60

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СТЕПГРАД

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MODELLING FLOOD WAVE TRANSFORMATION IN ULOG HPP RESERVOIR

Abstract:

This paper analyses the role of the Ulog Hydropower Plant reservoir in mitigating flood waves in the Neretva River basin. A mathematical simulation model was applied to transform flood waves with return periods of 10, 20, 50, and 100 years, under different initial reservoir water levels. The results indicate that complete retention of flood waves cannot be achieved solely by operating the Hydropower plant (HPP) and Small Hydropower plant (SHPP) units, however, appropriate operational strategies can significantly reduce peak discharges. The need for timely reservoir pre-release and coordinated operation of evacuation structures is emphasized. It is concluded that reliable forecasts and robust operational models are essential for effective active flood protection.

Keywords: reservoir, hydropower plant, flood wave, active flood protection

МОДЕЛИРАЊЕ ТРАНСФОРМАЦИЈЕ ПОПЛАВНОГ ТАЛАСА У АКУМУЛАЦИЈИ ХЕ УЛОГ

Сажетак:

У раду је анализирана улога акумулације ХЕ Улог у ублажавању поплавних таласа на сливу ријеке Неретве. Примјеном математичко-симулационог модела извршена је трансформација поплавних таласа повратних периода 10, 20, 50 и 100 година, при различитим почетним котама нивоа у акумулацији. Резултати показују да није могуће потпуно ретензирање поплавних таласа само радом агрегата ХЕ и МХЕ, али је одговарајућим управљањем могуће значајно смањити вршне протоке. Истакнута је потреба за благовременим претпражњењем и координисаним радом евакуационих органа. Закључено је да су прогнозе и поуздани управљачки модели кључни за ефикасну активну заштиту од поплава.

Кључне ријечи: акумулација, хидроелектрана, поплавни талас, активна заштита од поплава

1. INTRODUCTION

Climate change and anthropogenic pressures have significantly altered hydrological regimes worldwide, leading to increased spatiotemporal variability in water resource availability and intensification of extreme events. Over recent decades, observational records have documented increasing trend in both frequency and magnitude of extreme events, a pattern that is particularly pronounced in the Mediterranean Basin to which the Neretva catchment belongs [1], [2].

Floods represent the most pervasive and economically devastating natural hazard, accounting for approximately 40% of all-natural disaster-related economic losses and affecting more people than any other environmental risk category. Beside economic effect, profound environmental and social consequences indicate the need for more efficient approaches to protection against the destructive effects of water and to water resources management [1].

Flood protection measures are generally categorized into structural and non-structural approaches. Structural measures are further divided into active and passive, depending on their operational requirements. Active measures imply planned and continuous water resources management, including the use of reservoirs and retentions in order to reduce peak flood discharges [3]. The application of active measures enables timely responses to incoming flood waves, thereby minimizing potential damage. Passive measures include the construction of linear protection systems such as levees, floodwalls, and similar structures, i.e., the construction of barriers that directly protect the defended area. Complementing these are non-structural measures, which focus on spatial planning to prevent the construction of inappropriate facilities in flood-prone zones, thus limiting the growth of potential damage. These measures also include activities such as public education and the development of policies aimed at reducing flood risk [4].

Reservoirs constitute the principal infrastructure for active flood control, functioning as temporary storage systems that attenuate flood waves by retaining inflow volumes and releasing them at controlled rates. From a hydrological engineering perspective, reservoir flood control operates on a fundamental principle: the storage of a portion of the flood wave volume within the reservoir pool enables reduction of peak discharge downstream, with the transformation efficiency governed by reservoir morphology, outlet works capacity, and operational protocols. This attenuation process not only protects downstream communities but also facilitates more rational water resource utilization by retaining water that might otherwise contribute to flood damage [5].

The scientific literature has intensively investigated flood wave propagation through reservoirs [6], [7] with few of then regarding pre-release strategies with varying initial water levels under real-world operational constraints [5], [8]. This paper presents the application of a simulation model for flood wave transformation using the Ulog Hydropower Plant (HPP) reservoir in the Neretva River basin as a case study. The Ulog HPP reservoir, situated in the upper Neretva catchment, provides an ideal setting for investigating flood transformation dynamics due to its operational characteristics and the availability of hydrological data. During the flood wave transformation, different initial reservoir water levels and the possibility of reservoir pre-release were analyzed.

The research objectives are to quantitatively evaluate the effects of different initial water levels on flood peak attenuation and to assess the feasibility and efficacy of pre-release strategies as an operational measure for enhancing flood control capacity. By systematically analyzing these factors within the constraints of existing infrastructure, this research aims to identify operational protocols that achieve minimal possible outflow discharges while maximizing beneficial use of available storage—contributing both to local flood management practices and to the broader scientific understanding of reservoir flood control optimization.

2. DESCRIPTION OF THE ANALYZED SYSTEM

The Ulog storage–diversion hydropower plant is part of the hydropower system on the Neretva River. The upper course of the Neretva River, where this hydropower plant (HPP) is located, is a mountainous area characterized by karst terrain and complex geological conditions. The powerhouse is located near the settlement of Ulog in the municipality of Kalinovik.

The Neretva River springs below Mount Jabuka (a branch of Zelengora Mountain) at an elevation of 1227 m a.s.l. and is formed from several smaller springs and streams. The river flows through Bosnia and Herzegovina and Croatia and discharges into the Adriatic Sea. Its total length is 230 km, of which 208 km lies within Bosnia and Herzegovina. The total catchment area amounts to 10380 km². Along its course from the source, the Neretva receives a large number of tributaries.

The catchment area of the Neretva River up to the Nedavić dam profile is 294 km². The river reach length is approximately 35.3 km, with an average longitudinal slope of about 1% and an average catchment elevation of approximately 1120 m a.s.l. [9].

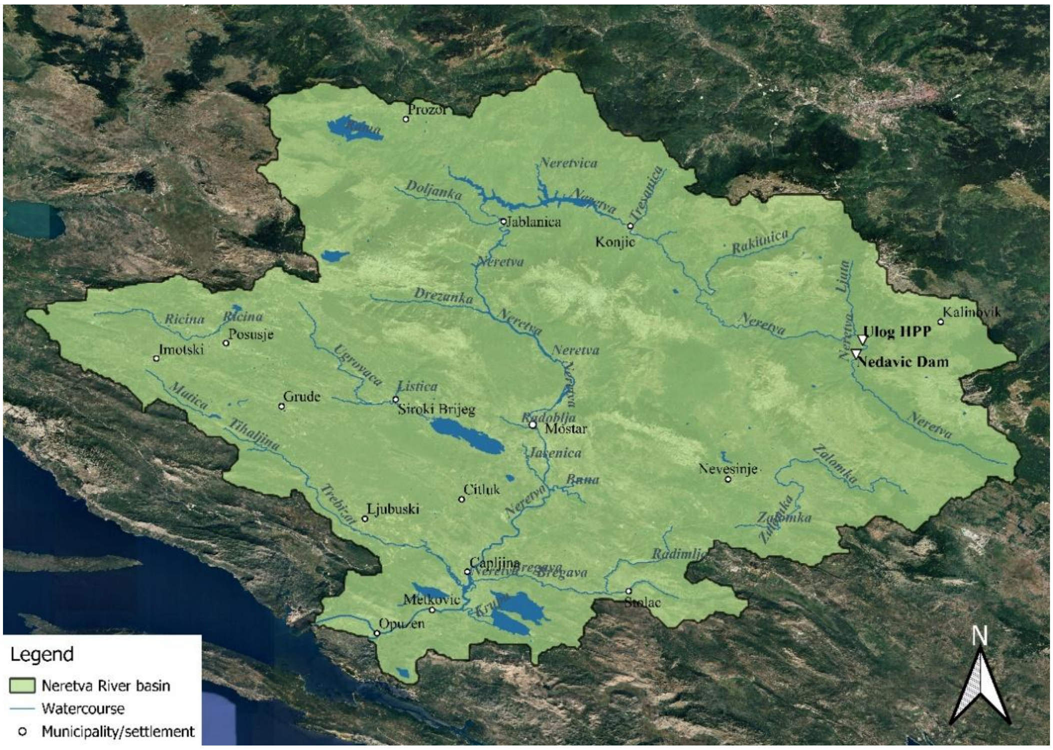

Figure 1. Neretva River catchment with the hydrographic network (created by author).

2.1. CHARACTERISTICS OF THE ULOG HYDROPOWER PLANT STRUCTURES

The data presented in this subsection were taken from the existing project "Reservoir and Hydropower Plant 'Ulog' Management Plan for the ORS (District) of the Trebišnjica River in the Republic of Srpska", completed in 2023 [10].

The Ulog HPP reservoir extends upstream from the Nedavić dam to just below the bridge on the local road in the settlement of Ulog. The normal water level is at elevation 641 m a.s.l. At this level, the reservoir length is approximately 4.90 km, while its surface area is 7.90 ha. The minimum operating water level is at elevation 622 m a.s.l. The total storage volume of the reservoir is 6.44×10^6 m³, while the useful storage volume is 5.90×10^6 m³.

The dam forming the Ulog HPP reservoir is located in the village of Nedavić and represents a high concrete arch dam. The structural height of the dam is 53 m, the dam crest elevation is 643 m a.s.l., while the riverbed elevation at the dam profile is 600 m a.s.l. The dam includes a frontal spillway with two spillway bays for flood evacuation, each 7.5 m in height and 6.5 m in width, with a spillway crest elevation of 633.5 m a.s.l. Both spillways are equipped with segment gates. The bottom outlet has a diameter of 1.20 m and a capacity of Q_{bo} =17 m³/s at the normal water level (641 m a.s.l.) and Q_{bo} =14.60 m³/s at elevation 631 m a.s.l. In addition, the dam includes an outlet for the environmental flow with a diameter of 0.3 m and a capacity of 0.52 m³/s.

Adjacent to the dam structure, a small hydropower plant is installed with a capacity of 180 kW and an installed discharge of 0.52 m³/s, corresponding to the environmental flow. The gross head of the facility is 38 m, and the average annual production amounts to 1.57 GWh/year.

The diversion system consists of a headrace tunnel, a surge tank, and a pressurized penstock. The headrace tunnel has a circular cross-section with a diameter of D_t = 4 m, and a length of 2302 m. The tunnel intake is located at elevation 612 m a.s.l., and the longitudinal slope of the tunnel is 1.75%. The surge tank is of shaft type and cylindrical shape with a diameter of D_{st} = 9 m. he penstock length is 330 m, with a longitudinal slope of 1.3%.

The powerhouse of the Ulog HPP is a surface structure and contains Francis turbines with an installed discharge of $Q_i = 2 \times 17.5$ m³/s and an installed capacity of $P_i = 2 \times 17.56$ MW. The expected average annual energy production is 82.34 GWh/year. The maximum gross head of the plant is 120.26 m, while the minimum gross head is 105 m. The switchyard is located on the roof of the powerhouse.

3. FLOOD WAVE TRANSFORMATION MODEL

Reservoirs mitigate the impact of flood waves by retaining a portion of the incoming flood wave volume within the empty storage space designated for this purpose, thereby reducing the peak of the outgoing wave released through the spillway and outlet structures. This process is referred to as flood wave transformation and is schematically illustrated in the Figure 2. [11].

The reservoir storage curve ($V = V(Z)$) defines the available volume for flood wave attenuation. The discharge capacity of the evacuation structures is represented by the discharge curve ($Q = Q(Z)$), which expresses the relationship between the outflow discharge and the reservoir water level. The objective of flood wave transformation is, for a given inflow flood wave defined by an inflow hydrograph ($Q_{in} = Q_{in}(t)$), and for known characteristics of the evacuation structures and the initial reservoir water level, to determine the outflow hydrograph ($Q_{out} = Q_{out}(t)$) and the reservoir water level hydrograph ($Z = Z(t)$) [11].

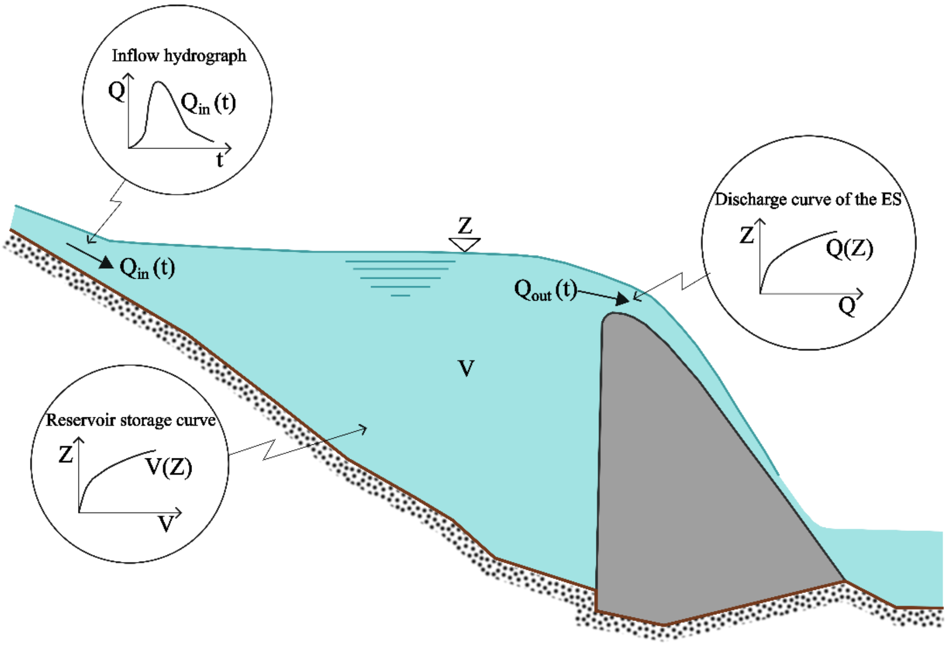

Figure 2. Flood wave transformation in a reservoir [6].

3.1. MATHEMATICAL MODEL

In the general case, the equations governing unsteady open-channel flow, known as the Saint-Venant equations (1871), are given as follows [12]:

Continuity equation:

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = 0 \tag{1}$$

Dynamic equation:

$$\frac{\partial Q}{\partial t} + \frac{\partial}{\partial x}(vQ) + gA\left(\frac{\partial h}{\partial x} - I_0\right) + gAI_e = 0 \tag{2}$$

where Q (m^3/s) is the discharge, A (m^2) is the flow cross-sectional area, v (m/s) is the velocity, h (m) is the flow depth, t (s) is time, x (m) is the longitudinal coordinate along the channel, I_0 (–) is the bed slope, I_e (–) s the energy grade line slope, and g (m/s^2) is the gravitational acceleration.

For the analysis of flood wave propagation in a reservoir, a simplified approach is used—specifically, the quasi-steady method—introducing the following assumptions:

- In the dynamic equation, inertial terms and terms related to velocity variation are neglected;
- The reservoir water surface is assumed to be horizontal, neglecting dynamic effects along the flow direction;

- The travel time of disturbances through the reservoir is neglected, assuming that the inflow is instantaneously distributed over the entire reservoir volume within a single computational time step.

Under these assumptions, the mathematical model for flood wave transformation can be written in simplified form [11]:
Continuity equation (mass balance) for the reservoir volume:

$$\frac{dV(t)}{dt} = Q_{in}(t) - Q_{out}(t) \tag{3}$$

the dynamic equation is replaced by the reservoir storage curve:

$$V = V(Z\ (t)) = V(t) \tag{4}$$

where $V(t)$ is the reservoir storage volume at time t, $Q_{in}(t)$ is the inflow discharge to the reservoir at time t, $Q_{out}(t)$ is the outflow discharge at time t, which is a function of the reservoir water level at the corresponding time.

3.2. NUMERICAL MODEL

By discretizing differential equation (3) for a selected time step Δt, the following equation is obtained [11]:

$$\frac{V(t+\Delta t) - V(t)}{\Delta t} = \frac{1}{2}\left[\left(Q_{in}(t+\Delta t) + Q_{in}(t)\right) - \left(Q_{out}(t+\Delta t) + Q_{out}(t)\right)\right] \tag{5}$$

where t denotes the beginning of the time step (previous time level), and $t + \Delta t$ represents the current time level.
The computation proceeds from the previous time level t, at which all relevant variables are known, toward determining the values at the current time level $t + \Delta t$, except for the inflow flood hydrograph, which is known in advance. Accordingly, equation (5) can be rewritten as a function of unknown variables [11]:

$$V(t+\Delta t) = C - \frac{\Delta t}{2} Q_{out}(t+\Delta t) \tag{6}$$

where

$$C = V(t) + \frac{\Delta t}{2}\left[Q_{in}(t+\Delta t) + Q_{in}(t) - Q_{out}(t)\right] \tag{7}$$

In equation (6), three unknowns appear: the reservoir volume $V(t+\Delta t)$ and the outflow discharge $Q_{out}(t+\Delta t)$, both of which are functions of the unknown reservoir water level $Z(t+\Delta t)$. To determine these three unknowns, three equations are available: equation (6), and the functional relationships $V(t+\Delta t) = V(Z\ (t+\Delta t))$ and $Q_{out}(t+\Delta t) = Q_{out}(Z\ (t+\Delta t))$. This system of equations is nonlinear, and its solution requires the application of an appropriate numerical method.

3.3. COMPUTATIONAL SCHEME FOR THE ULOG RESERVOIR

The previously presented mathematical and numerical models of flood wave transformation are applicable to the Ulog reservoir with the additional definition of specific variables, which are illustrated in Figure 3. For the analyzed reservoir, the continuity equation, i.e., the mass balance equation, can be expressed as:

$$\frac{\Delta V}{\Delta t} = \bar{Q}_{in} - \bar{Q}_{HPP} - \bar{Q}_{SHPP} - \bar{Q}_{sp} - \bar{Q}_{bo} \tag{8}$$

where:
$\Delta V = V(t+\Delta t) - V(t)$ represents the change in storage volume during the computational time interval Δt;
$\bar{Q}_{in} = \frac{Q_{in}(t+\Delta t)+Q_{in}(t)}{2}$ is the average inflow discharge during the considered time interval;
$\bar{Q}_{*} = \frac{Q_{*}(t+\Delta t)+Q_{*}(t)}{2}$ is the average outflow discharge during the considered time interval, where the indices denote:
HPP – hydropower plant, $SHPP$ – small hydropower plant, sp – gated spillway, bo – bottom outlet.
According to Equation (8), the total outflow discharge from the reservoir is:

$$\bar{Q}_{out} = \bar{Q}_{HPP} + \bar{Q}_{SHPP} + \bar{Q}_{sp} + \bar{Q}_{bo} \tag{9}$$

It should be emphasized that this outflow discharge is not equivalent to the discharge at the Ulog HPP downstream profile, which is increased by lateral inflows from the catchment. Furthermore, since the facility is a diversion-type hydropower plant, the discharge immediately downstream of the dam is given by:

$$\bar{Q}_{out,\mathrm{DAM}} = \bar{Q}_{SHPP} + \bar{Q}_{sp} + \bar{Q}_{bo} \tag{10}$$

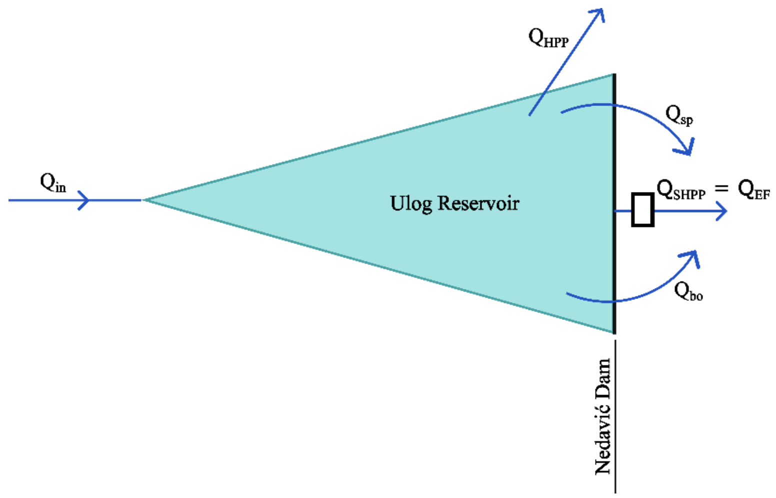

Figure 3. Computational scheme of flood wave transformation in the Ulog reservoir (drawing by author).

With regard to the outflow discharge and its regulation, the available data for this study included the discharge curve of the bottom outlet for a fully opened gate, as well as the discharge curve of the frontal spillway as a function of the reservoir water level and gate opening. These curves were obtained from the technical documentation and management plan defined in the respective project study [10].

The discharge through the hydropower plant, considering the use of Francis turbines, ranges from half of the installed discharge of a single turbine (8.75 m^3/s) to the total installed discharge of the hydropower plant (35 m^3/s). During operation, the small hydropower plant operates at its installed discharge of 0.52 m^3/s.

The inflow hydrographs to the reservoir, i.e. the flood waves, were also adopted from the project's technical documentation [10].

4. RESULTS AND DISCUSSION

4.1. PRE-RELEASE ANALYSIS

Pre-release operations depend on hydrological forecasts, operational rules, and catchment characteristics, and must be carefully coordinated with other reservoir users, such as water supply, irrigation, and power generation, as excessive lowering of the reservoir level may negatively affect these systems. It is assumed that there are no limitations on the daily rate of water level drawdown. Pre-release is performed from the normal operating level NOL = 641 m a.s.l. which is equal to maximum operating level MOL, down to selected elevations, with the spillway crest elevation being SCE = 633.5 m a.s.l. and minimum operating level minOL = 622 m a.s.l.

The following scenarios are considered:

- Scenario 1 – Ulog HPP and SHPP units operating at full installed capacity – energy-efficient pre-release;
- Scenario 2 – Ulog HPP and SHPP units, together with the bottom outlet, operating at full installed capacity – accelerated pre-release;
- Scenario 3 – Ulog HPP and SHPP units and the bottom outlet operating at full installed capacity, with partially opened spillway gates – rapid pre-release.

For the first scenario, the considered reservoir inflows were the long-term average river discharge Q_{avg} = 11.56 m^3/s and Q = 20 m^3/s. For the second scenario, the inflows were Q_{avg} = 11.56 m^3/s, Q = 20 m^3/s, and Q = 30 m^3/s, and for the third scenario considered inflow was Q = 40 m^3/s.

The gate opening increment was set to 0.75 m per hour, with the following gate openings considered: 0.75 m, then 1.5 m (where the gate opens to 0.75 m during the first hour and to 1.5 m during the second hour, remaining at that level thereafter). In the same manner, a gate opening of 2.25 m was considered, achieved in the third hour and then maintained, as well as a gate opening of 3.00 m, achieved in the fourth hour and then maintained.

Pre-release diagrams for the analyzed scenarios are presented in the following figures. The results indicate that, in the second scenario, with the bottom outlet operating at full capacity, the time required to lower the reservoir to the characteristic elevations is reduced by approximately 50% compared to the first scenario.

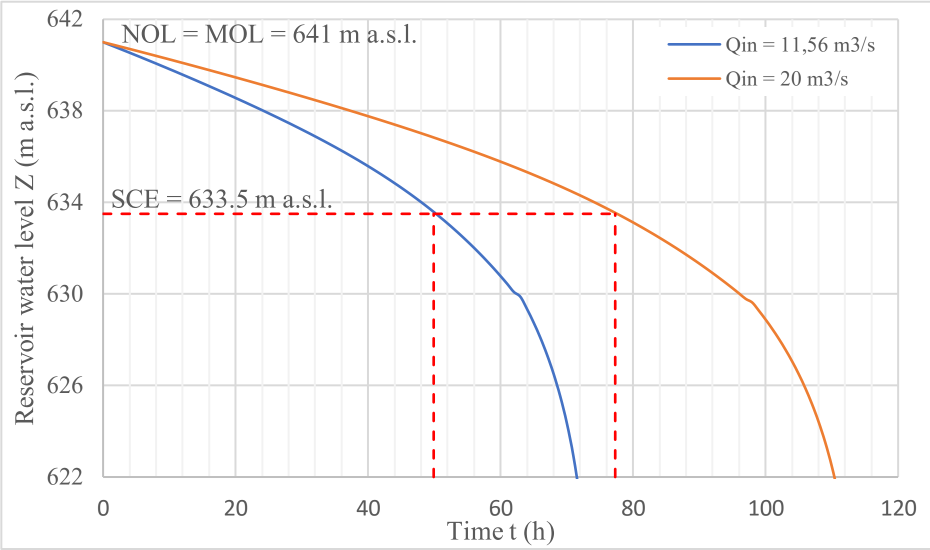

Figure 4. Reservoir pre-release diagram – first scenario (created by author).

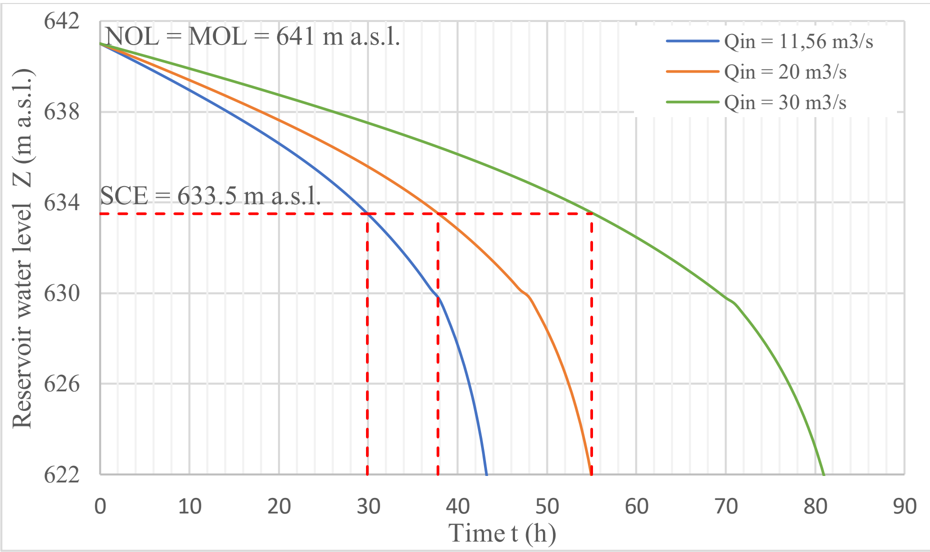

Figure 5. Reservoir pre-release diagram – second scenario (created by author).

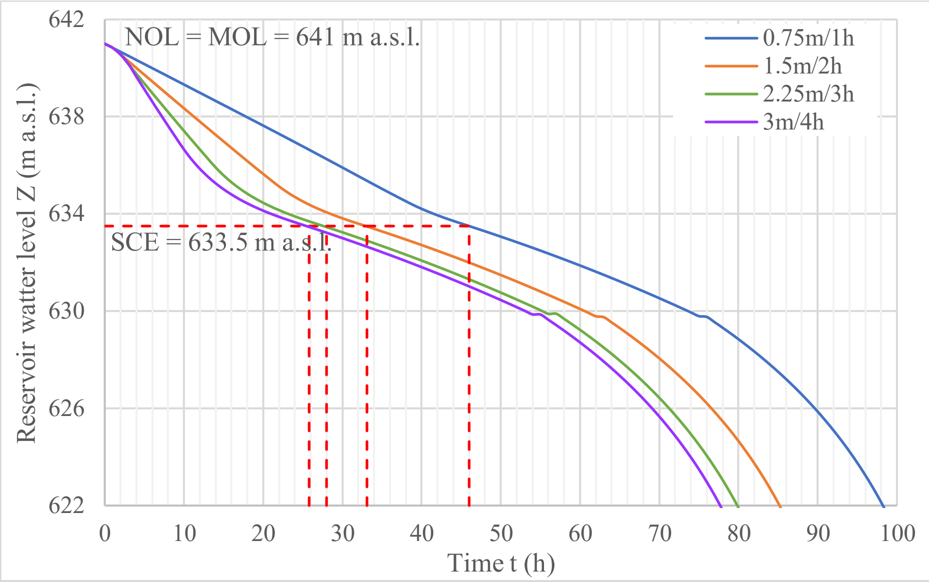

Figure 6. Reservoir pre-release diagram – third scenario (created by author).

4.2. RESULTS OF FLOOD WAVE TRANSFORMATION IN THE RESERVOIR

The transformation of flood waves in the reservoir was carried out using the previously established mathematical and numerical model. The analysis of flood wave attenuation potential was performed for adopted flood waves with return periods of 10, 20, 50, and 100 years. In order to assess the impact of operational decisions and the potential use of the Ulog reservoir for active flood protection, three different initial reservoir water levels were considered: 622 m a.s.l., 633.5 m a.s.l., and 641 m a.s.l., corresponding to the previously defined characteristic water levels.

The calculations were performed under the following conditions and assumptions:

- No limitation on the daily drawdown of the reservoir water level;
- Minimum reservoir water level minOL = 622 m a.s.l.;
- Maximum reservoir water level NOL = MOL = 641 m a.s.l.;
- The operation strategy aims to utilize inflow as much as possible for hydropower generation;
- Gate opening increment is 0.50 m;
- During operation of the bottom outlet, the gate is fully open;
- The small hydropower plant operates only when neither evacuation structures nor the spillway nor the bottom outlet are active, and operates at the installed discharge of 0.52 m³/s, corresponding to the environmentally acceptable flow;
- The operation strategy aims to minimize the discharge through evacuation structures;
- The operational strategy aims to ensure that the rate of change (increase) of the outflow discharge is lower than the rate of change of the inflow flood wave
- The operation strategy aims to achieve a reservoir water level at the end of the analyzed period approximately equal to NOL = MOL = 641 m a.s.l.

In the following, the transformation of flood waves and the variation of the reservoir water level for the considered initial reservoir levels are graphically presented. It is important to note that in the presented diagrams, Q_{out} refers to the outflow discharge from the reservoir defined by Equation (9), and not to the discharge immediately downstream of the dam, which is defined by Equation (10).

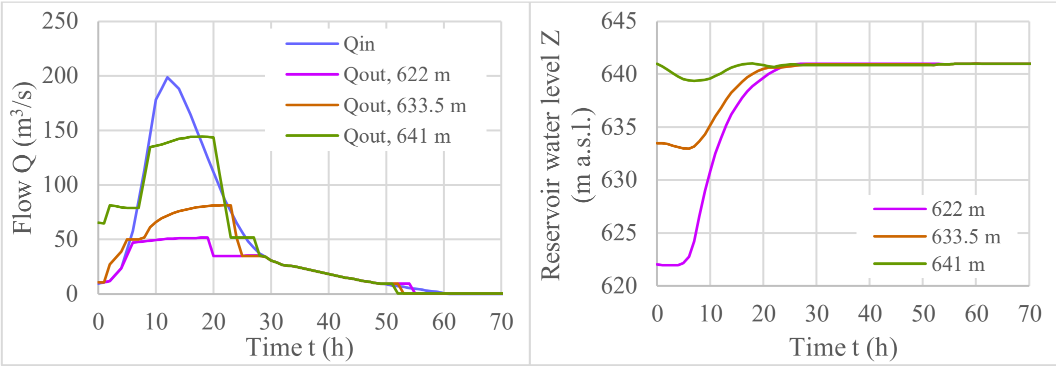

Figure 7. Flood wave transformation (left) and reservoir water level variation (right) for a 10-year return period (created by author).

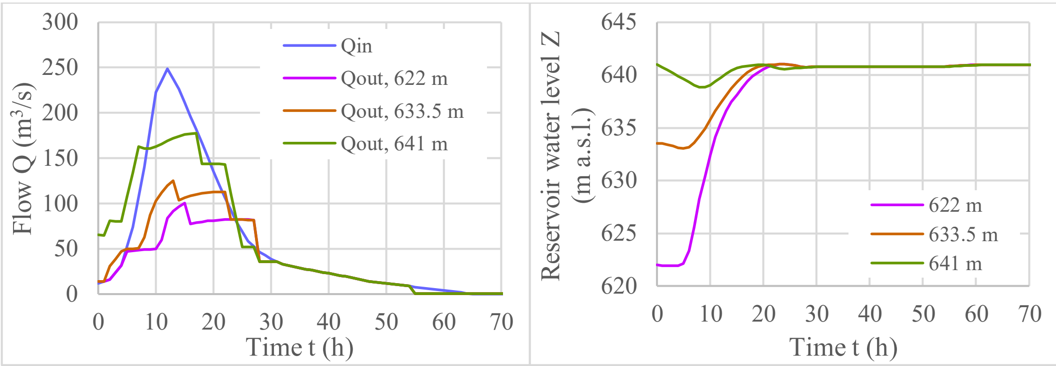

Figure 8. Flood wave transformation (left) and reservoir water level variation (right) for a 20-year return period (created by author).

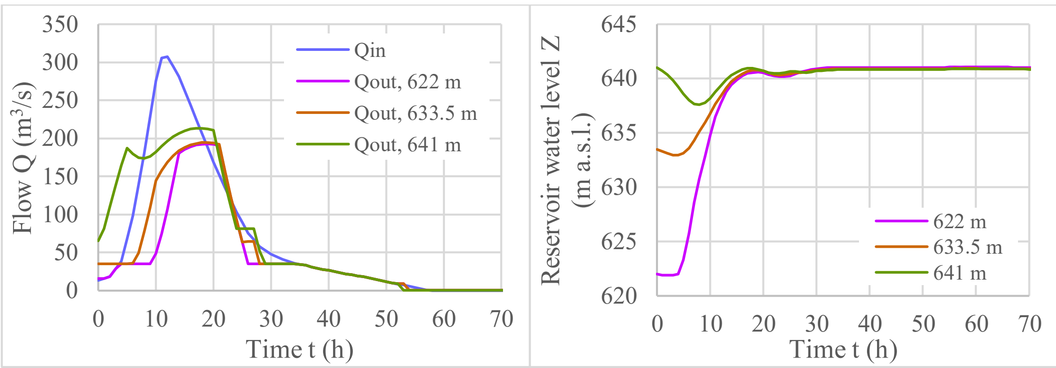

Figure 9. Flood wave transformation (left) and reservoir water level variation (right) for a 50-year return period (created by author).

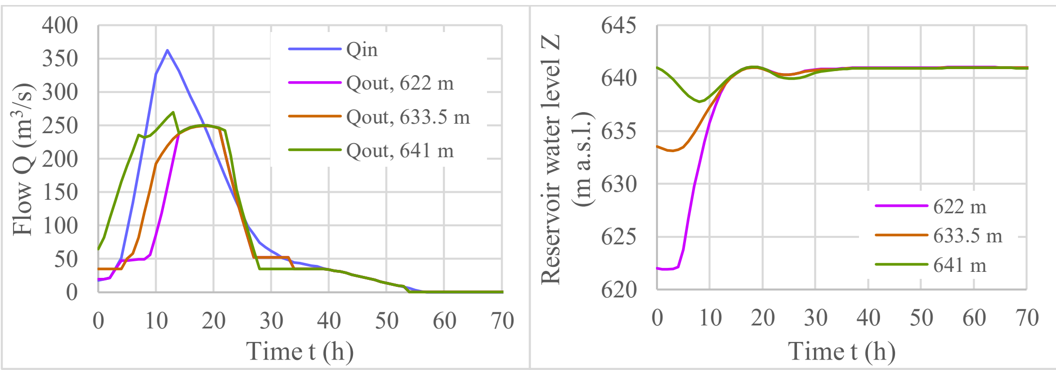

Figure 10. Flood wave transformation (left) and reservoir water level variation (right) for a 100-year return period (created by author).

The calculation results indicate that it is not possible to retain the analysed flood waves solely by operating the hydropower plant units, without additional spilling and releases through the bottom outlet. For a clearer presentation of the results, the following figure shows the maximum inflow discharge and the maximum outflow discharges for the analysed return periods. The outflow discharges shown in the figure refer to the total outflow discharge defined by Equation (9) and the discharge immediately downstream of the dam defined by Equation (10).

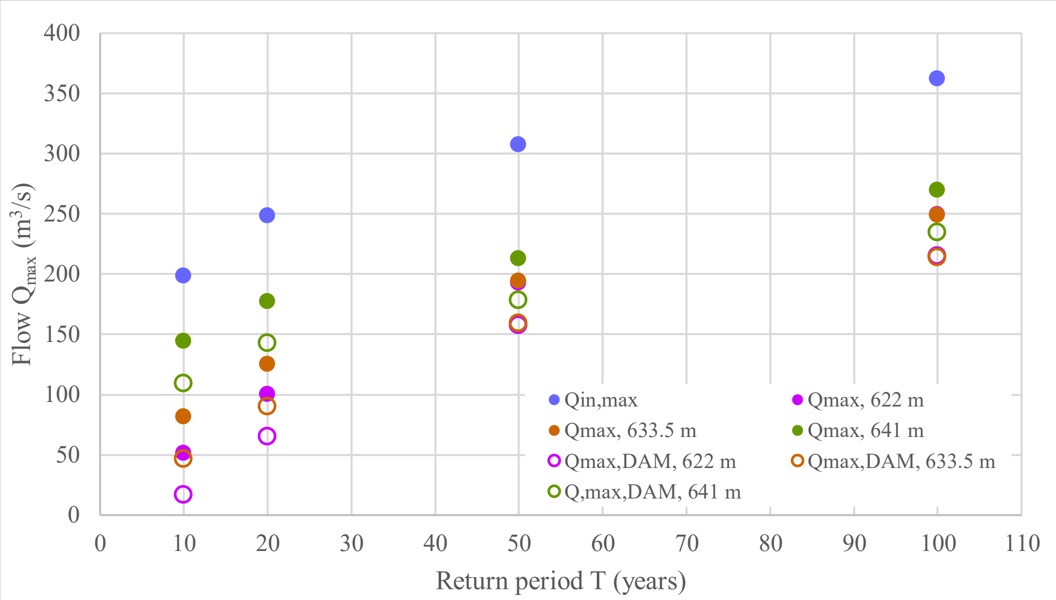

Figure 11. Maximum inflow and outflow discharges (created by author).

The peak discharge of the inflow flood wave with a 10-year return period amounts to 198.7 m³/s, while the maximum outflow discharges range from 51.7 m³/s to 144.4 m³/s, depending on the initial reservoir water level. This corresponds to a reduction of the peak discharge between 74% and 27.4%, which is considered significant. The maximum discharge immediately downstream of the dam ranges from 16.7 m³/s to 109.4 m³/s, representing a reduction between 91.6% and 45%. During the transformation of this flood wave, the bottom outlet was active for all analysed initial reservoir water levels. Additionally, for the initial water level of 633.5 m a.s.l., the gated spillway was also in operation with a maximum gate opening of 0.5 m, while for the initial water level of 641 m a.s.l. the maximum gate opening was 1.5 m.

During the transformation of the flood wave with a 20-year return period, both the bottom outlet and the gated spillway were in operation, with maximum gate openings of 1.0 m, 1.5 m, and 2.0 m for initial reservoir water levels of 622 m a.s.l., 633.5 m a.s.l., and 641 m a.s.l., respectively. The peak inflow discharge amounts to 248.7 m³/s, while the achieved reduction of the outflow discharge ranges from 59.6% to 28.6%, resulting in maximum outflow discharges between 100.4 m³/s and 177.5 m³/s. The maximum discharges at the dam profile range from 65.4 m³/s to 142.5 m³/s, depending on the initial reservoir water level, corresponding to a reduction between 73.7% and 42.7%.

The peak discharge of the flood wave with a 50-year return period amounts to 307.3 m³/s, with achieved reductions ranging from 37.3% to 30.6%. The corresponding outflow discharges range from 192.7 m³/s to 213.2 m³/s, depending on the initial reservoir water level. The discharge immediately downstream of the dam ranges from 157.7 m³/s to 178.2 m³/s, resulting in a reduction between 48.7% and 42%. During the transformation of this flood wave, the gated spillway was in operation with a maximum gate opening of 2.5 m, while the bottom outlet was active only for the initial reservoir water level of 641 m a.s.l.

For the flood wave with a 100-year return period, both the bottom outlet and the gated spillway were required to be in operation for all three analysed initial reservoir water levels. The maximum gate opening was 3.5 m for the initial water level of 641 m a.s.l., while for the remaining initial water levels the maximum gate opening was 3.0 m. A reduction of the outflow discharge between 31.1% and 25.6% was achieved, with maximum outflow discharges ranging from 249.1 m³/s to 269.6 m³/s, while the peak inflow discharge amounts to 362.4 m³/s. The discharge at the dam profile ranges from 214.1 m³/s to 234.6 m³/s, depending on the initial reservoir water level, corresponding to a reduction between 40.7% and 35.3%.

It is interesting to note that for the 50-year return period flood wave and initial reservoir water levels of 622 m a.s.l. and 633.5 m a.s.l., similar maximum outflow discharges are obtained, although the volumes of the outflow hydrographs differ. A slightly larger flood wave volume occurs for the initial reservoir water level of 633.5 m a.s.l., which is expected. The same behavior is observed for the 100-year return period flood wave. This is evident from the presented flood wave transformation diagrams as well as from Figure 11. Furthermore, it is noticeable that the difference in maximum outflow discharge for the initial reservoir water level of 641 m a.s.l. compared to the other two initial levels is considerably smaller for flood waves with higher return periods. This difference is approximately 20 m^3/s for both the 50-year and 100-year return period flood waves, which is relatively small compared to the magnitude of the maximum outflow discharges.

The above results indicate that it is not necessary to maintain lower reservoir water levels during periods when flood waves of lower probability are expected. At the mean annual discharge of 11.56 m^3/s, it is possible to lower the reservoir water level from 641 m a.s.l. to 633.5 m a.s.l. in approximately 2.1 days solely by operating the hydropower plant and small hydropower plant units, or in about 1.2 days if the bottom outlet is also used. Even when the reservoir inflow is higher, for example 40 m^3/s, it is possible to lower the reservoir water level from the normal operating level to 633.5 m a.s.l. within 1.1 to 1.9 days by operating the hydropower plant units and the evacuation structures, depending on the degree of gate opening at the gated spillway.

The reduction of the reservoir water level is considered only down to 633.5 m a.s.l., since similar results are obtained for this initial level as for lower levels. With adequate precipitation forecasts and simulation-based mathematical models for flood wave generation, appropriate operational decisions can be made and timely pre-release of the reservoir can be performed. It is evident that effective flood mitigation and rational use of water resources require the implementation of an integrated monitoring and decision-support system.

The results of flood wave transformation for the 10-year and 20-year return periods show that the initial reservoir water level has a greater influence on flood wave attenuation. The difference in maximum outflow discharge between initial levels of 622 m a.s.l. and 633.5 m a.s.l. is approximately 25 m^3/s and 30 m^3/s, while the difference between 633.5 m a.s.l. and 641 m a.s.l. is about 52 m^3/s and 63 m^3/s for the 10-year and 20-year return periods, respectively, which is significant. It should be noted that for the initial reservoir water level of 641 m a.s.l., even lower maximum outflow discharges could be achieved if a higher rate of increase of the outflow hydrograph were allowed compared to the inflow hydrograph. Nevertheless, it is still not necessary to maintain reservoir water levels below the normal operating level, as timely pre-release of the reservoir can be performed, provided that appropriate forecasting and simulation models are available.

For easier interpretation of the results, a summary of the main findings is presented in tabular form below.

Table 1. Summary of flood wave transformation results for different return periods and initial reservoir water levels

T (years)	$Q_{in,max}$ (m^3/s)	$Z_{initial}$ (m a.s.l.)	$Q_{out,max}$ (m^3/s)	$Q_{out,DAM,max}$ (m^3/s)	Reduction – Q_{out} (%)	Reduction – $Q_{out,DAM}$ (%)
10	198.7	622	51.7	16.7	74.0	91.6
		633.5	81.5	46.5	59.0	76.6
		641	144.4	109.4	27.4	45.0
20	248.7	622	100.4	65.4	59.6	73.7
		633.5	125.1	90.1	49.7	63.8
		641	177.5	142.5	28.6	42.7
50	307.3	622	192.7	157.7	37.3	48.7
		633.5	194.5	159.5	36.7	48.1
		641	213.2	178.2	30.6	42.0
100	362.4	622	249.8	214.8	31.1	40.7
		633.5	249.1	214.1	31.3	40.9
		641	269.6	234.6	25.6	35.3

The analysis indicates that the efficiency of the reservoir in attenuating peak discharges decreases with increasing flood return period. While the initial reservoir water level plays a significant role for flood waves with return periods of 10 and 20 years, differences in maximum outflows become less pronounced for 50 and 100-year floods. This points to the limited capacity of the reservoir for active flood mitigation during extreme events and highlights the importance of timely operational decisions and reliable inflow forecasts.

5. CONCLUSION

This study analyzed the HPP Ulog system within the Neretva River basin as a factor in active flood protection. The objective was to perform flood wave transformation while considering existing constraints and infrastructure capacities, aiming to minimize outflow discharge and maximize the energetic utilization of available water resources. Based on the conducted analyses, the following conclusions were drawn:

- It is not possible to fully retain the analyzed flood waves by operating only the Ulog HPP and SHPP units. This is expected, as the installed discharge capacity of the Ulog HPP (35 m^3/s) is only about three times higher than the mean annual discharge of 11.56 m^3/s.
- During high-flow conditions, both the bottom outlet and the gated spillway must be operational. However, it is not necessary for these evacuation structures to operate simultaneously; instead, appropriate operational strategies should be defined to achieve the required flood attenuation.
- By applying suitable operational decisions, it is possible to reduce the outflow discharge by 25.6% to 74% relative to the peak inflow discharge, depending on the return period and the initial reservoir water level. Consequently, the discharge immediately downstream of the dam can be reduced by 35.3% to 91.6% relative to the maximum inflow discharge.
- Similar levels of flood attenuation can be achieved for an initial reservoir water level of 633.5 m a.s.l. and lower levels, particularly for flood waves of lower probability. Higher initial water levels result in slightly higher maximum outflow discharges and larger outflow volumes. Given that, at mean annual inflow conditions, pre-release of the reservoir from 641 m a.s.l. to 633.5 m a.s.l. can be achieved within approximately 2.1 days by operating only the HPP and SHPP units, it is not necessary to maintain reservoir water levels below the normal water level of 641 m a.s.l.
- The applied mathematical simulation model does not represent a tool for real-time operational control, but rather serves as a framework for analysing different reservoir management scenarios. The obtained results enable an assessment of the system sensitivity to initial reservoir water levels and the operation of evacuation structures, and provide a basis for defining management guidelines under high-flow conditions.
- The obtained outflow hydrographs do not represent the hydrographs at the Ulog HPP profile. Further research should include detailed analyses of flood wave propagation from the dam profile to the HPP profile, taking into account intermediate inflows, after which the outflow discharge from the Ulog HPP should be added to the resulting hydrograph.
- For the most efficient operation of this and similar systems, it is essential to establish a comprehensive monitoring and information system, accompanied by the development of advanced operational and decision-support software.

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