A CASE STUDY OF FLOOD MANAGEMENT BY A CASCADE SYSTEM OF RESERVOIRS

Jovanovic, M., Dr.Sc[1]
Obuskovic, Z., B.Sc, Huseinspahic, Z., B.Sc[2]

Abstract. This paper presents results of a hydraulic study which has been undertaken in order to investigate the effects of canalization of the lower Drina river on the flood flow regime. Two possible flood control schemes have been analysed by mathematical modelling, including partial pre-emptying of reservoirs, and free or controlled discharge of excess water into the floodplains. The results of calculation are presented for the flood wave of return period 1000 years, illustrating the specific aspects of this kind of flood management.

1. INTRODUCTION

According to a proposed hydropower production scheme for the lower Drina river in Yugoslavia, a cascade system of four dams is to be constructed, and four reservoirs, named Kozluk, Drina I, Drina II, and Drina III are thus to be formed along the 83 km long river reach (Fig. 1) [4].

The lower Drina valley is characterized by wide floodplains with widths varying from several hundred meters at the upstream end of the considered reach, to several kilometers at the confluence of Drina and Sava rivers, at the dowstream end (Fig. 1). The floodplains consist mainly of farm land, but several villages, and one larger town are potentially endangered by extreme flood events.

The reservoirs are to be formed within levees 200-1000 m apart. The dams are designed as gravity structures with spillways 80 m wide (Drina I–Drina III), and 160 m (Kozluk). All spillways are to be equipped with radial gates, and the maximal spillway head is 8.6 m. The design discharge of all power plants is 800 m^3/s.

A special feature of the system are two auxiliary side spillways located at the upstream end (Fig. 1), allowing for the excess flood water (in respect to the designed capacity of the regulated river channel), to be evacuated into the floodplains. Both

[1]University of Belgrade, Faculty of Civil Engineering
[2]Energoprojekt Co., Belgrade

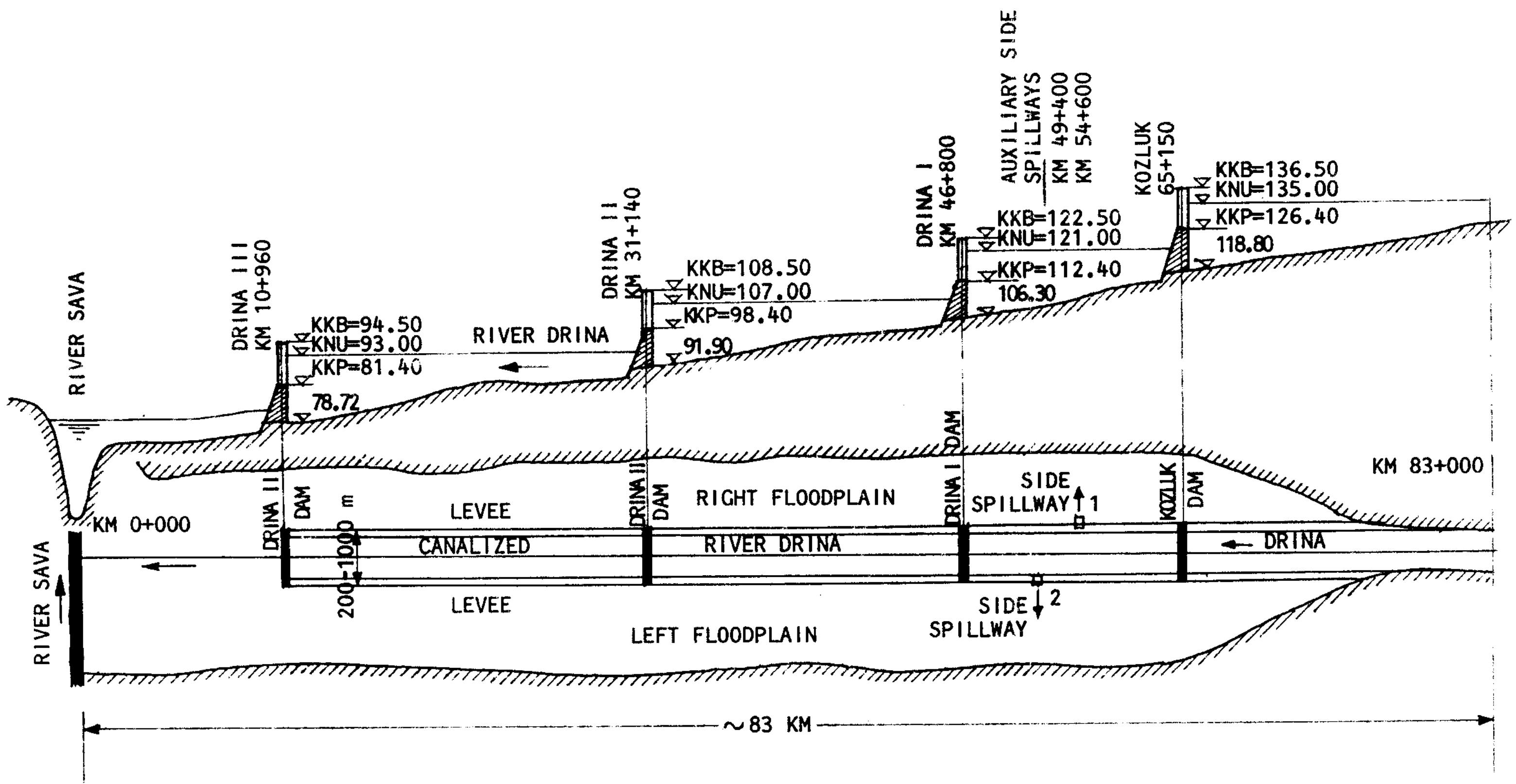

Fig. 1. Schematic layout of the cascade of reservoirs on the lower Drina river [4]

side spillways, 4 m high and 40 m wide, can operate either as free side weirs, or as control structures with flow regulated by gates.

This paper presents some of the results of a hydraulic study [4], based on numerical simulation of transformation of predicted 100, 500 and 1000-year floods, with the following objectives: (a) to determine the present retention capacity of the floodplains for the considered waves, (b) to see in what way would the proposed power system affect the flow regime, especially at the confluence of the two rivers, and (c) to determine an optimal strategy of flood control in respect to the following three conditions:

(1) avoiding the coincidence of extreme flow rates at the confluence of the two rivers by allowing an appropriate speed of flood wave propagation;

(2) least possible discharging over the auxiliary spillways into the floodplains;

(3) continuous functioning of the power production system.

2. MATHEMATICAL MODEL

All calculations have been performed by one-dimensional unsteady flow model, since only global transformation of flood waves is required, and application of a two-diemnsional model would require more detailed topographic data not presently available. However, the de St. Venant flow equations, are modified in such a way that the main river channel and the floodplain flows can be treated separately [2],[3]:

$$\frac{\partial S(A+A_o)}{\partial t} + \frac{\partial Q}{\partial x} - q = 0 \qquad (1)$$

$$\frac{\partial (SQ)}{\partial t} + \frac{\partial (\beta Q^2/A)}{\partial x} + gA(\frac{\partial Z}{\partial x} + I_f) = 0 \qquad (2)$$

The notation used in equations of conservation of mass and momentum (1)–(2) is given in the appendix. The conveyance factor in the friction slope term:

$$I_f = \frac{n^2 Q|Q|}{A^2 R^{4/3}} = \frac{Q|Q|}{K^2} \ , \qquad (3)$$

can be evaluated as follows:

$$K = K_r + K_l + K_d \qquad (4)$$

$$K_r = A_r R_r^{2/3}/(n_r S^{1/2}) \qquad (5)$$

$$K_l = A_l R_l^{2/3}/n_l \qquad (6)$$

$$K_d = A_d R_d^{2/3}/n_d \qquad (7)$$

where subscripts "r", "l" and "d" designate river channel, left and right floodplains, respectively. A special depth variable parameter – the "sinuosity factor" (S), is used as a weighted ratio of the flow path along the river channel and floodplains, in order to ensure mass conservation [2], [3]:

$$S_M = \frac{\sum_{j=2}^{j=M} \Delta A_{rj} S_j + \Delta A_{lj} + \Delta A_{dj}}{A_{rj} + A_{lj} + A_{dj}} \quad , \tag{8}$$

S_j is the sinuosity factor for the portion of the flow between the "j" and "j+1" depths, and the corresponding cross-sectional area difference $\Delta A = A_{j+1} - A_j$.

The momentum coefficient for velocity distribution is calculated as [3]:

$$\beta = \frac{1.06 K_l^2/A_l + K_r^2/A_r + K_d^2/A_d}{(K_l + K_r + K_d)^2/(A_l + A_r + A_d)} \quad , \tag{9}$$

The system of partial differential equations (1)–(2) with appropriate initial and boundary conditions is solved using the well-known 4 point implicit finite difference Preissmann scheme, the details of which can be found in numerous specialized literature (for instance [1]).

In addition to the inflow hydrograph and the rating curve as the standard boundary conditions at the upstream and the downstream boundaries, several inner boundary conditions are introduced in the cross-sections of dams and side spillways, as those are locations where discontinuities in the gradually varied unsteady flow occur. The inner boundary conditions are defined by the steady flow continuity equations of mass and energy, the later being expressed in the form of well-known hydraulic expressions for free or submerged spillway flow, or gate-controlled flow.

3. RESULTS

3.1. Transformation of flood waves in natural conditions

The given mathematical model has been used to calculate transformation of flood waves of 10, 20, 100, 500 and 1000-year return periods in natural conditions (in the river channel as is presently). As an example, the 1000-year discharge hydrographs, calculated at the proposed dam locations and the confluence, are shown in Fig. 2. For this particular case, the ratio of peak discharges of the outflow and the inflow hydrographs is about 0.7, which indicates the storage capacity of the floodplains, or the degree of flood wave attenuation. The ratios corresponding to other analysed flood waves are in the range 0.7–0.8 [4]. The calculated times of flood propagation, checked against available data registered during past flood events at gauging stations on Drina and Sava rivers, can be considered realistic.

When analysing the wave transformation through the system of reservoirs, a special attention has been given to the imperative condition that no coincidence of floods at the confluence of the two rivers is to occur. According to hydrologic records and some previous studies, it has been concluded that flood waves of the river Drina reach the confluence 1.5–3.5 days earlier than flood waves of the river Sava. Not only that this tendency is to be retained in case of any future river regulation, but it is advisable to increase this time delay if possible. This has been investigated by using various initial and boundary conditions when simulating flood propagation through the proposed cascade of reservoirs. For example, two flood control options have been investigated considering the 1000-year flood wave [4]:

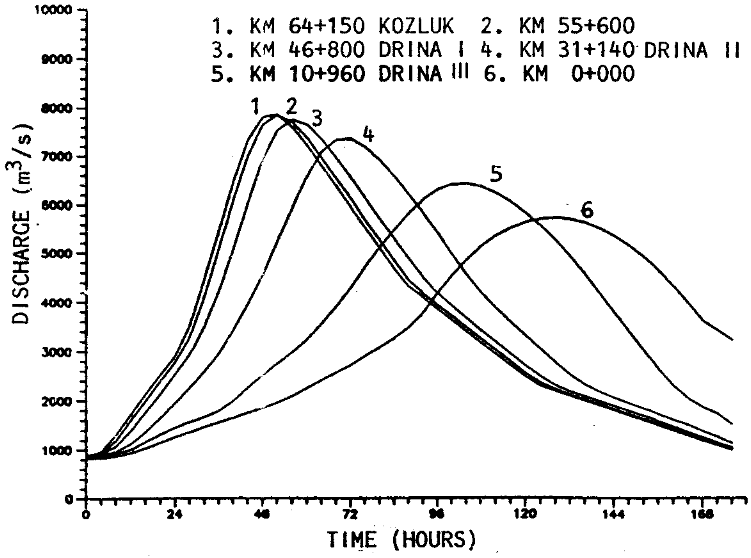

Fig. 2. Calculated 1000-year flood discharge hydrographs for natural conditions

(i) All reservoirs are to be pre-emptied up to the spillway crest elevations, and all gates are to be completely raised, thus allowing free flow on all spillways, including the two auxiliary ones. In this way the incoming flood wave will be divided into three smaller waves: one, propagating along the canalized river bed (reservoirs), and two, propagating along the left and right floodplains, as the excess water is discharged over side spillways into the floodplains. This flow distribution is not controlled and is effectuated in a natural way, according to inflow rates.

(ii) The normal reservoir water surface elevations (KNU in Fig.1) are to remain unchanged by letting the excess water discharge over the side spillways into the floodplains, but in such a controlled way that the quantities of water in two floodplains are to be equal. (This condition is imposed if flooding damages are to be minimized).

3.2. Flood control scheme (i)

The results obtained for the flood control scheme (i) are presented in Fig. 3 and Fig. 4. Fig. 3-a depicts calculated discharge hydrographs for the two side spillways. The apparent difference in these hydrographs is due to the chosen location of spillways. The effects of side discharging on the maximal flow rates in the canalized river reach is shown in Fig. 3-b. The maximal discharges are decreased for about 1700 m^3/s by the upstream side spillway, and for about 1400 m^3/s by the downstream one. Due to limited storage capacities of reservoirs, the part of the 1000-year flood wave propagating along the cascade of reservoirs Drina I-III is not much attenuated (Fig. 3-c). Stage hydrographs calculated immediately upstream and downstream from each dam show that maximal stage elevations would not significantly exceede

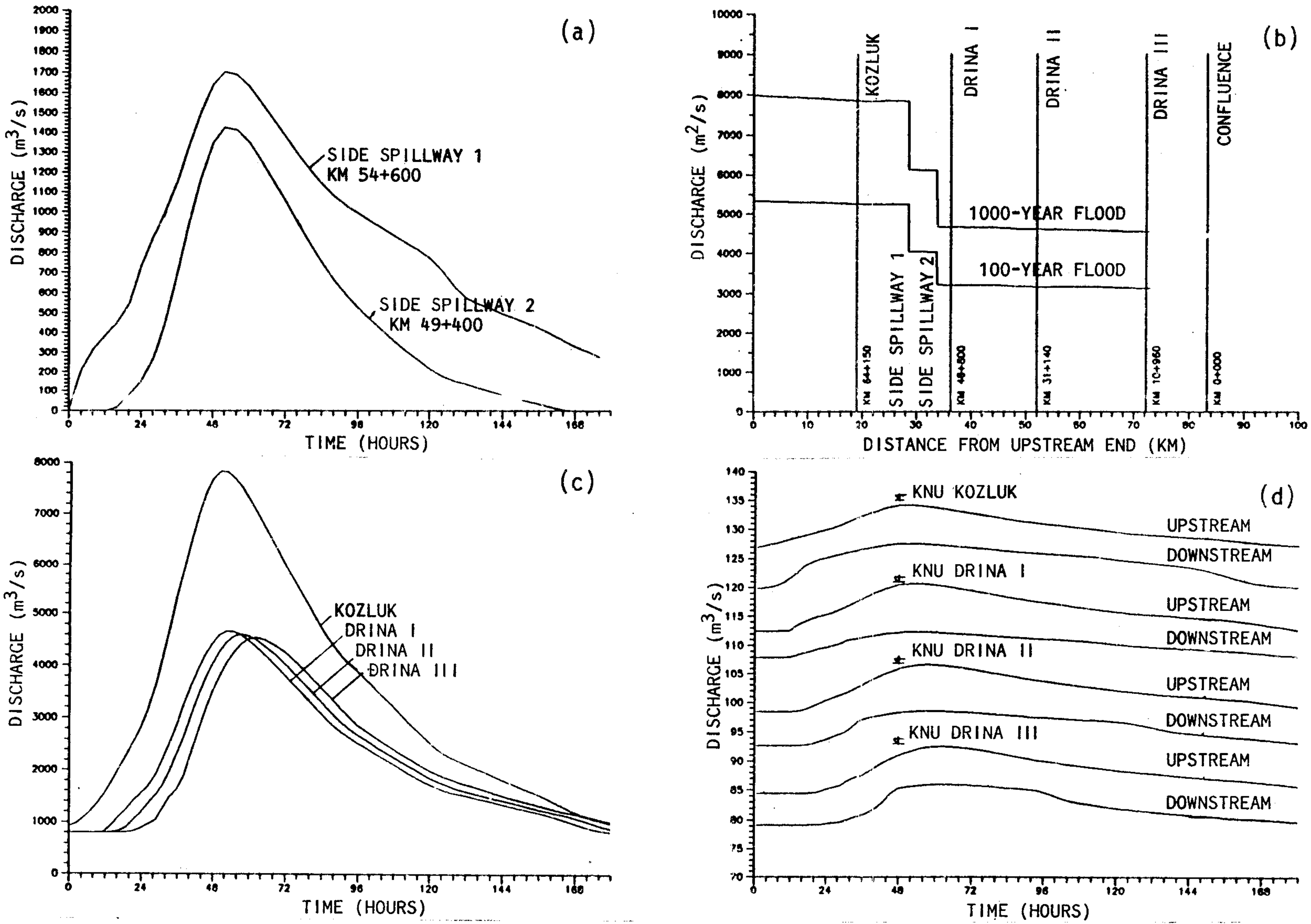

Fig. 3. Calculated transformation of the 1000-year wave according to the flood control scheme (i) – (a) side spillways discharge hydrographs; (b) maximal discharges in the canalized river; (c) discharge hydrographs; (d) stage hydrographs

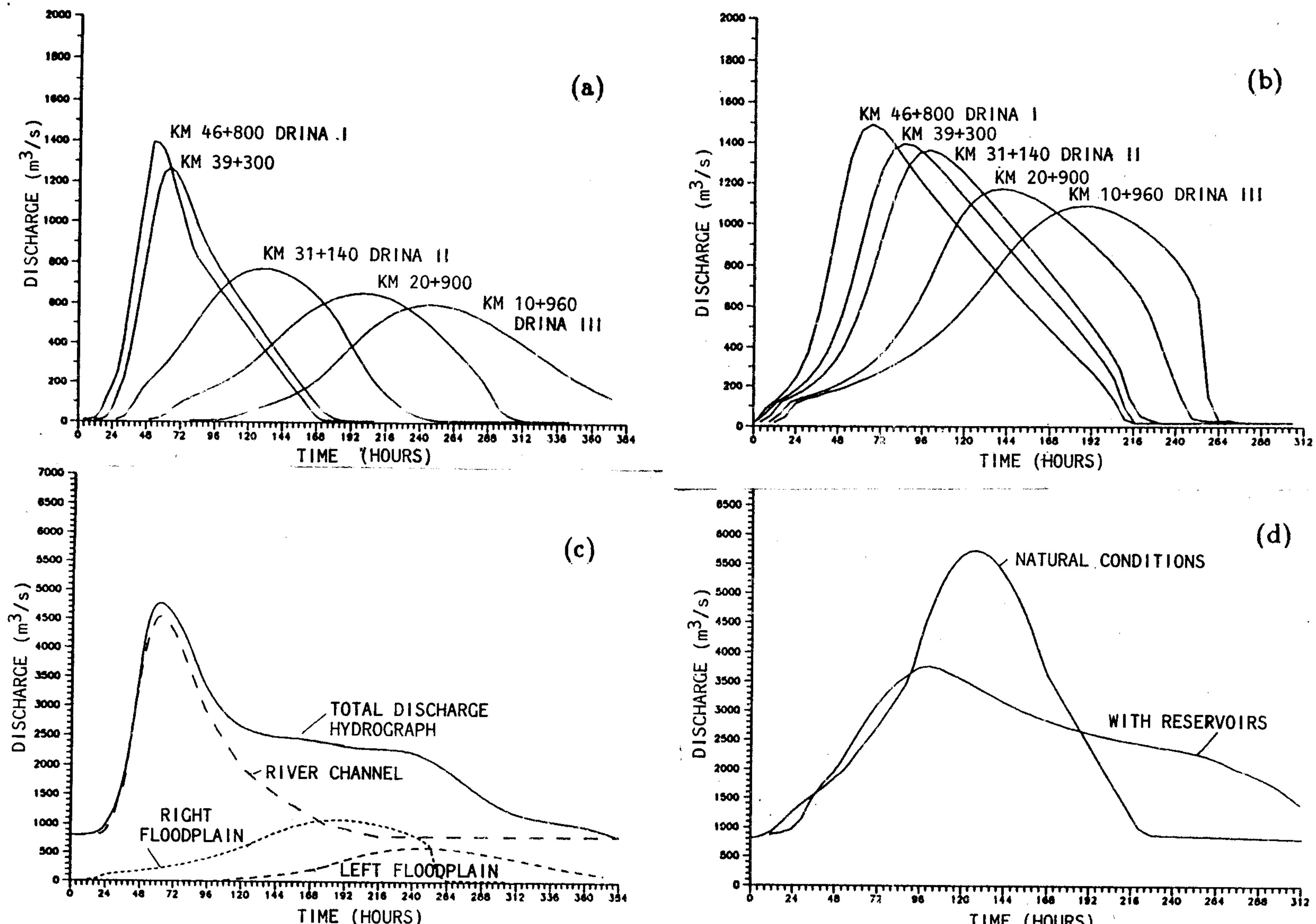

Fig. 4. Calculated transformation of the 1000-year wave according to the flood control scheme (i) – (a) left floodplain; (b) right floodplain; (c) superposition of discharge hydrographs; (d) transformed flood waves at the confluence

the normal reservoir elevations (Fig. 3-d), which means that the safety of dams and levees would not be endangered if this flood control scheme is applied.

The transformation of those parts of the 1000-year flood wave which are propagating along the floodplains are shown in Fig. 4-a and Fig. 4-b. Due to different size of floodplains, the velocities of propagation are different, the one in the right floodplain being greater. The three separate waves into which the 1000-year flood wave has been divided, meet at the end of the regulated reach – the cross-section Drina III (Fig. 4-c). The resulting wave has a smaller peak value than the one corresponding to natural conditions (without reservoirs). The same is true at the confluence (Fig. 4-d). The maximal discharge is decreased for 1650 m3/s, and occur 24 hours earlier, which means that the regime of extremely high flows at the confluence would be improved.

3.3. Flood control scheme (ii)

As mentioned before, according to this scheme the water surface levels are to be kept constant and equal to the normal reservoir elevations (KNU). This can be achieved by a synchronous action of all gates in the system, ensuring controlled discharging of water into the floodplains over the two side spillways. The maximal allowable discharge through the reservoirs, corresponding to the maximal capacity of spillways, would be 4080 m^3/s. Discharges exceeding this limiting value are to be transferred to the left and right floodplains (Fig. 5), with maximal values of 1960 m^3/s in each floodplain.

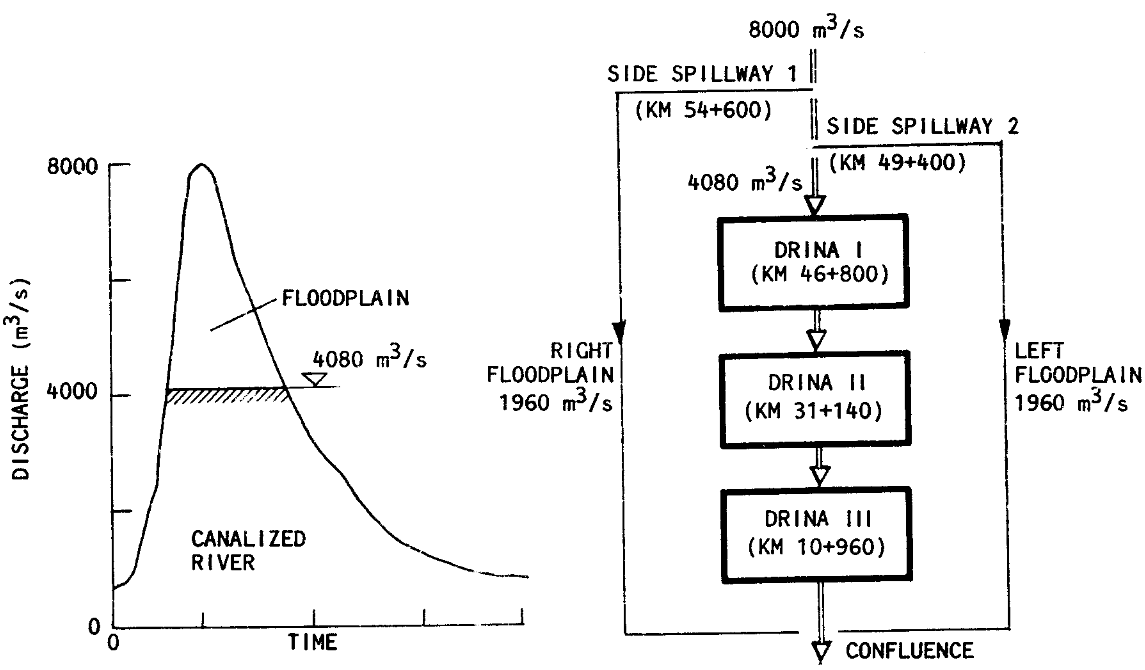

Fig. 5. Schematic representation of flow limiting in the regulated river channel in case of the 1000-year flood wave

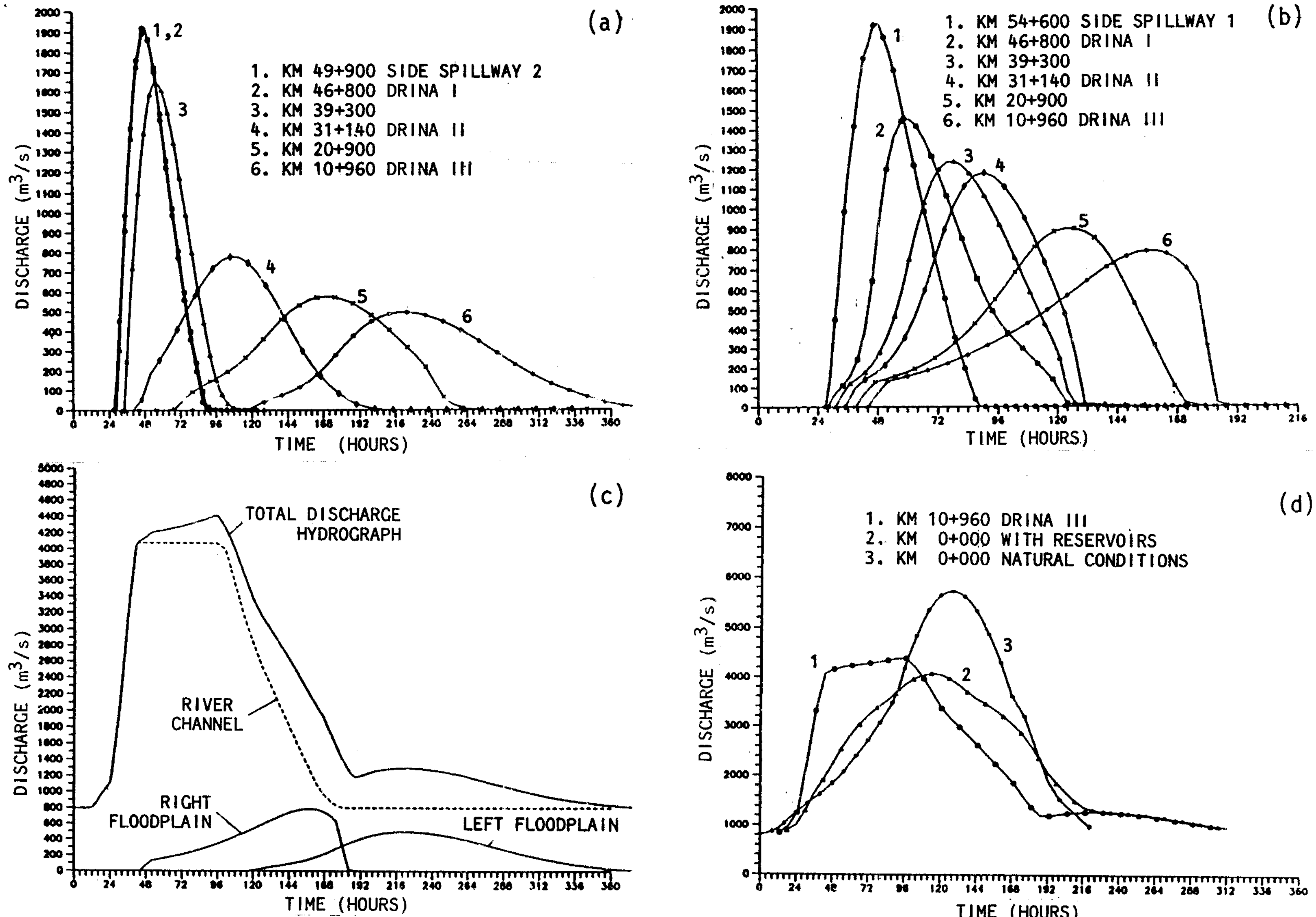

Fig. 6. Calculated transformation of the 1000-year wave according to the flood control scheme (ii) – (a) left floodplain; (b) right floodplain; (c) superposition of discharge hydrographs; (d) transformed flood waves at the confluence

The calculated results according to this flood control scheme are shown in Fig. 6-a through 6-d. Analysing the discharge hydrographs at the confluence (Fig. 6-d), it can be concluded that the peak of the 1000-year flood wave can be decreased by the given system of reservoirs for about 1600 m^3/s, and the time of propagation decreased for 12 hours.

4. CONCLUSIONS

The accomplished hydraulic analysis lead to the following conclusions:

1. There exists a considerable retention capacity of floodplains in the lower river Drina basin in natural conditions.

2. If a system of reservoirs is formed, there are two possible flood control shemes as shown in the case of the 1000-year flood wave: the first is based on the pre-emptying of reservoirs and free discharging of the excess water into floodplains, while the second is based on retaining the normal water levels in the reservoirs, and controlled discharging into floodplains.

3. Both flood control schemes ameliorate the flow conditions at the confluence of rivers Drina and Sava; by the given system of reservoirs the maximal discharges are decreased and any coincidence of peak flood discharges in the two rivers is successfully avoided.

References

1. Cunge, J.A., Holly, F.M., Verwey, A., "Practical Aspects of Computational River Hydraulics", Pitman, London, 1980.
2. De Long, L.L., "Mass Conservation: 1-D Open Channel Flow Equations", Journal of Hydraulic Engineering, ASCE, Vol. 115, No.2, Feb. 1989.
3. Fread, D.L., "The NWS Dambrk Model", National Weather Service, 1988.
4. Jovanovic, M., Obuskovic, Z., "Study of Hydro Power Potentials of Lower Drina River", Energoprojekt Co., Belgrade, 1991. (in Serbian)

Notation:

x	....	longitudinal distance (along floodplains)
t	....	time
Q, Z	....	discharge and water surface elevation
q	....	unit lateral inflow or outflow
A, A_o	....	active and inactive cross-sectional areas
I_f	....	boundary friction slope
S	....	sinuosity factor [2]
β	....	momentum coefficient for velocity distribution
n	....	Manning roughness coefficient
R, K	....	hydraulic radius, conveyance
g	....	acceleration due to gravity