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PHYSICAL MODELLING OF LOCAL SCOUR AROUND BRIDGE PIER

Mojtaba Karimaee Tabarestani* & S. Amin Salamatian

Department of Civil Engineering, Shahab Danesh Univeristy, Qom, Iran

*Corresponding Author: *m.karimaee @aut.ac.ir*

Abstract: Accurate estimation of local scour around bridge pier is important for economic and safe design of bridges. Due to the complexity of the phenomenon and weak relationship between laboratory and field data, the conventional method such as laboratory-derived equations may fail in determining the accurate scour depth. In the present study, a new method was developed to determine the temporal variation of scour depth and final scour extension around bridge pier. Accordingly, using the rules of physical modelling for rivers, a simple method was established for physical modelling of local scour around bridge pier. To demonstrate the accuracy of the method, a typical prototype was chosen and the method was applied for determining different dimensions of the model. Based on the sediment density scale, low density sediment with ρs=1.05 kg/m3 was considered in the model for scour experiment. The results of the experiment including temporal variation of scour depth, equilibrium time and scour depth and final extension of scour hole were then scaled up to the prototype. Finally, empirical equations were utilized to predict maximum scour depth for the typical prototype. Results showed that the value of equilibrium scour depth from the present method was in the range of empirical equations prediction.

Keywords: *Bridge pier, local scour, river with movable bed, physical modelling, model and prototype*

1.0 Introduction

Every year, many bridges around the world fail because of scouring around their piers or abutments (Richardson and Davis, 1995). As illustrated in Figure (1), this phenomenon is due to a complex vortex system which forms around the pier or abutment. These vortices consist of a horseshoe vortex initiated from the down flow at the upstream face of the pier and wake vortices downstream of the separation point at the sides of the pier (Raudkivi, 1998). Briefly, the approach flow velocity goes to zero at upstream face of a pier and this result in an increase in pressure. As the flow velocity reduces from surface to the bed, the dynamic pressure on the pier face also decreases downwards. This pressure gradient drives the flow down the pier resembling a vertical jet (Figure 1). When this down flow impinges the stream bed, it digs a hole in front of the pier and rolls up and by interaction with the coming flow; it forms a complex vortex system. This vortex extends downstream and passes the sides of the pier. Due to its similarity to a horseshoe, this vortex is called horseshoe vortex. The horseshoe vortex deepens the scour hole in front of the pier until the shear stress on the bed material becomes less than its critical shear stress. The accelerating flow at two sides of the pier creates two slots in the streambed, which facilitates the transport of removed sediment from the scour hole at the upstream perimeter of the pier. The separation of the flow at the sides of the pier creates the so called wake vortices. These vortices are unstable and shed alternatively. These vortices act as little tornadoes lifting the sediment from the bed and forming their own scour hole (Karimaee and Zarrati, 2013).

In order to study the influence of a particular river structure such as a bridge pier on the topography of river bed, three procedures were chosen: computation method, using empirical equations and physical modelling. Computation procedure possible only after a number of simplifying assumptions and the solution will naturally be only approximate and the results will always require checking and testing by laboratory investigations because of the complexity of flow field around the structure (Novak and Cabelka, 1981). On the contrary, in the case of bridge pier, many predictive equations for equilibrium scour depth have been suggested. Nevertheless, most of them are based on laboratory experiments, which have been carried out with simple channel and bridge geometry, or solely on field measurements which produce a wide range in values of the equilibrium scour depth. So, there may be a weak relationship between laboratory and field data, and overestimation of scour depth may occur when laboratory-derived equations are used to calculate the local scour depth in the field for bridge foundation design. Indeed, physical modelling may be the only possible method of solution most especially for more complicated cases.

Physical modelling developed rapidly into an engineering tool of general recognition for the solution of hydraulic engineering problems since 1885 (Kobus, 1980). Generally, a physical scale model is completely similar to its real-world prototype and involves no scale effects if it satisfies mechanical similarity implying the following three criteria (Novak and Cabelka, 1981 & Heller, 2011):

- Geometric similarity;
- Kinematic similarity;
- Dynamic similarity.

Geometric similarity requires similarity in shape, i.e. all length dimensions in the model are λ times shorter than of its real world prototype. Model lengths, areas and volumes therefore scale with λ, λ^2 and λ^3, respectively, in relation to the prototype. Kinematic similarity implies geometric similarity and in addition indicates a similarity of motion between model and prototype particles. It requires constant ratios of time, velocity, acceleration and discharge in the model and its prototype at all times. Dynamic similarity requires in addition to geometric and kinematic similarities that all force ratios in the two systems are identical. In fluid mechanics, a large number of force ratios were defined which have to be considered. Exact model similarity would consequently require a model operating in a miniature universe where all physical parameters are scaled including geometry, fluid properties, characteristics of the structure and also the atmospheric pressure. However, due to many restrictions such as economic aspects, it is impossible to consider the entire forces ratio in physical modelling. Therefore, the most relevant force ratio affect the process is selected and scale effects due to the others have to be negligible (Chanson, 2009 & Heller, 2011).

Generally, to obtain dynamic similarity, at first important parameters affect the phenomenon must be listed and dimensional analysis must be consider to determine dimensionless parameters (Novak *et al.,* 2010). Dimensional analysis is a most useful tool in experimental fluid mechanics, allowing for the implicit formulation of criteria for dynamic similarity in a simple and direct manner. The dimensionless parameters include the geometrical ratios as well as the force ratios. Similarity requires that each of these dimensionless parameters quantitatively agree between model and real-world prototype.
Due to sediment movement around bridge pier, local scour is a kind of river phenomena which can be physically modelled in a group of river with movable bed. Nevertheless, it may be a tough effort to model this phenomenon because of principle limitations such as model size as well as practical limitations such as measuring methods and data collection (Kobus, 1980).

In the present study, a new method was presented to determine the temporal variation of scour depth and final extension of scour hole around bridge pier. In this method, by considering the physical modelling principles, a way for physical modelling of scouring around a typical bridge pier as prototype was developed. By applying the scales derived, the model in the laboratory was prepared and local scour experiment was carried out. Finally, the results of the experiments were scaled up to the prototype.

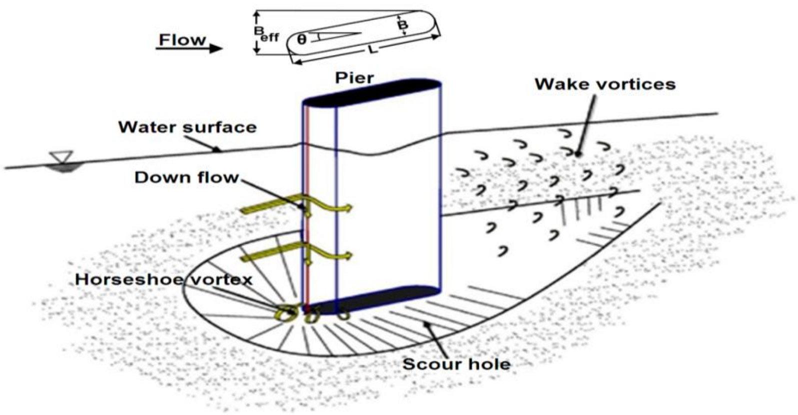

Figure 1: Schematic vortex structures around circular pier

2.0 Physical Modelling of Flow and Sediment Material

Flow and sediment transport in an open channel can generally be represented as a function of several variables. The specific sediment transport rate T_s, expressed as mass per unit time and unit width, depends on the following variables (Kobus 1980):

$$T_s = f\left(\rho_w, \nu, g, \rho_s, d, h, S\right) \tag{1}$$

where ρ_w and ρ_s are flow and sediment density respectively, ν is water kinematic viscosity, g is gravitational acceleration, d is the transported material, h is water depth and finally S is the slope of transporting flow. Applying dimensional analysis, five dimensionless parameters can be derived from Equation (1) which are shown in the following equation:

$$T_* = f\left(\frac{h}{d}, \frac{\rho_s}{\rho_w}, \frac{u_* \cdot d}{\nu}, \frac{\rho \cdot u_*^{\ 2}}{g \cdot \Delta\rho \cdot d}\right) \tag{2}$$

where T_* is dimensionless sediment transport number which is defined as

$$T_* = \frac{T_s}{\rho_s . d . u_*} \tag{3}$$

The parameter u_* in Equations (2) and (3) is shear velocity which is defined by the bottom shear stress (τ_0) and the density of the fluid (ρ_w)

$$u_* = \sqrt{\frac{\tau_0}{\rho_w}} = \sqrt{g.h.S} \tag{4}$$

The first term in the right hand side of Equation (2) is important in considering surface tension effects, which are generally not considered to be important when modelling the river bed (Maynord, 2006). The second term is the ratio of the sediment density to water density and it represents the buoyant force on the sediment. The third term in right hand side of Equation (2) is particle Reynolds number (Re_*) which was introduced by Shields in 1936. This parameter is the Reynolds number with the sediment diameter used for the characteristic length scale. The last term is a modified form of Froude number (Fr_*) named as Shields or Mobility number, containing the ratio of the specific weights of the fluid and submerged sediment.

In physical modelling techniques, sediment material is scaled such that the material will move in the same manner for both the prototype and the model. To do this one should assume that insipient motion and re-suspension of the sediment occur in the same manner for both. To accomplish this, Shields number similarity must be maintained. In addition, the ratio of the inertial forces of the sediment movement and the viscous forces in the laminar sub-layer of the water on the sediment, for each the prototype and model, must be equal. This is done by equating the Particle Reynolds Number.

Shields (1936) demonstrated from his experiments that the beginning of sediment motion has a unique relationship between Re_* and Fr_*. He also showed that, while Re_* is larger than 220, the river bed is in hydraulically rough condition and this parameter has negligible effect on sediment motion. This is due to insignificant effect of fluid viscosity on sediment transport motion. Moreover, conventional sediment transports presented by Mayer-Peter and Mueller (1948) or Einstein (1950) do not consider the influence of the particle Reynolds number which are valid for high values of the Re_*, i.e. for large grain sizes and large slopes.

The above theoretical consideration demonstrates that sediment transport in rivers is dependent on the dimensionless parameters Re_* and Fr_*. If sediment motion is to be correctly modelled, then the model sediment and the model geometric and dynamic scales have to be chosen in order for the dimensionless parameters to have the same value in model and in Prototype. Nevertheless, most Froude scale models usually consider Fr_* in their similarity but pay little attention to Re_*. If this is the case, then there will be scale effects that could eventually lead to misinterpretation of the results, as well as flawed designs.

After sediment material, the scale variables for flow characteristics such as flow velocity must be determined. For open channels, the flow is governed by the balance between inertial and gravitational forces. The ratio of these forces is called the Froude Number (Fr) which is:

$$Fr = V \Big/ \sqrt{g \cdot h} \tag{5}$$

where V is flow velocity. Practically, for every physical model, Froude number similarity must be maintained (Waldron, 2005). Furthermore, this parameter is also important in local scour around bridge piers (Richardson and Davis, 1995).

Normally, to physically study river phenomena, in order to improve the similarity of the flow processes as well as the time consumption and costly construction of river models which cover large areas, vertical distortion models are used. This means that the horizontal scale is not equal to the vertical scale. As a result, special care must be taken to ensure that the important physics impacting the processes in the prototype are being properly replicated in the model. As noted before, the important processes are determined from analysis of the problem being studied and then similitude theory is used to derive the appropriate non dimensional parameters (Equation 2). If X_r is the scale number of horizontal length such as bridge pier width and Z_r is the scale number of vertical lengths such as flow depth, then the distortion factor n is:

$$n = \frac{X_r}{Z_r} \tag{6}$$

where subscript r represents the ratio of prototype to model lengths. If the kinematic viscosity and specific weight of water is assumed to be equal in model and prototype, then the scales relationships of modelling can be obtained according to Re_* and Fr_* as is shown in the following:

$$(Re_*)_r = \left(\frac{u_* \cdot d}{\upsilon}\right)_r = 1 \tag{7}$$

$$(Fr_*)_r = \left(\frac{\rho \cdot u_*^{\;2}}{g \cdot \Delta\rho \cdot d}\right)_r = 1 \tag{8}$$

By using Equation (4), following relationships can be calculated, respectively:

$$(Re_*)_r = 1 \quad \Rightarrow \frac{g_r^{1/2}.Z_r^{1/2}.S_r^{1/2}.d_r}{\upsilon_r} = \frac{Z_r^{1/2}.d_r}{n^{1/2}} \tag{9}$$

$$(Fr_*)_r = 1 \quad \Rightarrow \frac{\rho_{wr} \times g_r}{\Delta\rho_r \times g_r} \times \frac{Z_r}{d_r} \times S_r = \frac{Z_r}{\Delta\rho_r \times d_r \times n} \tag{10}$$

Since g is equal in prototype and model, therefore g_r = 1 in above relationships. On the other hand, by placing Equation (6) in above relationships, following equations can be obtained, respectively:

$$X_r^{-\frac{1}{2}} \cdot Z_r \cdot d_r = 1 \tag{11}$$

$$X_r^{-1} \cdot Z_r^{2} \cdot d_r^{-1} \cdot \Delta\rho_r^{-1} = 1 \tag{12}$$

In above equations, d_r and $\Delta\rho_r$ are the sediment properties scales for grain size and the sediment density (under submerged condition), respectively. Equations (11) and (12) are applied to choose the geometry and sediment characteristics of model. This provides a system of two equations with four scale variables X_r, Z_r, d_r and $\Delta\rho_r$. Therefore, two scales can be chosen, freely. The choice of these geometric and sediment properties scales are determined by several practical aspects. For example the horizontal length scale (X_r) of the model is determined by the available laboratory space. By freely choosing two scales, other scales can then be determined from Equations (11) and (12) as illustrated in Table (1). For example, according to the first row of this table, while X_r and Z_r are chosen freely, parameter d_r can be calculated from Equation (11) as $d_r = X_r^{\frac{1}{2}} \times Z_r^{-1}$ and parameter $\Delta\rho_r$ can be obtained from Equation (12) as $\Delta\rho_r = X_r^{3} \times Z_r^{-\frac{3}{2}}$.

Table 1: Determination of scale numbers

Chosen Scale Numbers	X_r	Z_r	d_r	$\Delta\rho_r$
X_r , Z_r	X_r	Z_r	$X_r^{\frac{1}{2}} \times Z_r^{-1}$	$X_r^{\frac{1}{2}} \times Z_r^{-\frac{3}{2}}$
X_r, $\Delta\rho_r$	X_r	$X_r^{\frac{1}{2}} \cdot \Delta\rho_r^{\frac{1}{3}}$	$\Delta\rho_r^{-\frac{1}{3}}$	$\Delta\rho_r$
Z_r , $\Delta\rho_r$	$Z_r^{2} \times \Delta\rho_r^{-\frac{2}{3}}$	Z_r	$\Delta\rho_r^{-\frac{1}{3}}$	$\Delta\rho_r$

In present study, Froude number similarity was basically selected since the phenomenon of local scour around bridge pier normally takes place in open channel flow. The Froude number governs the water surface profile through the bridge and hence the pressure gradient in the vicinity of the piers. Since the gravitational acceleration is similar in the model and prototype, then Froude number similarity can be written as:

$$(Fr)_r = \frac{V_r}{(g_r \cdot Z_r)^{0.5}} = 1 \tag{13}$$

accordingly:

$$V_r \times Z_r^{-0.5} = 1 \tag{14}$$

After determining flow depth in the model by using vertical scale variable Z_r (Table 1), the flow velocity in the model can be obtained from Equation (14).

Finally, the time scale for the sediment transport can be determined by equating the dimensionless sediment transport number T_* (Equation 3) in prototype and model. The scale relationship result of Equation (3) can be obtained as follows:

$$t_{s,r} = \frac{X_r^{5/2}}{Z_r^{2}} \times \Delta\rho_r \qquad (15)$$

There is also another time scale which can be determined from the Froude number similarity (Equation 14). This is the time scale for hydraulic process and it differs from time scale for the sediment transport (Equation 15).

Finally, in order to show the accuracy of the present method, a typical prototype is chosen whereas it is a representative of many extant bridges constructed on a river near seaside with very mild slope, low velocity and small sediment grain size. Usually, the bed of these rivers is hydraulically in smooth or transitional condition with Re_*<220. Therefore, in sediment transport modelling of this typical prototype, similarity of the particle Reynolds number (Re_*) should be considered. Table (2) illustrates the characteristics of the selected typical prototype as well as some extant bridges and their river conditions sites in United States presented by David *et al.* (2005).

The values for the different parameters in typical prototype are in ranges of other bridges. The flow intensity parameter in the prototype (V/V_c; V is flow velocity and V_c is critical velocity for bed material movement prepared by Shields 1936) is 0.95 which shows that the river bed material is in threshold condition. The threshold is the flow condition that is just adequate to initiate the motion of sediment particles at the bed surface. According to Raudkivi (1998) maximum scour depth around a bridge pier occurs at threshold of stream bed material.

Table 2: Typical prototype characteristics and some similar real cases

Parameter	Typical prototype	Knik River(AK)	Assawoman Bay (DE)	Choptank River (MD)	Killbuck Creek (OH)
Pier diameter (m)	1.5	1.5	0.8	1.2	0.8
Flow velocity (m/s)	0.37	1.6	0.3	0.2	0.7
Flow depth (m/s)	3	3	3.2	1.5	2.2
Bed sediment median size (mm)	0.31	1.8	0.18	0.38	0.19
Sediment density (gr/cm^3)	2.65	2.65	2.65	2.65	2.65
Critical velocity for bed material (m/s)	0.4	0.672	0.336	0.349	0.334
Flow discharge in unit width (m^2/s)	1.12	4.8	0.96	0.3	1.54

3.0 Determination of Model Dimensions

There are four parameters that must be estimated for physical modelling of the typical prototype X_r, Z_r, d_r and $\Delta\rho_r$. According to Table (1), if two of these parameters are known, then the other remaining parameters can be evaluated. The known parameters are selected by laboratory restriction or other economic considerations. Previous studies have shown that, the duration of scour experiments with natural sand (ρ_s=2.65 g/cm^3) is very long which may cause experimental problem. Alabi (2006) studied the local scour around cylindrical bridge pier with natural sediment. In some cases, his tests lasted for more than 500 h to approach equilibrium condition. A method to overcome this problem is to utilize low density sediment in the experiments instead of natural sediment. Low density sediment could decrease the equilibrium time of scouring around the piers. Oliveto and Hager (2002) and Zokaie *et al.* (2013) utilized low density sediment with density in their experiments to study time development of scouring around bridge piers. In the present study, based on the experiments of Zokaie *et al.* (2013), artificial plastic material with density of 1.05 g/cm^3 was considered. Maynord (2006) noted a lower limit of 1.05 g/cm^3 for the density of model sediment due to its lightness. Consequently, the scale of sediment density parameter is:

$$\Delta\rho_r = \frac{(\rho_s - \rho_w)_p}{(\rho_s - \rho_w)_m} = \frac{(2.65-1)}{(1.05-1)} = 33 \tag{16}$$

On the other hand, based on laboratory limitation, the diameter of bridge pier is considered as 0.04 m. The width of bridge pier for typical prototype is 1.5 m (Table 2), therefore, the horizontal length scale is calculated as:

$$X_r = \frac{X_p}{X_m} = \frac{1.5}{0.04} = 37.5 \tag{17}$$

Consequently, parameters d_r and Z_r can be calculated by known scales $\Delta\rho_r$ and X_r. According to the scales relations in third row of Table (1), one can obtain:

$$Z_r = X_r^{\frac{1}{2}} \cdot \Delta\rho_r^{\frac{1}{3}} = (37.5)^{\frac{1}{2}} \cdot (33)^{\frac{1}{3}} = 19.6 \tag{18}$$

$$d_r = \frac{1}{\Delta\rho_r^{\frac{1}{3}}} = \frac{1}{33^{\frac{1}{3}}} = 0.31 \tag{19}$$

Moreover, to calculate time scale of sediment movement, Equation (15) can be applied as shown below

$$t_{s,r} = \frac{X_r^{5/2}}{Z_r^{\ 2}} \times \Delta\rho_r = \frac{(37.5)^{5/2}}{(19.6)^2} \times 33 = 740 \tag{20}$$

Finally, after calculating model dimensions and flow depth with the above considerations, flow velocity and discharge can be calculated using Equation (13). Table 3 shows the different dimensions and flow and sediment characteristics in the model.

Table 3: Model dimensions and sediment characteristics

Parameter	Value
Pier diameter (m)	0.040
Flow velocity (m/s)	0.083
Flow depth (m/s)	0.153
Bed sediment median size (mm)	0.970
Sediment density (gr/cm^3)	1.050
Critical velocity for bed material (m/s)	0.072
Flow discharge in unit width (m^2/s)	0.013

4.0 Experimental Results

Results of the considered experiment were extracted from a study presented by Zokaie *et al.* (2013). The experiment condition was exactly similar to Table (3). Since the stream bed in model and prototype are in threshold condition (V/V_c=0.95), the experiment was carried out in clear water condition without upstream discharge of transported sediment. The test was continued until no motion of particles was observed and in other words, equilibrium scour depth (d_s) was achieved. Other information about the experiment was presented in the study of Zokaie *et al.* (2013).

Figure (2) shows the experiment with the pier and scouring hole around it. In Figure (3), the dimension of scoring hole around the pier is shown schematically. According to this figure, the width and length of the scouring hole is about 0.3 and 0.42 m respectively. Moreover, the experiment demonstrated that equilibrium scour depth at the pier nose equal to 9.2 cm was achieved after 6 hr. Figure (4) and Table (4) show the time development of scour depth at the pier nose. As shown in Table (4), about 90% of equilibrium scour depth occurred in just 30% of equilibrium time or about 2 hour from the beginning of the experiment.

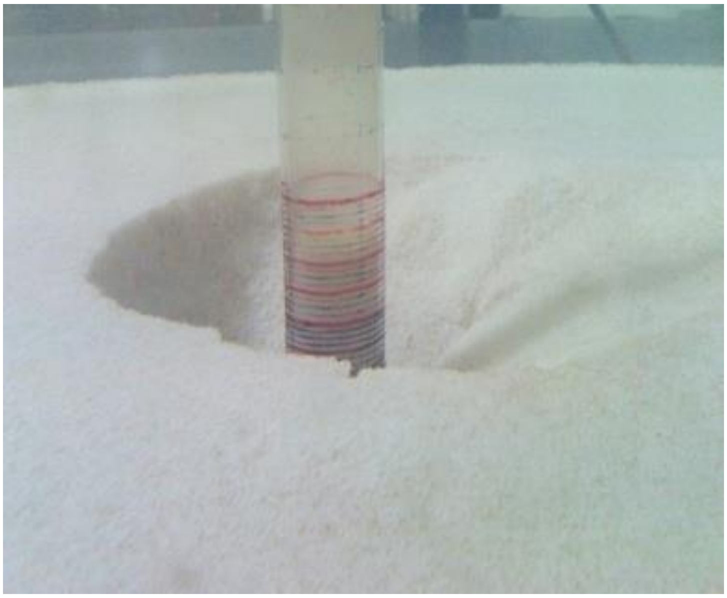

Figure 2: Experiment with the pier and scouring hole around it

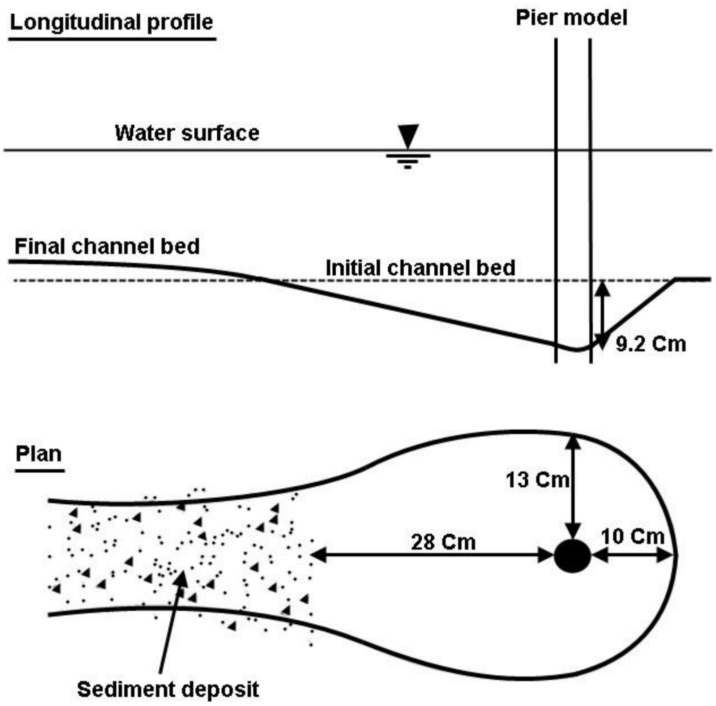

Figure 3: Dimensions of scoring hole around the pier

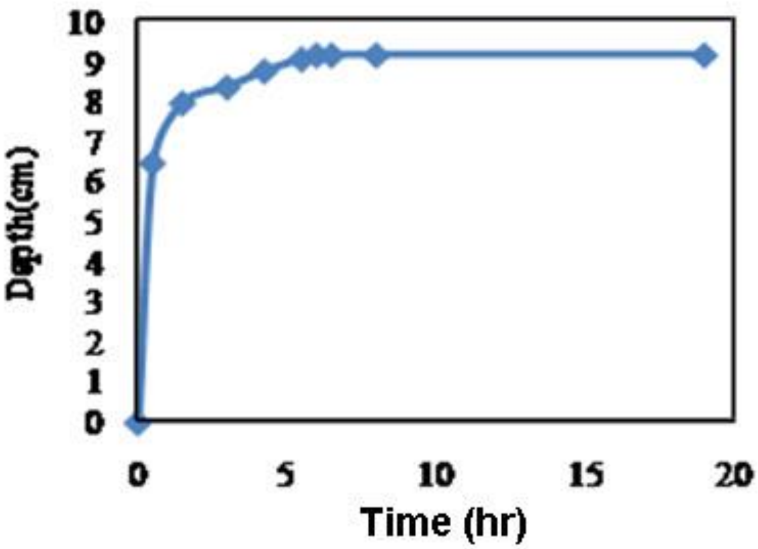

Figure 4: Development of scour depth at the pier nose

Table4: Scour depth at any time in the model

Time (hour)	Depth(cm)
0.0	0
0.5	6.5
1.5	8
3.0	8.4
4.3	8.8
5.5	9.1
6.0	9.2
6.5	9.2
8.0	9.2
20	9.2

5.0 Scale up Results from Model to Typical Prototype

In order to compute the time development of scour depth in the prototype, vertical length scale (Z_r) and time scale ($t_{s,r}$) were utilized. Results showed that, equilibrium scour depth at the pier nose and equilibrium time in the prototype is 1.8 m and 185 days, respectively. Table (5) shows the time development of scour depth at the pier nose.

Furthermore horizontal length scale (Equation 18) was used to obtain the dimensions of scour hole in plan. Figure (5) shows the extension of scour hole around the bridge pier in the prototype.

In order to validate the equilibrium scour depth at the pier nose in the prototype, the observed scour depth for extant bridges (see Table 3) are given in Table (6). The average of difference between scour depth in the prototype and the values of extant bridges is about 100%. It should be noted that, the observed scour hole in the extant bridges may not be in equilibrium condition.

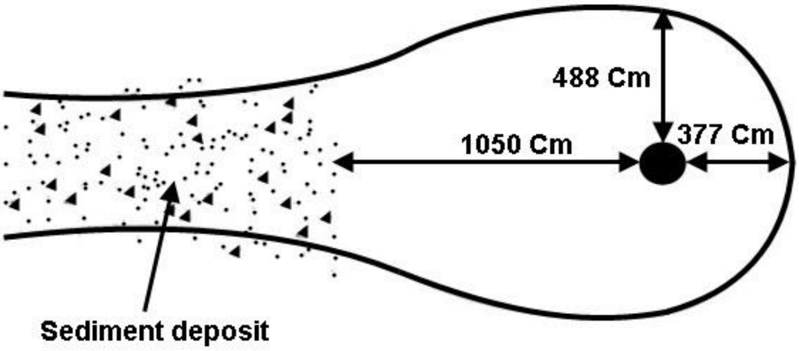

Figure 5: scouring region around the bridge pier in prototype

Table 5: Scour depth at any time in the prototype

Time(day)	Depth(m)
0	0
15.42	1.27
46.25	1.57
95.5	1.65
131.04	1.72
169.6	1.78
185	1.81
200.4	1.81
246.7	1.81
616.67	1.81

Table 6: Observed scour depth in different bridge sites

Bridge site	Scour depth (m)
Knik River(AK)	1.1
Assawoman Bay(DE)	1.0
Choptank River(MD)	0.9
Killbuck Creek (OH)	0.6
Typical Prototype	1.8

6.0 Calculating Scour Depth Using Empirical Equations

Various parameters are effective for scouring process around bridge pier in alluvial bed and numerous equations for predicting equilibrium scour depth have been presented by considering these parameters and experimental or field data. In this study, in order to find out the accuracy of present physical modelling method, equilibrium scour depth in the prototype was estimated by different empirical equations. Table (7) shows the empirical equations and calculated equilibrium scour depths. As shown in this table, equilibrium scour depth calculated by the present method is approximately in the range of the values computed by majority of the empirical equations. However, there is almost high difference between scour depth computed by the present method and Melville and Sutherland (1980) which is about 85%. Study prepared by Johnson (1995) showed that, the Melville and Sutherland (1988) equation over-predicted to a greater value than any other empirical equations.

Table 7: Equilibrium scour depth for the prototype computed by different methods

Investigation	Equation	Equilibrium scour depth (m)
Blench , 1969	$d_s = 1.8b^{0.25}q^{0.5}\left(\frac{h}{V}\right)^{0.25} - h$	1.78
Coleman , 1971	$\frac{V}{\sqrt{2gd_s}} = 0.6\left(\frac{V}{\sqrt{gb}}\right)^{0.9}$	1.24
Jain , 1981	$\frac{d_s}{b} = 1.4\left(\frac{h}{b}\right)^{0.3}\left(\frac{Vc}{\sqrt{gh}}\right)^{\frac{1}{4}}$	1.35
Richardson et al. , 1991	$\frac{d_s}{b} = 2K_1K_2K_3\left(\frac{b}{h}\right)^{0.65} Fr^{0.43}$	1.33
Melville & Sutherland , 1988	$\frac{d_s}{b} = 2.4K_hK_dK_\sigma K_sK_\alpha$	3.35
Present Study	Physical modelling	1.81

7.0 Conclusion

In present study, by considering principles in Physical River modelling with movable bed and an important parameter in scouring around bridge pier phenomenon called Froude number, a method for physical modelling of scouring around bridge pier for a particular prototype was presented. The prototype is chosen whereas it is a representative of many extant bridges. These bridges were constructed on rivers near seaside with very mild slopes, low velocities and small sediment grain sizes. Usually, the beds of these rivers are hydraulically in smooth or transitional condition. The conditions of the flow and bed material in prototype were effectively simulated for model. By utilizing the scales derived with modelling principles, the experiment on the model was prepared. From the experiment, equilibrium scour depth at the pier nose and equilibrium time for scour hole was measured as 9.2 cm and 6 hour, respectively. Subsequently, the results of experiments including time development of scouring at the pier nose and dimensions of scouring hole were scaled up to the prototype. Results showed that, equilibrium scour depth at the pier nose and equilibrium time in the prototype is 1.8 m and 185 days, respectively. Finally, empirical equations were utilized to predict the equilibrium scour depth for the typical prototype. Results showed that the value of equilibrium scour depth from the physical modelling was in the range of empirical equations.

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