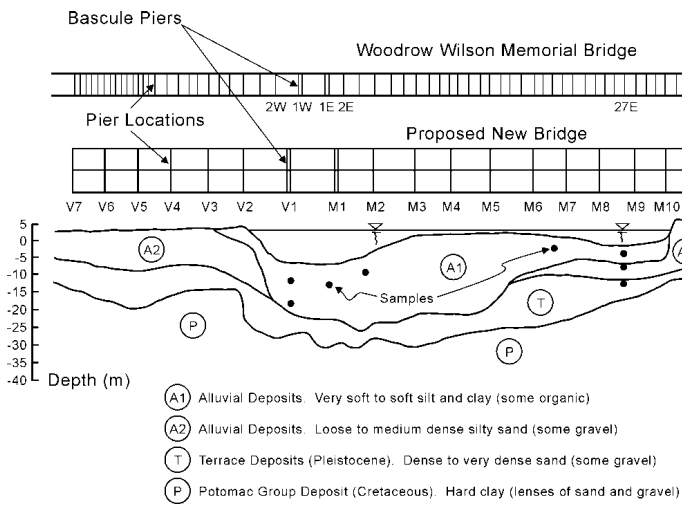
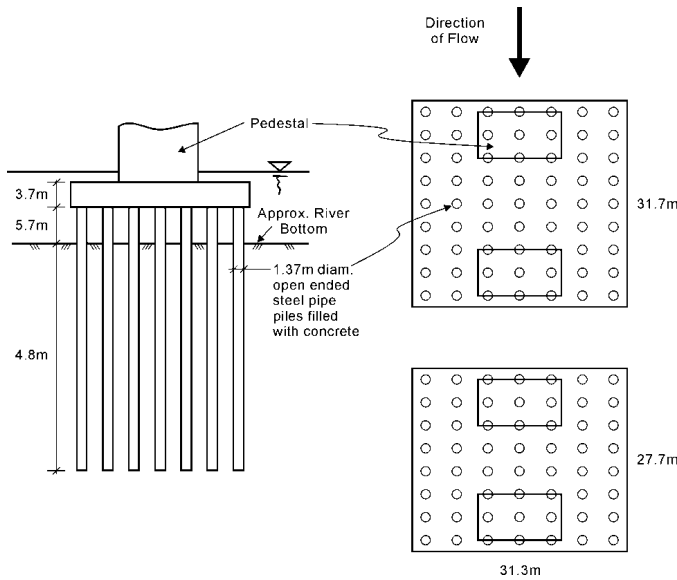Fig. 1 Estimated Soil Stratigraphy at the Woodrow Wilson Bridge Location

A1 Alluvial Deposits. Very soft to soft silt and clay (some organic)

A2 Alluvial Deposits. Loose to medium dense silty sand (some gravel)

T Terrace Deposits (Pleistocene). Dense to very dense sand (some gravel)

P Potomac Group Deposit (Cretaceous). Hard clay (lenses of sand and gravel)

Fig. 2 Foundation of Pier M1 of the Replacement Woodrow Wilson Bridge Considered for Scour Calculations
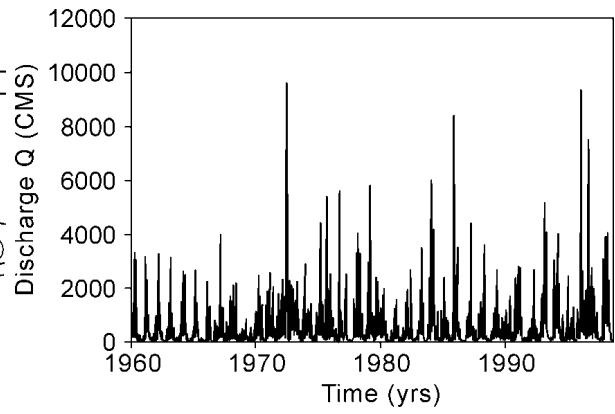
Fig. 3 Discharge Hydrograph at Woodrow Wilson Bridge (01/01/1960-09/31/1998)
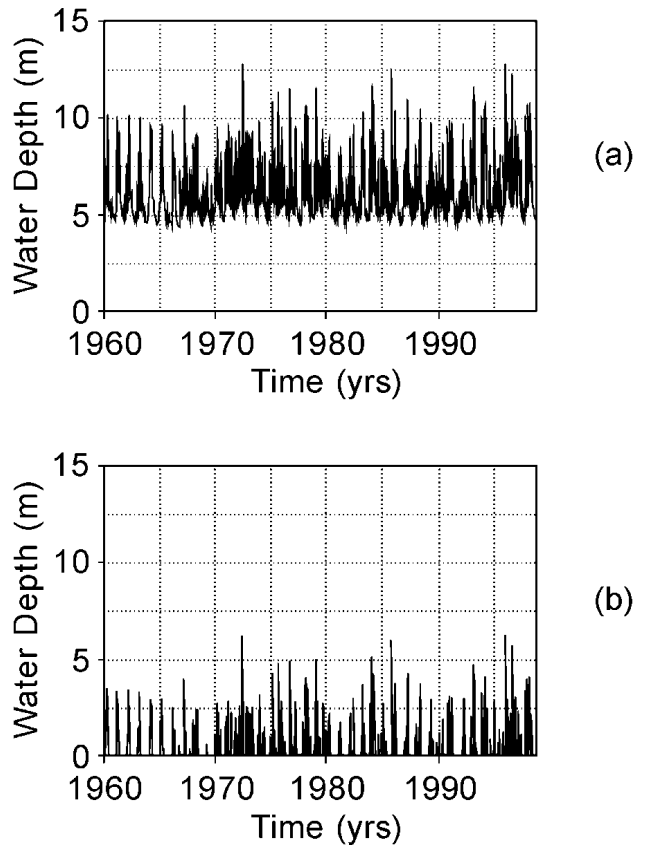
Fig. 4 Water Depth Hydrograph for the Existing Woodrow Wilson Bridge: (a) Pier 1E, (b) Pier 27E
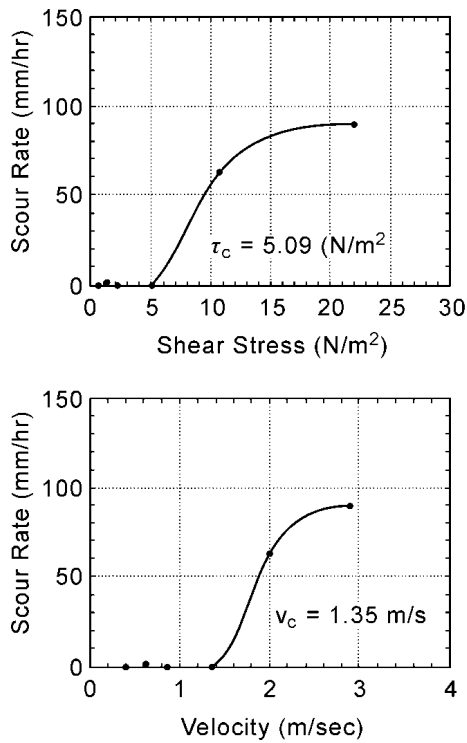
Fig. 5 Erosion Function for a soil sample taken near Pier 27E of the Existing Woodrow Wilson Bridge (2.6 – 3.2 meters depth): (a) Scour Rate vs. Shear Stress, (b) Scour Rate vs. Velocity
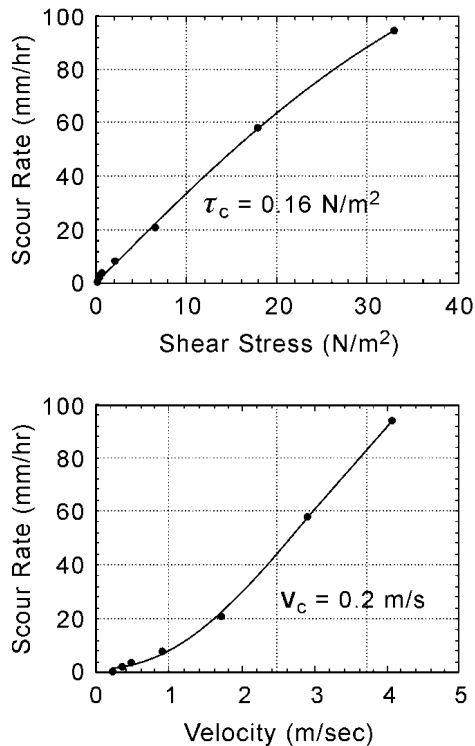
Fig. 6 Erosion Function for a soil sample taken near Pier 27E of the Existing Woodrow Wilson Bridge (11.2 – 11.7 meters depth) : a) Scour Rate vs.Shear Stress, b) Scour Rate vs. Velocity
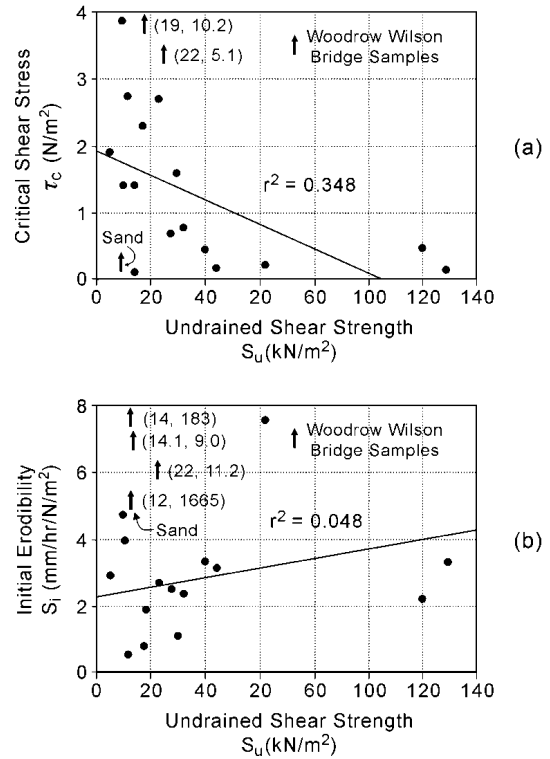
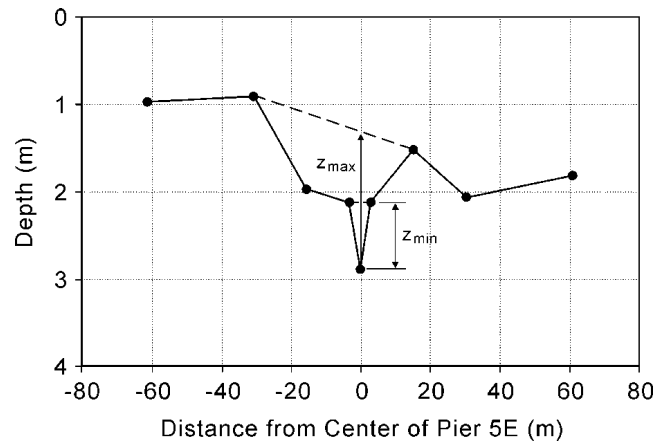Fig. 7 Lack of Correlation Between Critical Shear Stress, Initial Erodibility, and Undrained Shear Stress (after Briaud et al. 1999 (b))

Fig. 8 Potomac River Bottom Profile Around Pier 5E of the Existing Woodrow Wilson Bridge
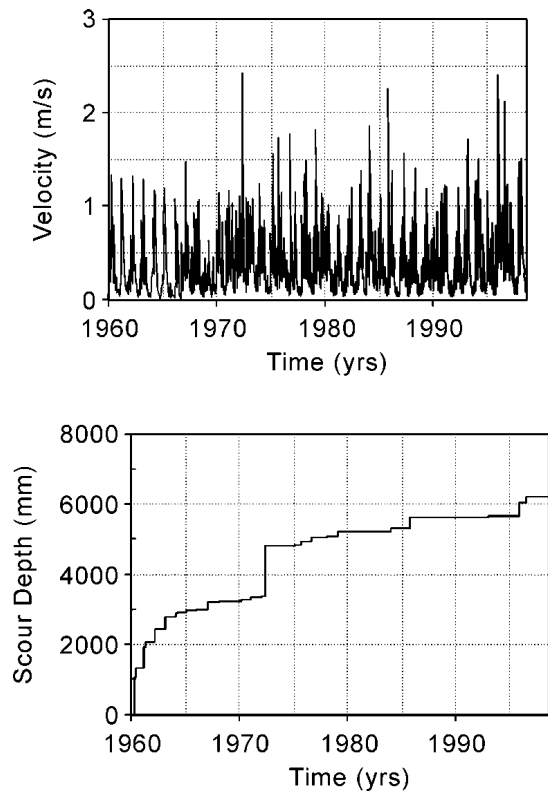
Fig. 9   Velocity Hydrograph and Predicted
Scour Depth vs. Time Curve for Pier
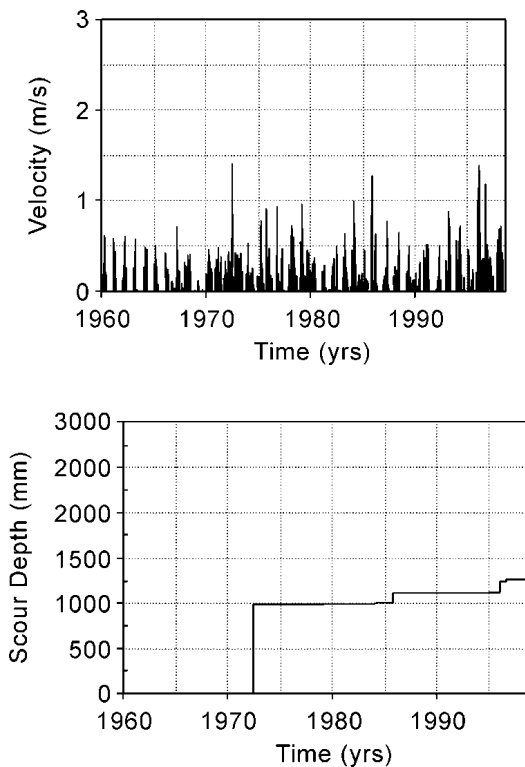1E of the Existing Woodrow Wilson Bridge

Fig. 10 Velocity Hydrograph and Predicted
Scour Depth vs. Curve for Pier 27E of the
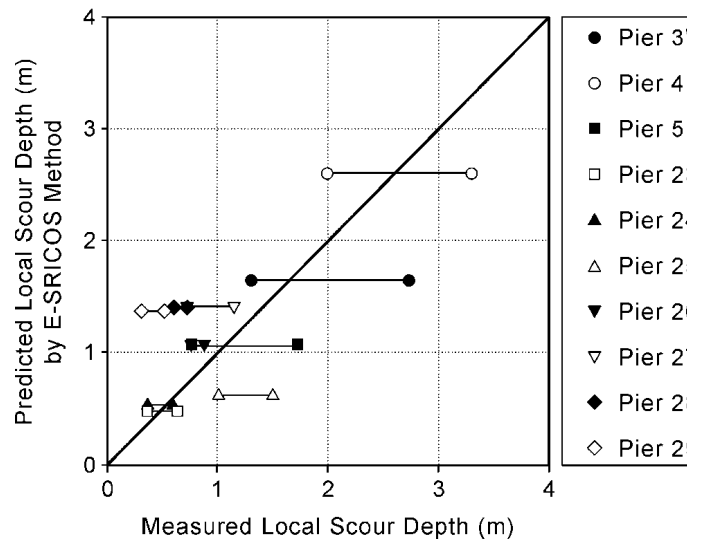Existing Woodrow Wilson Bridge

Fig. 11 Comparison of Measured and
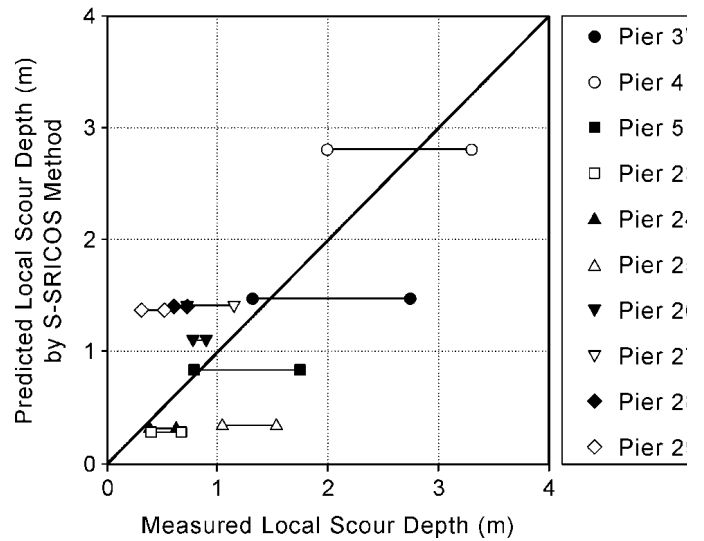Predicted Scout Depth (E-SRICOS Method)

Fig. 12 Comparison of Measured and
Predicted Scour Depths (S-SRICOS Method)