Role of Dams and Reservoirs in a Successful Energy Transition – Boes, Droz & Leroy (Eds)

Flood control across hydropower dams: The value of safety

Christina Ntemiroglou, Georgia-Konstantina Sakki & Andreas Efstratiadis
Department of Water Resources and Environmental Engineering, National Technical University of Athens, Greece

ABSTRACT: Hydropower reservoirs inherently serving as major flood protection infrastructures, are commonly occupied with gated spillways, to increase both their storage capacity and head. From an operational viewpoint, during severe flood events, this feature raises challenging conflicts with respect to combined management of turbines and gates. From the perception of safety, a fully conservative policy that aims to diminish the possibility of dam overtopping, imposes to operate the turbines in their maximum capacity and, simultaneously, opening the gates to allow uncontrolled flow over the spillway. Yet, this practice may have negative economic impacts from three aspects. First, significant amounts of water that could be stored for generating energy and also fulfilling other uses, are lost. Second, the activation of turbines may be in contrast with the associated hydropower scheduling (e.g., generation of firm energy only during peak hours, when the market value of electricity is high). Last, the flood wave through the spillway may cause unnecessary damages to downstream areas. In this vein, this fpaper aims to reveal the problem of ensuring a best-compromise equilibrium between the overall objective of maximizing the benefits from hydropower production and minimizing flood risk. In order to explore the multiple methodological and practical challenges from a real-world perspective, we take as example one of the largest hydroelectric dams of Greece, i.e., Pournari at Arachthos River, Epirus (useful storage 310 hm^3, power capacity 300 MW). Interestingly, this dam is located just upstream of the city of Arta, thus its control is absolutely crucial for about 25 000 residents. Based on historical flood events, as well as hypothetical floods (e.g., used within spillway design), we seek for a generic flood management policy, to fulfil the two aforementioned objectives. The proposed policy is contrasted with established rules and actual manipulations by the dam operators.

1 INTRODUCTION

According to recent statistical data by ICOLD, there exist more than 12100 large dams worldwide for hydropower generation. In particular, in the European Union, the majority of large dams (39%) are hydroelectric, while the share of hydropower over the total renewable electricity generation exceeds 40% (Wagner et al., 2019). By definition, all reservoirs, apart from their main water uses, they also contribute to the reduction of flood risk to downstream areas, since even under full storage conditions, inflow floods are attenuated. In about one third of large dams globally, particularly the hydroelectric ones, the spillway system also comprises control gates. This feature provides flexibility to the operators, since they allow to exploit the surplus inflows in order to increase both the storage and the head. On the other hand, the gate management turns to a quite complex task, since decisions are taken under pressure and highly uncertain conditions. Yet, a wrong policy may have significantly negative aspects, either in terms of economy or safety.

More specifically, the conflict between safety and economy arises from the controversial role of spillway gates. For instance, an unnecessary or too early open of gates during an

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incoming flood event may result to quite important losses of potential hydropower production. On the other hand, a too late open may cause significant flood damages downstream, and under extreme conditions also put the dam itself under the risk of overtopping. We underline that in the case of hydroelectric reservoirs, the overall flood management also includes the emergent activation of turbines to release the surplus water, which may be in contrast with the associated hydropower scheduling (Efstratiadis et al., 2021).

The real-time gate management during flood events is recognized as a multiobjective problem of major complexity. This is typically expressed through multistage opening rules, by taking advantage of real time monitoring data and, occasionally, some kind of flood forecasting inputs (e.g., Sordo-Ward et al., 2017; Nematzadeh and Hassanzade, 2021; Albo-Salich and Mays, 2021; Soriano et al., 2022; Salehi et al., 2022). Their definition is addressed either through simulation or optimization. Common simulation-based tools are the so-called Volumetric Evaluation Method (Giron, 1988) and its variants, e.g., the K-Method (Sordo-Ward et al. 2017). All these are based on specific assumptions about the reservoir management and do not employ optimization.

On the other hand, optimization methods aim at determining suitable flood control policies (e.g., Bagis and Karaboga, 2004; Karaboga et al., 2008). These mainly account for safety criteria to ensure minimal risk of dam overtopping and flood damages across downstream areas. Yet, only few methods highlight the impacts of flood routing to hydropower production, which is the main economic scope of hydroelectric reservoirs (Zargar et al., 2016; Liu et al., 2017). Furthermore, in most of literature approaches, the optimization does not explicitly incorporate the management of turbines (e.g., Jordan et al., 2012), since emphasis is given to the spillway gates.

Key issue is the definition of flood conditions for optimizing the operational rules. The typical case is to use either a set of historical flood events (e.g., Malekmohammadi et al., 2009; Jordan et al., 2012; Feng and Liu, 2014; Nematzadeh and Hassanzade, 2021; Liu et al., 2017) or empirically-defined flood hydrographs that correspond to specific return periods (e.g., Salaehi et al., 2022). These may range from medium-frequency events (e.g., few decades) up to extreme ones that are used in the hydrological design of spillways, also including the so-called Probable Maximum Flood. There also exist some cases where a mixing of historical and empirical hydrographs is applied (e.g., Chou and Wu, 2015; Zargar et al., 2016; Amirkhani et al., 2016).

There are also few attempts that make use of synthetic inflows that are derived through stochastic approaches. For instance, the flood events by Bianucci et al. (2015) have been obtained by coupling the RainSimV3 synthetic rainfall time series model with a deterministic hydrological model. Soriano et al. (2022) use a cascade of three models. First, they generated long time series of precipitation data at the sub-daily scale, which are inputs to a continuous hydrological simulation model. Finally, the sample of annual maximum hydrographs have been considered as inflow hydrographs to a reservoir simulation model that employed the VEM.

In this research, we develop a more comprehensive simulation-optimization context that seeks for an equilibrium between safety and economy across a wide range of flood events. The decision variables are expressed in terms of a small set of characteristic reservoir stage values, which are mapped to spillway gates opening and turbine activation. On the other hand, the objective function accounts for the amount of potentially energy loss due to water releases through the spillway (economy criterion) and the distance of the maximum flood level from the dam crest and other crucial levels, such as the top of gates (safety criterion).

As a proof of concept, we investigate the flood control policy of Pournari dam, at Arachthos River, Epirus. The combined hydropower production and flood management problem is highly challenging, since the dam is located just upstream an urban area, i.e., Arta. Taking as example a number of historical and synthetic flood events, we optimize its operational rules and contrast them with the running ones.

2 STUDY AREA

2.1 *Brief description and characteristic quantities*

In order to explore the multiple methodological and practical challenges of the spillway gate control problem from a real-world perspective, we take as example one of the largest hydroelectric system of Greece, i.e., Pournari, at Arachthos River, Epirus. The system was established in 1980 and operates by the Public Power Corporation (PPC). The location of the dam, just upstream the city of Arta, is shown in Figure 1. The drainage area upstream of the dam is 1794 km^2, with average altitude +854.0 m and average slope 25% (Koutsoyiannis et al., 2010). The mean annual inflow during years 1981-2021 is 50 m^3/s (1580 hm^3).

Pournari complex comprises an earthfill dam with central clay core, of 107 m height and maximum length of 580 m, that creates a reservoir area of up to 20.6 km^2. The spillway control is made by three arched gates of 12.5 m width and 12.5 m height each one (total width 37.5 m), and total outflow capacity of 6100 m^3/s. The gross storage capacity at the spillway crest level (+107.5 m) is 505 hm^3, yet it reaches 885 hm^3 at the maximum level of +120.0 m. According to the design study of the system, the maximum flood level is at +125.5 m, namely 1.5 m lower than the top of the dam (+127.0 m).

The hydropower station is equipped with three Francis-type turbines of total power capacity 300 MW, that produce 437 GWh, on mean annual basis. The reservoir produces peak energy, and operates four hours per day, on average. Yet, during the summer period, the turbines are operating up to eight hours. Since their discharge capacity is 500 m^3/s, the total discharge that can be conveyed through the spillway and the turbines is up to 6600 m^3/s.

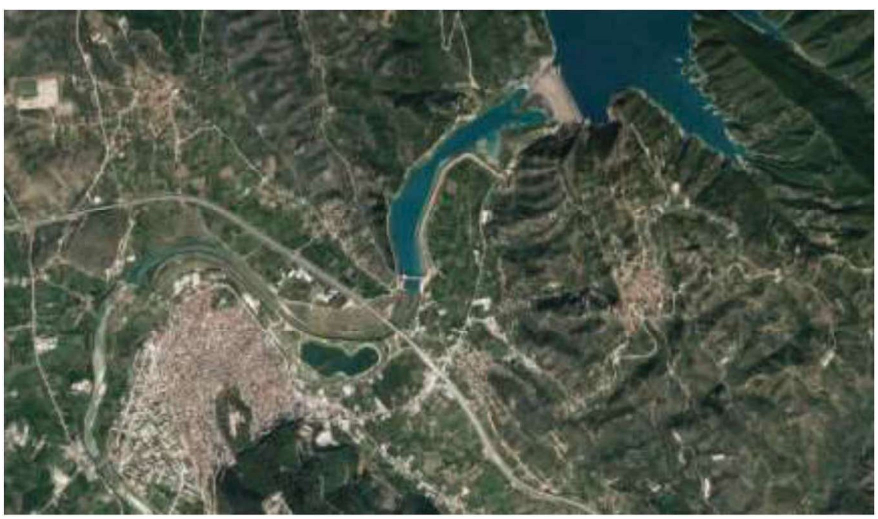

Figure 1. Location of Pournari Dam, just upstream of the city of Arta (Source: Google Earth).

2.2 *The flood management challenge*

In general, the overall flood management policy of the large Greek dams operated by the PPC focuses on three objectives, i.e., protection of river side areas, safety of hydroelectric installations, and maximization of hydropower generation (Leris, 2008). Their implementation in practice is based on the expertise of each dam staff and empirical manipulations that are made in real time.

The case of Pournari dam is the most challenging, since this is located just upstream of an urban area, namely the city of Arta. In this vein, its flood control is absolutely crucial for the safety of 25 000 residents, as well as the protection of the historical stone bridge of 17th century, which is a worldwide recognized heritage monument. Currently, the sole specific constraint is an alarm stage at +118.0 m, which implies the full opening of the three gates. We remark that formerly, this stage was higher, namely at +120.0 m (top of gates). Yet, this rule was abandoned because it was too difficult and highly uncertain, and thus risky, to open the gates under such strong hydrostatic pressure, while the time of reaction in case of malfunction would be extremely limited. As result of this more conservative policy, the system losses a retention capacity of about 40 hm^3 (i.e., storage difference between +118.0 to +120.0 m), which corresponds to a potential energy loss of up to 8.2 GWh (about 2% of mean annual energy production).

2.3 *The disastrous flood events of 2005 and 2015*

The floods of December of 2005 and January 2015 are of significant interest, since they are considered as two of the most disastrous events during the lifetime of the dam. Their evolution, as well as the evolution of the reservoir level, are shown in Figures 3 and 4. The accumulated inflows during the two events were estimated at 222 and 262 hm^3, respectively, and the associated peak flows are estimated up to about 1700 and 2100 m^3/s, respectively (Roilos, 2018).

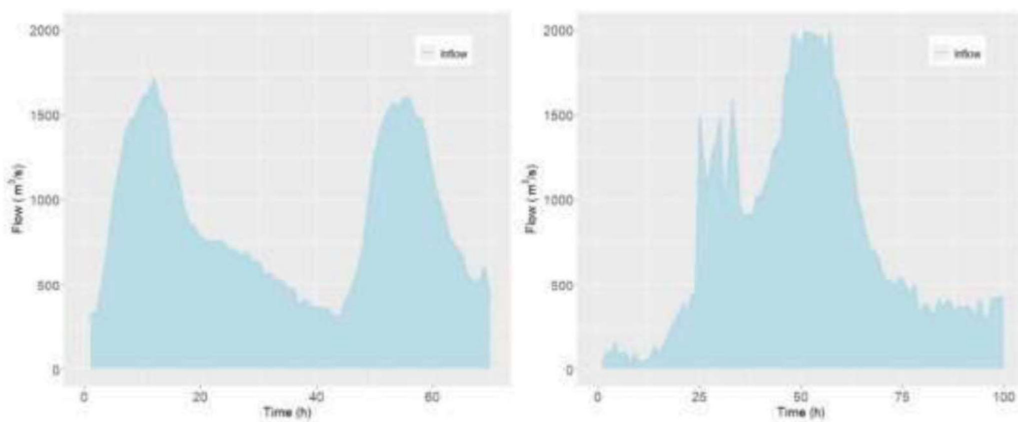

Figure 2. Evolution of inflows during the flood events of 2005 (left) and 2015 (right).

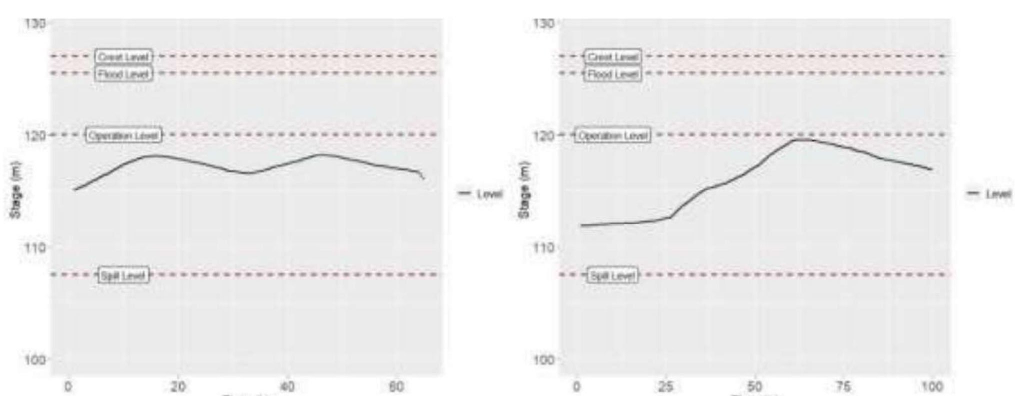

Figure 3. Evolution of reservoir level during the flood events of 2005 (left) and 2015 (right).

The first event lasted from 28/12/2005 to 1/1/2006, and initiated when the reservoir was at a quite high level of 115.5 m. As shown in Figure 3, left, the inflow hydrograph exhibited two almost equal peaks within a 40 hours interval. We remark that double peak floods are particularly difficult to handle, and rarely only accounted for in the design context (Gioia, 2016). The afternoon of 27/12/2005, by facing a sharp increase of the reservoir level by 20 cm per hour, the dam operators adopted a quite conservative policy. In particular, when the reservoir level reached +118 m, they partially opened the spillway gates to release 300 m^3/s (800 m^3/s in total, also considering other 500 m^3/s from the turbines). Grace to this manipulation, they ensured the safety of the dam and its installations, besides the unexpected arrival of a subsequent, equally extreme, event (Leris, 2008). On the other hand, from a power production perspective, the adopted policy was clearly sub-optimal, as the estimated loss of energy was up to 7.5 GWh.

The second flood event lasted from 30/1/2015 to 2/2/2015 and produced an even higher flood peak (2050 m^3/s) and flood volume, as well. Although the reservoir level at the beginning of the flood was quite lower than in 2005 (113.0 m, in contrast to 115.5 m), and although the operators opened the gates at the emergent level of +118.0 m, the flood eventually reached +119.5 m, thus close to the top of the gates.

3 METHODOLOGY

3.1 *Input data, governing equations and assumptions*

Let a hydroelectric reservoir, the spillway control system of which comprises n_G similar gates of height h_g, which are installed over an ogee-type crest, at elevation z_c. The reservoir's geometry is described through a power-type storage-elevation function, i.e.:

$$s = \kappa(z - z_0)^{\lambda} \tag{1}$$

where z is the reservoir level, z_0 is a datum level (e.g., minimum pool level or river bed elevation) and κ, λ are constants. On the other hand, the energy production is expressed as:

$$E = \psi\ r\ (z - z_\mathrm{p}) \tag{2}$$

where r is the water release through the turbines (for convenience, we apply lowercase letters for water volumes and capital ones for fluxes), z_p is the power station level (penstock outlet), and is the so-called specific energy, which is defined as follows:

$$\psi = \rho\ g\ \eta\ h_\mathrm{n}/(z - z_\mathrm{d}) \tag{3}$$

where ρ is the water density (1000 kg/m^3), g is the acceleration of gravity (9.81 m/s^2), η is the electromechanical system's efficiency (overall efficiency across turbines, generators and transformers), and h_n is the net head, namely the available hydraulic energy at the turbines (i.e., gross head minus friction and minor losses across the conveyance system). In the generic case, ψ is a nonlinear function of head and discharge, but under some premise (e.g., turbine operation close to its nominal capacity) is can be handled as approximately constant. Its theoretical maximum is 0.2725 GWh/hm^4, are refers to an ideal hydropower system of unit efficiency and zero hydraulic losses (Efstratiadis et al., 2021).

The discharge capacity, which depends on the gross head, z - z_d, and the properties of the conveyance system (length, diameter, roughness, etc.), can be also approximated as:

$$U = \alpha(z - z_\mathrm{d})^{\beta} \tag{4}$$

where α and β are constants.

Finally, by considering fully open gates, the discharge over the spillway crest is given by:

$$Q = c \ \sqrt{2g} \ L \ h^{3/2} \tag{5}$$

where g is the acceleration of gravity, c is a discharge coefficient, which accounts for energy losses, and depends on the ogee shape and the head, L is the effective length of the spillway crest, and h is the upstream energy head above the spillway crest. If the top of the ogee is at elevation z_s, the head is estimated by:

$$h = z - z_s + V^2/2g \tag{6}$$

where v is the flow velocity at the entrance section, which is function of q. The combination of (5) and (6), as well as the nonlinearity of term c, which is typically expressed by means of empirically-derived nomographs (US Bureau of Reclamation, 1987), results into to quite complex numerical problem. This can be significantly simplified by writing the theoretical formula (5) as:

$$Q = c^* \sqrt{2g} \ L \ (z - z_s)^{\xi} \tag{7}$$

where c^* is an equivalent discharge coefficient, which may be assumed approximately constant, and ξ is a shape parameter that exceeds the theoretical value of 3/2, thus allowing to omit from the head function (6) the kinetic energy term, V^2/2g. Under this premise, the discharge over the ogee is explicitly expressed as function of the actual level, z.

If the gates are partially only opened, the discharge calculations become even more complex. In order to facilitate computations, this case is not examined here.

3.2 *Simulation model*

We consider the reservoir operation during a flood event, which is represented by a sequence of given inflows i_t. If n is the length of simulation, the reservoir dynamics is described through the water balance equation, written in the discretized form:

$$s_t = s_{t-1} + i_t - r_t - w_t \tag{8}$$

where r_t are the controlled releases through the turbines, w_t are the spill losses, and s_t is the reservoir storage at the end of time step t (all quantities are given in volume terms).

Starting from a given initial level z_0, which corresponds to a storage value, s_0, through eq. (1), the estimation of the unknown outputs r_t and w_t can be explicitly employed, by expressing the turbine and spillway control policies by means of parametric operational rules.

In particular, the turbine control is associated with two different operational modes, hereafter referred to as *normal* and *emergent*, which are determined through two characteristic level thresholds, z_{e_o} and z_{e_c}. During the rise of the flood, if the pool level exceeds the first threshold, i.e., zz_{e_o}, the system is set under emergent operation conditions, and thus the power station is forced to operate in its maximum capacity. In contrast, during the recession of the flood, when the level falls below the second threshold, thus $z < z_{e_c}$, the system returns to its normal operation.

Under normal conditions, the release policy follows a standard energy production schedule, by means of energy production targets, e_t^*, during the time period n. In the case of dams providing peak hydropower, target energy is set to zero, except for few hours per day. We highlight that the power production scheduling is independent of the flood control policy, since it is specified a priori, on the basis of the strategic management of the hydropower system. Hence, the targets e_t^* are inputs to simulation and not parameters to optimize, as made with thresholds, $z_{e_o}$and z_{e_c}.

Regarding the gate control, provided that the system comprises n_G gates, we introduce three thresholds, which are symbolized z_o^k, denoting a progressive opening of gates, and a generic threshold z_c for determining the closure of all gates. In particular, during the rising of the flood, each time the reservoir level exceeds the associated threshold z_o^k, the gates are appropriately manipulated to release a specific percentage, a_k, of the corresponding spillway capacity, which is estimated by the analytical formula (5) or its approximative expression (7). Under this premise, the outflow through the spillway at level z, where $z_o^k \geq z > z_o^{k+1}$, is given by:

$$w = a_k \; Q(z) \; \Delta t \tag{9}$$

We remark when the pool level exceeds the upper threshold $z_o^{n_G}$, all gates are opened, to allow operating the spillway at its full capacity, thus under free flow conditions ($a_{n_G} = 1$). Next, during the recession of the flood, when the pool level falls below, all gates are closed.

At the beginning of each time step, t, the model updates the reservoir level, z_t, thus determining whether the system is under normal or emergent conditions, and also estimates the turbine and spillway capacity, through eqs. (4) and (7), respectively. In the first case, all gates are closed, thus $w_t = 0$, while the water released through the turbines is adapted to fulfill the energy target, e_t^*, by applying eq. (2). If the system is under emergent conditions, the full capacity of turbines is used, and thus the water releases are set equal to $r_t = U \; \Delta t$, where Δt is the time interval of computations. Besides, and according to both the state of the flood (rising or falling) and the value of z_t with respect to associated thresholds, the model recognizes the state of the system, in order to employ the appropriate control of gates, namely progressive opening or closing.

3.3 *Performance metrics*

As already discussed, the flood control of hydroelectric reservoirs is subject to two conflicting criteria, namely safety and economy. The first one refers to two aspects, the safety of the dam, per se, and the safety of the floodplains downstream of the dam, which may be put in danger according to the intensity and duration of flows released through the spillway. The dam safety criterion is quantified by determining a set of characteristic elevation thresholds, and compute their distance from the maximum level reached during the flood event, as derived through the simulation model. These may include the dam crest elevation, the maximum flood level, as specified within the spillway design study, the top of the gates, etc. On the other hand, the floodplain safety criterion can be expressed is dimensionless terms, as the ratio of the maximum value of outflow to the discharge capacity of the spillway system.

Regarding the economy criterion, this also involves two aspects. The first is the deviation from the normal power generation scheduling, due to the operation of their turbines in their full capacity whenever the pool level exceeds the emergency threshold, z_{e_o}, thus the production of secondary instead of firm energy. The market value of secondary energy is lower than the firm one, which is some kind of long-term economic loss for the system. The unnecessary generation of surplus energy can be introduced within the economy metric by means of a small penalty term. Yet, the most important impact to its economic performance refers to the direct loss of water, and thus hydropower, due to the opening of gates. This quantity can be easily estimated, by computing the potential energy that would be produced by the overflowing water, i.e.:

$$E_{L,t} = \psi \; w_t(z_t - z_\mathrm{p}) \tag{10}$$

In order to provide an overall performance measure that ensures good equilibrium among the various safety criteria as well as the two economy-related criteria, it is essential to assign suitable weighting coefficients to the corresponding distance and energy metrics. The derived metric can be next set as the objective function of a global optimization problem, as discussed hereafter.

3.4 *Optimization*

The extraction of generic flood management rules, which are efficient across different conditions, both in terms of safety and economy, imposes to configure the optimization problem in a scenario-based context. In this vein, we must use as inputs a number of inflow data sets, and seek for an overall optimal policy, which maximizes the average performance measure across all scenarios. Crucial modelling decisions involve: (a) the formulation of inflow scenarios, either based on observed flood events or synthetically generated, (b) the assignment of the initial condition of the system, i.e., the reservoir level at the beginning of the flood, and (c) the definition of the normal operation scheduling of turbines (e.g., activation during peak demand hours). In order to simulate the system under a wide range of potential states, the optimization problem should run for combined scenarios of inflows, initial levels and hydropower production schedules.

An alternative option is to determine different rules according to the initial state of the system, which is an absolutely crucial input (cf. Gabriel-Martin *et al.*, 2019). In this respect, the optimized turbine and gate management are adapted to the reservoir level at the beginning of the flood event. Apparently, the higher is this level, the more conservative is expected to be the associate rule.

Regardless of the formulation of the objective function and the state scenarios, the underlying optimization problem comprises a set of parameters, i.e.: (a) the two turbine control thresholds, z_{e_o} and z_{e_c}, (b) the n_G gate opening thresholds, z_o^k, (c) the $n_G - 1$ spillway capacity ratios, a_k (we remind that at threshold, $z_o^{n_G}$, this ratio is by definition unit), and (d) the closing threshold for all gates, z_c. In this vein, the total number of parameters to optimize is $2(n_G + 1)$.

For convenience, the turbine operation in their full capacity is employed by priority with respect to gate opening, while the upper threshold $z_o^{n_G}$ cannot exceed the top level of the gates. Therefore, the level thresholds are forced to satisfy the sequence $z_{e_o} < z_o^1 < \cdots < z_o^{n_G} < z_c + h_g$ and $z_o^{n_G} > z_c > z_{e_c}$, while for the flow ratios we set $a_1 < \cdots < a_{n_G - 1} < 1$.

4 CASE STUDY

4.1 *Problem setup*

The proposed framework is applied to determine an optimal flood control policy of Arachthos dam. In this context, we run the optimization problem against four characteristic synthetic input hydrographs that correspond to return periods of 5, 10, 50 and 100 years (Figure 5). The data were retrieved from the flood engineering study by Koutsoyiannis et al. (2010), also involving: (a) the derivation of areal intensity-duration-frequency (also referred to as ombrian) curves over the upstream catchment, based on a comprehensive statistical analysis of observed rainfall maxima, (b) the construction of 48-hours design storm events for the aforementioned return periods, by applying the worst profile approach, and (c) the implementation of the NRCS-CN method to transform rainfall to flood runoff, (d) the routing of the derived runoff of the upstream area to the dam site, through the unit hydrograph theory, and (e) the addition of a constant baseflow, which is also considered as increasing function of return period.

The power plant scheduling imposes to release a constant discharge equal to nominal capacity of turbines (500 m^3/s), during peak electricity demand hours. Two time-blocks are considered, i.e., from 8:00 to 12:00 am and from 18:00 pm to 22:00 pm. Following the rationale of section 3.4, we formulate a large number of scenarios, by means of randomly selected input hydrographs with random time arrivals, and under random initial level values. We remark that the assignment of random time arrivals becomes essential, since the power production policy refers to specific intraday periods. Regarding initial levels, these range from the ogee crest elevation (+107.5 m) up to +118.0 m, which is the alarm stage imposed by the operator of the dam (see section 2.2).

The formulation of the optimization problem follows the principles of section 3.4. Since the dam is equipped with three gates (n_G = 3), the number of control variables to optimize is 8.

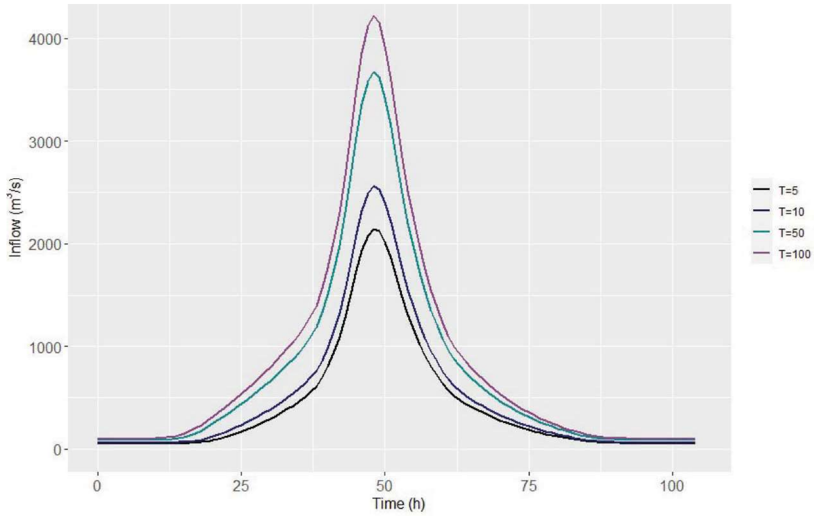

Figure 4. Input hydrographs for four characteristic return periods (T = 5, 10, 50 and 100 years).

4.2 *Optimized flood management policy*

The optimized management policy of the major flood control components of the dam (turbines and spillway gates) is expressed in terms of six characteristic level thresholds and two ratios, i.e.:

- If the reservoir level exceeds z_{e_o}= 117.84 m, the turbines are forced to operate in the maximum capacity, thus releasing 500 m^3/s in continuous time, while at the same elevation threshold ($z_o^1 = z_{e_o}$) the gates are manipulated to release a_1 = 3.3% of the associated discharge capacity of the spillway system.
- If the level further rises up to $z_o^2 =$ 118.45 m, the outflow ratio through the spillway increases to a_2 = 17.7%.
- The three gates are fully opened, thus establishing free flow conditions through the spillway, at level $z_o^3 =$ 119.20 m.
- Provided that the evolution of the flood event is clearly under recession, the three gates are closed when the level falls lower than $z_c =$ 117.65 m.
- The power generation system returns to its normal operational policy when the level is further decreased to $z_{e_c} =$ 116.70 m.

We remark that the activation level of +117.84 m is very close to the legal threshold of 118.0 m, which has been empirically established by the PCC. Yet, at this threshold the dam operator is forced to open all gates to operate the spillway in its full capacity, while our rule only considers a quite limited water release through the spillway, in order to save energy.

4.3 *Evaluation of optimized rules with respect to historical flood events*

In order to evaluate the optimized management policy in practice, and contrast it with the real-time manipulations by the dam operator in the field, we reconstruct the inflow events of 2005 and 2015 (section 2.3) and run the associated simulation model, by staring from the same initial level. The theoretical evolution of the reservoir stage compared to the real evolution as demonstrated in Figure 4. Summary information is also provided in Table 1.

This analysis further reveals the effectiveness and efficiency of the proposed policy. This allowed to release less water from the spillway, thus retaining significantly larger amounts of potential energy in the reservoir, and also causing less severe damages downstream. Focusing to the more severe event of 2015, our rules would result to a slightly higher maximum stage, while the risk of dam overtopping would remain negligible.

Table 1. Actual vs. simulated data by considering the flood events of 2005 and 2015.

	Flood 2005	Flood 2015
Pool level at the beginning of the flood (m)	115.06	111.83
Maximum observed inflow (m^3/s)	1712	2095
Maximum observed level (m)	116.79	119.55
Maximum simulated level (m)	118.21	119.66
Actual loss of energy (GWh)	12.0	6.4
Theoretical loss of energy (GWh)	3.3	1.0

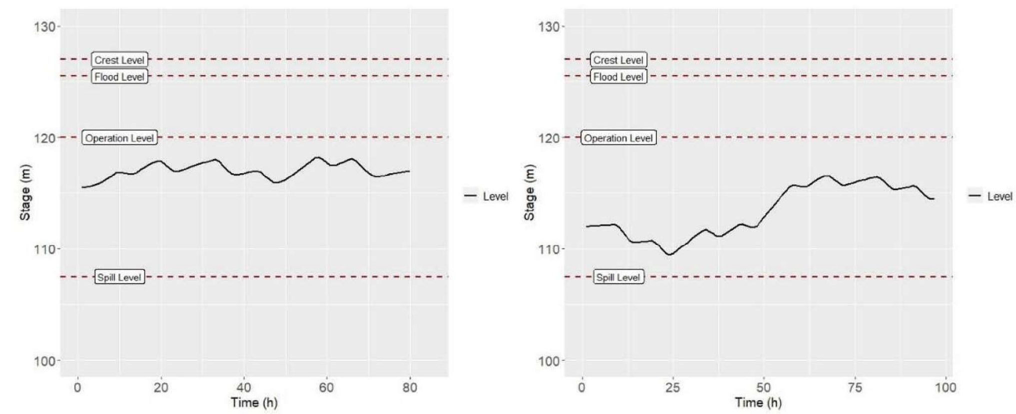

Figure 5. Simulated evolution of reservoir level, driven by the inflow hydrographs of 2005 (left) and 2015 (right), and by applying the optimized flood control policy.

5 CONLUSIONS

The optimal operation of hydroelectric dams that are equipped with gated spillways, during severe flood events, is a challenging problem, which is subject to the highly conflicting criteria of economy and safety. In this vein, we developed a generic simulation-optimization method that allows for establishing simple yet effective rules, for the conjunctive control of turbines and gates.

The control rules are expressed in terms of a small number of level thresholds and discharge capacity ratios. In order to ensure generality, the methodology is tailored in a scenario-based context, namely by formalizing a global optimization problem driven with a wide spectrum of potential states of the system, i.e., inflow hydrographs, arriving at random time stamps, and under varying pool level conditions. This accounts for the multiple aspects of the real-world management problem, namely the safety of the dam per se, and the downstream areas, as well, and the potential loss of energy due to water release through the spillway.

The effectiveness of the proposed context has been demonstrated in the case of Pournari dam, which is the most crucial of the country, in terms of flood risk awareness. Fruitful conclusions were obtained by contrasting the derived policy with its real-time implementation during the two most severe events of the time life of the dam so far. In fact, this would save quite large amounts of energy, also being marginally only riskier.

In terms of practical implementation, a strong advantage of the proposed operational policy is its simplicity. Actually, the sole external information to the dam staff is the reservoir stage, which is an easily measured quantity that can be manually retrieved through conventional instruments.

Potential improvements are twofold. First, the optimization may run with a large number of stochastically-generated flood events instead of few deterministically-derived hydrographs corresponding to specific return periods. We remark that the common engineering practice for producing "design" floods results to hydrographs of specific bell-type shape, as shown in Figure 5, while in reality a hydrograph shape is irregular (cf. Figure 3). This will make the method much more generic, since the less the number of scenarios examined and the more specific is the hydrograph shape, the more dependent are the optimized rules to the model inputs.

Second, the information provided to the dam operator could be substantially enhanced by also accounting for real-time monitoring data over the upstream river basin, as well as short-term hydrometeorological forecasting products (provided that the time response of the basin is large enough to take advantage of such information). This will allow to adapt the operational policy to the running conditions, thus ensuring an even better equilibrium between economy and safety.

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