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http://ascelibrary.org/doi/abs/10.1061/%28ASCE%29WR.1943-5452.0000867

Multipurpose design of the flow control system of a steep water main

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Abstract

This work presents the technical characteristics and the regulation system of a complex water supply system (WSS) in an Italian Alpine valley. The WSS faces multiple challenges: water supply over a large area, hydropower generation, and coordination between multiple local sources and networks. The development of an optimal feedback-control algorithm for the supervisory control system was key to guarantee the operation of this modern WSS. This regulation scheme and the rationale for its development are described in this paper. A customized numerical model of the WSS was developed in order to test the operating rules through suitable numerical simulations. Results show that the proposed algorithm satisfies the objectives of the WSS and respects its tight constraints. The analysis of the case study evidences the advantages of coordination between municipal water networks, quantifies the hydropower generation potential in the WSS, and highlights the key role of automation and remote control in modern water systems. Finally, the case study presented here provides an efficient technical solution for the hydraulic regulation of a high pressure water main connecting a cascade of small tanks in mountain regions.

INTRODUCTION

In regions where water resources are commonly available, as in the case of the north

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of Italy, water shortages are mostly due to failures in local water supply sources. As a consequence, an effective strategy is the creation of intermunicipal water networks which connect multiple local water systems (Massarutto, 2000). In this way, the production and treatment of drinkable water can be performed in a small number of facilities. It follows that: (i) the use of the best quality water is privileged, (ii) the cost of water treatment is reduced, and (iii) the resilience of the system is increased thanks to the diversification of water sources. In addition, a growing awareness about renewable energy is increasingly leading to the integration of small hydropower plants in water systems. In this way the excess potential energy of water is converted into electric power (e.g., Filion et al., 2004; Carravetta and Giugni, 2009; Fontana et al., 2011) and optimal pressure values are maintained in the network (Tricarico et al., 2014; Fecarotta et al., 2015). Pumps as turbines (PAT) are an innovative, low-cost, and reliable solution for energy production in water systems where pressure and flow conditions are variable and the available power is limited (Carravetta et al., 2012, 2014; Lydon et al., 2017). On the other hand, the installation of traditional turbines (e.g., Pelton turbines) is better suited to large transmission pipelines and especially to mountainous regions where energy potential is high (Afshar et al., 1990; Möderl et al., 2012; Sitzenfrei and Rauch, 2015).

Modern water supply networks are thus becoming complex systems that coordinate many local facilities and sources and pursue multiple purposes (Yazdani and Jeffrey, 2011). The first purpose is to reliably provide drinking water to consumers. The second purpose is to efficiently manage water and energy resources. In order to achieve these purposes, modern water systems are generally controlled by a regulation algorithm implemented in a supervisory control system (e.g., Cembrano et al., 2000; Giacomello et al., 2013).

In this framework, the goal of this study is to present an optimal control algorithm developed for the operation of a newly designed water supply system (WSS) in an Alpine valley in the north of Italy. The WSS consists of a 80 km-long water main that runs along the valley connecting 20 municipal water supply networks. The system takes water from a hydropower plant and supplies the valley population with 250-500 l/s. The water main starts at 1260 m above sea level and ends at 400 m with an 860 m difference in altitude. Four inline tanks are present along the water main and the excess water pressure is converted in hydropower by three turbines. A key characteristic of this system is that the size of the inline tanks is severely constrained by the topography of the valley. As a result, the tanks are particularly small in comparison with the daily volume delivered by the water main. The first target of the WSS is to reliably provide high quality water to the local municipal systems when local sources fail or their water quality is low, and when the water treatment in the main plant is cheaper than in the local plants. The second target is to generate hydroelectricity. Tailoring the optimal hydraulic control to achieve these two targets is not trivial and presents both conceptual and technical difficulties. At the conceptual level, it is necessary to develop a robust control strategy to be implemented in the supervisory control system in order to guarantee the objectives of the WSS. The technical difficulties concern the availability of control devices (valves and turbines) that can perform the proposed regulation.

Over the last years, several studies have focused on different aspects concerning the optimal regulation of water supply systems. In this context, numerical simulations allow a realistic representation of complex water systems, which involve economic, social and engineering issues (e.g., Jain et al., 2005). Therefore, the vast majority of water system planning and managing studies is based on numerical modeling approaches (e.g., Rani and Moreira, 2010). A widely adopted approach (e.g., Lund and Guzman, 1999) is to define operating rules based on engineering targets, and to check the response of the modeled system. In this approach, a centralized regulation system is usually implemented and possible coordination mechanisms for multiple water storing facilities at large scale are proposed by various authors (e.g., Anghileri et al., 2013; Ficchí et al., 2016). In fact, coordination in operations increases the system efficiency and resilience, especially in a context of adverse conditions such as climate change and increasing water demand (e.g., Marques and Tilmant, 2013).

However, little research has been done on the optimal design and management of WSSs in mountain areas characterized by a high pressure water main which connects numerous local water systems. This article fills the gap by presenting the rationale for the development of a multipurpose control system for WSSs in mountain regions. Differently from the current literature, several management issues are considered at the same time: (i) the regulation of a steep water main with small inline tanks, (ii) the energy recovery from pressure dissipation, and (iii) the coordination between many local WSSs.

Referring to the specific case study, the hydraulic constraints and the regulation objectives are formulated as a set of mathematical conditions (see the section "Hydraulic Constraints"). The operating rules for the optimal hydraulic and energy regulation are then developed according to these constraints (see the section "Management Rules"). Flow balance equations, triggering thresholds for the tank levels and a centralized management approach are the main tools for the regulation system. Beside the specific case study, the proposed solution approach provides guidance for the design of the control system for modern WSSs in highly populated mountain regions, where the available storage volume constrains the system reliability, several local water systems have to be networked, and energy recovery can be performed.

CASE STUDY

The WSS will be in service along an Alpine valley in northwestern Italy with a total population of 115 000 inhabitants distributed in 20 municipalities. These municipalities are very different in terms of size, population, and economic activities. The upper valley is characterized by small towns with a permanent population ranging between 300 and 3000 inhabitants. Tourism is the main economic activity and during the ski season population can increase by one order of magnitude. The municipalities of the lower valley enclose most of the valley permanent population and host a range of industrial and commercial activities. In particular, the main town (marked with V in Fig. 1) hosts 50000 inhabitants and several industrial activities. Currently, water is provided by local water supply networks operating independently one from each other. A typical local network (see inset in Fig. 1) is characterized by a local storage tank collecting water from springs or wells and supplying the local population. Groundwater pumped from wells is often necessary to satisfy the local water demand. For each local network, the amount and the temporal pattern of daily water demand as well as the relative contribution of each local source (e.g., water wells) are known from historical data (see Fig. S1 and Table S2 in the Supplemental Data).

Over the last decades, numerous WSSs in the north of Italy suffered from unexpected failures. In particular, water availability in the study area was strongly affected by water scarcity in 2003 and 2006 (Carrera et al., 2013). These severe droughts demonstrated the vulnerability of the existing water supply infrastructure. Moreover, strong criticalities in water supply occur every year during the ski season, when the water demand increases for the presence of tourists. Additional critical issues concern the quality of water. Water from springs is often unusable after rain events, due to high levels of turbidity. Finally, high concentration of sulphate in the aquifer are common in some municipalities. Thus, groundwater is frequently an unsuitable water source. In order to solve these problems, the water utility company and the local authorities decided to employ part of the water stored in a high-altitude Alpine reservoir. In order to distribute this water in the valley, a WSS with a 80 km-long water main was designed and built (Fig. 1). The ultimate goal of the WSS is to provide high quality water to the local systems when: (i) local sources fail to satisfy local demands (e.g., pump breakage, unexpected peaks in water demand, etc.), (ii) the quality of local water is not satisfying (e.g., high concentration of sulphate), and (iii) the cost of treatment in the main plant is lower than the cost of treatment and pumping in the local plants.

Characteristics of the hydraulic system

The new WSS takes water from a 3 500 000 m^3 reservoir located at an altitude of 1900 m a.s.l. This reservoir stores water with high physicochemical quality and is currently used for hydroelectric purposes. Thanks to an agreement between the hydropower and the water supply companies, the flow released from the hydroelectric power plant is constant during the week but varies in the range 250-500 l/s according to a monthly schedule (see Table S1 in the Supplemental Data). This discharge is collected downstream of the hydroelectric plant and is transferred to the water potabilization plant (WPP). After rather mild treatments, water is stored in a first water tank (S1 in Fig. 1) and then delivered to the municipal water systems of the valley by a pressurized water main. The water main is a 700 mm diameter ductile iron pipe with a roughness of 0.1 mm and with thrust-resisting joints. In the upper valley the water pipe is characterized by steep slopes (2% on average, with 10% in the steepest profile). Three intermediate inline tanks (S2, S3, and S4 in Fig. 1) split the water main in order to limit the static water pressure in the pipes. Downstream of the last tank (S4 in Fig. 1) the water main runs for 50 km along the lower part of the valley, where 14 of the 20 supplied municipalities are located. A key feature of the system is that the topography of the valley severely constrained the construction of tank S2, that has an area of 80 m^2 and a height of 4 m. As a result, small changes in the inflow or outflow discharge induce large changes in water levels. The rate of level variations $\dot{h}$ is given by

$$\dot{h} = \frac{dh}{dt} = \frac{1}{\Omega} \cdot \Delta Q \tag{1}$$

where h is the level in tank, t is time, Ω is the tank area and ΔQ is the net flow to the tank. For tank S2, $\Delta Q \sim 100$ l/s and, thus $\dot{h} \sim 1$ mm/s. It follows that over a time interval of 5 minutes (that is required for adjusting the flow rate with the installed valves or turbines) the level of S2 varies as much as 40 cm which corresponds to 10% of the tank height.

The whole system is monitored and controlled by a supervisory control system (SCS). The flow rates into and out of the tanks and the flow rate delivered to the local water systems are measured by electromagnetic flow meters, whereas water levels in tanks are measured by ultrasonic level sensors. The acquired data are processed by a decision algorithm that calculates the target values for the control devices. The operating rules implemented in the decision algorithm have been developed in order to optimize the operations of the WSS and are presented in the next section "Management rules". The active elements controlled by the SCS are turbines and valves. Pelton turbines with electronically controlled Doble needles adjust the flow that enters into the intermediate tanks. Needle valves with electric actuators regulate the flow from the water main to the local water systems. The time interval for flow adjustment is longer than 5 minutes. This slow regulation results in smooth transitions from one steady state to another one, without large pressure and flow fluctuations (Boulos et al., 2005). In this way, water hammer and hydraulic resonance (e.g., Riasi et al., 2010) in the system are prevented.

Characteristics of the energy recovery system

As illustrated in Fig. 1, three turbines (T1, T2, and T3) are located along the water main just upstream of the inline tanks. The hydrostatic heads are 255, 265, and 130 m for T1, T2, and T3, respectively. Each turbine has a capacity of 500 l/s that corresponds to the maximum flow rate of the supply system. Two four-jet Pelton turbines (T1 and T2) with a power of ∼1 MW are installed before S2 and S3. A single-jet Pelton turbine rated at ∼500 kW (T3) is installed before S4. The turbines have electronically controlled Doble needles. A hydraulic actuator regulated by a Programmable Logic Controller (PLC) allows to independently regulate each needle and, thus, to independently open each nozzle. The algorithm that controls the nozzle opening is developed by the manufacturer, and allows the turbine to operate with optimal efficiency and minimum mechanical weariness. The flow through the turbine can be adjusted within the range 0-500 l/s. In order to prevent excessively frequent adjustments of the regulation device, the turbine regulates the discharged flow in a discrete way. The turbine flow (Q_T) can take the discrete values

$$Q_T = k \frac{Q_N}{10 \cdot J}, \tag{2}$$

where k is a positive integer, Q_N= 500 l/s is the nominal flow rate of the turbine and J is the number of jets of the turbine (J = 4 for T1 and T2 and J = 1 for T3). Eq. (2) indicates that the actual flow through each jet varies at steps of 10% of the maximum flow rate through the jet.

Hydraulic constraints

High quality water supply, reduction of water supply costs, and hydropower generation are the main targets of the WSS. These targets are achieved if the hydraulic constraints presented below are satisfied. These constraints entail significant challenges in the regulation of the system and concern: (i) the water level in tanks, (ii) the number of operations performed by the control devices, and (iii) the coordination with the local water systems.

Water level h in tanks must satisfy three conditions

$$h > h_{\mathrm{MIN}}, \quad h < h_{\mathrm{MAX}}, \quad h \to h_{\mathrm{MAX}}, \tag{3}$$

where h_{MIN} and h_{MAX} are the minimum and maximum water level in tanks, respectively. The first inequality avoid emptying of the intermediate tanks in order to prevent air from entering the water main. The presence of air in the pipeline can induce disruption of the flow, pressure spikes associated with column rejoining (e.g., Bergant et al., 2006; Malekpour and Karney, 2014), and reduced turbine efficiency. The second inequality avoid water overflow from tanks. In fact, these water losses reduce the hydropower generation and represent a waste of high quality water. Finally, the third relation means that the level in the storage tanks has to be maintained as high as possible in order to have a sufficient water reserve in case of network failures or unexpected water consumptions.

Turbines and valves adjust the flow in the whole system. The number of their operations must be minimized because a change in the status of a control device (i.e., an operation) results in the reduction of the lifespan of electromechanical components and in additional energy consumption. Therefore, the number of operations performed by the control devices must be as low as possible, namely

$$n_T = \min[\mathbf{n_T}], \quad n_V = \min[\mathbf{n_V}] \tag{4}$$

where n_T and n_V are the number of turbine and valve operations performed in an optimal regulation, while $\mathbf{n_T}$ and $\mathbf{n_V}$ are vectors that collect the number of possible operations performed in any regulation that satisfies conditions (3).

Pumping and potabilization costs make some local sources very expensive. One of the main purposes of the WSS is to replace the most critical local water sources (in terms of quality and cost) with water provided by the water main. Hence, coordination between the water main and the local networks is required for an optimal management of the available water resources in the system. Therefore, the total cost of water production (from local sources and at the WPP) must be minimized, namely

$$E = \min\big[\mathbf{E}\big], \qquad \mathbf{E_j} = \sum_{i=1}^{n_L} e_i \overline{Q}_i \tag{5}$$

where E is the total cost of water production performed in an optimal regulation, $\mathbf{E}$ is a vector that collect the total cost of water production performed in any regulation that satisfies (3), e_i is the mean unit cost (€$/m^3$) for water production from the i-th water source, $\overline{Q}_i$ is the mean flow production from the i-th source, and n_L is the number of local sources.

METHODS

A simulation model of the system was developed in MATLAB to study the behavior of the WSS under different operating rules and technical characteristics of the system components. This model consists of a coupled hydraulic and decision model. The hydraulic model calculates flow rates, pressures at junctions and water levels in tanks. The decision model simulates the operations of the SCS.

The hydraulic model (see Fig. 2) is a in-house developed MATLAB code consisting of a system of $(M{+}N)$ nonlinear equations, where N is the number of nodes and M is the number of elements (i.e., pipes, valves, pumps, and turbines). The following equations are used to model the hydraulic behavior of the system. The *flow-head loss relation* in the m-th pipe that connects nodes n and *n+1* reads

$$h_{n+1} - h_n - \hat{r} \cdot \left| Q_m^{\hat{a}-1} \right| Q_m - \hat{b} \left| Q_m \right| Q_m = 0; \tag{6}$$

where h is the nodal head, Q is the flow rate in the pipe, $\hat{r}$ is the resistance coefficient, $\hat{a}$ is the flow exponent, and $\hat{b}$ is the minor loss coefficient.

For pumps, turbines and valves the *flow-head loss relation* for the m-th element reads

$$c_{c,m} Q_m + c_{b,m} \cdot (h_{n+1} - h_n) + c_{f,m} \cdot (Q_m - \tilde{Q}_m) + c_{r,m} \cdot [h_{n+1} - h_n - f(Q_m, h_{n+1}, h_n)] = 0, \tag{7}$$

where the set of coefficients $\mathbf{c}= \{c_c, c_b, c_f, c_r\}$ indicates if the m-th element is closed ($\mathbf{c}=\{1,0,0,0\}$), by-passed ($\mathbf{c}=\{0,1,0,0\}$), imposes the flow rate $\tilde{Q}_m$ ($\mathbf{c}=\{0,0,1,0\}$) or imposes a flow rate depending on the nodal heads ($\mathbf{c}=\{0,0,0,1\}$). In this last case, the term $f(Q_m, h_{n+1}, h_n)$ must be specified as a generic nonlinear function that describes the hydraulic characteristics of the element (e.g., pump and turbine performance curves from the technical documentation). Moreover, *flow continuity* at nodes must be satisfied, namely

$$\sum_{m=1}^{M_n} Q_{m,n} - Q_{d,n} = 0, \tag{8}$$

where $Q_{m,n}$ is the flow from the m-th element into the node n, M_n is the number of elements connected by node n, and $Q_{d,n}$ is the flow demand of a local municipality at node n. The system of nonlinear equations (6)-(8) is completed with boundary conditions defining the piezometric head (h_n) of the node connected to the tank, namely

$$h_n = h_s. \tag{9}$$

where h_s is the hydraulic head of the s-th tank. Inertial effects in the system are negligible due to slowly varying boundary conditions, i.e., long closure time of valves (5 minutes). For this reason, unsteady formulations of the pipe hydraulics (e.g., Nault and Karney, 2016) are not required and the time evolution of the system is modeled by a succession of steady-states with duration Δt and whose boundary conditions at each instant are obtained from mass balance equations for the tank levels (as in Rossman, 1993). In the studied WSS, tank level variations are negligible compared to the piezometric head in pipes. Thus, the evolution of level in tanks does not affect significantly the water heads and the flows in the system. For this reason, the constant tank level assumption in the steady-state solution is valid, and a more refined model formulation is not required (Todini, 2011; Giustolisi et al., 2012). Moreover, it should be noted that in steady state simulations Δt represents the time interval during which the boundary conditions are assumed to be stationary. When pulsating stochastic water demands are applied at nodes of water distribution systems, this assumption is valid for Δt of the order of minutes because with a lower Δt the average value of the demand would not be representative (Giustolisi et al., 2012). Differently, in the present study, water demands are applied directly at local tanks and the stochastic fluctuations are balanced by the water volume stored in tanks. Thus, shorter time steps, of the order of tens of seconds, can be used.

Starting from the initial time $t = t_0$, the time evolution algorithm follows the steps below:

1. boundary conditions (9) for the initial istant $t = t_0$ are specified;
2. the Trust-region dogleg algorithm implemented in MATLAB is applied for the solution of the hydraulic problem, i.e., Eqs (6)-(8). The solution consists of the flow rates through the elements (Q_m for $m = 1, ..., M$) and the piezometric head at each node (h_n for $n = 1, ..., N$) at $t = t_0$;
3. depending on the water level in tanks, nodal heads, and flow rates in pipes, the decision model adjusts the status of valves and turbines (e.g., opening/closure of valves, regulation of the flow rate discharged by the turbines). By means of logical

operating rules (see following sections), the input data are processed and the parameters $\mathbf{c}$=$\{c_{c,m},c_{b,m},c_{f,m},c_{r,m}\}$ and $\tilde{Q}_m$ in (7) are updated for the calculation at time $t_1 = t_0 + \Delta t$;

4. the level of the s-th tank (h_s) at time $t_1 = t_0 + \Delta t$ is updated as $h_s(t_1) = h_s(t_0) + \Delta h_s(t_0)$, with $\Delta h_s(t_0) = \left[\sum_{m=1}^{M_s} Q_{m,s}(t_0) + Q_{p,s}(t_0) + Q_{d,s}(t_0)\right]\Delta t/\Omega_s$, where Ω_s is the tank area, $Q_{m,s}$ is the flow from the m-th element into the s-th tank, M_s is the number of elements connected to tank s, $Q_{d,s}$ and $Q_{p,s}$ are the flow demand and the inflow at tank s from local sources;
5. water demand (Q_d) and water inflow (Q_p) at local tanks are updated at time $t_1 = t_0 + \Delta t$, according to the data provided by the remote monitoring system;
6. steps 2-5 are repeated for the solution of the hydraulic problem at successive time steps.

The key advantages offered by this numerical model are: (i) to define customized and time-dependent nonlinear functions that describe the system components (e.g., the term $f(Q_m, h_{n+1}, h_n)$ in (7)); (ii) to implement sensitivity and performance analyses for different sets of operating rules that enforce the hydraulic constraints (3)-(5); (iii) to implement an algorithm with varying time step. This last point is crucial for the correct computation of the timing of turbine operations that are triggered by target tank levels (see next section). In fact, due to the small area of the water tanks, fast variations of the water levels occur. As a result, the crossing of target levels can be detected with a sufficient precision only adopting small time steps. More in detail, the water level computed with a time step Δt results in a maximum error ϵ_h equal to $\epsilon_h = \int_t^{t+\Delta t} \dot{h}\, \mathrm{d}t$. For the tank S2, $\dot{h} \sim 1$ mm/s (see Eq. (1)) and thus the time step Δt must be shorter than 10 s to keep $\epsilon_h < 1$ cm. However, too short constant time steps would lead to long and unaffordable computation times and would be in conflict with the hypothesis of steady-state conditions. For this reason, the solution algorithm adopts a variable time step, whose duration is increased or decreased when tank levels vary more or less slowly, respectively.

MANAGEMENT RULES

A feedback-control algorithm for the control of the active elements in the WSS is presented. Firstly, a regulation scheme for the operations of the turbines is recommended. Then, the developed algorithm for the control of the flow to the local networks is introduced. These rules were developed in order to satisfy the requirements defined in the subsection "Hydraulic constraints".

Reservoir regulation

The operating rules for the regulation of the flow between the inline tanks have been developed in order to satisfy the hydraulic constraints (3) - (4) that concern the water level in tanks and the minimization of the frequency of the control operations. The rationale behind the proposed regulation is: (i) to reduce as much as possible the variations of the tank level, and (ii) to keep the level of tanks as high as possible. In order to explain how points (i) and (ii) are actually implemented in the SCS, the level regulation of tank S1 by the turbine T1 is analyzed (see Fig. 3a,b). The level of S1 (upstream tank) remains constant if

$$Q_{T1} = Q_{IN,S1} - \sum_{i=A}^{D} Q_{EX,i}, \tag{10}$$

where $Q_{IN,S1}$ is the flow into S1 from the WPP, $Q_{EX,i}$ is the flow supplied to the i-th municipality located between S1 and S2 (i.e., municipal local networks A-D), and Q_{T1} is the flow through the turbine T1. $Q_{IN,S1}$ and $\sum_{i=A}^{D} Q_{EX,i}$ are boundary conditions. The only way to satisfy Eq. (10) and to keep the tank level constant is to adjust the term Q_{T1}. The local water demand ($\sum_{i=A}^{D} Q_{EX,i}$) exhibits a great variability over time. Thus, the flow through T1 should be continuously updated to maintain a perfectly constant level in S1. However, this continuous adjustment of the flow through the turbine is not possible for technical reasons, as explained in section "Hydraulic Constraints". In order to reduce the number of flow adjustment operations, the water level in tanks is allowed to vary at most of about 1 m (a modest oscillation compared to the total tank height of 5.1 m). More in detail, the tank level is allowed to vary between two regulation thresholds $h_{U,1}$ and $h_{L,1}$, where subscripts "U" and "L" refer to upper and lower thresholds, respectively (see Fig. 3). The actual value of these thresholds is selected on the basis of technical considerations, and will be detailed in the following sections. Adjustments of the turbine opening are performed only when the water level in S1 exceeds $h_{U,1}$ or goes below $h_{L,1}$ (see Fig. 3a,b). When the water level goes below $h_{L,1}$, the value of Q_{T1}, calculated using (10), is rounded down to the nearest discrete value of discharge that can be regulated by the turbine (see Eq. (2)). In this way, the flow rate leaving S1 is slightly lower than the flow rate that precisely satisfies Eq. (10) and the water level in S1 slowly rises. On the other hand, when the level exceeds $h_{U,1}$, the value of Q_{T1} evaluated with (10) is rounded up to the nearest discrete value given by Eq. (2) and the level in S1 slowly decreases. The flow regulation from S2 to S3 is performed by T2. The control parameters are Q_{T1} (that plays the same role of $Q_{IN,S1}$ in the regulation of T1), Q_{T2}, Q_E, Q_F, and h_2. The same operating rules described before are followed (see Fig.3c). Flow from S3 to S4 is regulated by T3. The involved parameters are Q_{T3}, Q_{LV} (i.e., the flow to the municipalities of the lower valley) and h_4. The upstream tank (S3) is nearly nine times bigger than the downstream tank S4 and therefore h_3 is less sensitive to flow rate fluctuations. For this reason, the updating of Q_{T3} is performed focusing on the level h_4 of the downstream tank, S4. As discussed in the next section, the water level in tank S3 is instead used to control the flow rate discharged to the lower valley, Q_{LV}.

Coordination between the WSS and the local water systems

Some local supply systems of the lower valley are affected by the following criticalities: (i) low quality of spring water (e.g., turbidity), (ii) low quality of groundwater (e.g., high concentrations of sulphate), and (iii) high cost of pumping operations. It is therefore convenient for technical and economic reasons to replace these local sources with the water conveyed by the water main. This replacement can be done whenever the municipalities of the upper valley consume less water than the WPP production and, then, there is an excess of water available for the lower valley. The water availability for the lower valley is estimated by monitoring the level of S3, which has by far the largest storing capacity of the system. More in detail, when the level of S3 exceeds the threshold $h_{U,3}$ (see Fig. 3e), the SCS performs the two following steps:

1. the surplus of water that is accumulated in the upper valley is evaluated as the difference between the inlet flow in S3 and the flow released from S4 to the lower valley (Q_{LV})

$$Q_{SUR} = Q_{IN,3} - Q_{LV} \tag{11}$$

   It is a key point that since water levels in S1 and S2 are kept almost constant, the flow $Q_{IN,3}$ is a good estimate of the difference between the water produced in the WPP and the water consumed by towns A-F in the upper valley;

2. a number (n_{OFF}) of local sources in the lower valley are turned off so that

$$\sum_{i=1}^{n_{OFF}-1} \overline{Q}_{P,i} \leq Q_{SUR} \leq \sum_{i=1}^{n_{OFF}} \overline{Q}_{P,i}\,, \tag{12}$$

   where $\overline{Q}_{P,i}$ is the mean flow production of the i-th local source. The priority of the sources to be turned off must be specified on the basis of technical and economic criteria.

When the tank level falls below the lower threshold $h_{L,3}$, the algorithm is similar to the previous case. Steps 1 and 2 are repeated, but the logical condition (12) is replaced by

$$\sum_{i=1}^{n_{OFF}} \overline{Q}_{P,i} \leq Q_{SUR} \leq \sum_{i=1}^{n_{OFF}+1} \overline{Q}_{P,i} \tag{13}$$

Moreover, steps 1 and 2 are repeated every 6 hours even if no threshold is crossed. This time interval approximately corresponds to the time required for significant variations of local consumption. In order to select the number (n_{OFF}) of local sources to be turned off, Eq. (12) or (13) is enforced again using the updated value of Q_{SUR}.

RESULTS

A 3-year-long simulation of the coupled hydraulic and decision models was performed in order to test the operating rules presented above under different scenarios. Input data are: (i) the scheduled discharge released from the WPP, (ii) the water consumption in the local water networks, and (iii) the flow available from the local sources.

The value of the regulation thresholds (Table 1) used in the model was obtained from a sensitivity analysis that will be described in the following section. The analysis was performed to find the threshold values that maximize the water storage in tanks and minimize the number of turbine operations.

In order to evaluate the effectiveness of the feedback-control algorithm, we verify that the hydraulic constraints (see section "Hydraulic Constraints") are respected. The first constraint (3) concerns the water level in tanks. Fig. 4a reports the temporal evolution of tank levels over a typical time period of four days. The water level in S1, S2, and S4 varies between the regulation thresholds. The level in these tanks controls the flow discharged by the turbine that is installed just upstream (for S3) or downstream (for S1 and S2) the tank. These discharges are regulated in order to fulfill the balance equation (10) and thus to keep the tank water level between the regulation thresholds. The regulation of the water level in tank S3 is different from those of the other tanks. The water level in S3 decreases up to the lower thresholds $h_{L,3}$ and then rises, exceeding the upper threshold $h_{U,3}$. This happens because the water level in S3 controls the flow delivered to the lower valley from tank S4. When h_3 falls below the lower threshold $h_{L,3}$ (24 April in Fig. 4a), the operations reported in (11) and (13) are performed and the level immediately rises. On the other hand, when the upper threshold $h_{U,3}$ is exceeded, operations (11)-(12) are performed. However, the level remains above $h_{U,3}$ because the water released from the WPP replaces all the critical water sources in the valley, and thus there are no other local sources to turn off (see Fig. 3e). The equilibrium between $Q_{IN,3}$ and Q_{LV} stabilizes the water level h_3 above $h_{U,3}$. Level h_3 starts to decrease when a reduction in the water flow from the WPP occurs or when the water demand in the lower valley increases. Notice that the temporal evolution of water levels in S1 and S2 follows a daily recurrent pattern. In fact, the only parameters that affect the evolution of h_1 and h_2 are the flow rates delivered to the municipalities of the upper valley, that present a daily consumption pattern. Instead, daily patterns are not observed in tanks S3 and S4. In fact, water level variations in S3 and S4 depend on the flow discharged to the lower valley (Fig. 3d,e) which is updated when the regulation thresholds in S3 are crossed or when a period of six hours has passed without any crossings. In order to have a global view of the behavior of the 3-year-long system dynamics, the relative frequency (RF) of the water level in the four tanks is evaluated (Fig. 4b). The average water level in tanks is high and guarantees an average total water reserve of 14830 m^3, 80% of which is stored in S3. Therefore, in case of a pipeline failure upstream of S3, the water stored in S3 can sustain the total downstream population ($\sim$ 100 000 inhabitants) for an average time of 8 hours. Actually, this duration underestimates the system resilience because most of the local sources of the lower valley are gradually reactivated (i.e., the well fields) when the water level in S3 decreases. In this situation, a higher cost for the water supply (i.e., pumping and water treatment costs) must be taken into account. Levels in tanks S1, S2, and S4 are always restrained between the physical bounds of the tanks, thus avoiding empty and overflow conditions during the whole simulation time. The RF distribution tails often extend beyond the regulation thresholds (Fig. 4b) because of the delay in the response of the water level to the flow regulation performed by the turbine. However, the clearance between the regulation thresholds and the tank physical limits prevents the occurrence of overflow and empty conditions. Finally, the average value of h_3 is higher than $h_{U,3}$ because the water provided by the WPP satisfies the full water demand of the lower valley for most of the simulation time.

The second constraint (4) concerns the minimization of the number of operations performed by the turbines. The histograms in Fig. 5 report the daily average number of flow rate variations with magnitude ΔQ_T performed by the turbines. The size of the histogram classes (12.5 l/s for T1 and T2 and 50 l/s for T3) corresponds to the smallest amount of change that can be regulated by the turbine, that is 10% the nominal flow rate through a jet. Turbines T1 and T2 display only mild flow rate variations ($\Delta Q_T < 150$ l/s and $\overline{\Delta Q}_T \simeq 30$ l/s) thanks to the high sensitivity of the four-jet Pelton turbine governing system. Differently, the flow rate through T3 is affected by larger variations ($\Delta Q_T < 400$ l/s and $\overline{\Delta Q}_T \simeq 125$ l/s). Finally, it should be noted that each turbine experiences less than 9 flow rate adjustments over a day. As will be discussed in the following section, the number and the extent of these variations are the compromise between the maximization of water storage in tanks and the minimization of the number of turbine operations.

The third constraint (5) concerns the minimization of the total cost of water production in local plants. Energy consumption and pump maintenance represent the major cost for the extraction of groundwater in local water supply systems. Therefore, a remarkable reduction of costs can be achieved by replacing the local groundwater with water from the WPP. Differently, local springs are generally high quality water sources, that only require mild and cheap treatments. However, during rain events turbidity greatly increases. Hence, local treatment can become very expensive and the exploitation of water from the WPP should be preferred. Fig. 6a shows the relative composition of water in the local networks before the realization of the new aqueduct. Spring water is the main source of water supply in the upper valley (A to F communities). In the lower valley (G to V communities), water is mainly extracted by wells. Fig. 6b exhibits the composition of water supply after the introduction of the new WSS. Groundwater is completely replaced, except for towns N (where technical issues prevent the total replacement of groundwater) and V. For technical and economic reasons, the local wells in V are the first ones to be reactivated when the flow released from the WPP is not sufficient to satisfy the water demand of the entire valley. The last bar in panels (a) and (b) of Fig. 6 gives the mean composition of the water supplied to the whole valley. After the realization of the new WSS, 62% of the water demand is provided by the WPP. Spring water remains an important water source (32%), whereas groundwater contribution decreases from 67% to 6%. This reduction results in a significant energy saving.

Fig. 7a illustrates the advantages of the new WSS in energetic terms, considering both hydropower generation and energy saving from reduced pumping. Energy saving from pumping is approximately 10 MWh per day. Energy generation from the installed turbines increases with the water flow released from the WPP, whose temporal pattern is reported in Fig. 7b. A maximum production of 57 MWh per day occurs between March and April, when the WPP delivers 500 l/s. Notice that even for a constant value of water flow rate released from the WPP (e.g., 500 l/s), energy generation decreases during touristic seasons (energy production is 57 MWh in March and only 47 MWh in January, see Fig. 7b). During touristic seasons the water demand in the municipalities of the upper valley (towns A to F) strongly increases and the water consumption upstream T1 is five times higher with respect to the remaining part of the year. As a consequence, the water volume that flows through the turbines considerably decreases.

SENSITIVITY ANALYSIS

The regulation thresholds reported in Table 1 were selected focusing on the reserve of water in tanks, and the number of turbine operations. Energy production from turbines was not considered in this analysis because the effect of level variations on the total heads at turbines is negligible and hydropower generation is hence unaffected.

The preliminary step was to understand the effect of changes in the regulation thresholds on the water volume stored in tanks and on the number of turbine operations. The upper threshold (h_U) was fixed, while the lower threshold h_L was varied as $h_L = h_U - \Delta H$, with ΔH taking values between 0.3 and 2.4 m. Fig.8 reports the number of turbine operations and the volume of water stored in tanks S1, S2, and S4 as a function of ΔH. As ΔH increases up to 1.5 m, the number of turbine adjustments decreases rapidly and for larger ΔH no additional reduction is obtained (Fig. 8a). As to the effect of ΔH on the water volume stored in tanks, the mean and minimum water storage volumes strongly decrease with ΔH (Fig. 8b-d). As a consequence, high values of ΔH induce a reduction of the system resilience in case of failures. In particular, when ΔH is larger than 1.2 m only half of the capacity is exploited for tanks S2 and S4. After these analysis, ΔH for tanks S1, S2, and S4 was set equal to 1.2 m as a compromise value between the optimization of turbines performance and the optimization of the system resilience. A similar sensitivity analysis was performed for the regulation thresholds in tank S3. However, the water volume stored in tank S3 presented no significant variations. According to the operations (11)-(13) the water released to the lower valley from S3 depends on prescribed temporal variations of municipal water demands and of water production at the WPP. Therefore, variations in the level thresholds $h_{U,3}$ and $h_{L,3}$ have little effects on h_3. Additional simulations were conducted to test other combinations of ΔH for tanks S1, S2, and S4 (Table 2). Fig. 9 reports the total number of turbine operations and the total minimum volume of water stored in all tanks for the different investigated scenarios. These results are reported in relation to those obtained with the adopted solution (Table 1 and scenario 2 in Table 2). The points lying on the dashed line represent the Pareto front of the nondominated scenarios. The solutions lying on the Pareto front are equivalent because none of the objectives can be improved without worsening the other ones. Among these nondominated solutions, the scenarios included in Region I present a greater water storage but a higher number of turbine operations, in relation to scenario 2. On the other hand, the scenarios in Region II result in a lower number of turbine operations but offer a lower water reserve. Among the Pareto optimal solutions, scenario 2 was selected for its central position in the domain of the possible solutions.

CONCLUSIONS

In this work a feedback-control algorithm has been proposed for the operation of a newly designed water supply system in an Italian Alpine valley. The operating rules have been developed in order to satisfy the tight hydraulic constraints imposed by the valley topography. In particular, the aqueduct is characterized by long and steep pipes and small inline tanks. Moreover, the regulation of the connections with the existing local water systems was taken into account.

The effectiveness of the proposed algorithm has been assessed through numerical simulations of the coupled hydraulic and decision model, showing that the hydraulic and regulation constraints are satisfied. In particular: (i) water storage in tanks is sufficient to guarantee a reserve of at least 8 hours in case of failures; (ii) the number of turbine operations is low (less than 9 flow rate adjustments over a day for each turbine) in order to extend the control devices lifespan; (iii) expensive groundwater is almost completely replaced by high quality water from the WPP, inducing a saving of more than 10 MWh/day; (iv) the turbines installed along the water main allow an energy recovery of around 40 MWh per day.

These results evidence that a comprehensive regulation of water supply systems allows for a multipurpose management of water resources on large-scale areas. Moreover, the advantages of connecting multiple municipal water systems and of integrating hydropower generation in water supply are highlighted in terms of water supply quality, resilience and cost, and in terms of renewable energy generation. The control algorithm developed for the automated remote control of the WSS can prove useful for the design of the flow regulation system in similar mountain water networks. In particular, the use of flow balance equations and triggering thresholds for the tank levels represents an efficient technical solution for the control of inline turbines with small storage tanks. Finally, the distribution of water resources to the connected municipalities based on (i) a centralized priority list and (ii) on the available water storage in the main tank can be a valuable approach for water management in similar extensive networks.

ACKNOWLEDGMENTS

We gratefully acknowledge SMAT Group for the financial support to this research and for providing valuable information. The Editor and three anonymous Reviewers are acknowledged for their suggestions that helped us to improve our work.

NOTATION

The following symbols are used in this paper:

$A, B, ..., V$	local water systems;
$\hat{a}$	flow exponent;
$\hat{b}$	minor loss coefficient;
$\mathbf{c}$	vector of the regulation coefficients;
E	optimal total cost of water production;
$\mathbf{E}$	vector of water production cost for different regulation schemes;
e_i	cost of the i-th water source;
$f(\cdot)$	generic non-linear function for pumps, turbines and valves;
h	tank level and nodal head;
h_U (h_L)	tank upper (lower) threshold;
i, j	running indexes;
J	number of turbine jets;
k	positive integer [0, 10· J];
M (N)	number of elements (nodes) in the network;
m (n)	running index for elements (nodes);
n_L (n_{OFF})	number of (deactivated) local sources;
$\mathbf{n_T}$ ($\mathbf{n_V}$)	vector of the number of turbine (valve) operations for different regulation schemes;
n_T (n_V)	minimum number of turbine (valve) operations;
Q ($\tilde{Q}$)	flow (flow imposed) in a element;
Q_d	flow demand at nodes or in tanks;
$Q_{EX,A...R}$	flow to the local water networks;
Q_{IN} (Q_{OUT})	incoming (outgoing) flow in a tank;
Q_{LV}	flow to the lower valley;
Q_N	turbine nominal flow;
$\overline{Q}_P$	mean flow production from the i-th local source;
Q_{SUR}	surplus of water in the upper valley;

Q_{Ti}	flow through i-th turbine Ti;
$\hat{r}$	resistance coefficient for pipes;
S	tank;
s	running index for tanks;
T	turbine;
T_w	waiting time between control decisions for the local sources;
ϵ_h	error in the computation of water level in tanks; and
Ω	tank area.

SUPPLEMENTAL DATA

Tabs. S1-S2 and Fig. S1 are available online in the ASCE Library (ascelibrary.org).

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List of Tables

Table 1. Threshold values in tanks for the triggering of (10) and (11)

Thresholds	S1	S2	S3	S4
h_U [m]	1264.5	1009.6	741	609.7
h_L [m]	1263.3	1008.4	739	608.5

Table 2. Combinations of ΔH in tanks S1, S2 and S4 studied in the sensitivity analysis. Scenario 2 corresponds to the values in Table 1.

Regulation range	1	2	3	4	5	6	7	8	9
ΔH_1 [m]	0.6	1.2	2.4	0.6	2.4	1.2	1.2	1.2	1.2
ΔH_2 [m]	0.6	1.2	2.4	1.2	1.2	0.6	2.4	1.2	1.2
ΔH_4 [m]	0.6	1.2	2.4	1.2	1.2	1.2	1.2	0.6	2.4

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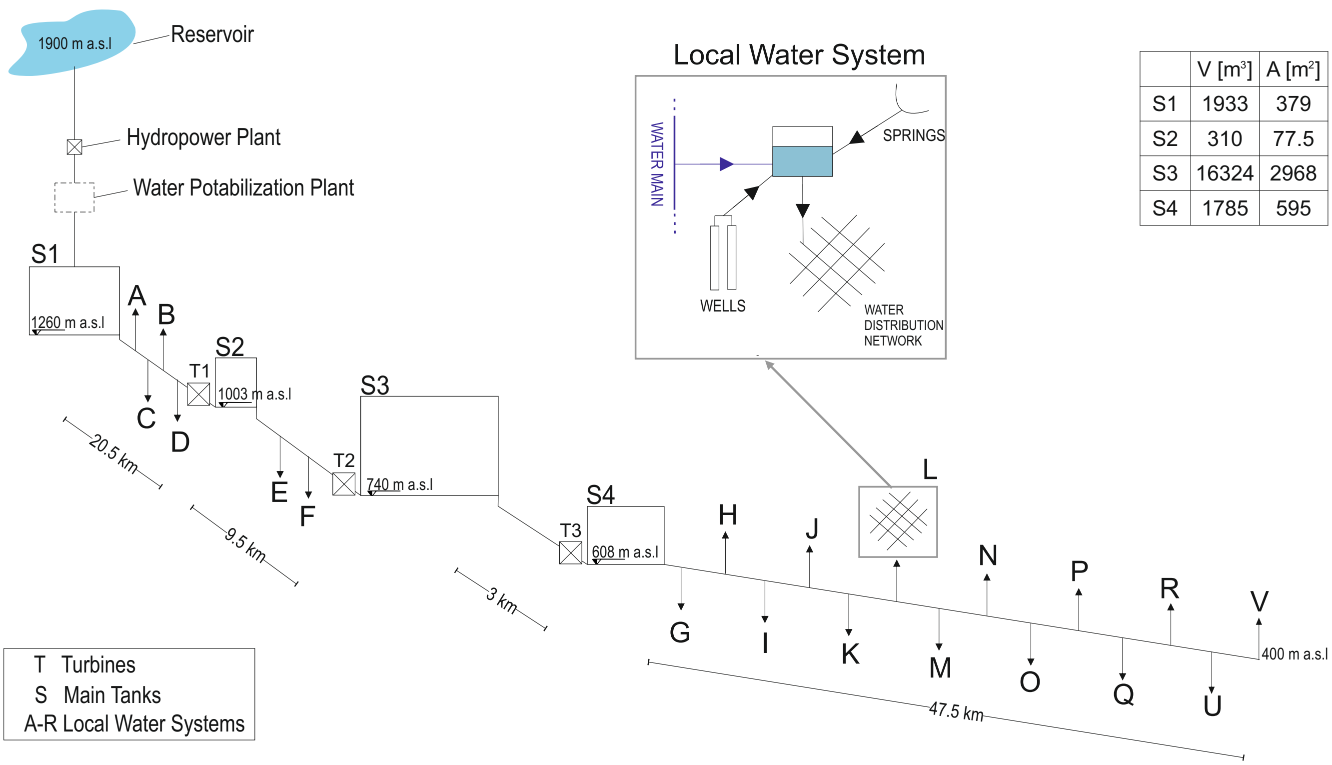

Figure 1. Scheme of the WSS. The capital letters (A,B, ... V) indicate the local water networks. The inset shows a typical local water system with a storage tank supplied by mountain springs, local wells and by the new water main.

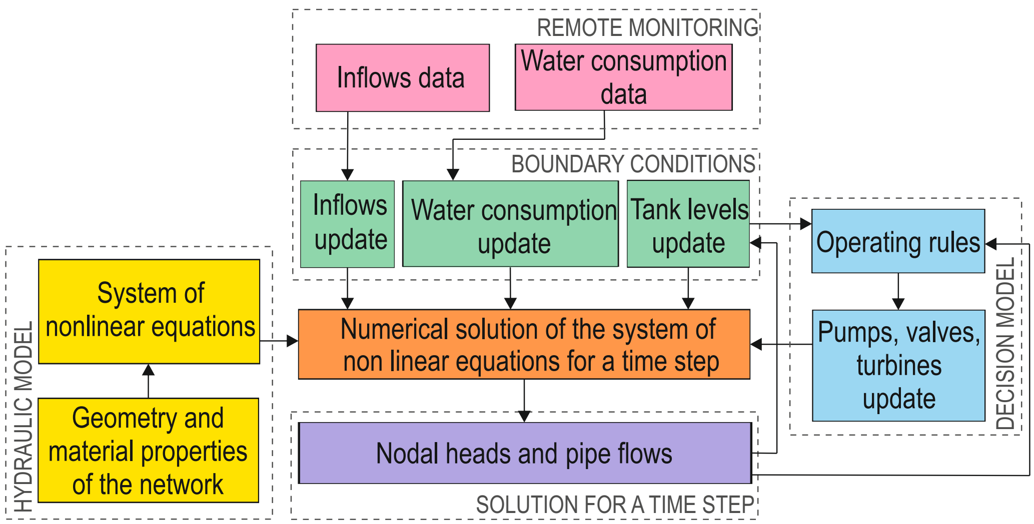

Figure 2. Scheme of the numerical model developed to simulate the operation of the new WSS.

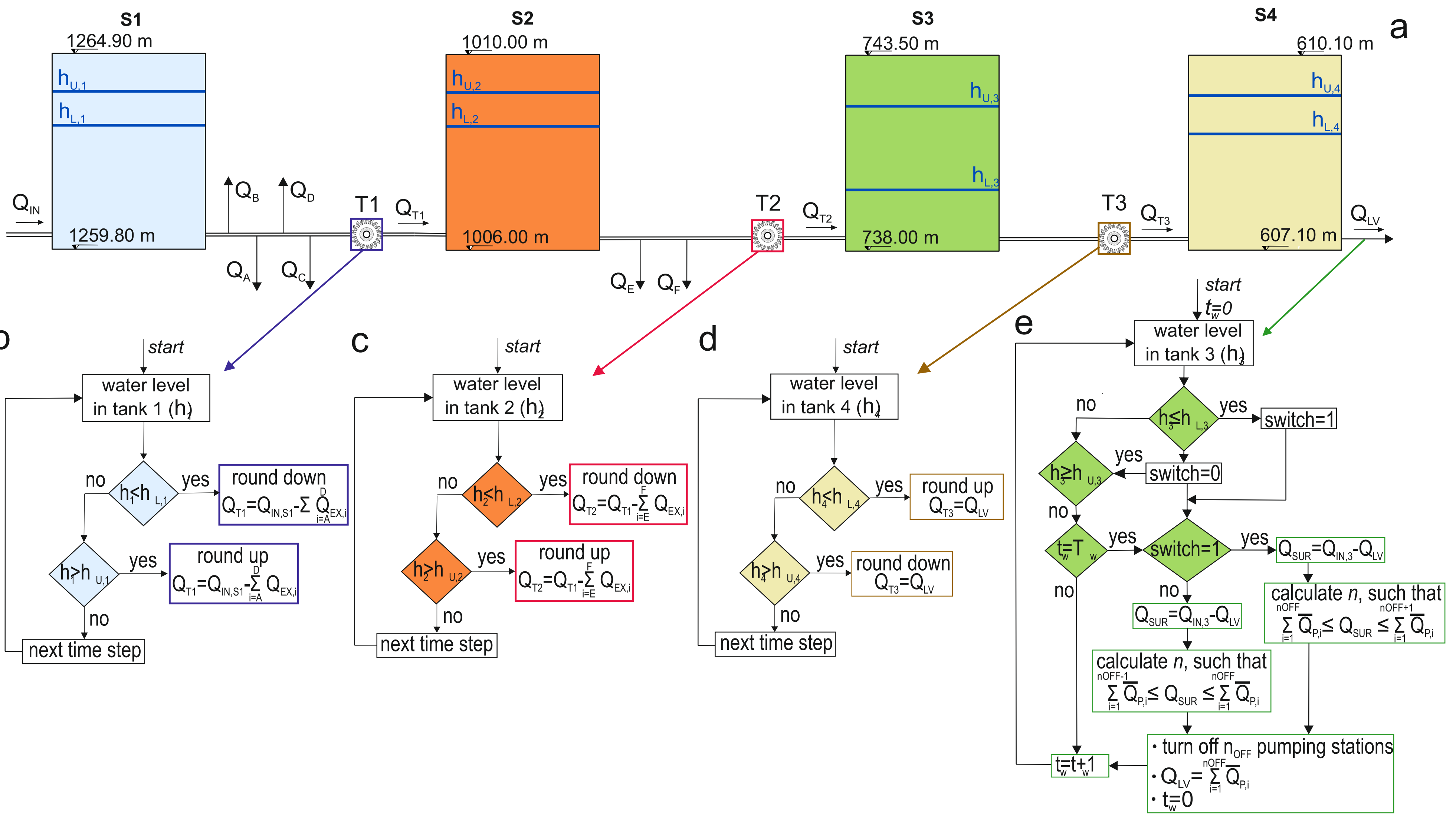

Figure 3. Scheme of the feedback-control algorithm of the new WSS. Panel (a) represents the scheme of the hydraulic system. The block diagrams report the operating rules for T1 (b), T2 (c), T3 (d) and for the control of the local sources in the lower valley (e).

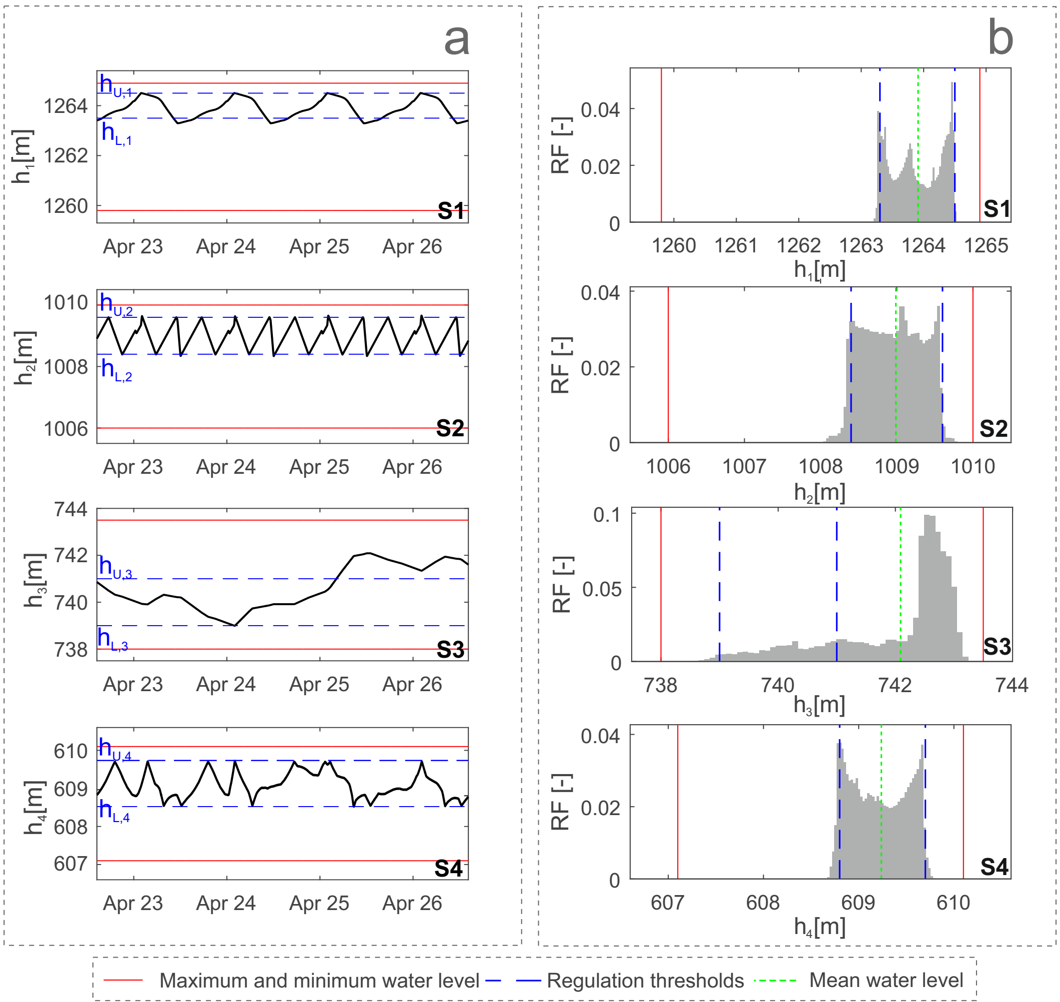

Figure 4. (a) Typical time series for a generic four days period. (b) Relative frequency of levels in tanks S1, S2, S3 and S4 for a 3-years simulation time.

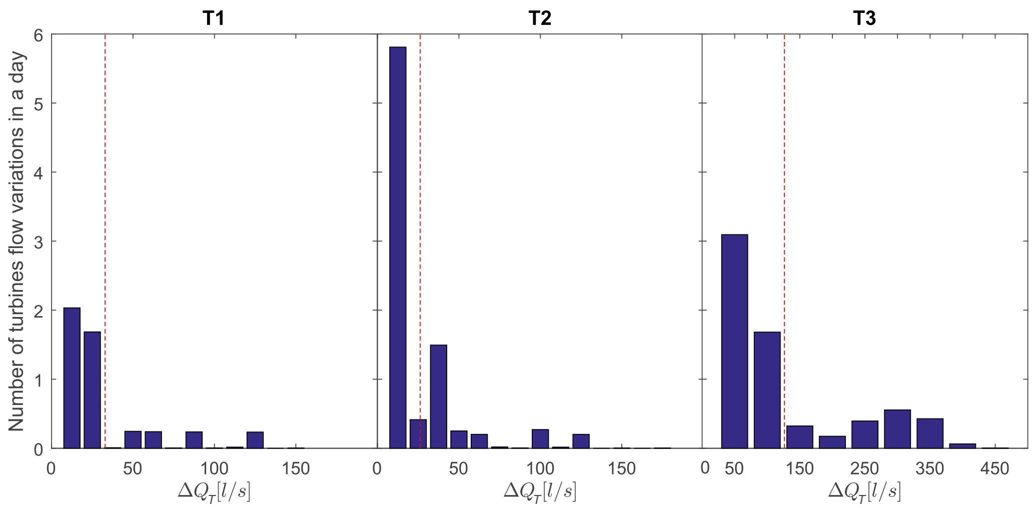

Figure 5. Daily number of flow rate variations with magnitude ΔQ_T for turbines T1, T2 and T3. The dashed line indicates the average magnitude of ΔQ_T.

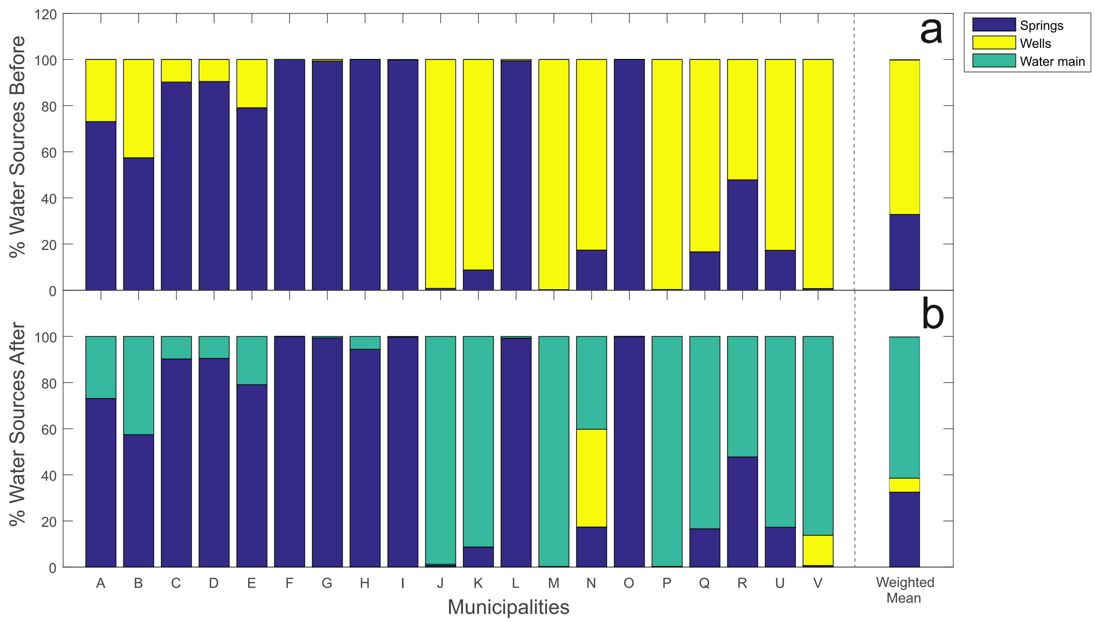

Figure 6. Composition of the water sources supplying each local network (a) without water from WPP and (b) with the water from WPP. The last bar on the right reports the mean composition in the entire valley.

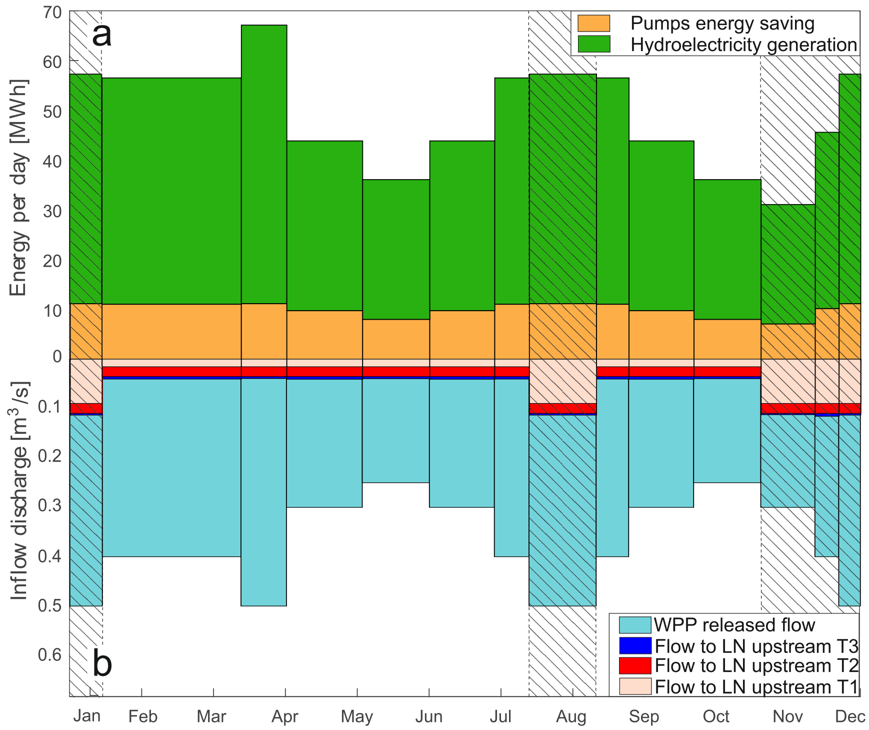

Figure 7. (a) Daily energy saving and hydropower generation over a typical year. (b) Released discharge from the WPP, flow delivered to the local networks upstream T1, T2 and T3. The hatched areas highlight touristic seasons.

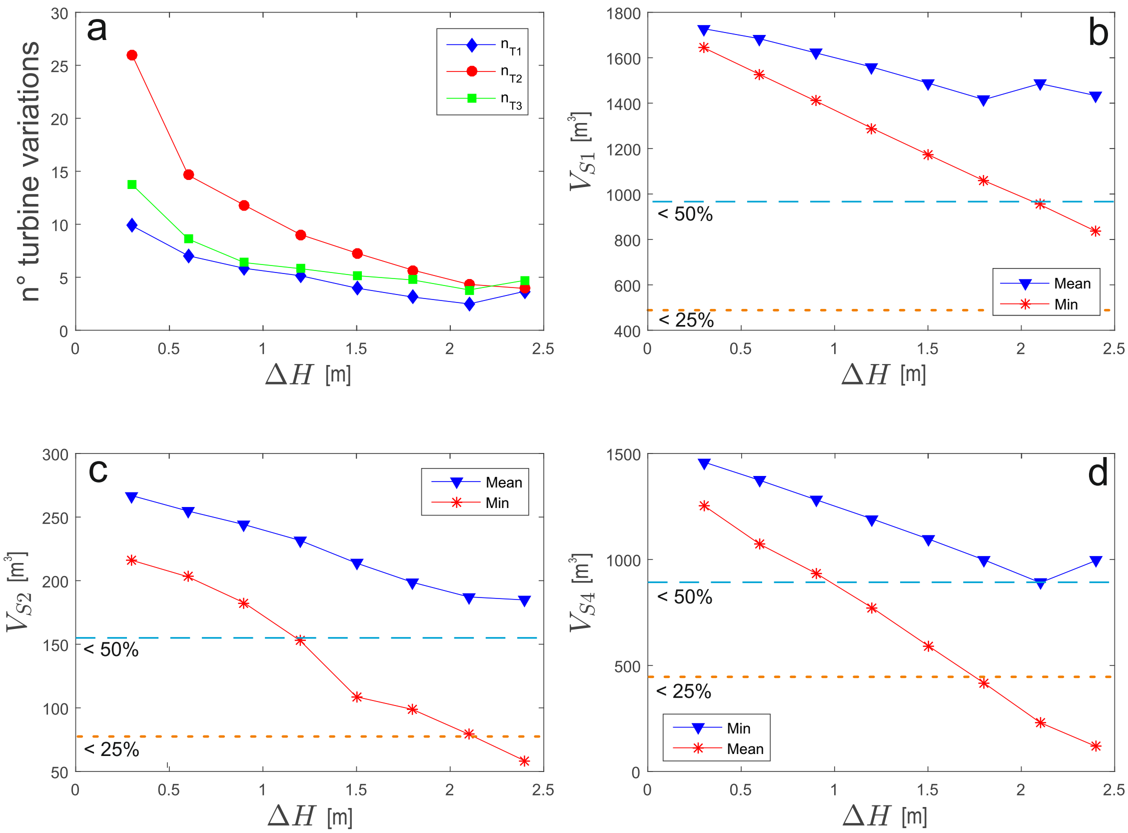

Figure 8. (a) Number of flow rate variations in a day for turbines T1, T2 and T3. (b)-(d) Minimum and mean water volume stored in tanks S1, S2 and S4 for different ΔH. The dashed and dotted lines mark the 50% and 25% of the total tank capacity, respectively.

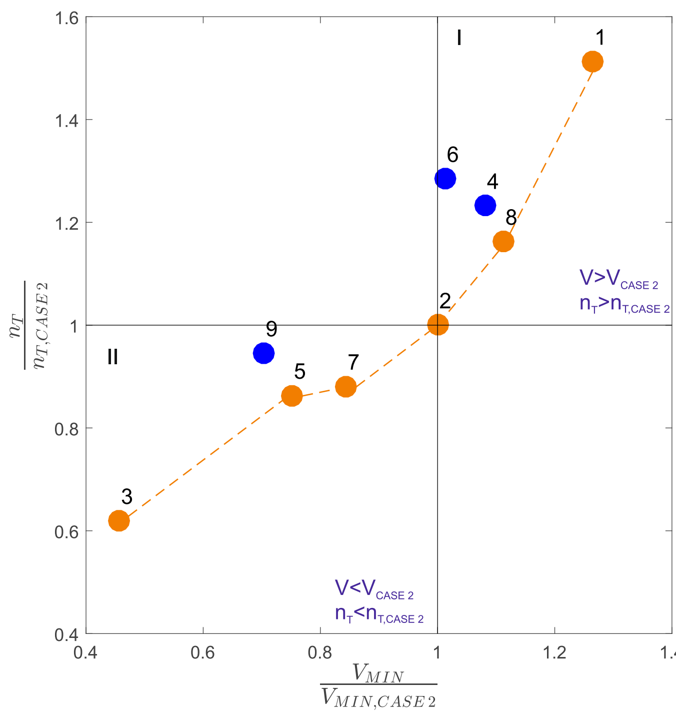

Figure 9. Number of turbine operations against minimum water storage in tanks for different combinations of ΔH_1, ΔH_2, ΔH_4, in relation to the case of Table 1. The points lying on the Pareto front (dashed line) are the nondominated solutions. Notice that scenario 2 corresponds to Table 1