Impacts of Urban Growth on Surface Water and Groundwater Quality (Proceedings of IUGG 99 Symposium HS5, Birmingham, July 1999). IAHS Publ. no. 259, 1999.

A model of the flood transformation in the Amudarya River

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Abstract On the basis of modelling the flood transformation for a river with a deformed channel, a technique has been developed for short-range forecasting of the Amydarya River discharge. The model takes into account the relationship between the lag time of the flood peak and the water discharge as well as the deformational instability of the river channel. A description of the model and techniques of flow adjustment with a strongly deformed river channel and dynamical changes of the hydraulic bonds are presented.

INTRODUCTION

The main water sources flowing into the Aral Sea are the Amudarya and Syrdarya rivers, the water of which is extensively used in the economy of the Central Asian states. The main and urgent problem is the fall of water level in the Aral sea of more than 20 m which has taken place during the last 10 years. A five day flow forecast of the Amydarya River is now required for the rational use and effective management of the water resources for routine operational purposes.

Modelling results on the flood transformation of the Amudarya River to date do not have the required accuracy to be applied for hydrological forecasting due to very complicated hydrological processes. For the purposes of hydrological forecasting, a "reservoir model" of river flow with a deformed channel has been used, based on a non-linear relationship between water discharge and hydraulic parameters of the channel.

THE STUDY AREA

The Amudarya River is considered to be one of the biggest rivers in the world. Its length is 2600 km with a watershed area of 465 000 km^2; average annual water discharge is more than 1500 $m^2 s^{-1}$. The river flows through the territories of Tadjikistan, Turkmenistan, Afghanistan and Uzbekistan and is an extremely complex natural feature with many specific features.

The water regime of the Amudarya River is characterized by significant variations in the intra-annual flow distribution. The peak of water discharge recorded at the Kerki gauge line is 9060 $m^2 s^{-1}$ (20 July 1958). However, the maximum values of discharge are mainly not more than 8000 $m^2 s^{-1}$ and 65% of the time the maximum values are within 5000–7000 $m^2 s^{-1}$. Maximum water losses in the river channel and flood bed occur in April–June, and especially in May and June when intensive increases in discharge values and rise in water horizons are observed. The water losses during this period, apart from the losses by evaporation and groundwater augmentation are determined by the filling-in of the channel and flood bed together with soil seepage.

The main water losses from the Amudarya River are regulated by the intensive use of river water for irrigation and in the middle reaches the water loss for irrigation is more than 300 $m^3 s^{-1}$ (May–August period) which is more than 10–20% of the discharge. In the lower reaches of the Amudarya River, below Tyuamuyun dam, intensive use of water occurs (about 50% from discharge) of more than 1.24 $m^3 s^{-1}$ per 1 km length of river. Water management (abstraction of fresh river water and return of water after use for irrigation) increases the mineralization of the river water to about 2 g l^{-1} during low flow in the summer period.

MATHEMATICAL MODEL OF AMUDARYA RIVER

The mathematical river model defined generally as a "reservoir model" serves as the basis for the proposed technique. This model is based on the balance equation and the equation of water flow movement and was designed specifically for Amudarya conditions. The scheme of calculation is advantageous, because the lag-time of the flood peak depends on the water discharge, and the strong deformational channel instability is also taken into account. The proposed model is a nonlinear mathematical model of the water flow which allows for calculation of the flood movement along the river length in a prescribed space and temporal scale. With a known inflow into the upper gauge line, the water discharge can be calculated along the whole river length. For the simulation of water flows the balance equation is used:

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = P \qquad (1)$$

where A = cross section area, Q = water discharge and P = lateral inflow per unit of length.

The Shazi equation without inertial components is widely used as the equation of water flow movement for simple (regarding the hydraulic phenomena) streamflows:

$$V = C\sqrt{KI} \qquad (2)$$

where V = flow velocity and C, K and I are hydraulic characteristics of the streamflow.

For the numerical realization of the model based on the use of equations (1) and (2) it is necessary to have a set of channel parameters characterizing the stream flow. However, the main requirement is the strict and statistically close relationship between the hydraulic characteristics of flow: cross section area, washed perimeter, hydraulic-radius and water level. The degree of correlation between the water discharge and other hydraulic flow parameters and water level in the river depends on the deformation stability of the river bed. Because the river bed is subject to strong deformation, there are severe difficulties for statistical correlation.

The finite-differential presentation of equation (1) with the absence of inertial components in equation (2) is similar to the successive set of balance equations for the river section:

$$\frac{dW_i}{dt} = Q_{i-1} - Q_i + P_i - V_i \tag{3}$$

where W_i = water storage in i-th section, Q_i, Q_{i-1} = output and input water discharges, respectively, P_i = inflow into section and V_i = water losses.

In general, water storage in the section can be presented as:

$$W_i = A_i L_i \tag{4}$$

where L_i is the length of the section. It is assumed in the analysis that the change in effective cross section for the section of finite length is negligible and can be neglected in the case of numerical realization, i.e. $dA_i/dL_i = 0$. It should also be mentioned that for the numerical realization of equations (1) and (2), a similar assumption is introduced, otherwise the identification of the equation (2) parameters (roughness coefficient and degree index) will encounter significant difficulties and does not ensure the required accuracy for the successive calculations.

If the water discharge out of the i-th section is assumed to be proportional to the water amount into the section, i.e. $Q = C \times W$, and also assuming a linear nature for the section, it can be written as $Q = L \times f(A)$. This approach does not need a single-valued relationship between the flow levels in the river and flow characteristics as it is impossible to introduce this assumption for the unstable flow. During the passing flood wave, the correlation curve for any gauge line will adopt a loop form which attest to the fact that for the same flood depth there are few discharge values and each flood can produce its own loop. With the application of a non-linear relationship, the correlation coefficient is markedly greater in comparison with the linear one. Taking all the above mentioned into account, equation (3) can be presented as:

$$L_i \frac{dA_i}{dt} = Q_{i-1} - Q_i + P_i - V_i \tag{5}$$

In essence, the identification of equation (5) lies in the estimation of the coefficients of correlation between the water discharge in the river and the area of the effective cross section. Special attention during the course of numerical realization for the model is paid to the selection of numerical values of coefficients characterizing the relationship :

$$Q = \alpha \times A^2 + \beta \times A \tag{6}$$

where α and β are parameters selected on the basis of the measured discharge values using the least square method.

The numerical solution for the model consists of the solution of the set of primary differential equations by differential methods. There are many such methods for the solution of such systems. The implicit methods are not appropriate because of the non-linearity of parameters on the right side of the equation. As for methods such as Runge-Kutt, Adams-Boshfort or two-stepwise ones, they are slightly more accurate compared with the implicit ones but much more time consuming in terms of calculations. This is why the implicit Euler scheme with optimization of designed time step is applied which leads to considerable time saving in terms of calculation and accuracy.

The well known test used for such models where the initial flood triangle form is prescribed and the transformation process is traced along the river length has been applied in our case. Figure 1 presents the results of the model tested with different initial conditions. The change of distances between the flood peaks for different water discharge is noticeable. Each equilateral flood during its passage down the river length takes a typical wave-shape form. For the elaboration of the forecasting method, an analysis of the model run on real data for the 1985–1996 period has been fulfilled.

The following criteria were used for the estimation of the forecasting method by the model: effectiveness of the forecasting technique, assumed error, reliability. The results of testing by these criteria are given in Table 1.

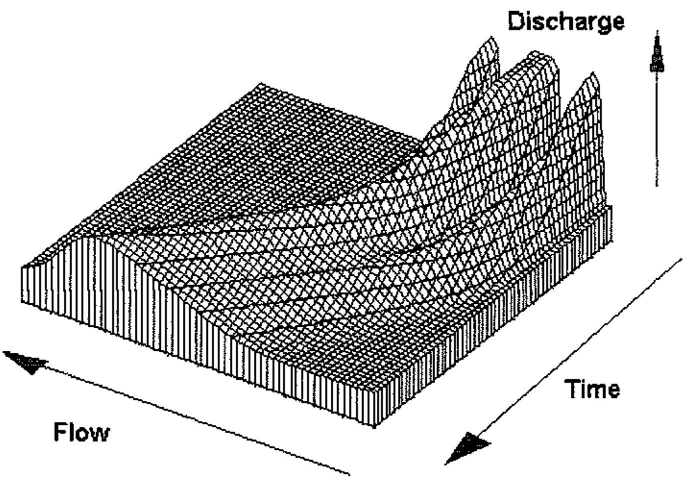

Fig. 1 Model calculations of the flood group passing along the river. Axes: X, length of the river; Y, time; Z, discharges.

Table 1 The results of testing of the model and method of forecast.

Years	Effectiveness	Assumed error ($m^3 s^{-1}$)	Reliability (%)
1985	0.32	364	91
1986	0.37	360	92
1987	0.23	362	98
1988	0.22	395	97
1989	0.32	320	95
1990	0.21	331	89
1991	0.28	357	94
1992	0.29	327	96
1993	0.27	364	92
1994	0.24	389	98
1995	0.28	391	97
1996	0.26	364	92

Acknowledgements The work was carried out within the programme of the Environmental Policy and Technology Project of USAID. The authors thank Dr Kalandarov and Mr Kuprovsky for valuable consultations and presentation of unique materials.