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ON THE PREDICTION OF THE MAXIMUM DEPTH OF A SCOUR HOLE AROUND CYLINDRICAL BRIDGE PIERS IN NON COHESIVE SOILS

By

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ABSTRACT

The requested predictions for the maximum depth of scour in non cohesive soil around bridge piers are calculated by using a semiempirical approach. The presented method was derived from the continuity equation of mass and a balance of the acting forces during scouring in a non-cohesive sediment bed. As part of the results, the calculated evolution of the maximum scour depth with time is also shown.

INTRODUCTION

In this paper we present a method to estimate the maximum depth of scour around a bridge pier and apply it to solve the cases 1, 2, 7 and 8 of the prediction event in which only non cohesive sediments are expected to be eroded. The method is based on equations similar to others of wide application like those proposed by Breusers et al (1977), Jain and Fischer (1980), Richardson (1987) cited in Yammaz and Cicekdag (2001) also known as CSU-equation, or Melville and Sutherland (1988). Our methodology is based on an equation derived by Zanke (1982a) and differs from the above mentioned methods in the fact that the maximum depth of scour is time dependent. Melville and Chiew (1999) derived an equation using dimensional analysis which also incorporates time as a variable. Nevertheless, these authors assumed an exponential law for the erosion rate and adjusted their parameters. Such a mathematical form had been previously suggested by other authors (see Hoffmans and Verheij, 1997) but the suggestions were only based on the fact that the observed scour depth was well correlated. Ting et al (2001) presented laboratory measurements of scour-depth-versus-time curves and fitted them with an hyperbola. These experiments were conducted on sand and clay mounted piers.

Zanke (1978a) proposed a general formula for sediment transport estimation. It is based on the forces acting on the soil particles and dimensional analysis. Under the assumption that the variation of the volume of a three dimensional scour hole is proportional to that of the scour surface perpendicular to the horizontal plane and directed to the pier, it is posible to solve three dimensional scour cases, such as scour around bridge piers, as the general bidimensional case. Zanke (1982a) rearranged his primitive function for bedload estimation into a general equation for the prediction of the maximum scour of depth around bridge piers. The resulting dimensionless parameters were determined based on laboratory measurements (Zanke 1982a) and on the data of Ettema (1980) cited in Zanke (1982b). A summary of the mathematical derivation of Zanke´s equation is given below. In this work, we implement an algorithm to calculate scour depth under unsteady flows. This is also briefly explained.

METHOD

It is well known that the variation of the solid discharge, Q_s, per unit of area, A, equals the variation of the scour depth, z, per unit of time, t. If the specific solid discharge per unit of width is q_s, then

$$\frac{\partial Q_s}{\partial A} = \frac{\partial Q_s}{\partial x \partial y} = \frac{\partial q_s}{\partial x} = \frac{\partial z}{\partial t} \tag{1}$$

$$t \approx \frac{A_l}{q_s} \tag{2}$$

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Where x means the principal direction of flow and A_l the scoured surface along the x direction. Zanke (1978b) corroborated Mosony's results (cited in Zanke 1982b) for a bidimensional scour case. They showed that the scoured surface is proportional to the second power of the scour depth, $A_l \approx z^2$. Measurements of bridge piers scour by Dargahi (1990) also confirm these results. The relationship used herein is

$$t \approx \frac{z^2}{q_s} \tag{3}$$

The specific solid discharge is then calculated by Zanke's bed-load transport formula (Zanke, 1978a), which was derived based on the fact that horizontal forces acting on soil particles, F_H tend to increment the bed-load when a critical trend is exceeded while vertical forces due to submerged weight , F_V tend to reduce it:

$$q_s \approx \left(\frac{F_H}{F_V}\right)^{\alpha} \tag{4}$$

$$q_s \approx \left(\frac{V^2 - V_{cr}^2}{w^2}\right)^{\alpha} \tag{5}$$

Where V is the velocity of flow, V_{cr} is the sediment entrainment velocity, w is the settling velocity of sediment particles in water and α is a coefficient. For cases in which flow depth exceeds 35 cm, and assuming a typical natural sand porosity of 30%, Zanke (1978a) proposed

$$q_s = 1.4 * 10^{-7} \left(V^2 - V_{cr}^2\right)^2 \left(\frac{D_*}{w}\right)^4 \nu \tag{6}$$

Where ν is the fluid cinematic viscocity, $D_* = \Delta g d_s^3 / \nu^2$ the sedimentologic diameter, $\Delta = (\rho_s - \rho_f)/\rho_f$ the relative density between the solid particles and the fluid, g the gravitational acceleration and d_s the representative sediment diameter.

The turbulence intensity in a scour hole due to the presence of a bridge pier is different to the case of an undisturbed flow. Based on Ettema's experimental data cited in Zanke (1982b), the latter related the flow velocity, V with an effective velocity, V_{ef} in front of the pier as follows

$$V_{ef} = \frac{V\omega}{1 + \frac{z}{b}} \tag{7}$$

Where ω represents an effective velocity increase due to the increment of turbulence caused by secondary flows. The term 1+z/b, where b is the pier diameter, is derived from the mass continuity condition. From eq. (3) follows

$$t \approx \frac{z^2}{q_s} \quad , \ V<V_{cr} \tag{8}$$

$$t \approx \frac{z^2}{q_s - q_0} \quad , \ V>V_{cr} \tag{9}$$

Where q_0 is the sediment specific discharge entering into the scour hole from the upstream bed-load. Following Eqs. (6) – (9), Zanke (1982b) proposed the following equation, valid for $0.4<V/V_{cr}<8.8$

$$t = \frac{1{,}94z^2}{\left(\frac{D_*}{w}\right)^4 \nu \left\{\left[\left(\left(\frac{V\omega}{1+\frac{z}{b}}\right)^2 - V_{cr}^2\right)\frac{V}{V_{cr}}\right]^2 - \left(V^2 - V_{cr}^2\right)^2\right\}} \tag{10}$$

The settling velocity of sediment particles in water, w and the sediment entrainment velocity, V_{cr} are calculated by Zanke's equations (Zanke, 1977b and Zanke, 1977a, respectively):

$$w = \frac{11\nu}{d_s}\left(\sqrt{1+0.01D_*^3} - 1\right) \tag{11}$$

$$V_{cr} = 1{,}4\left(2\sqrt{g\Delta d_s} + 10{,}5\frac{\nu}{d_s}\right) \qquad , h > 0.35m \tag{12}$$

Since the equilibrium depth of scour is reached when time tends to infinity, from Eqs. (8)-(10) it follows that

$$\frac{z}{b} = \left(\frac{V}{V_{cr}}\omega - 1\right) \qquad , V<V_{cr} \tag{13}$$

$$\frac{z}{b} = \left(\frac{\omega}{\left(\frac{V_{cr}}{V} + \frac{V_{cr}^2}{V^2} - \frac{V_{cr}^3}{V^3}\right)^{0.5}} - 1\right) \qquad , V>V_{cr} \tag{14}$$

Because the erosion capacity or scour rate of a given flow is time dependent, i.e.:$\partial z/\partial t \neq$ constant, different scour depths will develop under the action of identical discharges depending upon initial geometrical conditions, $z_{t=0}$. Therefore, the initial depth of scour should be considered in depth scour computations.

The time evolution of the maximum scour depth caused by an unsteady flow can be treated as a succesion of different constant discharges flowing over the bed during given periods. It is neccesary to determine an equivalent initial time at which the new discharges start to erode. This equivalent initial time represents the period that a new discharge needs to erode the actual or initial scour depth. Figure 1 shows a flow diagram for the computation of scour depth under unsteady flow. The equivalent initial time is called t_{dummy}.

To automatise the presented methodology we approximated Ettema's data (Ettema, 1980 cited in Zanke, 1982b) by a polynom of order six with a correlation coefficient, r^2=0.999. This regression is valid in the range of the original data, 0.01< z/b < 2.5.

RESULTS

As mentioned before, we only solved those cases without cohesive soils, i.e. cases 1, 2, 7 and 8 of the prediction event. The predictions are summarized in Table 1. In the cases 1 and 2, we assumed a particle density of 2.65 t/m^3.

In the flume case 1 the scour depth at the end of the first day is estimated to be about 64% of the equilibrium depth scour, which should be about 0.24 m after 3.67 years subjected to the given conditions. In the second case, it is estimated that the resulting scour depth should take only about 2.15 days to develop, when subjected to a constant velocity of to 0.35 m/s, and about 36 days when subjected to a constant velocity of 0.25 m/s. Figure 2 shows the calculated time evolution of the maximum depth of scour for cases 1 and 2.

To calculate the scour depth of both Bridge site cases, we made a spline interpolation of the given hydrograph, in order to obtain hourly discharges. We correlated discharge with velocity and depth data by fitting a potential curve as proposed by Leopold and Madock (1992, pp. 215). The correlation coefficients for the cases 7 and 8 were r^2=0.992 and r^2=0.994 respectively. Because the fitted values varied in a wide range, the regression allowed us the estimation of the hourly velocities without need of extrapolation.

Figure 3 shows the prediction of the maximum scour depth in the bridge site case 7 during the 8.3.93 flood. In the calculations, we assumed that at the beginning of the flood there was no scour depth.
The prediction of the maximum scour depth in the bridge site case 8 during the 5.1.91 flood is showed in figure 4. It is possible that this flood did not cause any erosion, since other important flood events had taken place before. We considered two cases: (a) the bridge presents no scour depth, z =0 during the summer and (b) the bridge presents no scour depth, z =0, at the beginning of the 5.1.91 flood. Quiet a different evolution of the scour depth in time resulted for cases (a) and (b), but the same maximum depth of scour after the 5.1.91 flood. In both situations it was z =1.76m.
To predict the scour depth after the next 50 years, we based on the idea of bankfull discharge as they are supposed to have a 1 to 5 years recurrence interval and they occur most of the time. Mathematically, the probability of excedance of a given discharge is equal to the inverse of its recurrence interval. A discharge with T=5 will be probably exceeded during the 1/5 of the time. For the present case, it means about 10 years.
We first calculated how long a discharge needs to be exceeded so that the equilibrium scour depth is reached. Under the given conditions, bankfull discharges will probably cause the equilibrium scour if they are exceeded during about 4 years, not necessarily uninterrupted. It is expected that under bankfull conditions the velocities range between about 0.85 and 1.1 m/s.
A discharge with the magnitude of the 500-year's one should take about 10 days to cause the equilibrium depth. The given hydrograph shows that floods take usually longer than 10 days to develop their rising and recession stages. In consequence, if within the next 50 years there is at least one 500-year flood we estimate that equilibrium scour depth associated with this extrem discharge will develop, z = 2.52 m.

Table 1- Predictions

Test description	Maximum depth of scour hole (m)
Flume case 1	0.15
Flume case 2	0.17
Bridge site case 7, 8-3-93 flood	10.06
Bridge site case 8, 5-1-91 flood	1.76
Bridge site case 8, 50 years prediction	2.52

DISCUSSION AND FINAL REMARKS

In this paper we estimate the maximum depth of a scour hole around bridge piers in non cohesive soils based on Zanke's attempt (Zanke, 1982a). The equations used herein, are not valid for cohesive soils. Following the results obtained by Ting et al. (2001) it is expected that similar equilibrium scour depth will develop on cohesive and non cohesive soils. Nevertheless, the rate of scouring is much slower in clay than in sand.
Our predictions are valid for single cylindrical piers in a sediment bed. The bridge site cases 7 and 8, include groups of two in line piles mounted on rectangular caissons. Breusers and Raudkivi (1991, pp. 85-86) indicate that, depending on the relative spacing between the piles, the expected scour depth at the front pile increases about 10-20% under the conditions given in the aforementioned cases (relative spacing of about 4). On the other hand, a foundation caisson should provide scour protection. Chabert and Engeldinger (1956) and Shen and Schneider (1970) cited in Breusers et al. (1977) investigated the effect of a circular caisson having a diameter three times the diameter of the pier and a variant of the caisson system in which the caisson is sourrounded by a vertical lip (cut-of sheet-pile) respectively. Their results showed that scours were reduced up to 35-50% of that reached with the pier alone. We estimate that the afore mentioned effects tend to cancel each other under the given conditions and therefore our bridge site case predictions should provide a reasonable estimate of the asked scour depth.
As all known sediment transport predictions are less exact, we assume our results of the time dependent scour elevation to be more insecure in time than in elevation.

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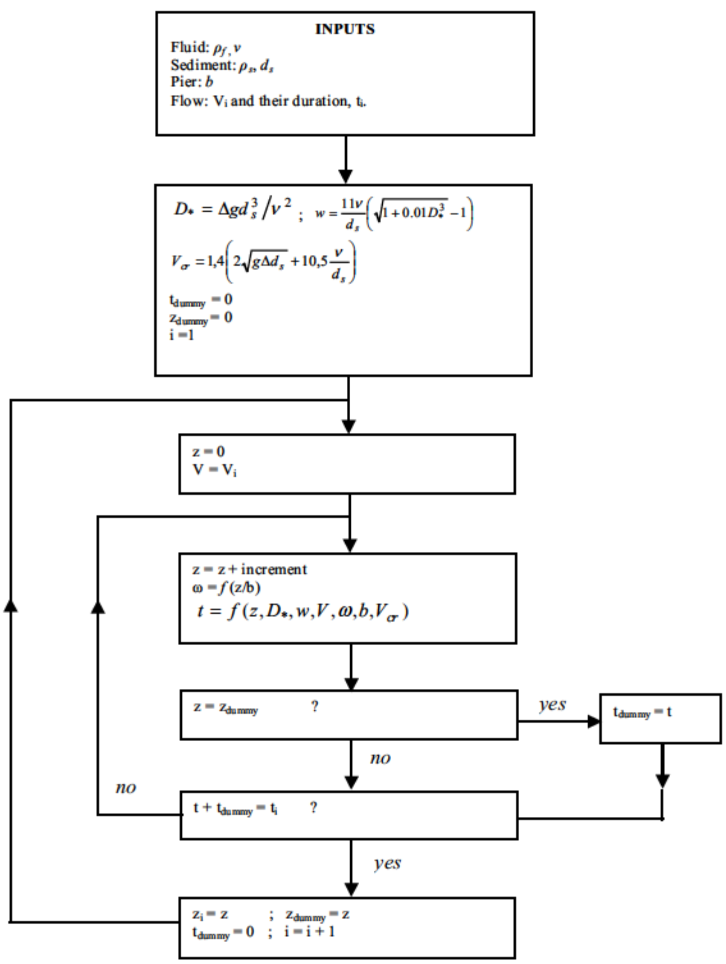

Fig. 1- Flow diagram for the computation of scour depth around a bridge pier under unsteady discharge.

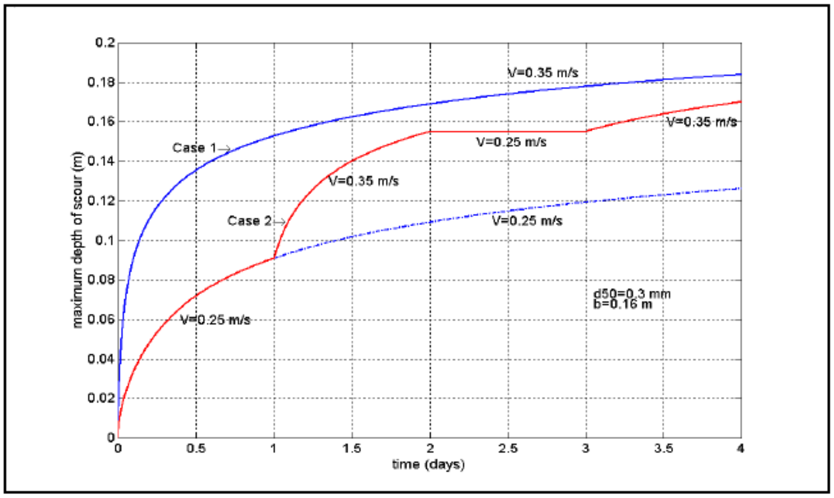

Fig. 2– Time dependence of the maximum depth of scour, for cases 1 and 2.

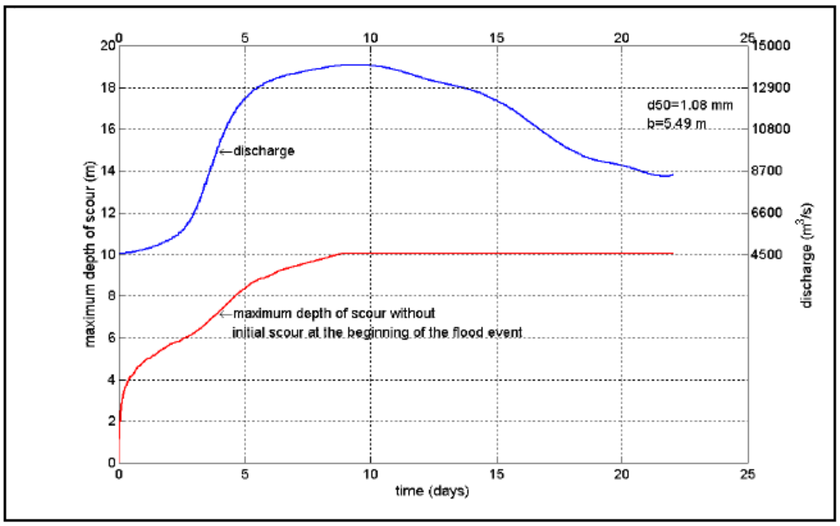

Fig. 3- Solution of the bridge site case 7, assuming no scour at the beginning of the flood event. Additionally, discharge is also plotted.

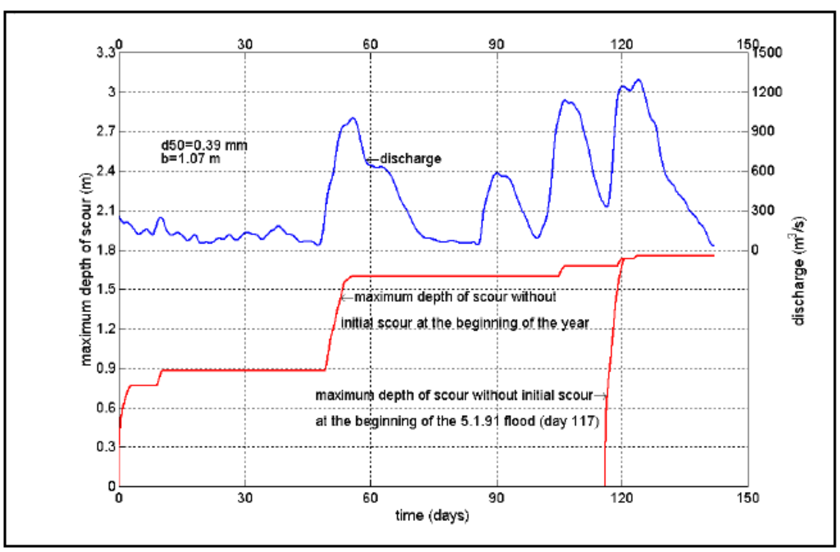

Fig. 4 - Solution of the bridge site case 8 without initial scour at the beginning of the flood event and without initial scour at the beginning of the year. Additionally, discharge is also plotted.