EXPERIMENTAL INVESTIGATION ON LOCAL SCOUR AROUND SPILL-THROUGH SPUR DIKES

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Abstract

An experimental study on local scour around a spill-through spur dike has recently started in a laboratory flume at the National Laboratory of Civil Engineering (LNEC) in Lisbon. The main objectives of this paper are the presentation of the results, the check of existing literature on maximum scour depth and the characterisation of scour holes in their horizontal extend. Two sets of runs for uniform flow regime, corresponding to two different mixtures of river sand, are discussed. Experiments were made under live-bed and with clear-water conditions, for which different equations, obtained by multiple regression analysis, are suggested allowing the prediction of the maximum scour depth. A tentative relation for the width of the scour holes is also presented.

1. Introduction

Spur dikes, obstacles protruding from river banks, are among the hydraulic structures that can be used in environmentally acceptable river training schemes. Thus, the evaluation of their performance and design criteria became again, recently, a topic of interest. Namely, the complexity of local scour mechanism on sand beds around the spur dikes warrants some more work, both analytical and experimental. Hence, an experimental study on this topic has recently started in a laboratory flume at the National Laboratory of Civil Engineering (LNEC) in Lisbon.

The first objective of this paper is the presentation of some results of the study, namely on the influence of the gradation curve on the final scour hole as well as reviewing existing literature on the prediction of its maximum depth. The second objective, motivated by prediction needs of areas or even volumes, is the characterisation of the lateral extension of the scour hole.

2. Experimental apparatus and procedure

The experiments were carried out in a flume which is 18 m long, (the test reach being 10 m long), 2 m wide and 0.6 m deep. The inlet system includes a constant head tank leading to a Bazin spillway used to measure the discharge. The maximum available discharge is 0.16 m^3/s. The experiments were run with movable bed. The flume is prepared for the study of localised river problems since a deeper bottom exists in the central reach, allowing an additional sand thickness of 0.4 m. The initial bed slope is adjustable to a prefixed value for each run, with the help of a carriage system. A sediment feeder exists at the upstream section of the flume that can be operated at constant adjustable rates. At the downstream end, a tailgate allows the regulation of the flow depth in the flume.

The measuring equipment includes an electronic limnimeter and a bed follower, installed in the carriage system. The water elevations can also be measured by two point gauges installed in side wells close to the upstream and the downstream end sections. The limnimeter is fixed in the carriage, at the central longitudinal axis of the flume. The bed follower is allowed to travel in the transverse direction at almost all the flume width, recording the cross sectional deformations of the bed. These two electronic instruments are integrated in an automatic data acquisition system.

The spur dike used in these experiments had a spill-through shape and was placed, perpendicular to the flow, 5 m downstream of the entrance of the flume. It is represented in Fig. 1. Its main dimensions, in the horizontal plan at which the sand bed is levelled, are 0.36 m along the flow direction and 0.37 m perpendicular to the flow direction.

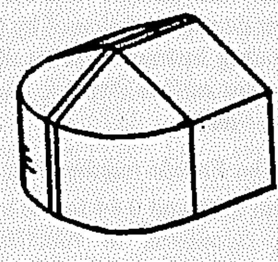

Fig. 1 - Spur dike

According to the experimental procedure followed in this study, the discharge, Q, was set constant for each run; a tentative water depth, h_0, was also set. Searching the uniform flow regime, the corresponding values of the bed slope, i_b, and the sediment discharge, Q_s, were estimated on the basis of a resistance relation (Brownlie 1983) and a sediment transport formula (Karim and Kennedy 1990). Prior to each run, the bed was levelled with the calculated slope. The tailgate was regulated such that an approximate value of selected h_0 was reached after the flow discharge being imposed. Finally, the flow was kept free to adjust the bed slope and the water depth. The flow response was monitored and attempts were made to reach the uniform equilibrium, acting in the rate of sediment feeding and in the water depth. The bottom profile was estimated by means of the regression line that can be adjusted to the points obtained for the cross section bed measurements. The evolution of the scour hole in time was monitored by the bed follower probe. The test was stopped when the evolution of the scour hole was negligible or oscillating around an equilibrium value. After the flow stopped, 29 cross section profiles were measured, in the zone of the local scour, in order to draw its final topography.

3. Results and discussion

Two series of runs, corresponding to two different mixtures of river sand, were carried out: series A1, with median grain size, D_{50}= 0.37 mm and gradation coefficient, σ_g= 1.58; series A2, with D_{50}= 0.96 mm and σ_g= 1.29.

The most relevant variables measured for each run are presented in Table 1, for the final (equilibrium) stage, where the bed slope, i_b, and the water surface slope, i_s, were accepted as similar. The table includes, for the undisturbed flow (free of disturbance induced by the spur dike), the values of the imposed discharge, Q, the corresponding hydraulic radius, R, mean velocity, U_0, the values of the bed slope, i_b, water surface slope, i_s, and sediment rate, Q_s (which was not measured in three runs), followed by the values of the water depth, h_0.

The Froude number was evaluated as $Fr = U_0/\sqrt{gR}$; it covers values between 0.23 and 0.64 in the lower flow regime. The slope of the energy line, J, was calculated in function of the bed and water surface slopes as $J = Fr^2 . i_b + (1 - Fr^2) i_s$. From Table 1, it can be seen that, with a few exceptions, J and i_b are reasonably similar, which means that uniform flow was satisfactorily achieved. Reynolds number, $Re = 4RU_0/\nu$, varies between 0.35×10^5 and 1.87×10^5 showing that the flow is turbulent. For the calculation of Re, ν was evaluated in function of the water temperature which varied between 10.6 °C and 20.5 °C during the study. The relative submergence, $Z = h_0/D_{50}$, varies between about 37 and 336.

Table 1

Run	h_0 (m)	Q $\times 10^3$ (m^3/s)	R (m)	U_0 (m/s)	i_b $\times 10^3$	i_s $\times 10^3$	Q_s $\times 10^6$ (m^3/s)	Fr	J $\times 10^3$	Re $\times 10^{-3}$	Z	U_0/U_c
A1.01	0.040	18.4	0.039	0.23	1.12	2.64	0.500	0.37	2.43	35	109	0.99
A1.02	0.043	24.0	0.041	0.28	2.22	2.23	2.400	0.45	2.23	47	115	1.22
A1.03	0.049	29.6	0.047	0.30	2.30	2.60	5.306	0.45	2.54	51	132	1.28
A1.04	0.056	27.6	0.053	0.25	2.18	2.20	0.900	0.34	2.20	52	151	1.03
A1.05	0.065	36.0	0.061	0.28	1.51	2.03	2.500	0.36	1.96	68	176	1.13
A1.06	0.070	44.4	0.065	0.32	2.50	2.10	-	0.40	2.16	80	189	1.28
A1.07	0.085	41.4	0.079	0.24	0.77	1.47	0.439	0.28	1.42	73	231	0.96
A1.08	0.092	54.0	0.084	0.29	1.13	1.88	2.906	0.32	1.80	100	249	1.14
A1.09	0.100	66.6	0.091	0.33	2.00	1.86	-	0.35	1.88	118	270	1.29
A1.10	0.112	55.2	0.101	0.25	1.53	1.29	-	0.25	1.30	93	303	0.94
A1.11	0.112	72.0	0.101	0.32	1.22	1.20	4.117	0.32	1.20	129	303	1.22
A1.12	0.124	88.8	0.111	0.36	1.40	1.57	7.069	0.34	1.55	154	336	1.34
A2.01	0.036	18.4	0.035	0.26	1.80	0.78	0	0.44	0.98	35	37	0.80
A2.02	0.040	24.0	0.039	0.30	1.91	1.40	0.750	0.48	1.52	46	42	0.91
A2.03	0.046	29.6	0.044	0.33	2.00	1.40	1.278	0.50	1.55	56	47	0.98
A2.13	0.043	35.2	0.042	0.41	2.24	2.80	6.722	0.64	2.57	67	45	1.24
A2.04	0.059	27.6	0.056	0.23	0.95	0.59	0	0.32	0.63	51	61	0.68
A2.05	0.057	36.0	0.054	0.32	1.08	1.08	0	0.43	1.08	67	59	0.93
A2.06	0.063	44.4	0.060	0.35	1.73	1.17	0.889	0.46	1.29	83	66	1.01
A2.14	0.069	52.8	0.064	0.38	2.30	2.14	3.333	0.48	2.18	98	72	1.10
A2.07	0.090	41.4	0.083	0.23	0.61	0.61	0	0.26	0.61	75	94	0.63
A2.08	0.093	54.0	0.085	0.29	1.67	1.07	0.333	0.32	1.13	98	97	0.80
A2.09	0.088	66.6	0.081	0.38	1.18	0.90	1.000	0.42	0.95	121	92	1.04
A2.15	0.089	79.2	0.081	0.45	1.78	1.50	6.056	0.50	1.55	144	92	1.23
A2.10	0.119	55.2	0.106	0.23	0.61	0.61	0	0.23	0.61	97	124	0.61
A2.11	0.120	72.0	0.107	0.30	1.10	0.33	0	0.29	0.40	127	125	0.80
A2.12	0.115	88.8	0.103	0.39	1.15	1.15	1.111	0.38	1.15	158	120	1.03
A2.16	0.114	105.6	0.102	0.46	1.90	1.90	6.056	0.46	1.90	187	119	1.23

In the last column of Table 1, the flow intensity parameter U_0/U_c is included, where the subscript "c" refers to the initiation of sediment motion.

The critical velocity was defined for each run, as a function of h_0 and the sediment characteristics (D_{50} and γ_s), following the formulations of Goncharov, Neill, and Garde, as presented in Garde and Raju 1985, p. 58.

The values of the most pertinent variables characterising the scour hole, namely its maximum width, L_s, its area, A_s, the excavated volume, V_s, and the maximum scour depth, h_s, are given in Tables 2 and 3 for series A1 and A2. The duration of each run, t_f, and the obstacle (spur) length, L, are also given. The referred geometric parameters were obtained from the representation of the scour topography by contour lines drawn at 1 cm intervals. The obstacle length was measured perpendicularly to the flow direction in the horizontal plan located at half depth between the initial bed and the water surface, as defined by some authors, e.g. Melville 1992.

Table 2

Run	t_f (h)	L (m)	L_s (m)	A_s (m^2)	V_s $\times 10^3$ (m^3)	h_s (m)
A1.01	14.00	0.350	0.23	0.79	0.76	0.085
A1.02	14.00	0.349	0.38	0.85	1.31	0.121
A1.03	12.50	0.346	0.20	1.00	1.52	0.139
A1.04	26.00	0.342	0.38	1.17	1.88	0.130
A1.05	3.00	0.337	0.33	1.34	2.96	0.155
A1.06	2.33	0.335	0.35	1.08	2.37	0.144
A1.07	19.00	0.327	0.33	0.75	2.33	0.158
A1.08	5.83	0.324	0.38	1.27	3.73	0.176
A1.09	2.00	0.320	0.33	1.02	3.21	0.166
A1.10	21.00	0.314	0.38	0.93	3.55	0.182
A1.11	5.50	0.314	0.38	1.29	4.69	0.191
A1.12	5.00	0.308	0.41	1.06	4.71	0.214

Table 3

Run	t_f (h)	L (m)	L_s (m)	A_s (m^2)	V_s $\times 10^3$ (m^3)	h_s (m)
A2.01	19.00	0.352	0.16	0.13	0.25	0.072
A2.02	5.67	0.350	0.33	0.33	1.12	0.145
A2.03	4.75	0.347	0.30	0.43	1.47	0.155
A2.13	6.58	0.348	0.33	1.85	1.08	0.156
A2.04	41.00	0.341	0.20	0.23	0.71	0.109
A2.05	56.00	0.342	0.36	1.23	5.42	0.229
A2.06	3.83	0.338	0.36	0.68	2.66	0.170
A2.14	3.50	0.336	0.33	1.52	2.80	0.165
A2.07	40.50	0.325	0.30	1.02	4.16	0.140
A2.08	26.33	0.324	0.43	1.33	6.84	0.243
A2.09	5.00	0.326	0.38	1.25	4.99	0.203
A2.15	7.25	0.326	0.40	1.63	5.78	0.228
A2.10	34.25	0.310	0.32	0.40	1.90	0.153
A2.11	17.50	0.310	0.43	1.03	5.83	0.213
A2.12	11.08	0.313	0.48	1.33	7.77	0.237
A2.16	6.83	0.313	0.46	1.22	7.71	0.246

The time duration varies between 2 and 56 hours. The tests that last longer are thoroughly the runs in which the bed movement was non-existent or incipient.

The maximum scour depth, h_s, reduced by the undisturbed flow depth, h_0, is presented in Fig. 2 in function of L/h_0. In this figure, the data were separated in three groups. The runs with $U_0/U_c \leq 0.9$ (neat clear-water) were included in the first graph; the runs with $0.9 \leq U_0/U_c \leq 1.15$ were chosen for the representation at the threshold condition and included in the second graph; the runs with the values of $U_0/U_c \geq 1.15$ (neat live-bed) were plotted in the third graph. The scour depth values were divided by the shape coefficient K_s in order to remove the shape effect and make the results comparable with those obtained for a vertical plate obstacle, as suggested by Melville 1992. A value of $K_s = 0.7$ was chosen in such a way that the envelope curve obtained by Melville 1992 for threshold conditions also serves as an envelope curve for the present threshold data ($0.9 \leq U_0/U_c \leq 1.15$). This value of K_s is between those recommended for semi-circular end ($K_s = 0.75$) and spill-through abutments with slope 1:1 ($K_s = 0.5$). $K_s = 0.7$ is close to the value of the semi-circular end spur, because, in this study, the spill-through spur presents a vertical wall bellow the initial bottom that shows up as scouring proceeds.

Recent literature states that the threshold condition leads to the largest scour depths and that the clear-water scour depths are considerably lower. This result can be confirmed, in some way, from the first plot in Fig. 2, where lower scour depths are clearly observed.

Authors like Breusers and Raudkivi 1991, p.76, or Melville 1992 state that, especially for bridge piers and under live-bed conditions, scour depths tend to decrease for $U_0/U_c > 1$ and then increase again for $U_0/U_c \geq 3.5$, to about the threshold value. From Fig. 2, it can be concluded that in the experimental range of this study ($0.61 \leq U_0/U_c \leq 1.34$), the scour depth under neat live-bed conditions, $1.15 \leq U_0/U_c \leq 1.34$, is statistical indistinguishable from the scour depth for threshold conditions. This conclusion does not contradict the mentioned literature, since the upper limit of the flow intensity reached in the present study is comparatively low.

A slight influence of D_{50} on the maximum scour depth seems to exist in the present study, as the results corresponding to series A1, in which a finer sediment was used, show a trend to be below the results corresponding to series A2 (cf. Fig. 2). This result is in agreement with Kandasamy 1989, p. 228, who mentioned that preliminary experiments indicate scour depth at abutments to be larger for coarser uniform sediment.

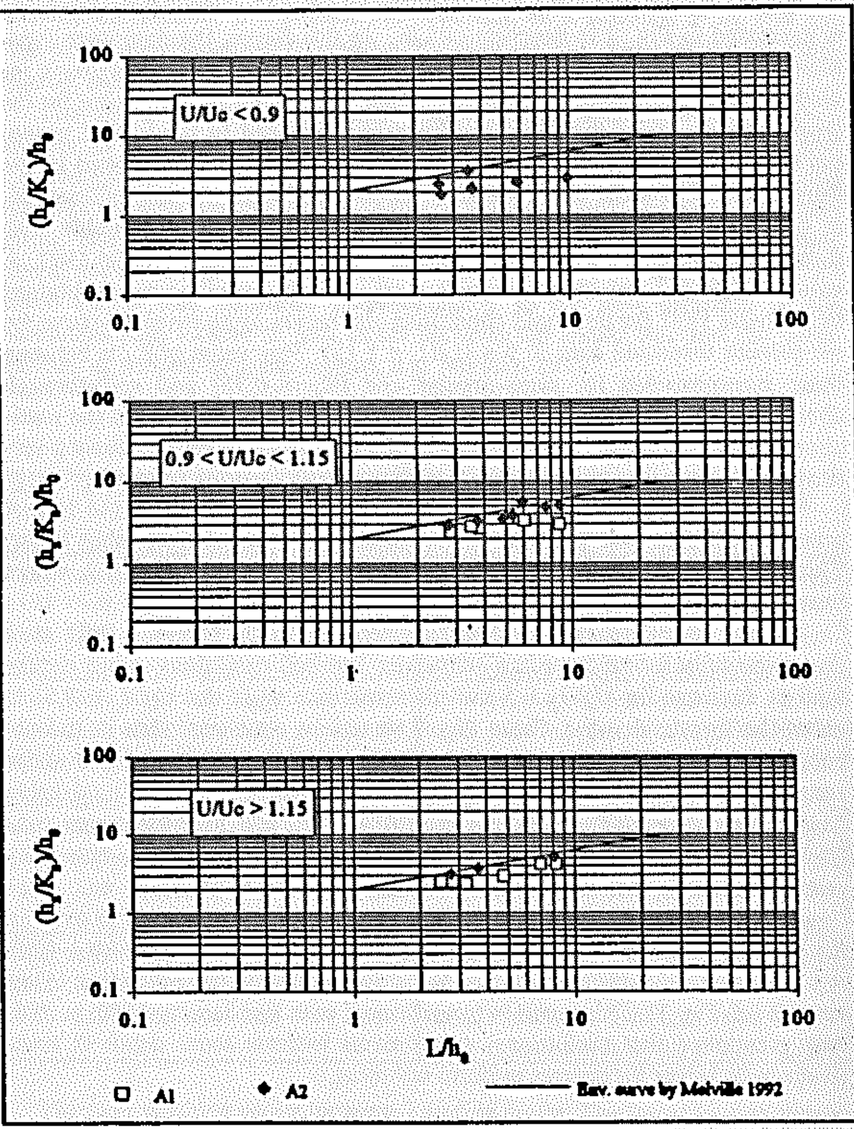

Fig. 2 - Maximum scour depth

It might also be argued that the mentioned trend could be explained by a slight armouring effect for series A1, because the gradation coefficient of this sand ($\sigma_g \approx 1.6$) is higher than the one of series A2 ($\sigma_g \approx 1.3$). However, this argument is not supported by Breusers and Raudkivi 1991, p. 78, stating that the effect of sediment grading appears to be negligible for $\sigma_g < 2$.

In the following discussion, the data corresponding to threshold conditions were split into the other two groups, i.e. clear-water and live-bed conditions, for sakes of comparison with relevant literature. In series A2, the set of clear-water runs correspond to those with $Q_s = 0$ and run A2.02, with plan bed and incipient motion; in the remaining runs of series A2 (classified as live-bed), sediment transport with the formation of dunes was observed. In series A1, although ripple formation was observed in all runs, four of them were classified as clear-water, since U_0/U_c was below 1.05; the remaining runs were included in the live-bed set. The adopted distinction criterion can be related with tf: it can be seen that the live bed tests lasted for less than 14 hours while clear-water tests lasted for more than 14 hours.

Results on the maximum scour depth, h_s, were analysed by regression techniques. One of the best regression curves for live-bed data (regression coefficient r = 0.91) was the following:

$$\frac{h_s}{h_0} = 2.25\left(\frac{L}{h_0}\right)^{0.33} Fr^{0.49} \tag{1}$$

which is quite similar to the equation of Liu et al. (cf. Chang 1988, p. 101) valid for live-bed conditions:

$$\frac{h_s}{h_0} = k'\left(\frac{L_1}{h_0}\right)^{0.4} Fr^{0.33} \tag{2}$$

Despite of the different definitions of L and L_1 (obstacle length measured perpendicular to the flow at the water surface level), predictions of eqs. (1) and (2) practically collapse to each other for the experimental range of this study, as soon as k' is taken as k' = 1.9. It should be noted that the power coefficients of both equations are of the same order of magnitude and that k' = 2.15 was suggested by Liu et al. for vertical walls. The adjusted value, being closer to 2.15 than to 1.1 (suggested for gentle side slope) is a new evidence that the used spur behaves mostly as a semi-circular end spur.

Proceeding the data analysis, the following equations were obtained by multiple regression:

$$\frac{h_s}{h_0} = 38.3\left(\frac{L}{h_0}\right)^{-0.56} Z^{0.23}\, Fr^{2.76} \qquad \text{for clear-water} \qquad (3)$$

$$\frac{h_s}{h_0} = 4.30\left(\frac{L}{h_0}\right)^{0.29} Z^{-0.21} \qquad \text{for live-bed} \qquad (4)$$

The parameters in the right hand side of eq. (3) reflect the influence of the main variables intervening in the phenomena: the spur length, the sand diameter ($Z = h_0/D_{50}$), the undisturbed flow velocity and depth, and gravity. All lengths are reduced by h_0; the gravity effect is considered by means of Froude number. This parameter does not appear in eq. (4) (where $r = 0.96$), for live-bed, since its influence was seen to be negligible, while it plays an important role for clear-water (see eq. (3) with $r = 0.93$). Higher power coefficients for the Froude number in equations for clear-water than in similar equations for live-bed can be found in several authors formulae (cf. comparison in rewritten formulae in Wong 1982). Liu et al., e.g., arises this power from 0.33 in live-bed equation to 1 in clear-water equation, when studying vertical wall abutments.

Fig. 3 shows the results of the maximum scour width, L_s, under live-bed conditions, written in dimensionless form. Although these are definitely preliminary results, there seems to be a clear trend for L_s to increase with h_0/L.

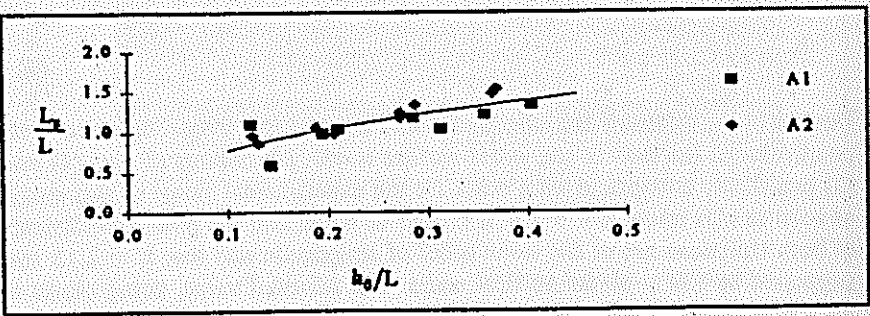

Fig. 3 - Maximum scour width

Similar plots were tried for A_s and V_s. Such plots have shown a considerable scatter and some additional effort on data analysis is needed.

4. Conclusions

In this study, the conformity with the envelope curve obtained by Melville 1992 for bridge abutments was illustrated for threshold and live-bed conditions, and the reduction of scour depths for clear water was identified. A slight influence of the gradation curve was also shown. The results of Liu et al. were confirmed to hold for live-bed; two alternative equations, eqs. (3) and (4), were obtained for the present clear-water and live-bed data. A new relation for the maximum width of the scour hole (Fig. 3) was tentatively established.

Acknowledgments

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