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Numerical Simulation of Scour Depth Variation Around Vertical Wall Abutments

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***Abstract*— The HEC-RAS one dimensional, free-surface numerical model was used to simulate bed formation in alluvial channels. Based on the hydraulic equations all the hydrodynamic parameters were firstly simulated and then a sediment transport simulation was performed. The model can compute the sediment transport using a hydrodynamic simplification, the quasi-unsteady flow assumption, which approximates a continuous hydrograph with a series of discrete steady flow profiles. The current research work aims to investigate scour depth variation in the region of vertical wall abutments. Numerical computations of scour depth variation around a vertical wall abutment are verified by comparing them with available measurements. Moreover the study reports experimental results of scour depth variation along the center flow line of a channel in the vicinity of a vertical wall abutment. The experimental measurements of bed variation for six different inflow discharges and three different abutment widths, after a specified flow duration, are graphically presented and can be used by other researchers in order to validate their simulations. Comparison between computed results and available measurements of scour depths is satisfactory along the center flow line as the numerical simulation is one dimensional. The one dimensional model gives an accurate estimation of the maximum scour depth near the obstruction and it is applicable for a safe design criteria for the assessment of bridge vulnerability which is very important in river engineering works.**

***Index Terms*—Bridge Abutment, Numerical Simulation, Bed Level Variation, Scour Depth.**

I. INTRODUCTION

Erosion, transportation and deposition of solid particles have shaped the present landscape of our world and can cause severe engineering and environmental problems. Local scour at bridge piers and abutments are the most common causes of bridge failure during catastrophic floods. Local scour around piers and abutments involves removal of material from around the obstructions and it is caused by an acceleration of flow and resulting vortices. The flow obstructed by an abutment accelerates and forms a vortex starting at the upstream end of the abutment and running along the toe of the construction. The flow pattern and the mechanism of local scouring around bridge abutments is very complex and many fluvial bridges suffered severe damage or collapsed during heavy floods. River structures are exposed to both contraction and local scour around bridge piers and abutments and erosion phenomena are surely one of the most important factors in bridge vulnerability and have been reported by various investigators. From the engineering view point, the accurate quantitative estimation of local scour process around hydraulic structures is necessary for the prevention of severe environmental problems and for a safe river design.

Extended research has been conducted to determine the depth and location of the scour hole that develops from the vortex that occurs at the abutment, and numerous abutment scour equations have been developed to predict this scour depth. Therefore, for the computational simulation of actual river problem, it is necessary to consider unsteady flow, bottom topography, sedimentation etc. The three-dimensional hydrodynamic models coupled with a proper sediment transport equation are expected to give more accurate results than other models. However, the semi-three-dimensional or two-dimensional or one dimensional models are very popular. Their popularity is due, not only on their simplicity (relatively to the three dimensional models), but also on the satisfactory approximation in rivers and canals. Numerous publications are reported for one-dimensional bed deformation in alluvial channels. Among them Bhallamudi and Chaudhry [1] developed an explicit numerical model to simulate one - dimensional scour and deposition. Klonidis and Soulis [7] and Farsirotou, Soulis and Dermissis [3] developed one dimensional models for bed morphology simulation. Biglary and Sturm [2] developed a two dimensional hydrodynamic model for the flow around a bridge abutment in order to compute the maximum velocity near the upstream corner of the abutment face. They used a finite volume difference scheme with a staggered grid. Richardson and Panchang [8] simulated with a three dimensional hydrodynamic model the flow occurring at the base of a cylindrical bridge pier. The results were compared with experimental observations. Kassem and Chaudhry [6] compared a two-dimensional fully coupled model with a semi-coupled one using the Beam and Warming alternative-direction implicit scheme to calculate bed variations in alluvial channels. Farsirotou, Soulis and Dermissis [4] presented an explicit, integral, two-dimensional, finite volume, numerical scheme to simulate bed morphology variation in alluvial channels. Farsirotou and Xafoulis [5] reported an experimental investigation on the effects of various parameters on local scour depth around vertical wall abutments.

The objective of this research work is to investigate bed formation in alluvial channels as well as in regions around abutments. For this purpose a one dimensional numerical model which combines hydrodynamic and sediment transport equations was applied. Numerical computations of scour depth variation around a vertical wall abutment were verified by comparing them with available measurements. A laboratory experimental procedure was established to simulate local scour around vertical-wall abutments in

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uniform sediments under clear water scour conditions. Comparisons of scour depth variation along the center flow line of the channel, in the region of the abutment, for different flow discharges and abutment widths show that the one-dimensional simulation model appears to give a reasonable estimation of the maximum local scour depth.

II. COMPUTATIONAL ANALYSIS

A. Hydraulic and sediment transport model

The Hydrologic Engineering Center's River Analysis System (HEC-RAS [9]) numerical model was used in order to perform a one dimensional sediment transport and bed level variation analysis. The HEC-RAS model can compute the sediment transport using a hydrodynamic simplification, the quasi-unsteady flow assumption, which approximates a continuous hydrograph with a series of discrete steady flow profiles. For each record in the flow series, flow remains constant over a specified time window for transport. It is obviously that before performing any sediment transport numerical simulation the river hydraulics are already determined.

B. Governing flow equations

Water surface profiles are computed from one cross section to the next by solving the energy equation to a body of water enclosed by two cross sections at locations 1 and 2 as:

$$h_2 + Z_2 + \frac{a_2 V_2^3}{2g} = h_1 + Z_1 + \frac{a_1 V_1^3}{2g} + h_e \qquad (1)$$

where h is the water depth at cross sections, Z is the elevation of the main channel inverts, V is the average velocity, a is velocity weighting coefficient, g is the gravitational acceleration and h_e is the energy head loss. The energy head loss between two cross sections is comprised of friction losses and contraction and expansion losses as:

$$h_e = LS_f + C\left(\frac{a_2 V_2^2}{2g} - \frac{a_1 V_1^2}{2g}\right) \qquad (2)$$

where L is the distance between cross section 1 and 2 along the direction of the flow, S_f is the friction slope between two cross sections, which is computed using Manning's equation as:

$$Q = \frac{1}{n} A R^{2/3} S_f^{1/2} \qquad (3)$$

where Q is the flow discharge, n is the Manning roughness coefficient, A is the flow area, R is the hydraulic radius (area/wetted perimeter) and C is expansion or contraction loss coefficient. The program assumes that a contraction is occurring whenever the velocity head downstream is greater than the velocity head upstream. When the change in river cross section is small and the flow is subcritical, the contraction and expansion coefficient are equal to 0.1 and 0.3, respectively. In more abrupt change, such as these occurring at bridges, the used values are 0.3 and 0.5, respectively.

The HEC-RAS sediment model solves the sediment continuity equation, known as the Exner equation, as:

$$(1-\lambda_p) B \frac{\partial z}{\partial t} = -\frac{\partial Q_s}{\partial x} \qquad (4)$$

where B is the channel width, z is the channel elevation, λ_p is the active layer porosity, t is the time, x is the distance along the flow direction and Q_s is the sediment discharge. Equation (4) states that the change of sediment volume in a control volume (aggradation or degradation) is equal to the difference between the inflowing an outflowing sediment loads. The sediment continuity equation is solved by computed a sediment transport capacity through the control volume associated with each cross section. If sediment transport capacity is greater than supply there is a sediment deficit and bed erosion. If supply exceeds transport capacity there is a sediment surplus and material deposition [9].

C. Sediment transport empirical relations

Since sediment transport is sensitive to many hydraulic and sediment variables there are different sediment transport empirical equations available in the literature. The sediment discharge may be predicted by one of the following bed-load formulae available in the HEC-RAS model [9]:

a. Ackers and White (1973),
b. Engelund and Hansen (1976)
c. Laursen-Copeland (1996),
d. Meyer-Peter and Muller (1968),
e. Toffaleti (1968),
f. Yang (1984) and
g. Wilcock (2001)

The transported sediment load computed by the different equations can vary by orders of magnitude, depending on the project material and hydrodynamics.

III. SCOUR PROCEDURE AROUND A BRIDGE ABUTMENT

The local scour variation around a vertical wall abutment in uniform sediments under clear water scour conditions is numerically simulated. The one-dimensional numerical results are compared with available experimental measurements observed by Farsirotou and Xafoulis [5] in a laboratory flume. In the laboratory of ¨Flooding Protection Hydraulic Works & Water Resources Management¨ of the Department of Civil Engineering T.E. of the Technological Educational Institute of Thessaly a prismatic open channel, of rectangular cross-section, of 0.078m wide and 6.0mm long was used. Three different sizes of vertical wall abutment models were used, with abutment width, b, transverse to the flow, equal to 0.036m, 0.048m and 0.051m. The experimental inflow discharges were equal to $0.0004m^3/s$, $0.0005m^3/s$, $0.0006m^3/s$, $0.0007m^3/s$, $0.0008m^3/s$ and $0.00095m^3/s$.

After the required time duration period is reached, the bed elevation measurements data were recorded using a water-level gauge moving in longitudinal and in transverse directions of the flume introducing an optical error of ~0.1mm. Measurement data were obtained along three different ¨flow lines¨ presented in Fig. 1:

a) the flow line located at a constant distance of 0.005m to the streamwise face of the abutment parallel to the flow direction (¨flow line¨ 1),
b) the flow line located at a constant distance of 0.005m to the upstream transverse face of the abutment transverse to the flow direction (¨flow line¨ 2) and
c) the center flow line of the channel located at an equal distance between the solid walls (¨flow line¨ 3).

All measurements along the hypothetical ¨flow line¨ 1 and 3 were obtained at successive distances of 0.02m while the measurements along the ¨flow line¨ 2 were taken at successive distances of 0.01m apart. The scanned bed variation region was included in an area extending 0.06 m upstream to the abutment to 0.06 m downstream to it, while in the transverse direction the length is extended along the total channel width (Fig. 1) [5].

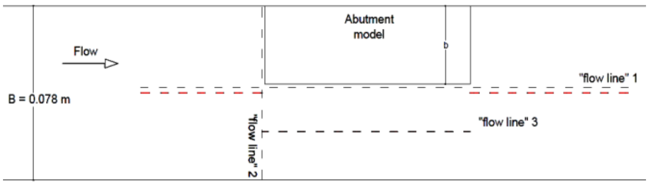

Figure 1. Measurement data locations

The used grid for the one-dimensional numerical computation had a spatial step, Δx, equal to 0.01m upstream, downstream of the abutment and in the abutment region. The Manning's roughness coefficient, n, was estimated as 0.02 and the initial bed slope was equal to 0.0. The bed material consisted of gravel which had a mean diameter of 0.002m.

Sediment transport numerical simulation was based on quasi-unsteady hydraulics which approximates a flow hydrograph by a series of steady flow profiles associated with corresponding flow durations. At the upstream cross section of the channel each experimental inflow discharge accompanied by a time duration, over which the flow is constant, was specified as an upstream hydraulic boundary condition. At the downstream cross section the slope of the energy grade line was used and the model determines a downstream depth for each discharge, solving Manning's equation. Moreover, the equilibrium load was specified, at the upstream cross section, as a sediment boundary condition. This means that at the upstream cross section the sediment transport capacity is computed, for each time step, and is used as the sediment inflow. Since load is set equal to capacity, there will be no aggradation or degradation at this cross section.

The sediment discharge was predicted, in the one dimensional numerical model, by Engelund and Hansen bed-load formula as [9]:

$$Q_s = 0.000025\gamma_w s V^2 \sqrt{\frac{d_f}{g(s-1)}}\left[\frac{\tau_o}{(\gamma_w s-\gamma_w)d_f}\right]^{3/2} B \qquad (5)$$

where γ_w is the specific weight of the water, s is the specific gravity of sediment, V is the average velocity, d_f is the particle fall diameter, B is the channel width and τ_o is the bed level shear stress given by:

$$\tau_o = \gamma_w R S \qquad (6)$$

where S is the channel bottom slope.
The particle fall diameter is computed as:

$$d_f = -69.07 d_s^2 + 1.0755 d_s + 0.00007 \quad \text{if } d_s \le 0{,}026\text{m} \qquad (7)$$

$$d_f = 0.1086 d_s^{0.6462} \quad \text{otherwise} \qquad (8)$$

Exner 5 sorting method was used to compute active layer thickness and vertical bed layer tracking assumptions and fall velocity of sediments was computed using the available in the model Ruby algorithm [9]. Those empirical relations were proved to be more reliable and the choice was made after several numerical test experimentation and comparison with experimental results.

IV. COMPARATIVE STUDY OF NUMERICAL SIMULATIONS WITH EXPERIMENTAL MEASUREMENTS

Comparisons between numerical simulation results and experimental measurements of bed level variation in the region of the vertical wall abutment of b=0.036m are shown in Figs. 2-7, along the center flow line (flow line 3, equal distance between upper and lower walls), at different inflow discharges, after t=30min. Figures 8-13 and 14-19 present the characteristic comparisons between numerical predictions and measurements of the bed elevation, along the center flow line, with abutment widths b=0.048m and b=0.051m, respectively. In all figures experimental measurements along the flow line 1 are also presented in order to provide the maximum scour depth at the abutment for each geometry and hydraulic condition.

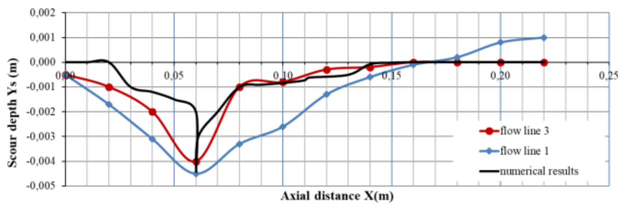

Figure 2. Comparison between computed and measured scour depth, at Q=0.0004m^3/s and b=0.036m, after t=30min

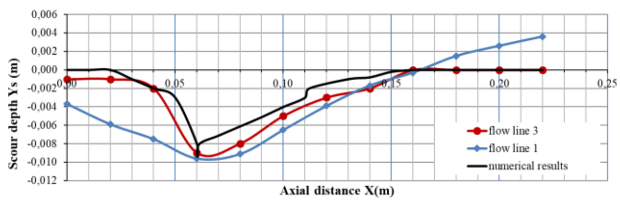

Figure 3. Comparison between computed and measured scour depth, at Q=0.0005m^3/s and b=0.036m after t=30min

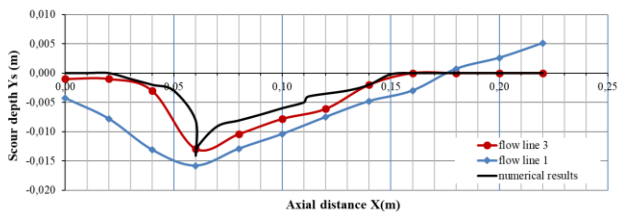

Figure 4. Comparison between computed and measured scour depth, at Q=0.0006m^3/s and b=0.036m after t=30min

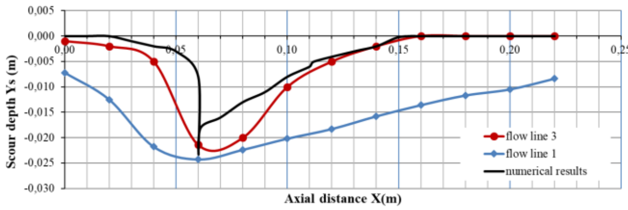

Figure 5. Comparison between computed and measured scour depth, at Q=0.0007m^3/s and b=0.036m after t=30min

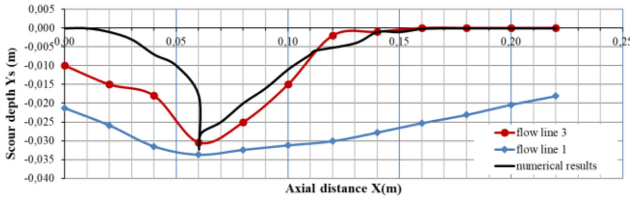

Figure 6. Comparison between computed and measured scour depth, at Q=0.0008m^3/s and b=0.036m after t=30min

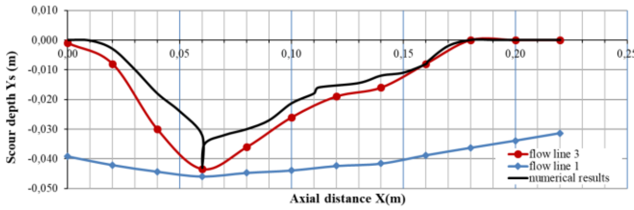

Figure 7. Comparison between computed and measured scour depth, at Q=0.00095m^3/s and b=0.036m after t=30min

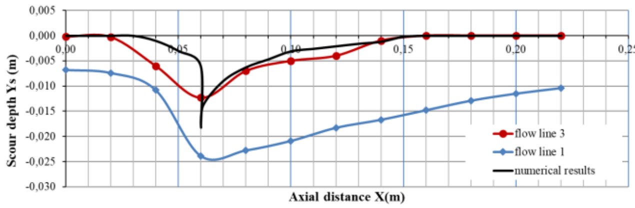

Figure 8. Comparison between computed and measured scour depth, at Q=0.0004m^3/s and b=0.048m after t=30min

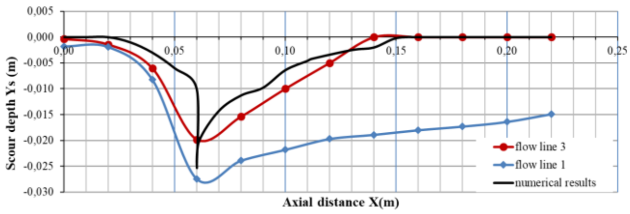

Figure 9. Comparison between computed and measured scour depth, at Q=0.0005m^3/s and b=0.048m after t=30min

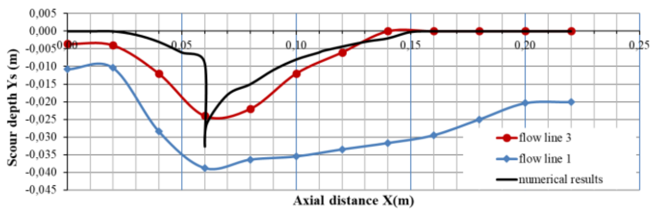

Figure 10. Comparison between computed and measured scour depth, at Q=0.0006m^3/s and b=0.048m after t=30min

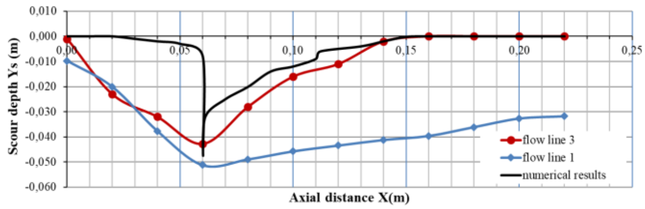

Figure 11. Comparison between computed and measured scour depth, at Q=0.0007m^3/s and b=0.048m after t=30min.

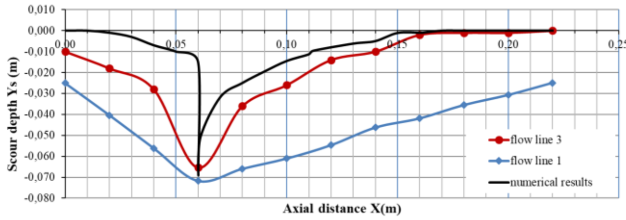

Figure 12. Comparison between computed and measured scour depth, at Q=0.0008m^3/s and b=0.048m after t=30min

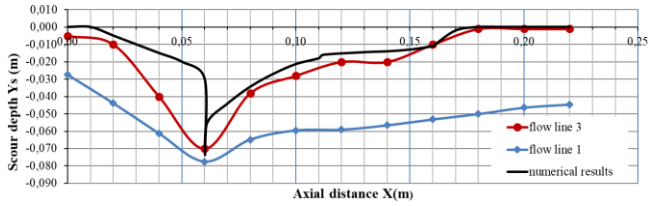

Figure 13. Comparison between computed and measured scour depth, at Q=0.00095m^3/s and b=0.048m after t=30min

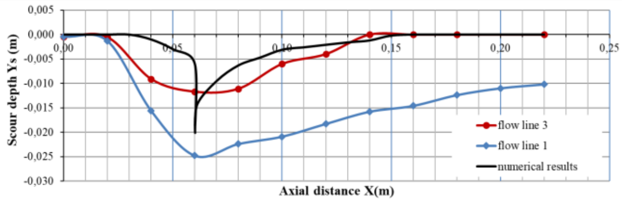

Figure 14. Comparison between computed and measured scour depth, at Q=0.0004m^3/s and b=0.051m after t=30min.

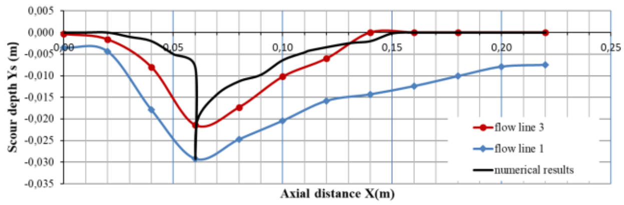

Figure 15. Comparison between computed and measured scour depth, at Q=0.0005m^3/s and b=0.051m after t=30min.

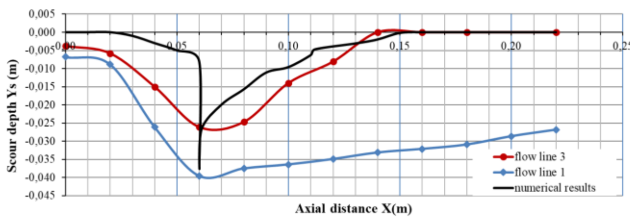

Figure 16. Comparison between computed and measured scour depth, at Q=0.0006m^3/s and b=0.051m after t=30min.

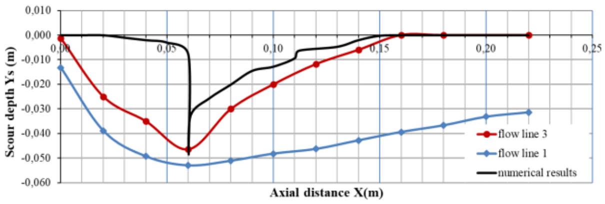

Figure 17. Comparison between computed and measured scour depth, at Q=0.0007m^3/s and b=0.051m after t=30min.

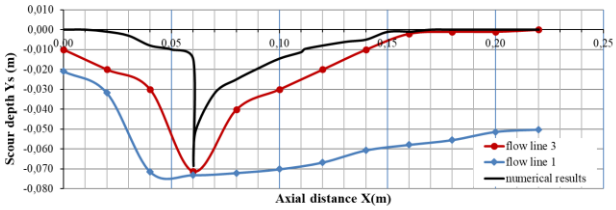

Figure 18. Comparison between computed and measured scour depth, at Q=0.0008m^3/s and b=0.051m after t=30min.

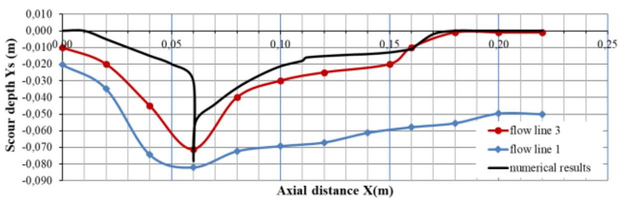

Figure 19. Comparison between computed and measured scour depth, at Q=0.00095m^3/s and b=0.051m after t=30min.

Measured scour depths in the upstream to the abutment region are under predicted by the numerical simulation. In general, good agreement between measured and computed bed erosion is obtained in the region and downstream of the abutment. The comparative performance between computed scour depths and measurements along the center flow line (flow line 3) gives satisfactory results. Differences between predictions and measurements in regions immediately upstream to the abutment are usually expected as in the flow high viscous effects co-existing with downward velocities in the close to the leading edge region of the abutment. It is in these regions that high viscous effects combined with three-dimensional flow restrict a one dimensional model to adequately predict the flow behavior. Thus, the use of a viscous three-dimensional model to simulate the flow is recommended. Experiments show that along the upstream to the abutment region and close to the vertical side of the abutment, erosion increases and a scour hole develops. The maximum scour depth occurs upstream of the abutment, at the upstream corner of the construction. At the downstream side of the abutment, the previously eroded material is deposited there and the scour depth is relatively small. This process is well numerically predicted by the one-dimensional hydrodynamic and sediment transport numerical model. The maximum scour depth at the upstream edge of the abutment, is influenced by the abutment width and by inflow discharge and increases with increase in the abutment width, normal to the flow direction and with inflow discharge increment. The computed maximum scour depth at the upstream edge of the abutment, for each geometry and inflow discharge, is adequately compared to the corresponding experimental measurements.

V. Conclusion

The one dimensional, quasi-unsteady, hydrodynamic and sediment transport HEC-RAS numerical model was applied to simulate bed level variation in the region of vertical wall abutments. Experimental measurements of scour depths were obtained at various locations near the abutments, for different inflow discharges and abutment widths. Comparison between computed results and available measurements of scour depths is relatively satisfactory along the center flow line as the numerical simulation is one dimensional. The HEC-RAS model gives an accurate and reliable estimation of the maximum scour depth near the obstruction and it is applicable for a safe design criteria for the assessment of bridge vulnerability. However, wherever possible, a two or three dimensional numerical model is necessary in order to give an improved validation methodology for complex river geometries with different constructions.

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