THE CALCULUS OF BACKWATER CURVE ON THE MAN-MADE CHANNEL

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In lucrare s-a efectuat studiul curbei de remuu ținând seamă de deversarea laterală [1]. Au fost impuse de condițiile de deversare laterală pentru o gamă de debite afluente în secțiunea amonte, corespunzătoare unor debite de evacuare impuse în secțiunea aval [2],[3]. S-au dedus relațiile de legătură între aceste debite, calculând debitul lateral total corespunzător. Au fost considerate diferite variante pentru înălțimea pragului deversor h_D. Nomograma rezultată din corelația debitului defluent cu cel afluent ne permite să estimăm atenuarea unui hidrograf de viitură.

This paper is devoted to the study of backwater slope under the condition of a lateral overflow [1]. The boundary conditions for lateral overflow have been imposed in the upstream for a wide range of input flow discharge corresponding to outflow discharge required in the downstream section [2],[3]. The correlations link between the above mentioned flows have been derived computing also the total discharge through the weir. Several options for the weir height h_D of the spillway have been taken into account. The resulting chart outflow versus input flow (for different values of h_D) allows us to estimate the mitigation of flood hydrograph.

Key words: backwater, outflow, mitigation

1. Introduction

Man-made embankment assumes the focusing of flow even for flooding situations in low-flow channel. For bed to support very high flood wave, the aggradations of dams is necessary. They are designed to retain the excess stock of water during transit. To avoid exaggeratedly elevated dykes, which in, most of the times are non operative, a more economic option is to supply along the river course foot plains or polder as they are known in the literature [4]. In case of high risk floods they are capable to take over the water stock in excess. Under normal circumstances they may be used for agricultural purposes.

The polder is usually designed to store the volume of water in excess coming from the increasing branch of the hydrograph. In this way the mitigation of the flood is efficient.

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The length of lateral overflow zone and the threshold weir are strongly connected with longitudinal channel position. Also the value of roughness in cross section and along the river bed is of importance.

In this paper we analyze the role played by these parameters for the flood discharge in a man-made river bed sector, equipped with lateral weir. We also obtain a correlation between upstream and downstream discharges, for different values of the threshold weir [5].

The lateral unit discharge q [m^3/s] is null along the channel, an exception being the situation with lateral overflow in polder. Using the particular formula for rectangular weir, we obtain:

$$q = k_0 \cdot m\sqrt{2 \cdot g} \cdot l_D^{\,3/2} \tag{1}$$

where k_0 [8] is an empirical coefficient $(k_0 = 1.5\ldots1.75)$ concerning the effect of streamline contraction, m is a discharge coefficient for the weir, $l_D = h_j - h_D$ [m] is the height overflowing nape (the height of free surface over threshold weir in the analyzed cross section), h_j [m] is the level of free surface and h_D [m] the level of threshold weir.

The mathematical model is founded on the fundamental equation of gradually variable flow, which, in finite differences can be expressed as:

$$-\frac{h_{n+1} - h_n}{dx} = (i - \frac{Q^2}{K^2} + \frac{\alpha \cdot Q}{g \cdot A^2} \cdot k_0 \cdot m \cdot \sqrt{2 \cdot g})/(1 - Fr) \cdot l_D^{\,3/2} \tag{2}$$

where $K = A \cdot C \cdot \sqrt{R}$ is discharge module, $Fr = \alpha \cdot Q^2 \cdot B / g \cdot A^3$ is the Froude number. $C = 1/n \cdot R^{1/6}$ represents the Chézy coefficient and B [m] is the width at free surface. The rectangular area is $A = B \cdot h$ [m^2]. The wetted perimeter is $P = B + 2 \cdot h$ [m], and hydraulic radius is $R = A/P$ [m]. Other characteristic quantities are: Q – discharge in cross section [m^3/s], dx - sector of river bed [m], i – slope (‰), α – Coriolis coefficient, g – gravitational acceleration [m/s^2].

If the level in the channel is smaller than the threshold weir, the calculus of backwater curve is performed starting from downstream data propagated toward the upstream. If the discharge increases step by step until we reach the lateral overflow region, the discharge upstream (Q_{up}) [m^3/s] is maximum and is almost constant until the overflow region is attainted. The computation of backwater curve is performed starting with downstream data, it arrives in the lateral overflow region where the curve has a decreasing slope along direction of flow and continues toward upstream.

The boundary conditions imposed downstream correspond to a junction condition with a high depth storage reservoir where the level is imposed and known [6]. The upstream condition is the flood hydrograph $Q(t)$ [m^3/s].

2. Computational algorithm

For a sector of river bed of arbitrary shape, of length dx [m], the Bernoulli relation is [6], [7]:

$$dH = dh_v + dh_l + dh_f \tag{3}$$

where: dH [m] is the variation of level between two cross sections, dh_v [m] is the change in kinetic energy, dh_l [m] loss of local charge and dh_f [m] is loss of longitudinal charge. The loss of local charge is due to the variation of cross section along the river with expression:

$$dh_l = \varsigma d\left(\alpha \frac{v^2}{2 \cdot g}\right)$$

where ς is the coefficient taking into account the local charge losses and v [m/s] is the current velocity of the fluid.

The loss of longitudinal charge has the expression:

$$dh_f = S_f \cdot dx$$

where S_f is the longitudinal slope on the sector dx, expressed according to Chézy equality:

$$S_f = \frac{v^2}{C^2 \cdot R} = \frac{v^2 \cdot A^2}{A^2 \cdot C^2 \cdot R} = \frac{Q^2}{K^2}$$

The change in kinetic energy is:

$$dh_v = d\left(\alpha \frac{v^2}{2 \cdot g}\right)$$

With the above notations, equality (3) becomes:

$$dH = (1+\varsigma) d\left(\alpha \frac{v^2}{2 \cdot g}\right) + S_f \cdot dx$$

Transposing this equation by means of finite differences for a sector of length dx, bounded by two cross sections upstream and downstream, it result the level upstream H_{up} (m):

$$H_{up} = H_{down} + S_{fm} \cdot dx + (1+\varsigma_m) \frac{\alpha}{2 \cdot g}\left(v_{down}^2 - v_{up}^2\right) \tag{4}$$

where

$$S_{fm} = \frac{1}{2}\left(S_{f\ down} - S_{f\ up}\right)$$

represents the mean slope for the sector, evaluated like an arithmetic mean between two slopes, upstream and downstream.

Depending on the velocity downstream v_{down} [m/s] and upstream v_{up} [m/s], the sector is:
- Convergent along the river ($v_{down}>v_{up}$), then the loss of local charge is negligible, $\varsigma_m=0$;
- Divergent along the river ($v_{down}<v_{up}$) the loss of local charge is no longer negligible, $\varsigma_m=-1$.

The algorithm is conducted according to the following steps [8]:

Step 1 (predictor): As a first approximation, the level of free surface in the neighboring cross section is expected to be

$$H_{down}^{(1)} = H_{\text{up}} + S_{f\,\text{up}} \cdot dx$$

Step 2 (correction): Using the equation of continuity, the speed and the hydraulic slope are inserted in relation (4):

$$H_{down}^{(2)} = H_{\text{up}} + \frac{1}{2}\left(S_{f\,up} + S_{f\,down}^{(1)}\right)dx + \left(1+\varsigma_m\right)\frac{\alpha}{2g}\left(V_{up}^2 - V_{down}^2\right)$$

Step 3 (test):

If $\left|H_{down}^{(2)} - H_{up}^{(1)}\right| < \varepsilon \qquad \left(\varepsilon = 0.01\ldots0.001\,[m]\right)$

the iterative process is considered completed and one moves to the next section.

If this inequality is not satisfied, one chooses

$$H_{down}^{(3)} = \beta \cdot H_{down}^{(1)} + \left(1-\beta\right)\cdot H_{down}^{(2)}$$

where a coefficient β (called the relaxing coefficient) with values in the range (0.6 ... 0.9) is introduced and then calculations from step 2 are performed.

3. Numerical results

The paper follows the evolution of backwater curve for a real case in a natural riverbed, upstream of hydropower Turnu, located on the Olt river. The flow on this section is under natural conditions, the river bed being with variable geometry along the flow. We also assume that the boundary conditions downstream (storage reservoir) is: $Fr<1$, discharge and the level at the junction with the storage reservoir have the same values as in natural flow case [9], [10].

In order to highlight the three situations analytically described above, we use a program written in Matlab 7.0.0. We consider the following particular cases.

Case 1: the natural river flow

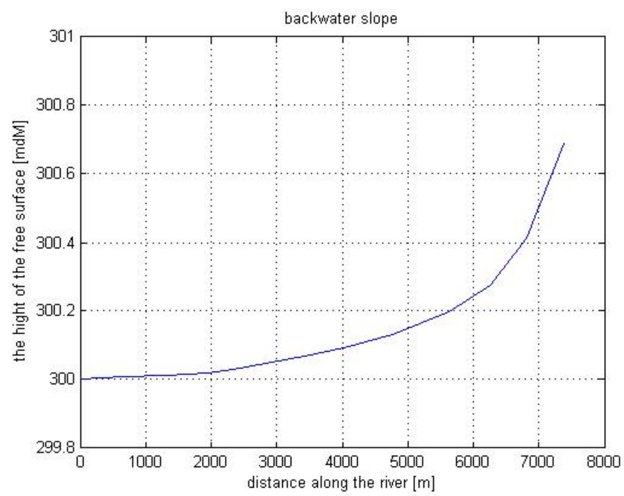

Fig. 1 The backwater on the natural flow

Case 2: flow in man-made channel

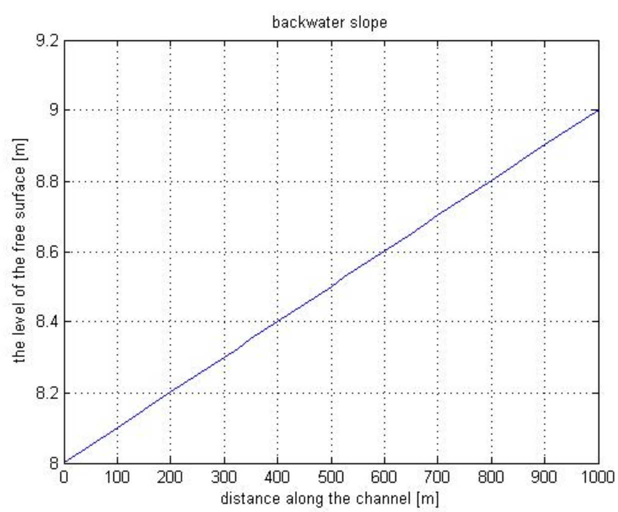

Fig. 2 The backwater for flow channel

Case 3: The mitigation flood for channel equipped with lateral weir

The velocity and the shape of backwater curve in the man-made cannel are shown in Fig. 3 and Fig. 4.

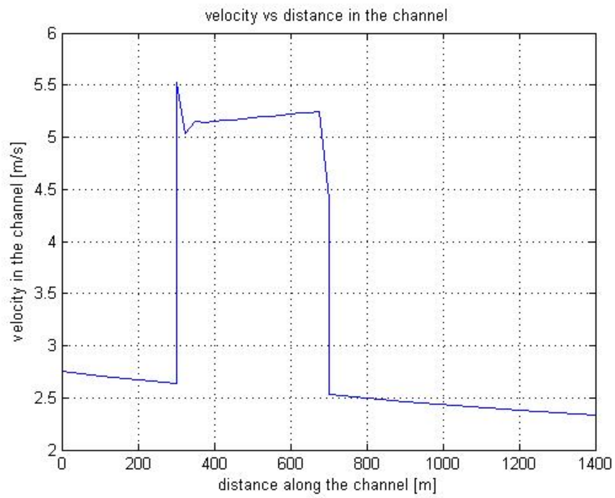

Fig. 3 The variation of velocity in the channel with lateral weir

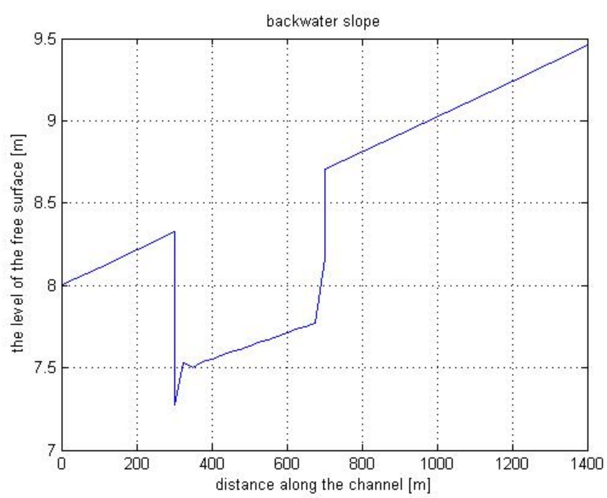

Fig. 4 The backwater in the channel with lateral weir

For several values of the weir threshold h_D=3-5-7 m, we computed the correlation $Q_{down} - Q_{up}$ (nomogram). The graphs (Fig. 5) display the fact that Q_{down} curves tend to flatten (the effect of lateral overflow).

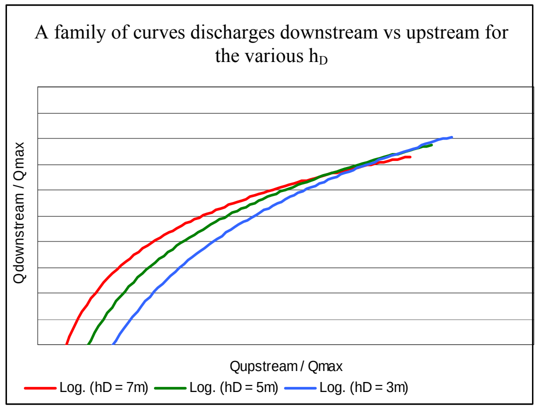

Fig. 5 The family curves (nomogram)

In case we do a wave flood hydrograph *Q(t)* (upstream), one can determine the attenuated hydrograph by means of the nomogram represented in Fig. 5. Every value of flood hydrograph upstream is mapped into a single value of the downstream hydrograph at a given value of the weir threshold (h_D). The numerical calculation has been done for a flood with increasing time of 12 hours and total duration of 36 hours (Fig. 6).

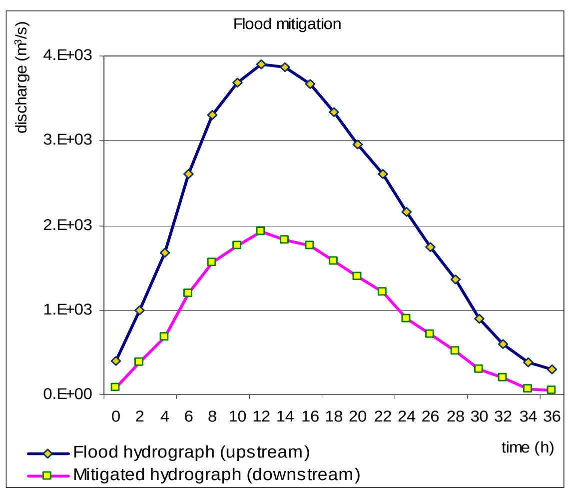

Fig. 6 The mitigation wave flood

Also, the stock of water in excess, leaked in the polder for the entire period of flood was computed for different values of weir threshold h_D. The dependence is shown graphically in Fig. 5. It is noteworthy that the stock corresponding to maximum rate threshold overflow leaked h_D = 4.5 m and a further decrease in these values has the effect of raising the stock leaked inside the polder (Fig. 7).

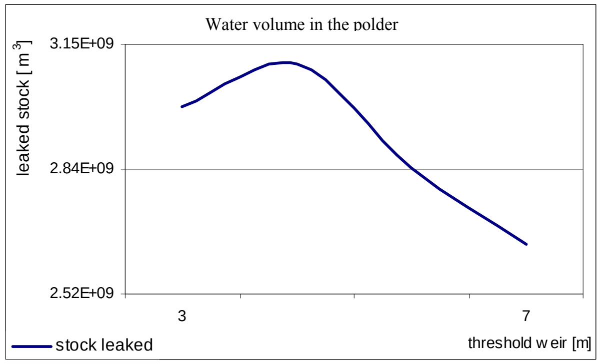

Fig. 7 The volume variation in the polder at different values h_D

4. Conclusions

One can also determine when the maximum mitigation flow occurs. The situation depends on the state of the polder whether it is filled up or not: i) if the polder is not filled, the maximum flow occurs at the same time as the mitigation maximum flow; ii) if polder is filled up, the mitigation maximum flow occurs with a time delay.

R E F E R E N C E S

[1] *I. Seteanu, Victoriţa Rădulescu and Diana Broboană,* Basic hydraulics, Ed. BREN, Bucureşti, 2000, ISBN 973-9493-53-X

[2] *M. Buyer, J. Vazquez and B. Bremond*, Numerical modelling of the hydraulic behaviour of lateral storm weirs in a transcritical regime, Rev.Sci.Eau 18/1(2005) 25/46

[3] *S. R. Chaurasia*, Direct Equations for Hydraulic Jump Elements in Rectangular Horizontal Channel Journal of Irrigation and Drainage Engineering, **vol. 129**, no. 4, July/August 2003, pp. 291-294

[4] *I. Seteanu and Diana Broboană,* Numerical models in hydraulics and power engineering, 146 pag., Ed. BREN, Bucureşti, 2000, ISBN 973-99604-4-8

[5] *A. Rezgui, Ben Maiz, N. and Moussa,* Fonctionnement hydrodynamique et écologique du Lac Nord de Tunis par modélisation numérique. **Rev. Sci. Eau** 21 (3) 2008: 349-361.

[6] *Ben Chie Yen*, Open channel flow resistance, Journal of Hydraulic Engineering, vol. 128, no 1, January 2002, pp. 20

[7] *A. Abdulrahman*, Best Hydraulic Section of a Composite Channel, Journal of Hydraulic Engineering, **vol. 133**, no. 6, June 2007, pp. 695-697

[8] *I. Seteanu, Victoriţa Rădulescu, N. Vasiliu and Diana Vasiliu,* Fluid Mecanics and Hydraulic Systems; Fundamentals and applications, Ed. The. Aplicată, 1999, ISBN 973-31-1316-6

[9] *C. C. Hsu, C. Tang, W. Lee and M. Shieh*, Subcritical 90° Equal-Width Open-Channel Dividing Flow, Journal of Hydraulic Engineering, **vol. 128**, no. 7, July 2002, pp. 716-720

[10] *M. Igouzal and A. Maslouhi,* Contribution to the management of a reservoir dam an the Sebou river (Morocco) using a hydraulic model, Rev.Sci.Eau 16/3(2003) 443-458