Protection of built and new plan dams against disastrous floods under the climatic change conditions

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Abstract Climatic changes exert already a great influence on rainfall processes about their extreme fluctuations and as a result disasters are often arising. The major disasters involve loss of live and a destruction of property, built dams and other structures. During the last 20 to 50 years the observations on the maximum water discharges in the rivers are to a great or less degree disturbed depending on the saturated ground water and the depth of the ground. On the other hand the constructed already dams have a smaller water discharge capacity throughout the overflow and other relieving structures as bottom outlets, for example they have 1 to 2 outlets instead their necessary numbers 2 to 25. Based on the experience with the flood in Bulgaria more reliable models are proposed for dam building, protection and river regulation.

Key words: flood; drainage; numerical modelling; correlative dependence; catchments basin; finite strips

INTRODUCTION

The river basin management processes focus our attention to determine a possible maximum of the high wave by observations. The determination of the input data by means of the maximum diurnal rainfall with a normative probability is a very important because the rainfall has not disturbed catchments characteristics. This information needs to provide model parameters for a future real high waves and prediction. The author’s algorithm consists of a new numerical method of a finite longitudinal and cross strips shown below (Fig. 1).

First the maximum rainfall of the catchments is determined for the observed period. Then the differential density function and the probability curve are constructed.

Finally an algorithm for the flood and drainage is presented with an appropriate example for a built rock fill Bulgarian dam.

INVESTIGATIONS OF THE RAINFALL DEPENDENCY AND THE CURVES OF THE DENSITY AND PROBABILITY FUNCTIONS

Initially a correlative dependence for an average annual maximum diurnal rainfall is established concerning every selected strip of the catchments. This parameter is calculated for the left and the right river bank respectively - $h_{N\max}^{l(r)}$ and for the catchments basin $\overline{h}_{N\max}^{c.b.}$ as well. The last dependence has the form:

$$\overline{h}_{N\max}^{c.b.} = b\, e^{a\overline{H}\, c.b.} \qquad (1)$$

where $\overline{h}_{N\max}^{c.b.}$ is the average annual max diurnal rainfall of the catchments basin (mm), a and b are the correlative dimensionless coefficients, and $\overline{H}_{c.b.}$ is the average elevation of the catchments basin (km).

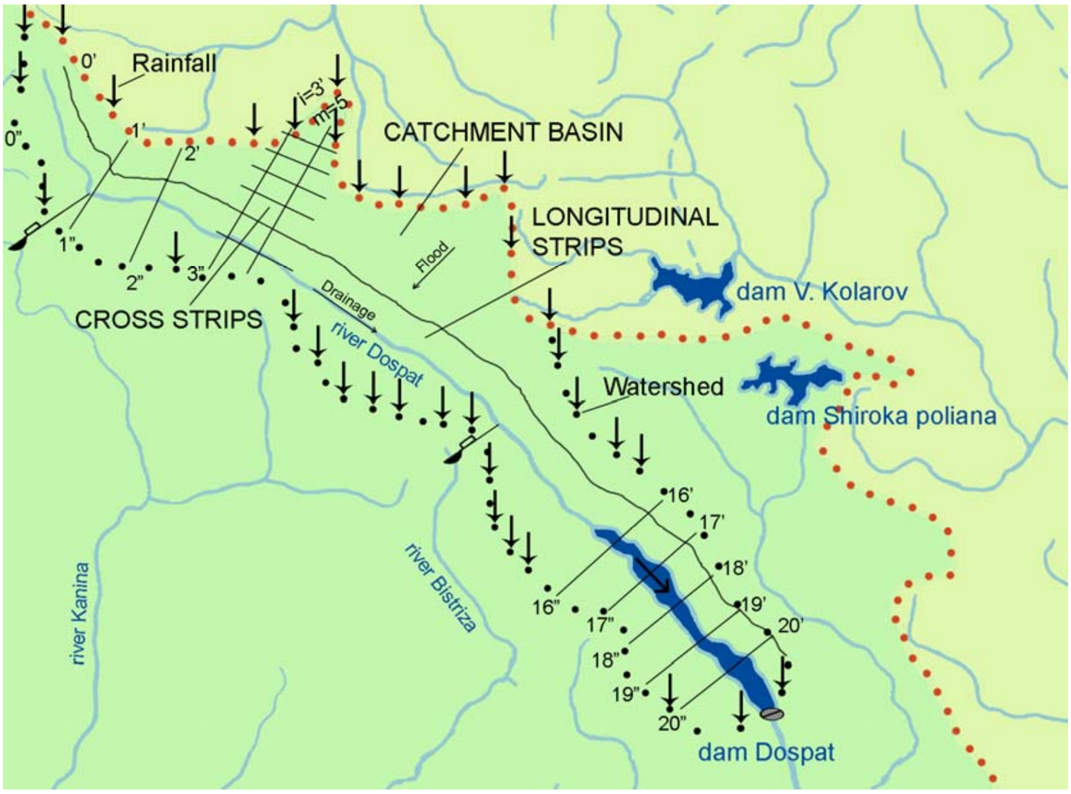

Fig. 1 Design of longitudinal and cross strips in the catchments basin to dam A

The average elevation $\overline{H}_{c.b.}$ of the catchments basin is calculated from the above sea level altitudes and the quadratures of the accepted cross strips. The cross strips are drawn orthogonally of the horizontals of the basin so that the cross strips and the stream lines coincide. In this instance a first order differential equations is applied for the solution of the unsteady flow in wide beds. The average elevation of the catchments basin $\overline{H}_{c.b.}$ is substituted into equation (1) using the next formula:

$$\overline{H}_{c.b.} = \frac{\sum_i (\overline{H}_i^{/} F_i^{/} + \overline{H}_i^{//} F_i^{//})}{\sum_i (F_i^{/} + F_i^{//})} \tag{2}$$

where i is the current co-ordinate for the cross strip, $\overline{H}_i^{/}$ and $\overline{H}_i^{//}$ are the average absolute elevations in the left and the right banks of the river, $F_i^{/}$ and $F_i^{//}$ are a share due from the catchments basin area i in the left and in the right side respectively (m^2).

In order to define the coefficients a and b in a formula (1) two rainfall gages are chosen. They are situated nearest the elevation $\overline{H}_{c.b.}$ by checking two correlative coefficients – general and true one. In this way the correlative dependence (1) can be evaluated as satisfactory, transferring the entry data of the rainfall gages to the dam A, as well as the annual and maximum diurnal rainfall during the observed years. The observations show that the probability density function and the cumulative probability curve have the more steep character, during the last 15 years, because of the climatic changes. In particular this fact is valid in the branch of the probability curve with normative probability of 1% to 0.01%. Distributions like these could be found by gamma function, maximum likelihood method (Fisher), Weibull distribution, etc. In this study a gamma density function is applied with one additional parameter $\hat{a}$ in the

limits $1 \le \hat{a} \le 7.824$. The bigger values have a larger skew ness for the catchments less than 250 km^2 and the smaller ones are valid for the catchments more than 500 to 1200 km^2. In this form the probability density is presented as the formulas (3) and (4):

$$P(X_i) = \frac{\gamma^\gamma}{\Gamma(\gamma)\,\overline{X}_0} e^{-\gamma \frac{X_i}{\overline{X}_0}} \left(\frac{X_i}{\overline{X}_0}\right)^{\gamma - 1} \quad (3)$$

$$\gamma = \frac{1}{C^2_{V\lambda_2}}\ , \quad C_{V\lambda_2} = \sqrt{-\hat{a}\,4.6\,\lambda_2} = \sqrt{-\hat{a}\,4.6 \frac{\sum_i \lg K_i}{n-1}} \quad (4)$$

where $\Gamma(\gamma)$ is a generalized gamma function at an argument γ, $C_{V\lambda_2}$ is the standard deviation at the mean value λ_2, λ_2 is the empirical mean value of K_i which is the modulus coefficient and the dimensionless quotient $\frac{X_i}{\overline{X}_0}$ means a modulus coefficient given by the mean record $\overline{X}_0$. For the gamma distribution at the additional parameter $\hat{a} = 7.82$ and with the normative probability 0.1% given in (Norms for Loads and Impact on Hydraulic Structures by Waves, Ice and Navigation, 1988) the average annual maximum diurnal rainfall $\overline{h}^{c.b.}_{N\max}$ is computed as $350.3\,mm\ m^{-2}$. In 1990 approximately the climatic changes were started to influence and they are increasing more and more all over the word. In the course of 2005 and 2007 years the rainfall in Bulgaria reached to 250 mm m^{-2} and many hydro technical structures were destroyed as a result of flood.

ALGORIHTM OF FLOOD, DRAINAGE AND PROTECTION OF DAMS

The flood and drainage flow are taken place during the period of upsurge. Details are given in (Nikolov, 2007). The transient process is described for a wide open bed in the rivers. For the solution of this unsteady motion the following equations are used:

$$i_{c.b.} - \frac{\partial h}{\partial s} = \frac{\partial}{\partial s}\left(\frac{v^2}{2g}\right) + \frac{\partial h_f}{\partial s} + \frac{1}{g}\frac{\partial v}{\partial t} \quad (5)$$

$$\frac{\partial h_f}{\partial s} = \frac{v^2}{C^2 R} \quad (6)$$

$$Q\,t = h_N\ F \quad (7)$$

where $i_{c.b.} - \frac{\partial h}{\partial s} = -\frac{\partial z}{\partial s}$ is the piezometric slope at the first section, $v^2(2g)^{-1}$ is the velocity height (m), h_f is the friction height (m), g is the gravity constant (m s^{-2}), C is the Manning formula, R is the hydraulic radius (m), Q is the water discharge (m^3 s^{-1}), t is the time (h), F is the mean area between two sections (m^2) and h_N is the average rainfall of the catchments (mm) for the time t.

On the bases of the recorded entry information the following data is necessary: the size of longitudinal and cross strips, the slope and the roughness of the terrain and the river, the maximum rainfall for 0.1% probability. After solving the above mentioned system, through the finite strips method, the unknown values are established: the minimum duration of the rainfall passing from the watershed to the river, the maximum water discharge and the shape of the high wave by balance modelling, the

values of the maximum 24 hours rainfall and the intensity in mm min^{-1}. The shape of the high wave – upsurge, detention and subsiding is implemented by the balance formulas and empirical data equalizing the rainfall volume and the high wave volume:

Upsurge: $$Q = Q_0 \left(\frac{t}{t_0} \right)^{1-\left(\frac{t}{t_0}\right)^2} 2^{-\left[1-\left(\frac{t}{t_0}\right)^{\frac{1}{2}}\right]} \qquad (8)$$

Subsiding: $$Q = Q_0 \; 2^{-\left[1-\left(\frac{t}{t_0}\right)^2\right]} \qquad (9)$$

where the upsurge time is $t = \sum \Delta t$ (h), $t = t_0 + \sum \Delta t$ is the time for subsiding (h) at the upsurge time t_0 corresponding to the initial rainfall flood and Q_0 is the water discharge with a probability of 0.1% (m^{3}s^{-1}). Two built Bulgarian dams are checked up by the new proposed method for the flood in the catchments and the drainage in the river. The both dams are designed in 1952 (Gravity) and in 1965 (Rock fill). They have relieving structures with relatively small discharges 800 m^3 s^{-1} and 500 m^3 s^{-1}. The results for the second dam are given below. The minimum duration for the maximum rainfall is: t_0 =2.828 hours and $\bar{h}^{c.b.}_{N\max}$ =350.3 mm. The maximum water discharge is given as $Q^{0.1\%}_{\max}$ =8285 m^3 s^{-1}.

The total maximum volume of the catchments basin is assumed to be $W^{c.b.}_{N\max} = 81.98 \times 10^6$ m^3. The overflow head is computed as H =2.75 m and the volume of the side channel spillway is $W_{sp.}$ =2.19$\times$ 10^6 m^3. The resulting retention height is assumed as $h_{ret.}$ =3.71 m and the velocity setup has a value of h' =1.89 m. The final result is given by the comparison between the retention height plus the velocity setup and the overflow head or the sum is: $h_{dest.}$ = 3.71+1.89-2.75=2.85 m.

where $h_{dest.}$ is the potential risk for destruction (m). The above mentioned result shows the possibility of destruction if the rainfall is bigger than 250-300 mm m^{-2} and if the built dam is constructed before 20 years and more. Concerning the new plan dams an iterative solution has to be done in order to receive optimal numbers and sizes of the relieving structures as overflows, spillways and outlet devices.

CONCLUSIONS

Appropriate local hydrological information is discussed to model the flood rainfall-runoff process in the river on the bases of a new method called finite strips. This method is expected to assess the relieving structures, the reduced high wave with lower section and tributaries along the whole length of the river. By automation and depending on the rainfall intensity the bottom outlets are opening so that when the lake is partial empty the outlets start closing later on and finally the lake is again full. Finally an inverse modeling process closes the system and verify the credibility of this protection.

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