FLOOD EFFECTS ON LOCAL SCOUR AT IMBABA BRIDGE

Nahla Sadek

Researcher, Nile Research Institute, National Water Research Center, Egypt
E-mail: Nahla_NRI@Yahoo.com.

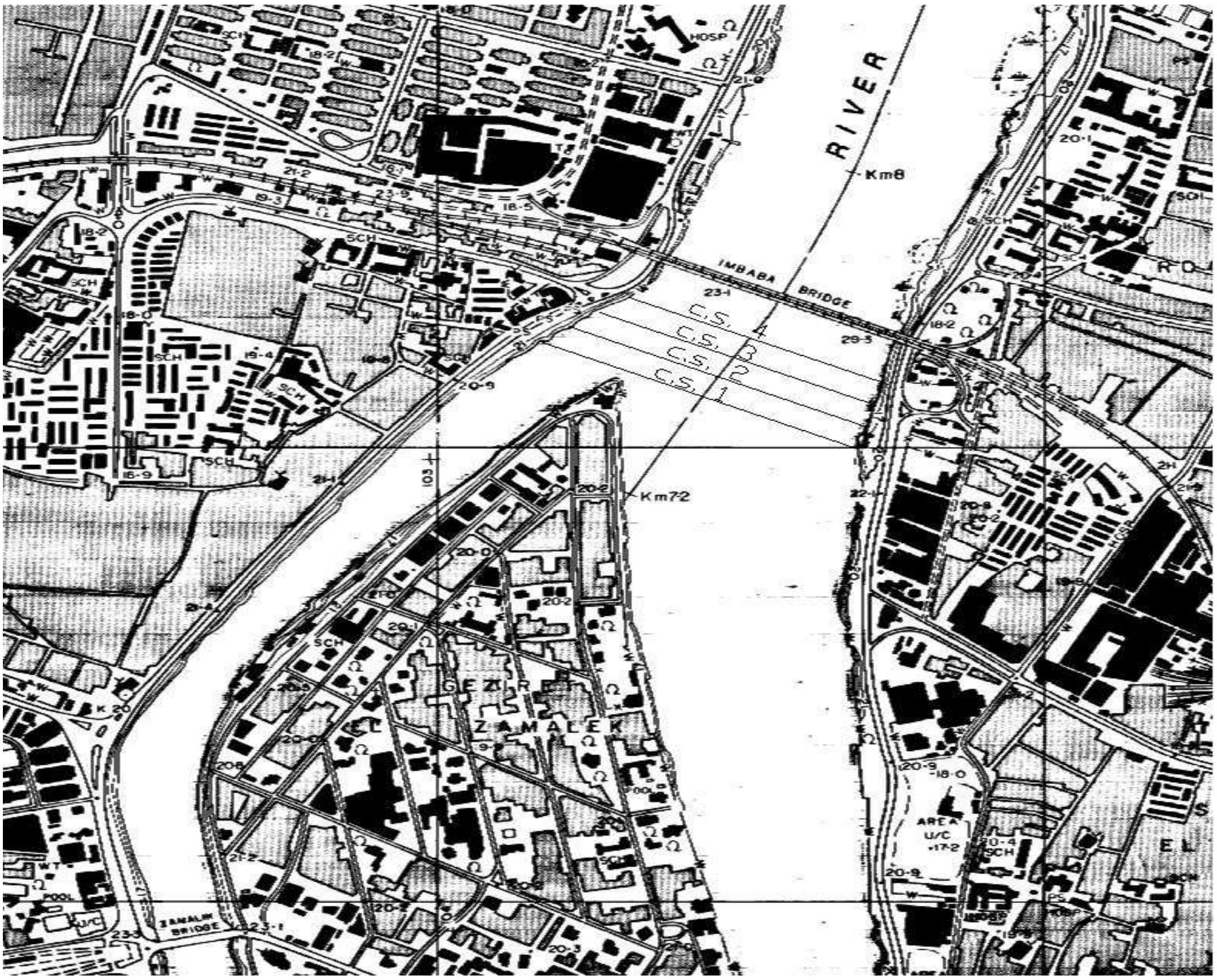
ABSTRACT

Construction of Aswan High Dam (AHD) in 1968 has reserved and controlled the Nile River discharge to protect Egypt not only from floods but also from drought. During the past few years, Egypt has experienced high floods. Consequently, high inflows have to be passed to the river downstream the dam according to dam operation rules. These inflows constitute risks to bridges and other structures on the Nile River especially in narrow river sections where general scour is expected. Imbaba Bridge (km 934.70 downstream Aswan Dam) is located upstream Delta Barrage (km 954 from Aswan). It is considered one of the major bridges across the Nile River. Some studies investigated local scour around Imbaba bridge pier based on laboratory flume data analysis that suffer from scale effects. In this study, HEC-RAS mathematical model was used to simulate the scour phenomenon around Imbaba bridge piers. The computation of bridge pier scour depth depends on flow velocity, flow depth, bed sediment characteristics and pier width. The main objective of this research paper is to predict, evaluate and analyze the future maximum excepted scour occurred on the Imbaba Bridge piers that would take place due to the occurrence of high releases during a high flood period.

Keywords: Nile River Bridges, Bridge local scour

INTRODUCTION

Imbaba Bridge is considered one of the most important bridges in Egypt since it transports the railway from South to North and it is located on the fourth reach. This reach extends from downstream Assuit Barrage until upstream Delta Barrages. The reach length's is about 410 km and the average width is about 500 m. It is considered the longest reach, relatively straight channel and meandering occurs locally and its sinuosity is about 1.1. Most towns are located on the right and bank protection works on the left in general confine the channel movement. Map 1 shows the location of Imbaba Bridge on the fourth Reach. This bridge was built before 1959 from about 150 years, therefore the 1959 flood or a similar flood magnitude could have caused the scour at Imbaba Bridge. The peak flood flows were released from AHD are dramatically reduced after its completion in 1968. Consequently, the scour hole at Imbaba Bridge is becoming stable under current flow conditions. Flow discharge is a very important factor influencing the scour depth. During the past few years, Egypt has experienced high floods such as flood of 1998. Consequently, high inflows have to be passed to the river downstream the dam according to dam operation rules which may affect different hydraulic structure along the river including local scour around bridge piers. In this study, the effect of high releases on Imbaba bridge pier scour is predicted and evaluated using HEC-RAS mathematical model.



Map 1: Imbaba Bridge location on the Fourth Reach

LOCAL SCOUR AROUND BRIDGE PIERS

Generally, scour is resulting from the capacity of water to carry sediment is greater than the incoming supply of sediment (ASCE, 1975). The local scour occurs at a pier, abutment, erosion control device, or other structure obstructing the flow. These obstructions cause flow acceleration and create vortices that remove the surrounding sediments. This can occur due to the partial or complete removal of the sediment load because a local increase in shear and hence in the carrying capacity of water or for both reasons. While it is generally recognized that, in the case of locality increased shears, an equilibrium depth of scour exists, no approach has been formulated to predict the depth of scour or guide a design engineer in determining the depth. The shape of the pier is very significant with respects to the strength of the horseshoe vortex at the base of the pier. The factors affecting the magnitude of the local scour depths at piers and abutments can be summarized as follows:

1 - Velocity of the approach flow,
2 - Depth of flow,
3 - Width of the pier,
4 - Length of the pier if skewed to flow,
5 - Size and gradation of bed material,
6 - The angle of attack, and
7 - Bed configuration.

COMPUTING PIER SCOUR

As a result of complexity of evaluating flow pattern around piers and shear forces generated by flow pattern, most of the estimates of scour depth have been obtained by experimental work. An example of these estimates is the Colorado State University (CSU) equation for predicting maximum pier scour depths, the equation is (ASCE, 1975 [1]):

$$\mathbf{Y_s/Y_1 = 2.0\ K_1\ K_2\ K_3\ K_4\ (A/Y_1)^{0.65}\ F_{r1}^{0.43}}$$

For round nose piers aligned with the flow where:
Y_s = Scour depth, m
Y_1 = Flow depth directly upstream of the pier, m
K_1, K_2, K_3, and K_4 = Correction factors for pier nose, angle of attack, bed condition, and armoring by bed material size respectively.
A = Pier width, m
L = Length of pier, m
F_{r1} = Froude Number directly upstream of the pier.
$Y_s \leq 2.4$ times the pier width (A) for $Fr \leq 0.8$
$Y_s \leq 3.0$ times the pier width (A) for $Fr > 0.8$

GENERAL OVERVIEW FOR THE PROPOSED MODEL

In this study, HEC-RAS a 1-D model is applied to perform the objective of this paper. The model was developed by U.S. Army Crops of engineering, USA in 2001 [6]. It is used for calculating and predicting different hydraulic parameters such as water surface profiles, water depth and average velocity …. etc., in natural and artificial channel for both sub-critical and supercritical flow. The steady flow system can handle a full network of channels, a dendrite system, or a single river reach. The steady flow component is capable of modeling sub-critical and mixed flow regime water surface profiles.

This model computes energy losses caused by structures such as bridges and culverts in three parts. One part consists of losses that occur in the reach immediately downstream the structure where an expansion takes place. The second part is the losses at the structure itself, which can be modeled with several different methods. The third part consists of losses that occur in the reach immediately upstream the structure where the flow is contracting to get through the opening. To be able to simulate the actual flow conditions, the whole reach (Reach 4- from downstream Assiut Barrage to upstream Delta Barrage) has to be modeled and analyzed.

The required information for running the program can be grouped under two main parts, geometric and hydrologic data.

The geometric data records used in these runs consists of the surveyed cross sections for this reach can be summarized as:
- 1 cross-section in the center of the Imbaba Bridge surveyed by HRI in 1992 (HRI report, 1992 [4])
- 41 cross-sections along the fourth reach with different spacing between series of cross-sections, which surveyed by NRI in 1997.
- 4 recent cross-sections directly located with 80 m upstream the bridge and the distance between two consecutive cross-sections was about 50 m, which surveyed in 2002 (NRI. Report, 2002 [5]).

The hydrologic data for this reach consists of discharge data at Salam gauge station which is located downstream Assiut Barrages at kilometer 544.77 from Aswan. The second part of the hydrologic data for the reach consists of the water levels at gauging stations. Twelve gauging stations along the reach are available on daily basis.

Bed material grain size for D50 - D90 and Bridge geometry; pier dimensions were used in this study.

Model Calibration and Verification

Using the available water surface profiles, the model was calibrated by trial and error to simulate the actual conditions and corresponding water levels according to different process discharges. Two actual flow conditions were considered for the calibration analysis; 171 and 350 mm^3/day. The Manning's roughness was adjusted to obtain fair calibration. Figure (1) shows the calibration process for the flow of 171 $m.m^3$/day at year 1995 and figure (2) describes the verification process for the flow of 350 mm^3/day using rating curve for different gauging stations along the fourth reach before AHD construction. From these two figures, we can conclude that there is a close relationship between measured and predicted water levels. This indicates that the selected roughness values along all selected sections are suitable for the simulation process. This roughness ranges between 0.015 – 0.031 which are considered relatively of wide range.

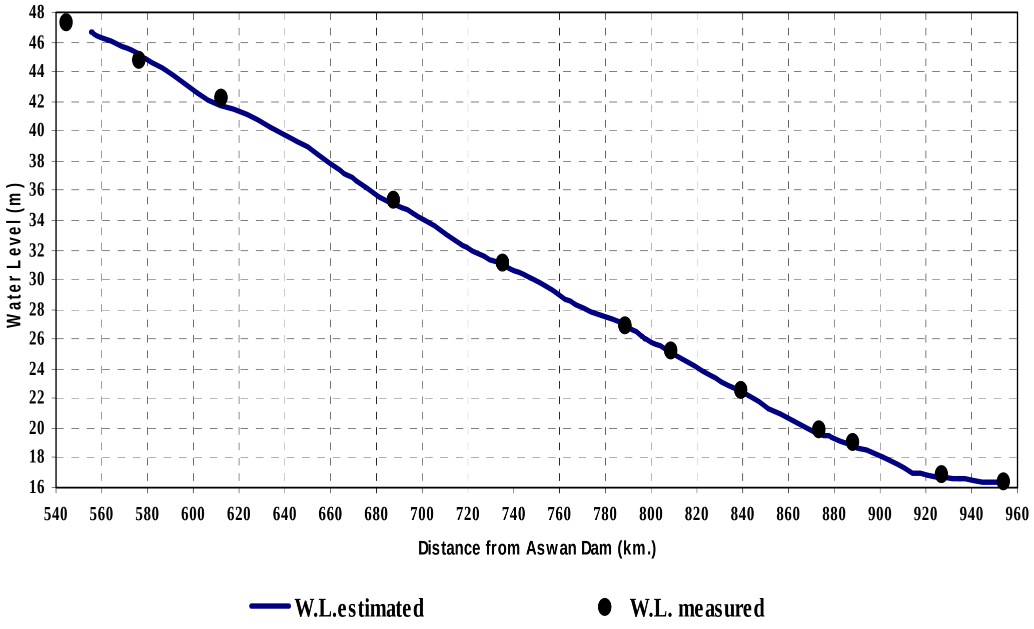


Figure 1: Model Calibration Results for inflow 171 mm^3/day along the fourth reach

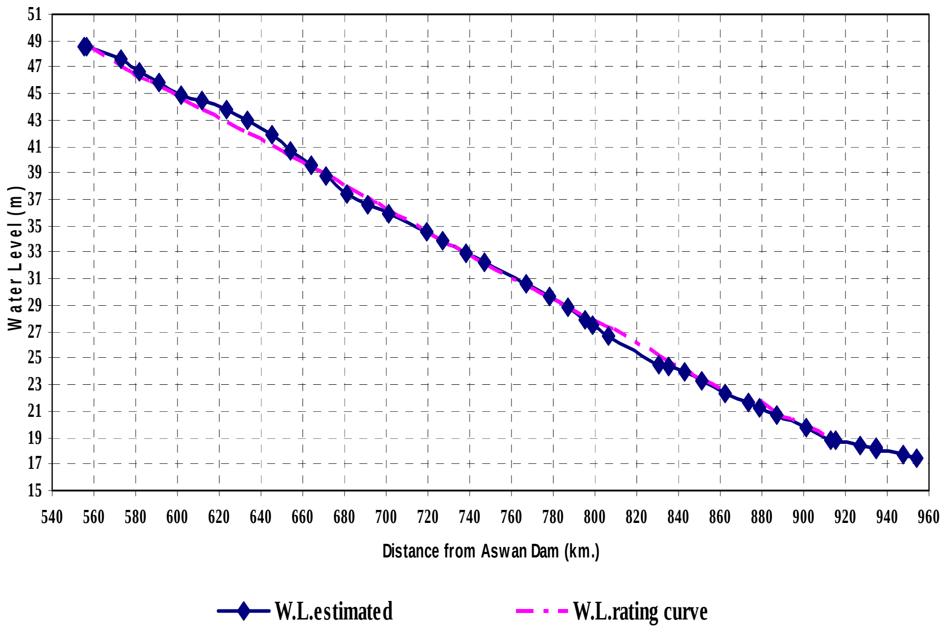


Figure 2: Model Verification Results for inflow 350 mm^3/day along the fourth reach

LOCAL SCOUR AT IMBABA BRIDGE

Imbaba Bridge is located in the backwater curve of Delta Barrage from about 150 years. This bridge is a steel one, with two levels one for railway and the second one for roadway. It has seven piers, the second pier from the left bank is circular with 10.6 m diameter and the other six piers are rectangular with 15 m length and 3.6 m width having rounded noses. The deep scour hole lies 97 m from the left bank and 32 m downstream of the centerline of the bridge with a bottom elevation of -8.3 m (HRI, Report 1992). Also, it is mentioned the mean velocity is not larger than 0.6 m/s since the bridge construction.

The most recent available scour investigation at Imbaba Bridge is that by HRI in 1992 which was extracted from map 2. While the oldest available data in 1982 by Nile Research Institute (NRI). Figure (3) shows the comparison between these cross-sections and the piers location. From this figure, it can be concluded that generally the local scour occurred around the all piers was during this period. However, the maximum value of local scour was found around the second pier from the left side because it is the largest pier. Due to the large size of this pier, the effect of the horseshoe vortex is very significant at the pier base. Moreover, the deposition that might have occurred upstream the piers after AHD might have caused change in the scour holes dimensions after their formation. Deposition of the Nile bed load occurs because Imbaba Bridge is located upstream the Delta Barrages where back-water curve effects appear. Figures (4) and (5) show the comparison between the recent available cross sections which were surveyed by NRI team in 2002 and the same location cross sections which extracted from Kenting Hydrographic survey in 1982 (NRI).

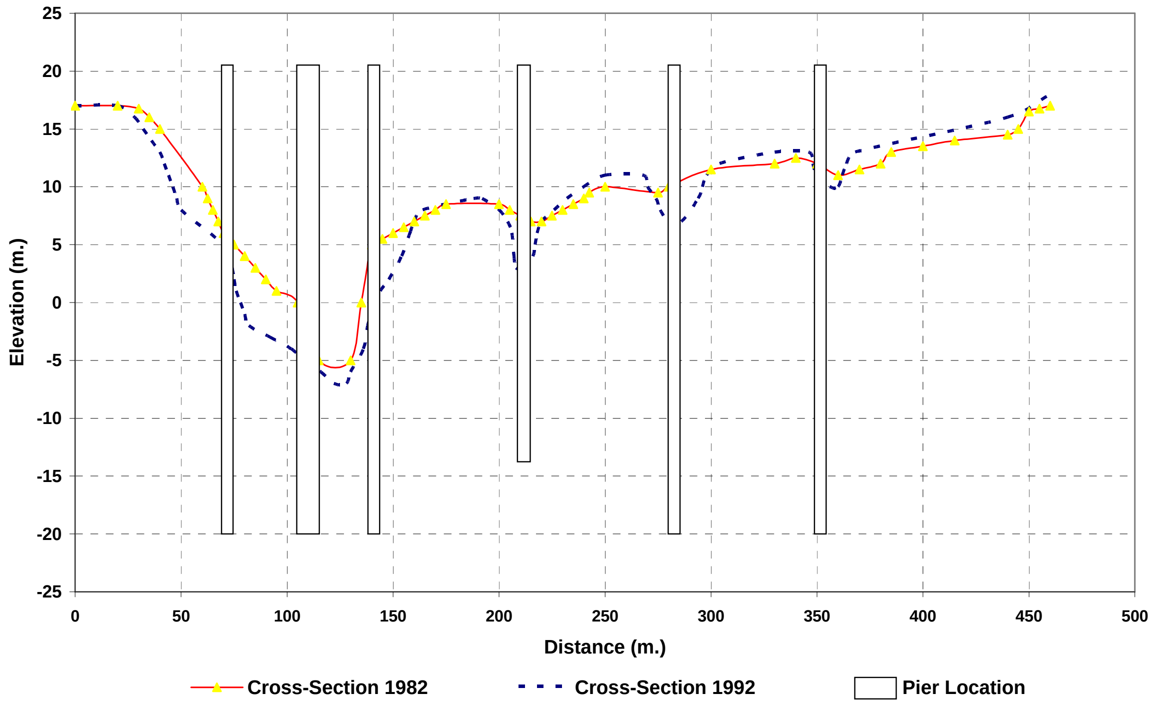
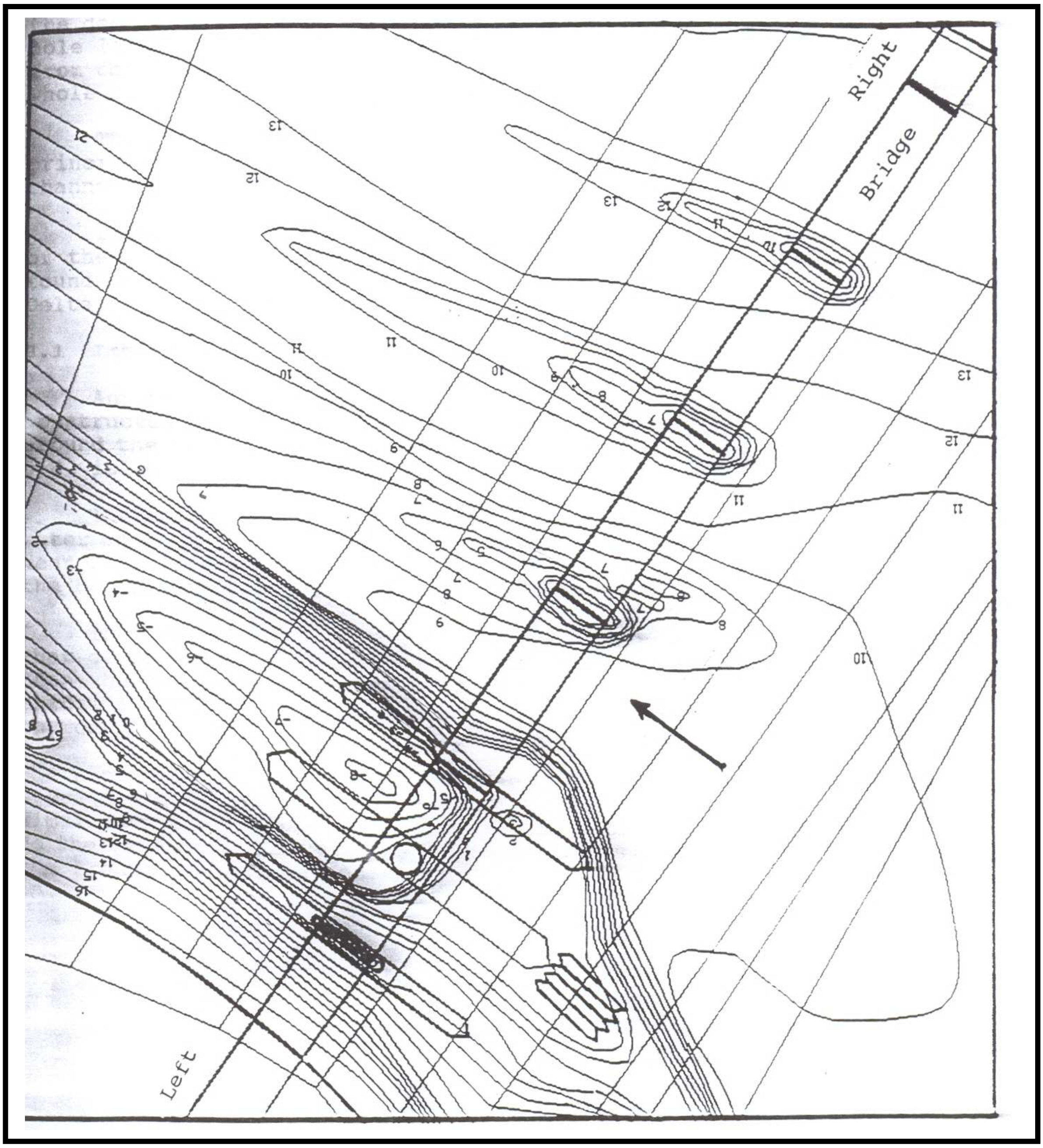


Figure 3: Cross-sections and the Piers location at the Center of Imbaba Bridge

Worth noting that no data about the dimensions of the scour holes at pre- AHD flood condition exists. This makes it difficult for obtaining the exact dimensions of the scour holes that are thought to be resulting from 1959 flood (881.5 mm^3/day) or from a different flood with nearly the same magnitude.



Map 2: Scour Investigation at Imbaba Bridge in 1992

LOCAL SCOUR PREDICTION

The study covered up to a maximum discharge of 171 mm^3/day and different high releases (250 – 300 and 350 mm^3/day) could be released from Aswan down to Cairo during high flood conditions. In this study it was decided to investigate the worst condition (clear water scour). This because the confluence scour can be very large when bed transport is low. It was simulate the average velocity, where most of the river course is nearly flat upstream the bridge. Figure 6 shows the model output for the simulated flow conditions for maximum discharge 350 mm^3/day. The CSU equation is used for the model for local scour computations. The round pier nose was considered for the analysis to simulate the conditions of the existing pier. In the current study, the median diameter of the bed material is D_{s50} = 0.115 mm, D_{s90} = 0.374 mm and the bed characteristics was fine sand as reported by (HRI, 1992).

The K terms are taken as follows:

- K_1 correction factor for pier nose is taken as 1.00
- K_2 if the flow direction upstream of the pier is perpendicular then the angle of attack of flow would be entered as zero and K_2 = 1.00
- K_3 correction factor for bed conditions 1.1 (clear water scour)
- K_4 armoring effect coefficient 1.0

Figure 7 explains model output local scour for six piers. The effect of different velocity values in estimating local scour for maximum discharge (350 mm^3/day) was analyzed. Worth noting the site of the eastern pier No 7 was not estimated to scour as it is located in a shallow area above the minimum water level. Table 1 illustrates the results of the local scour estimation for the 6 piers related to different velocity. The velocity ranges between the estimated velocity by the model 1m/s and the common velocity should be occurred in the back water curve region upstream Delta Barrage 0.6 m/s (HRI, 1992). The results of this table indicated that the maximum local scour is predicted of 9.66 m and 7.76 m related to velocity 1 m/s and 0.6 m/s respectively for the second pier at distance 104.33 m from the left side. Table 2 shows scour depth estimated for the sixth piers according to different discharges (200, 250 and 300 mm^3/day). From this table, it can be concluded that local scour depth affected by increasing discharge especially for the second pier which should be taken into consideration during future local scour protection.

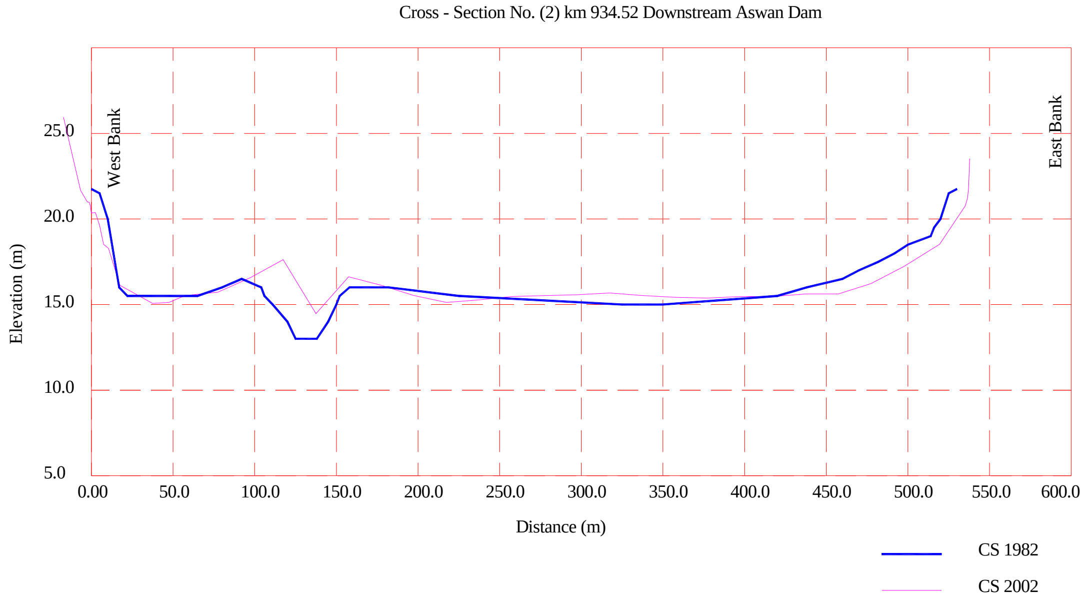


Figure 4: Cross section changes No. 2 at the bridge (1982 and 2002)

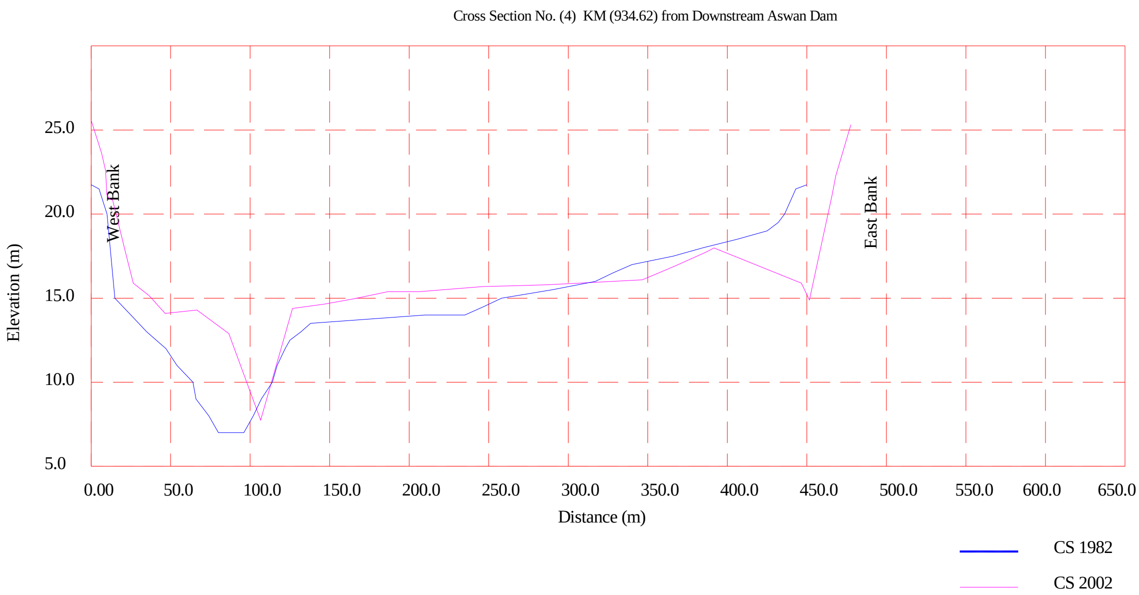


Figure 5: Cross section changes No. 4 at the bridge (1982 and 2002)

Table 1: Computed local scour according to different velocities for a flow of (350 m.m^3/day)

Pier No.	Distance from Left Side (m)	Scour depth (m) according to different velocities				
		V=1m/s	V=0.9m/s	V=0.8m/s	V=0.7m/s	V=0.6m/s
1	69.33	4.58	4.37	4.16	3.93	3.67
2	104.33	9.66	9.23	8.78	8.29	7.76
3	139.33	4.47	4.27	4.06	3.84	3.59
4	209.33	4.17	3.99	3.79	3.58	3.35
5	279.33	3.94	3.77	3.58	3.38	3.17
6	349.33	3.43	3.22	3.02	2.79	2.67

Table 2: Computed local scour according to different discharges

Pier No.	Distance from Left Side (m)	Scour depth (m) according to different discharges Q (mm^3/day)		
		300	250	200
1	69.33	4.36	4.18	3.87
2	104.33	9.21	8.84	8.19
3	139.33	4.25	4.08	3.77
4	209.33	3.95	3.78	3.49
5	279.33	3.72	3.56	3.27
6	349.33	3.17	2.98	2.79

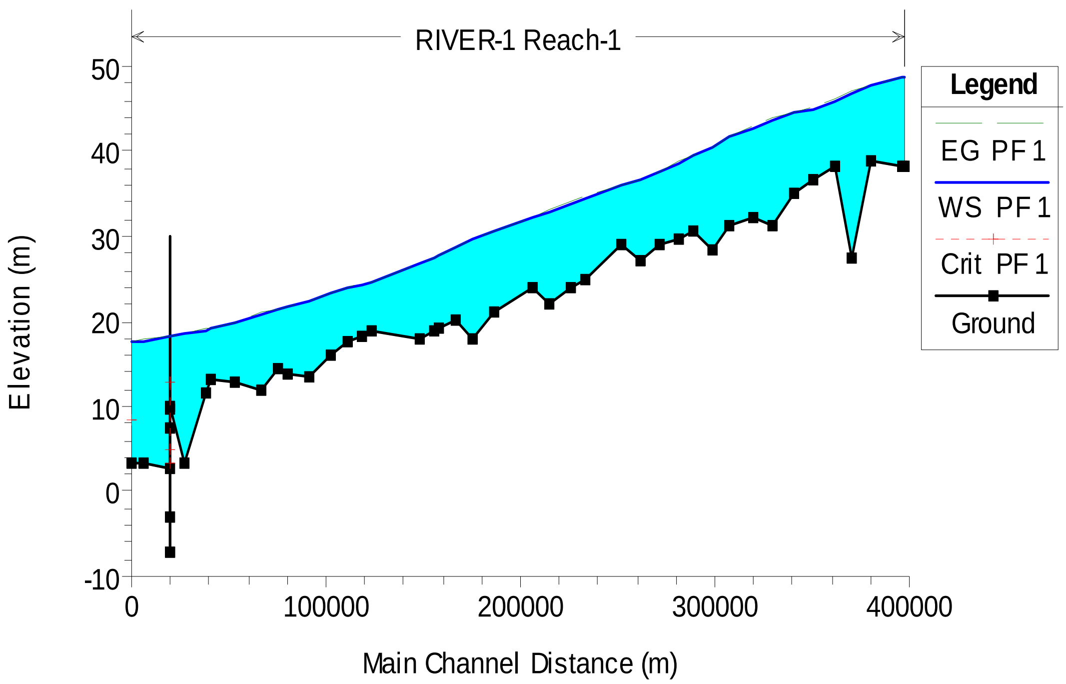


Figure 6: The model output for the computed water level (Q=350 mm^3/day)

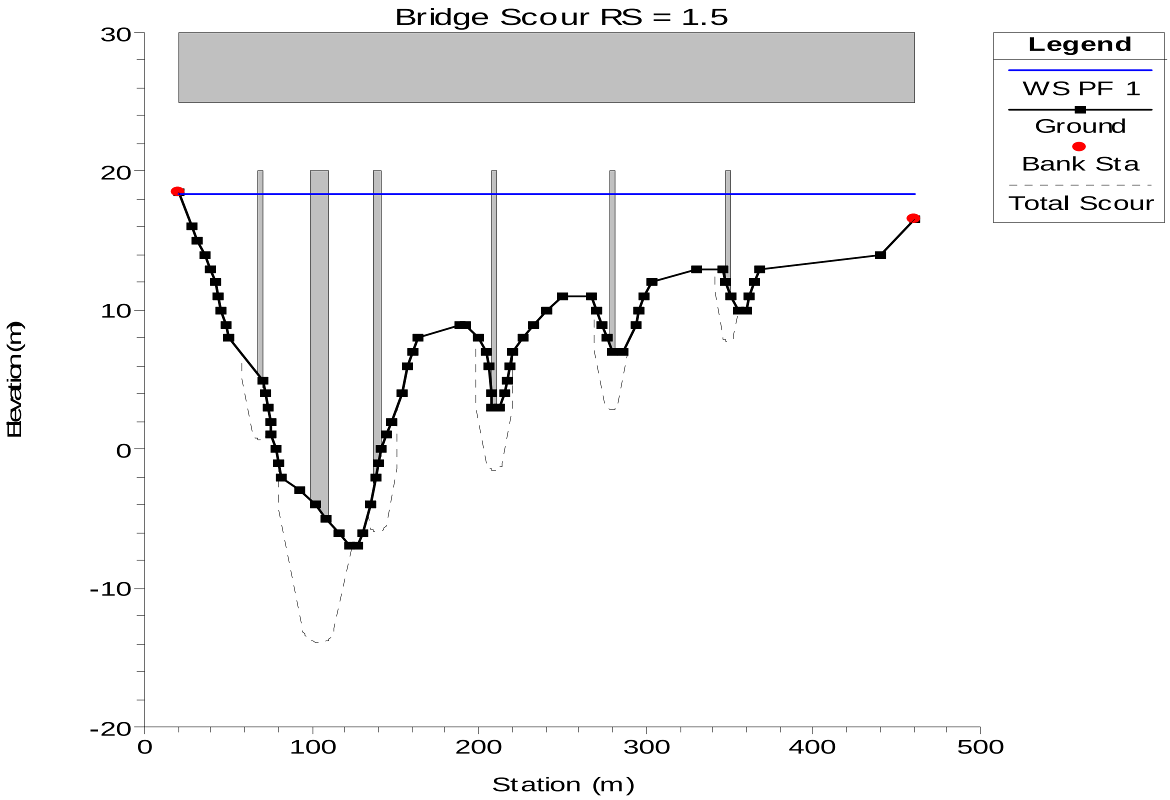


Figure 7: The model output local scour for maximum discharge (Q=350 mm^3/day)

CONCLUSIONS

Local scour monitoring is very important to avoid major damages that may occur to bridges and to be able to predict the future conditions. The monitoring of the local scour occurred at the Imbaba Bridge was analyzed during this research. The local scour at the bridge site was simulated using HEC-RAS computer model. The model calibrations for flows of 171 and 350 m.m^3/day show close relationship of the measured and computed results. The monitoring of Imbaba Bridge local scour shows a local scour depth of about -8.3 m since its construction up till 1992. In this study, the oldest available cross-section in 1982 and the recent available cross-section in 1992 at the center of the bridge and the piers location are analyzed and evaluated. From this results, it can be concluded that generally the local scour occurred around all piers was during this period. However, the maximum value of local scour was found around the second pier from the left side because it is the largest pier. Due to the large size of this pier, the effect of the horseshoe vortex is very significant at the pier base.

In addition, the effect of different velocity values in estimating local scour for maximum discharge (350 mm^3/day) was analyzed. The maximum local scour is predicted of 9.66 m and 7.76 m related to velocity 1m/s and 0.6 m/s respectively for the second pier which located at distance 104.33 m from the left side. The expected local scour in cases of passing a discharge of 300 – 250 -200 m.m^3/day is expected. It is concluded that local scour depth affected by increasing discharge especially for the second pier which should be taken into consideration during future local scour protection. The implementation of a monitoring program for bed level changes at the bridge site is recommended.

REFERENCES

[1] ASCE, 1975, “Sedimentation Engineering”. ASCE Manuals and Reports on Engineering Practice-No.54.

[2] Bruce W. Melville, 1984, “Live-Bed Sour at Bridge Piers” J., Hyd., Div., ASCE, Vol. 110, No. 9, (1984).

[3] Hydraulic Research Institute, 1981, “Study of Nile River Bed at the Cairo Bridges”, in Arabic, Cairo, Egypt.

[4] Hydraulic Research Institute, 1992, “Monitoring Local Scour in the Nile River at Imbaba Bridge”, in Arabic, Cairo, Egypt.

[5] Nile Research Institute, 2002, “Study of Electric Cable under Nile River Bed in Imbaba Region”, NRI report, NWRC, Egypt.

[6] US Army Corps of Engineers, 2001 HEC-RAS River Analysis System Computer Model. The Hydrologic Center, Davis, California.