Reservoir Operation Optimized for Hydropower Production Reduces Conflict with Traditional Water Uses in the Senegal River

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DOI
10.1061/(ASCE)WR.1943-5452.0001076

Publication date
2020

Document Version
Final published version

Published in
Journal of Water Resources Planning and Management

Reservoir Operation Optimized for Hydropower Production Reduces Conflict with Traditional Water Uses in the Senegal River

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Abstract: Manantali is a dam located on the Senegal River and is mainly used for hydropower production. Before the dam's construction, the annual river flood alimented the flood recession agriculture, a practice based on natural irrigation and fertilization of the flood plain, used traditionally by the local populations downstream. Analysis of the actual reservoir operation shows that annual floods have been largely reduced for the benefit of hydropower production. Moreover, the Senegal River Basin authority is evaluating the construction of different new dams, which could reduce even further the water available for flood support, given that the current operational focus is on satisfying hydropower demand. This study investigates the effects of an optimal reservoir operation strategy that maximizes hydropower production only, analyzing the results of this strategy in terms of effects on the two main objectives, i.e., hydropower production and flood support. The problem of finding optimal reservoir operation strategy is solved by applying the stochastic dual dynamic programming method. Results show the existence of a release strategy in which both objectives improve (+9% for hydropower and +7% for flood production) with respect to the historically observed operation. This solution, however, may require the electric system to compensate for the variability in energy supply along the year. **DOI: 10.1061/(ASCE)WR.1943-5452.0001076.** *© 2020 American Society of Civil Engineers.*

Introduction

The Senegal River is the second longest river in West Africa. Its drainage basin extension is 270,000 km^2, over Guinea, Mali, Senegal, and Mauritania. The basin can be divided into three parts: the upper basin, the valley downstream of Bakel, and the delta. The Senegal River has three main tributaries: Bafing, Faleme, and Bakoye, which together produce about the 90% of its flow. The natural discharge of the Senegal River is extremely variable, following the tropical rainfall seasonality, with a marked difference between the dry season, from January to June, and the rainy one, from July to October, when most rain falls in the mountainous upper basin (Albergel et al. 1997).

Manantali is a large dam located in the upper Senegal River, mainly used for hydroelectric production (Fraval et al. 2002). Manantali was completed in 1987 and started to produce electricity in 2003. Its benefits are shared among the riverine countries. The maximum reservoir volume of Manantali is 12×10^9 m^3, the installed capacity is 205 MW, and the average residence time is about 1 year. Manantali is mainly operated for hydropower purposes. Operating the reservoir for hydroelectric production reduces the annual river variability, with positive effects also for irrigated agriculture and navigation, which are expected to become more important uses in the future (OMVS 2011).

Operating the reservoir for energy production, however, is in conflict with flood support. Energy production, as currently operated, requires a regular release: before the flood peak, the water level in the reservoir is slowly drawn down, such that the flood peak can be absorbed in the reservoir and the flow is contained as long as possible below the capacity of the turbines. This is to avoid spillage releases. After the flood period and in the rest of the year, water is progressively released, such that the reservoir is maintained at high levels while producing sufficient electricity. On the contrary, flood support may require high flows, exceeding the capacity of the turbines, resulting in a loss of energy production.

Flood support requires an annual flood that provides favorable conditions for flood recession agriculture. Flood recession agriculture is a profitable production system used traditionally by the local populations, which does not require any purchased inputs: every time the floodplain is inundated and fertilized by the flood, vast areas are systematically sown with sorghum. Apart from agricultural production, the annual flood provides liveable conditions for forested ecosystems, livestock breeding, fish reproduction, and groundwater recharge (Varis and Fraboulet-Jussila 2002). Bader et al. (2003) mapped the trade-off between hydropower production and flood support. These solutions are integrated in the operational rules of Manantali, which recommends to produce an artificial flood by spilling part of the water from the reservoir every year in which the water level in the reservoir is sufficiently high (OMVS 2011). This artificial flood contributes, together with the inflow of other uncontrolled tributaries, to meet the flood support objective at Bakel (Fig. 2).

Notwithstanding the recommendations, the analysis of past reservoir operations seems to indicate that flood support objective received less priority than energy production. Fig. 1 presents the observed releases from Manantali for which data are available, i.e., from 1988 to 2011. Fig. 1 shows a regular annual peak of spillage release, indicating how the artificial flood has been produced

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Note. This manuscript was submitted on March 2, 2018; approved on November 27, 2018; published online on February 8, 2020. Discussion period open until July 8, 2020; separate discussions must be submitted for individual papers. This paper is part of the ***Journal of Water Resources Planning and Management***, © ASCE, ISSN 0733-9496.

every year until 2004. In 2002, Manantali was commissioned and started to produce electricity. The trajectory of releases after 2004 show that spillage release has been terminated, and turbine release has been kept regularly between 100 and 400 m^3/s, and always below the maximum turbine capacity, which is about 450–480 m^3/s. From the moment the reservoir was operative, in fact, producing an artificial flood would have subtracted water from energy production. The trend emerging from Fig. 1 indicates that given the current operational management of Manantali, the dam could be a driver of the alteration of the natural hydrological regime downstream. Presently, however, the potential consequences of depriving the Senegal valley of the traditional annual flood are only partially known.

The Senegal River and its basin face rapid changes (Fraval et al. 2002). A development plan under evaluation envisages the construction of new dams and development of new irrigation areas in the lower Senegal River valley. Fig. 2 indicates location and dimension of planned new dams and run-of-river dams that the Organisation pour la Mise en Valeur du Fleuve Sénégal (OMVS), the Senegal River Basin authority, is considering constructing.

The conflict between hydropower and traditional uses of water will be further exacerbated if new dams are built on this river. These new dams will enhance the controllability of the river flow, possibly reducing even further the water made available for flood support if this water use is not included in their operational management.

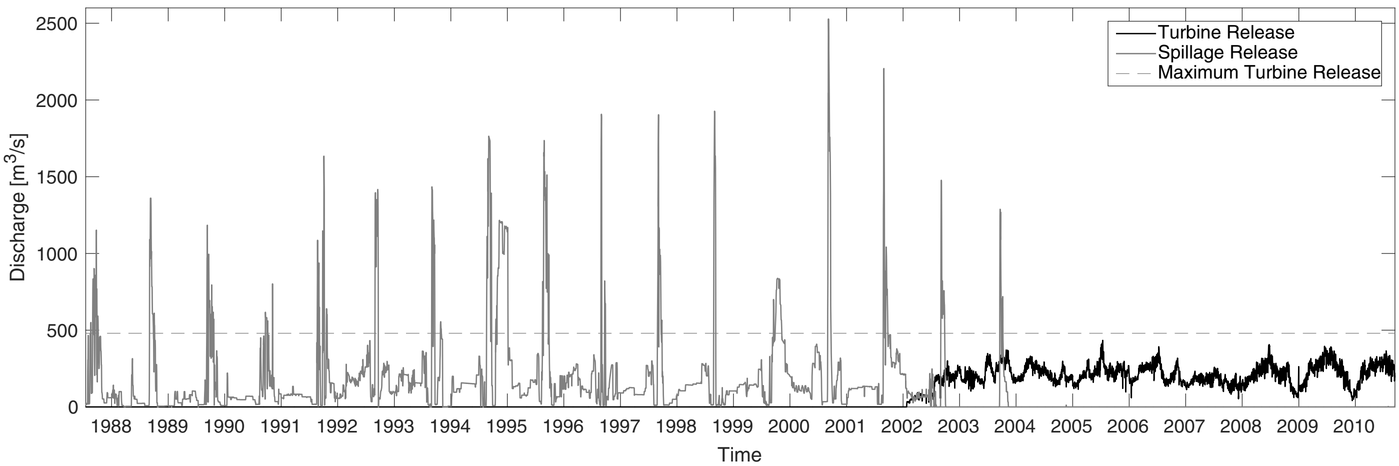

Fig. 1. Historical operation of Manantali, 1988–2011: observed turbine releases (solid line) and spillage release (gray line).

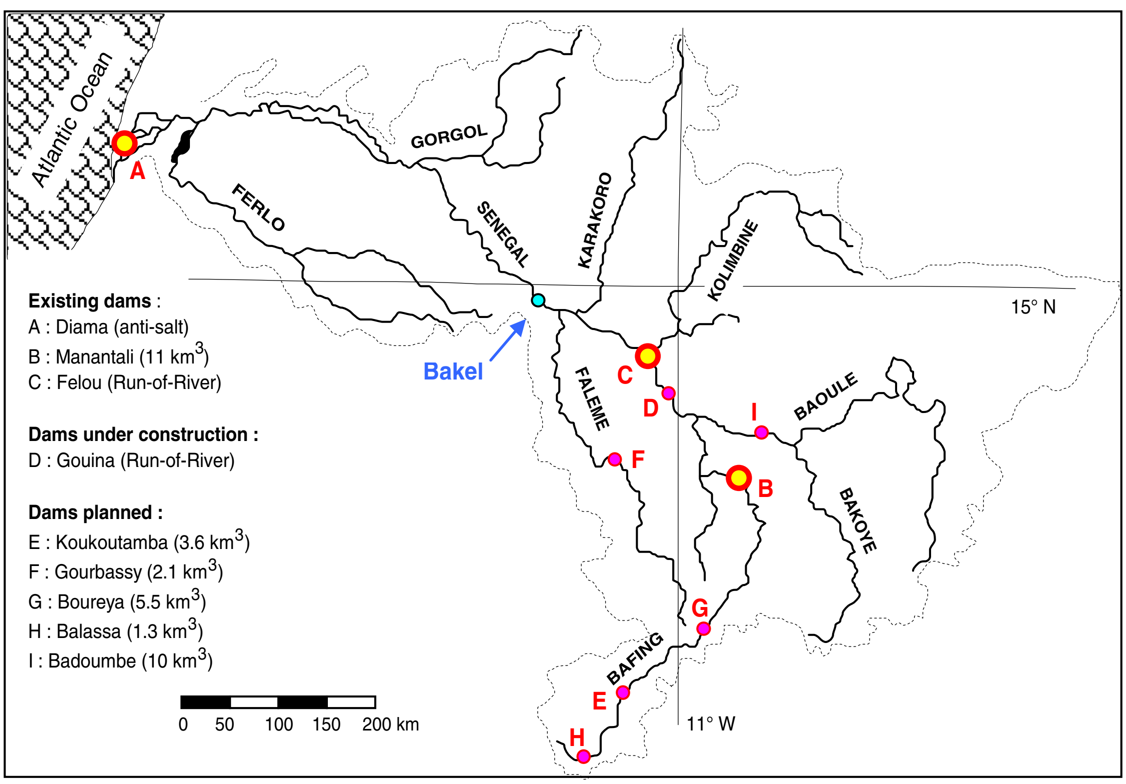

Fig. 2. Map of the Senegal River: existing and planned dams.

Managing the ongoing development processes will require a more accurate understanding of the limits of the system and of the long-term dynamics of change in this basin. If the current reservoir operation is maintained and other tributaries will be dammed, in fact, existing natural equilibria could be broken in the future, possibly leading to irreversible negative consequences for the socioecological system. The apparently unsolvable conflict between hydropower and traditional water uses in the Senegal River motivates this research.

The contribution of this research is investigating the existence of a reservoir operation strategy where electricity production is the main objective, with positive benefits for flood support being produced as secondary effect. The analysis of the historical reservoir operation shows the dominance of the energy production objective; therefore, a single objective problem is considered in this study. The question of this study is not about which trade-offs of performances are possible on this system, as is often the case in this type of studies (Mendes et al. 2015), but rather when hydropower production is the main objective, is the coexistence with flood production objective still possible? An optimization procedure is employed to identify the desired reservoir operation strategy. Results from the optimization provide an upper boundary of achievable performance in terms of hydropower production, offering a reference point to which the actual reservoir operation can be compared and assessed. Initial results of this analysis, albeit using a less realistic setting, have been presented by Raso et al. (2014a).

In this study, the problem of optimal reservoir operation is solved by applying the stochastic dual dynamic programming method (SDDP) (Pereira and Pinto 1991; Shapiro 2011; Tilmant et al. 2008). SDDP is an algorithm derived from stochastic dynamic programming (SDP) (Stedinger et al. 1984). The application of the SDDP method provides a realistic estimation of system performances, which includes the operational variability and the system adaptation over time (Raso et al. 2019; Guan et al. 2018; Tilmant et al. 2014; Goor et al. 2011; Macian-Sorribes et al. 2017). SDDP was selected for use in thid study because of the following advantages: (1) producing a faster solution, (2) not requiring variables discretization, and (3) easily allowing the extension of the analysis to a multireservoir system. The latter advantage, in particular, keeps open the possibility to enlarge the system boundaries and include all the reservoirs that are planned on the Senegal River.

The paper is structured as follows. The "Application" section presents the SDDP method, the model, and the objectives. In the "Results" section, results are presented and discussed. In the "Conclusions" section, the authors draw the conclusions.

Application

This section describes the setup of the optimization problem. First, the optimization method is described, upon which the setup of the system model and the operational objectives depend. Second, a system model made of a streamflow process model and a reservoir model at the daily time step is defined; the streamflow process model generates the inflow to the reservoir. Third, performance and operational indicators are defined; the performance indicators will be used to compare different reservoir operation strategies, and the operational indicator is to be included in the optimization procedure. The operational indicator, given in Eq. (6), is the time-step objective function included in the optimization.

Optimization Method

In reservoir operation, present benefits must be balanced with future uncertain ones (Soncini-Sessa et al. 2007; Castelletti et al. 2008). After each release decision, new information becomes available that partially reduces uncertainty. Optimal reservoir operation can be framed as a multistage stochastic programming (MSP) problem (Birge and Louveaux 1997; Shapiro and Andrzej 2003; Raso et al. 2014b), which, for a long horizon, is conveniently solved by SDP (Bellman and Dreyfus 1966; Stedinger et al. 1984). SDP decomposes the MSP problem in separable step-by-step optimal decision problems related by the Bellman equation. Differently from MSP, complexity in SDP increases linearly with the horizon length; hence, it can be used to solve long horizon problems.

Eq. (1) shows the Bellman equation for the system under investigation:

$$H_t(v_t, q_t) = \max_{r_t, s_t}[g_t(\cdot) + \mathop{\mathbb{E}}_{q_{t+1}}[H_{t+1}(v_{t+1}, q_{t+1})]] \tag{1}$$

where t = time index; $g_t(\cdot)$ = time-step objective function; $\mathbb{E}$ = average operator; v_t and q_t = reservoir volume and the inflow to the reservoir; r_t and s_t = turbine release and spillage release, where v_t and q_t are the system states and r_t and s_t are the controllable variables; and H_{t+1} = cost-to-go function, i.e., the average cost for leaving the system in the state $[v_t, q_t]$. The variables in Eq. (1) are related via a system model and subject to a set of constraints. Eq. (1) is solved backward, from the final time step to the initial one.

In this analysis, the SDDP (Pereira and Pinto 1991; Shapiro 2011) algorithm is employed. In SDDP, the time-step problem must be linear because problem linearity ensures the convexity of the cost-to-go function. SDDP is solved iteratively by a forward and backward phase. In the backward stage, at each t, the optimization finds the minimum average cost to pass from $[v_{t-1}, q_{t-1}]$ to $[v_t, q_t]$, adding the extra cut $l_k(\cdot)$ to the approximation of the cost-to-go function $\mathcal{H}_t(v_t, q_t)$ such that $\mathcal{H}_t(\cdot)$:max$\{\mathcal{H}_t(\cdot), l_k(\cdot)\}$. In the forward stage, the approximate problem is solved for the entire horizon to find the optimal trajectories that will be used in the next backward phase. By successive iterations, $\mathcal{H}_t$ converges to the real cost-to-go function H_t, as demonstrated, under mild conditions, by Philpott and Guan (2008) and Linowsky and Philpott (2005). The SDDP procedure run for this experiment used seven stochastic extractions backward and 10 stochastic extractions forward on a 4-year optimization horizon at the daily time-step.

Model

The streamflow process model is designed to statistically reproduce the hydrological variability of the inflow. SDDP requires the model to be linear, but it allows parameters to be time-variant and residuals to have any distribution. This study employs the multiplicative streamflow process model presented by Raso et al. (2017), calibrated on the observed inflow to the reservoir, i.e., the discharge data at the station of Soukoutali, in Mali.

Eq. (2a) represents the model as used in the forward phase, and Eq. (2b) its linearized version as used in the backward phase

$$q_t = \alpha_\tau \cdot q_{t-1}^{\phi_\tau} \cdot \xi_t \tag{2a}$$

$$q_t = [\rho_\tau \cdot q_{t-1} + \kappa_\tau] \cdot \xi_\tau \tag{2b}$$

where $\xi_t \sim \ln \mathcal{N}(0, \sigma_\tau)$; and parameters α_τ, ϕ_τ, σ_τ, ρ_τ, and κ_τ are defined and identified as by Raso et al. (2017). The model in Eqs. (2a) and (2b) is a periodic autoregressive model with lognormal multiplicative residuals, which ensures that discharge values are nonnegative.

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The reservoir is represented by the continuity equation [Eq. (3)], plus constraints

$$v_t = v_{t-1} + \Delta t \cdot (q_t - r_t - s_t - e_t - l) \tag{3}$$

where v_t = reservoir volume; q_t = inflow; r_t = turbine release (i.e., release through the turbines); s_t = spillage release (i.e., release through the spillways); e_t = evaporation from the reservoir; and l = other losses. The volume is the state, the releases are the controllable variables, and inflow and evaporation are the uncontrolled forcings. Within the range of normal operation, i.e., water level between 185 and 210 m above see level (a.s.l.), the reservoir can be assumed cylindrical. The evaporation is the product of specific evaporation from the reservoir surface. The specific evaporation is periodic, as defined by Bader et al. (2003), and the reservoir surface S is considered constant. The other losses are estimated from the inversion of the mass balance. Other losses are considered constant over time.

The inequalities in Eq. (4) define the constraints to the reservoir model

$$v_{\min} \le v_t \le v_{\max} \tag{4a}$$

$$r_{\min} \le r_t \le r_{\max}(v_t) \tag{4b}$$

$$s_{\text{safety}}(v_t) \le s_t \le s_{\max}(v_t) \tag{4c}$$

where volume, turbine release, and spillage release are constrained between a minimum and a maximum value. The volume is constrained by the physical characteristics of the reservoir. The maximum turbine release and spillage release depend on the reservoir water level. For safety reasons, the minimum release through the spillage is regulated by a legal constraint, s_{safety}, that obliges the operator to spill water if the water level in the reservoir exceeds a certain thresholds. The reservoir water level is univocally related to the volume; therefore, the maximum turbine release and maximum spillage release constraints can be defined as function of the volume. These constraints are included in the optimization by linear cuts, as in Fig. 3.

Objective

Eq. (5) defines the time-step performance indicator that quantify the objective for hydropower production

$$E_t = c \cdot \Delta h_t \cdot r_t \tag{5}$$

where Δh_t = hydraulic head, i.e., the difference between water level in the reservoir and water level immediately downstream of the reservoir; and c = multiplying factor proportional to $g \cdot \rho \cdot \xi \cdot \Delta t$, where g is the gravitational constant, ρ is water density, ξ is turbine efficiency, and Δt is time-step length. The turbine efficiency is considered constant and set at 0.9, and the downstream water levels are considered constant and set at 155 m. Water level in the reservoir depends on the reservoir volume according to the storage curve function. Eq. (8), to be included in the linear programming

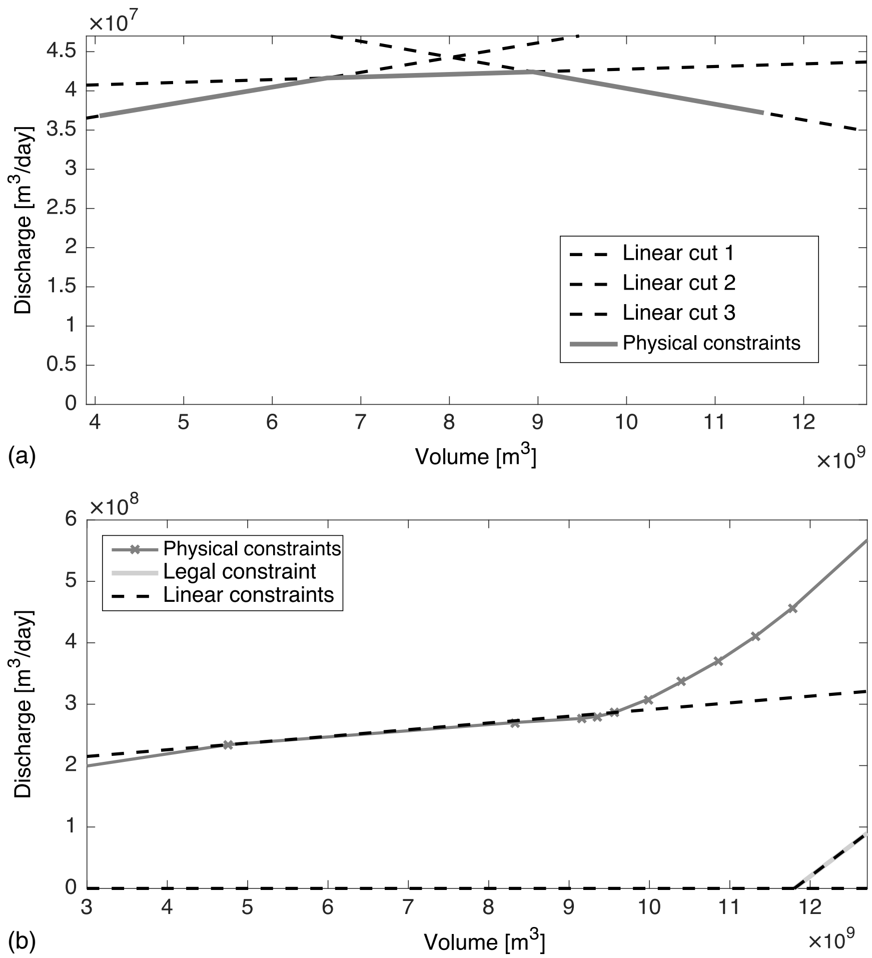

Fig. 3. Constraints depending on the reservoir volume. Continuous lines show physical and legal constraints and dashed lines indicate linear cuts included in the optimization: (a) maximum turbine release; and (b) maximum spillage release.

problem, is linearized with a Taylor expansion at a nominal point, as by Raso et al. (2017).

Eq. (6) defines the operational indicator for energy production that will be used as time-step objective function in the optimization procedure

$$E_t^o = r_t + \frac{r_0}{S \cdot \Delta h_0} \cdot v_t \tag{6}$$

where S = reservoir surface; Δh_0 = nominal hydraulic head; and r_0 = nominal turbine release. The values of constants in Eq. (6) are defined in the Notation. In Eq. (6), the energy production objective becomes a linear equation, the sum of turbine release and reservoir volume, where the importance of maintaining a high hydraulic head depends on the physical characteristic of the reservoir.

Performance Indicators

The system objectives considered in this study are hydropower production, flood support, and continuity of production. The performance indicators for these objectives, i.e., J_E, J_F, and J_C, are the sum of the respective time-step objectives over the simulation horizon, as defined in Eq. (7)

$$J_E = \frac{1}{N_y}\sum_t^H E_t \quad (\text{GWh/year}) \tag{7}$$

$$J_F = \frac{1}{N_y}\sum_t^H F_t \quad (\text{m}^3/\text{year}) \tag{8}$$

$$J_C = \frac{1}{N_y}\sum_t^H C_t \quad (\text{day/year}) \tag{9}$$

where t = time daily index; H = length of the simulation horizon; N_y = number of years; and E_t, F_t, and C_t = time-step objective functions for hydropower production, defined in Eq. (5), flood support, defined in Eq. (10), and continuity of production, defined in Eq. (11).

Eq. (10) defines the time-step performance indicator for flood support

$$F_t = (q_t^B - q^F)^+ \cdot \Delta t \tag{10}$$

where $q_t^B = r_t + s_t + q_t^L$ is the daily discharge at Bakel, the confluence point, in which q_t^L is the lateral discharge, sum of discharge at Oualia and Kidira stations; and q^F = threshold value. The operator $(\cdot)^+$ returns the maximum value between its argument and zero. Indicator 10 quantifies the flood support objective as proportional to the volume of water that exceeds the threshold q^F. The volume of water above this threshold is considered to reach the floodplain. The threshold is set at 1,300 m^3/s, which is the lowest value beyond which a flood occurs at Bakel according to Bader et al. (2003).

Eq. (11) defines the time-step performance indicator for energy continuity

$$C_t = 1 \quad \text{if } E_t > E_c, 0 \text{ otherwise} \tag{11}$$

where E_c = threshold for daily energy production, below which the daily energy production is considered unsatisfactory. E_c is fixed at 1,800 MWh/day, which corresponds to about the 30% quantile of daily historical operation.

Results

The effects of operational rules obtained from the a SDDP configuration that maximizes the electricity production indicator is analyzed. These rules are tested on the system model using as inflow the observed inflow data from January 1, 2004, to December 31, 2011, corresponding to the period in which the reservoir was operational for electricity production and for which data on observed release and reservoir volume are available. The simulation based on SDDP rules is compared with the historically observed reservoir operation trajectories. The authors acknowledge that the historical reservoir operation is influenced by many other contingent elements, limiting the meaningfulness of a comparison with rules identified by an optimization procedure. Observed reservoir operation trajectories, however, can still be a useful reference point to interpret the results from SDDP and identify promising strategies.

Table 1 presents the performance indicators for the simulations using rules derived from SDDP and historical data. Analyis of Table 1 indicates how the rules issued from SDDP lead to an improvement of electricity production of about 70 GWh/year, i.e., about 9%, and an improvement in flood support of about $40 \times 10^6\ \text{m}^3/\text{year}$, i.e., about 7%, but a reduction of production continuity of 70 days, i.e., about 20%. This finding shows that the application of SDDP operational rules can improve, with respect to the historically observed operation, both performance indicators of energy and flood production even when the objective of reservoir operation is the maximization of energy production only, but at the cost of the continuity of production. These results can be explained by examining the details of the simulations.

Fig. 4(a) displays the turbine release over the year for the 8-year simulation period. This plot shows that for both the historical and the SDDP simulation, the turbine presents an annual periodicity. The turbine release for the SDDP configuration (solid line), however, has a more irregular trend than the historical one. In SDDP simulation, most of the turbine release occurs from about Day 100 to Day 270, corresponding to the period just before and during the wet season. The analysis of SDDP rules reveals a strategy that draws down the reservoir level sufficiently to store the peak of the inflow in the reservoir and reduce spillages, without unnecessarily lowering the hydraulic head.

The interannual variability of inflow is handled by the short-term adaptation. The short-term adaptation adjusts the release decisions to the present conditions of the system states. The system states are made of the present reservoir level and the present inflow. Fig. 4(a) shows how in wetter years, when the inflow is larger and/or the reservoir level is higher, the turbine release continues until Day 270 at maximum level. The turbine release in the latter period contributes to flood production, even without spillage release. In drier years, instead, when the inflow is smaller and/or the reservoir level is lower, turbine release continues until Day 230, such that the reservoir level rises up as close as possible to the maximum level.

Fig. 4(b) displays the reservoir water level over the year for the 8-year simulation period. This plot shows how both the SDDP and the historical simulations draw down the reservoir before the annual peak of discharge enters the reservoir. Then the higher discharge in the wet season replenishes again the reservoir. In the SDDP

Table 1. Table of results per configuration and performance indicator

Configuration	Hydropower (GWh/year)	Flood support ($\times 10^6\ \text{m}^3/\text{year}$)	Continuity (day/year)
Historical	780	610	250
SDDP	850	650	180

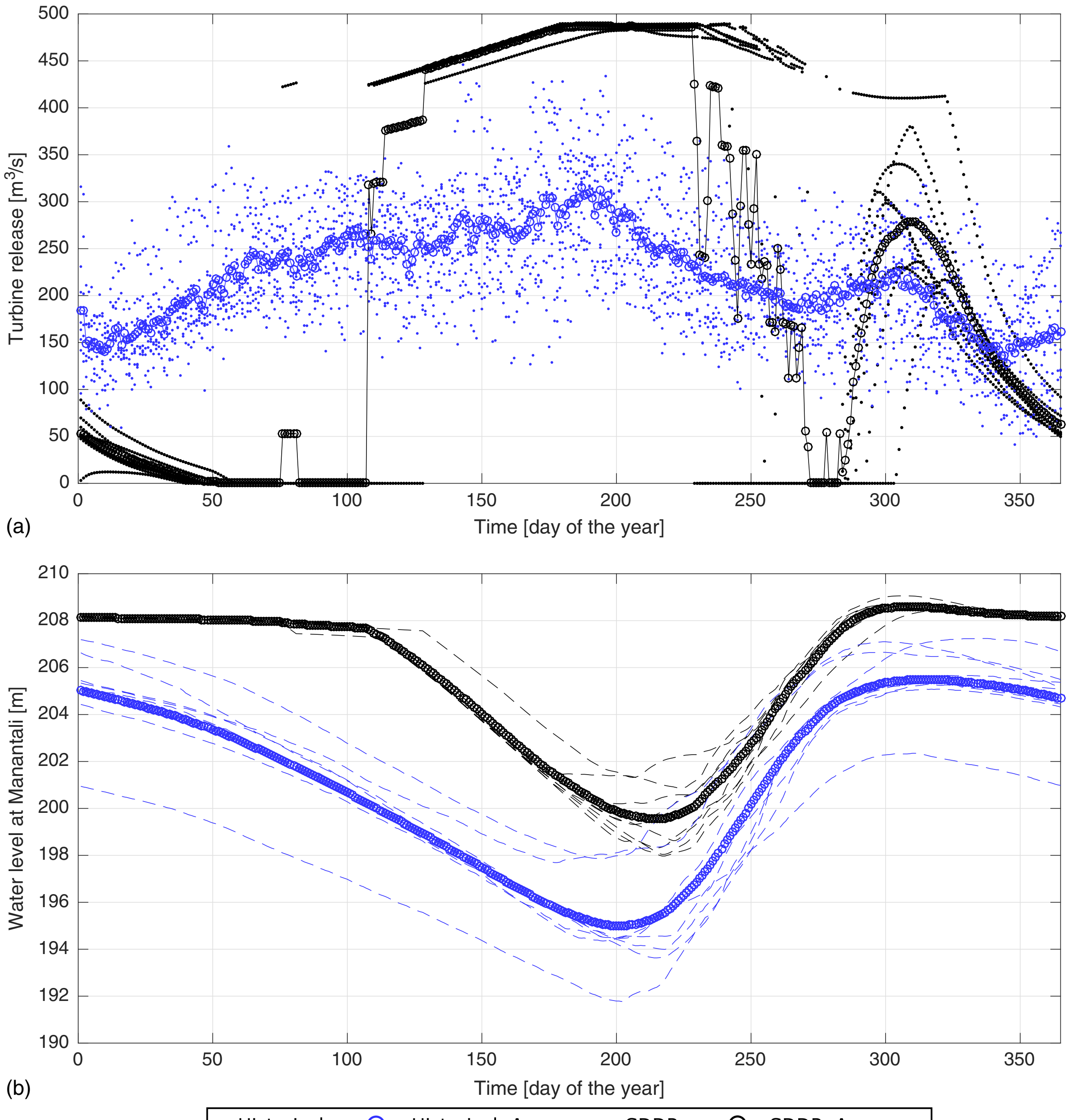

Fig. 4. (a) Turbine release; and (b) water level at Manantali. Solid lines show results from SDDP maximizing energy production.

configuration the water level is kept higher for the entire simulation period. In some years, the higher water level leads to the exceeding of the safety level at 208 m, which forces the operator to spill some water. The loss in electricity production due to this spillage release is, however, largely compensated for by the higher hydraulic head. The higher hydraulic head permits the production of the same electricity with less turbine release, increasing the efficiency of the reservoir. The SDDP procedure finds the optimal trade-off between the advantage of hydraulic head and risk of spillage, such that hydropower is maximized, also taking also into account the real-time adaptation capacity of the system.

Fig. 4(a) shows how applying the operational rules as in SDDP would result in an irregular discharge, and hence in an irregular dispatching of electricity. This may have unwelcome consequences for the river water system and the regional electrical system, whose possible implications are to be further understood. Irregular discharge on the river may have negative consequences on the capacity to maintain an environmental flow (Marques and Tilmant 2018), and the irregular dispatching of electricity may require other sources to compensate it, or it may cause power outages (Andersen and Dalgaard 2013).

Irregular discharge, which emerged as a potential problem of the optimized solution, was investigated with an additional experiment. In this experiment, the consequences of enforcing continuous production are estimated. The experiment is equivalent to the one described in section "Application," but setting a constraint $r_{\min}$ on minimum turbine release at 170 $\mathrm{m^3/s}$. This value corresponds to about the 20% quantile of turbine release in the historical operation. Results from this experiment estimate hydropower production at about 850 GWh/year, i.e., equivalent to the experiment without continuity constraint, and flood production at about 630×10^6 $\mathrm{m^3/year}$, calculated on the indicators in Eq. (7). If compared with the values of the same indicators for the unconstrained case (Table 1), one can see that there is no reduction on hydropower production and a limited reduction on flood production. A possible explanation is that the increase in hydropower production with respect to observed operation is mostly due to the higher hydraulic head, which is equivalent for the case with or without continuity constraint, and that flood production is largely influenced by lateral discharge.

Conclusions

This study demonstrated the possibility of reducing the conflict between hydropower production and traditional water uses on the Senegal River. Specifically, the contribution of this study is the identification of reservoir operational rules that have positive benefits for the flood support objective while optimizing only the hydropower production objective for the Manantali reservoir, in the Senegal River. These rules were identified by setting up a SDDP problem. The solution was found using SDDP, indicating an improvement of 9% on hydropower production and 7% on flood production with respect to the observed release. The meaningfulness of this improvement, however, is limited by the partial comparability between performances obtained from the SDDP simulation and actually observed performances. A more meaningful comparison, which will be made in a future study, would benchmark performance obtained from SDDP simulation to performance obtained from simulation using current reservoir operational rules.

A single-objective problem is considered in this study because the analysis of the past reservoir operation shows the dominance of the energy production objective. The analysis of past releases of Manantali, in fact, seems to indicate hydropower production as the main objective, which led to a noticeable reduction of water availability for the annual flood, traditionally used for flood recession agriculture. Results from this study indicate that the application of operational rules issued from SDDP can improve both performance indicators of energy and flood production even when the objective of reservoir operation is maximizing energy production only. The SDDP procedure finds a good trade-off between the advantage of keeping a high hydraulic head, which increases the efficiency of water use, and the risk of having to spill water. The release strategy found by SDDP suggests maintaining the turbine release at maximum level during the flood period if the inflow to the reservoir is sufficiently high. This strategy contributes to the flood support objective even if this was not included in the optimization. The SDDP strategy, however, suggests an average release over the year that is much more irregular than the historical one. The irregular release can have negative consequences for the continuity of the electricity supply.

Despite the advantages that improved reservoir operation can bring, the proposed operational rules must be analyzed from the aspect continuity of production. The irregular production that the SDDP strategy suggests poorly fits with the regular demand of electricity. In the period of the year when Manantali does not supply electricity, other sources should compensate for it. The negative effects of irregular electricity production will be more severe if more reservoirs will be built, when a larger share of the electricity supply will be produced by hydropower. This could imply the need of integrating water management policies with energy policies or the need to connect the reservoir to a subcontinental electric pool (Gnansounou et al. 2007). Future research could investigate this aspect further.

The relevance of the findings of this research can be better understood when framed in terms of the clash between (1) the strong and increasing demand of electricity in the region, and (2) the commitment of the Senegal River authority to sustainable river management. The coexistence of hydropower and traditional water uses may be possible, but this would require a radically different reservoir operation and an appropriate integration of Manantali within the electric system. On the Senegal River, construction of additional reservoirs will make the river discharge more controllable, also making design of reservoir operation rules a more complex problem. Future research could test whether the results presented in this paper maintain their effectiveness when applied to a multireservoir system. In a system made of several reservoirs, in fact, a coordinated operational management could be employed to synchronise the peak discharge, hence preserving a satisfactory level of flood support.

Acknowledgments

Luciano Raso's research was funded by the AXA Research Fund.

Notation

The following symbols are used in this paper:

c = conversion factor [Eq. (5)] ($1{,}000\ \mathrm{kg/m^3} \times 9.81\ \mathrm{m/s^2} \times 0.9 \cdot 10^{-6}\ \mathrm{MW/W} \times 2.77 \times 10^{-4}\ \mathrm{h/s}$);
E_t = daily energy production (MWh);
E_t^o = daily energy production, linearized;
e_t = evaporation ($\mathrm{m^3/s}$);
F_t = flooded volume ($\mathrm{m^3}$);
$g(\cdot)$ = time-step objective function;
$H(\cdot)$ = cost-to-go function;
$\mathcal{H}(\cdot)$ = cost-to-go function approximation by Bender's cuts;
l = losses ($30\ \mathrm{m^3/s}$);
N = optimization horizon ($T \times 4$);
q_t = inflow ($\mathrm{m^3/s}$);
q_t^B = discharge at Bakel ($\mathrm{m^3/s}$);
q_t^D = flood threshold at Bakel ($13{,}000\ \mathrm{m^3/s}$);
$r_{\max}$ = maximum turbine release ($\mathrm{m^3/s}$);
r_t = turbine release ($\mathrm{m^3/s}$);
r_0 = nominal turbine release ($400\ \mathrm{m^3/s}$);
S = reservoir surface ($2.6 \times 10^8\ \mathrm{m^2}$);
$s_{\max}$ = maximum spillage release ($\mathrm{m^3/s}$);
s_{safety} = minimum spillage release ($\mathrm{m^3/s}$);
s_t = spillage release ($\mathrm{m^3/s}$);
T = period length (365);
t = time index;
$v_{\max}$ = maximum reservoir volume ($1.5 \times 10^{10}\ \mathrm{m^3}$);
$v_{\min}$ = minimum reservoir volume ($3.9 \times 10^9\ \mathrm{m^3}$);
v_{safety} = reservoir volume safety limit ($1.18 \times 10^{10}\ \mathrm{m^3}$);
v_t = reservoir volumes ($\mathrm{m^3}$);
α_τ = periodic moving average parameter of the streamflow process model;
Δh_0 = nominal hydraulic head (50 m);
Δt = time-step length (86,400 s);
κ_τ = autoregressive additive parameters of the linearized streamflow process model;
$\rho_{\tau,i}$ = autoregressive multiplicative parameters of the linearized streamflow process model;
σ_τ = variance of errors of the streamflow process model;
τ = periodic time index over the year; and
ϕ_τ = periodic autoregressive coefficient of the streamflow process model.

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