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

S.Fellini,¹ R.Vesipa,² F. Boano³ and L.Ridolfi⁴

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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22 of Italy, water shortages are mostly due to failures in local water supply sources. As a
23 consequence, an effective strategy is the creation of intermunicipal water networks which
24 connect multiple local water systems (Massarutto, 2000). In this way, the production and
25 treatment of drinkable water can be performed in a small number of facilities. It follows
26 that: (i) the use of the best quality water is privileged, (ii) the cost of water treatment
27 is reduced, and (iii) the resilience of the system is increased thanks to the diversification
28 of water sources. In addition, a growing awareness about renewable energy is increasingly
29 leading to the integration of small hydropower plants in water systems. In this way the excess
30 potential energy of water is converted into electric power (e.g., Filion et al., 2004; Carravetta
31 and Giugni, 2009; Fontana et al., 2011) and optimal pressure values are maintained in the
32 network (Tricarico et al., 2014; Fecarotta et al., 2015). Pumps as turbines (PAT) are an
33 innovative, low-cost, and reliable solution for energy production in water systems where
34 pressure and flow conditions are variable and the available power is limited (Carravetta
35 et al., 2012, 2014; Lydon et al., 2017). On the other hand, the installation of traditional
36 turbines (e.g., Pelton turbines) is better suited to large transmission pipelines and especially
37 to mountainous regions where energy potential is high (Afshar et al., 1990; Möderl et al.,
38 2012; Sitzenfrey and Rauch, 2015).

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

45 In this framework, the goal of this study is to present an optimal control algorithm
46 developed for the operation of a newly designed water supply system (WSS) in an Alpine
47 valley in the north of Italy. The WSS consists of a 80 km-long water main that runs along
48 the valley connecting 20 municipal water supply networks. The system takes water from

49 a hydropower plant and supplies the valley population with 250-500 l/s. The water main
50 starts at 1260 m above sea level and ends at 400 m with an 860 m difference in altitude. Four
51 inline tanks are present along the water main and the excess water pressure is converted in
52 hydropower by three turbines. A key characteristic of this system is that the size of the
53 inline tanks is severely constrained by the topography of the valley. As a result, the tanks
54 are particularly small in comparison with the daily volume delivered by the water main.
55 The first target of the WSS is to reliably provide high quality water to the local municipal
56 systems when local sources fail or their water quality is low, and when the water treatment
57 in the main plant is cheaper than in the local plants. The second target is to generate
58 hydroelectricity. Tailoring the optimal hydraulic control to achieve these two targets is not
59 trivial and presents both conceptual and technical difficulties. At the conceptual level, it is
60 necessary to develop a robust control strategy to be implemented in the supervisory control
61 system in order to guarantee the objectives of the WSS. The technical difficulties concern the
62 availability of control devices (valves and turbines) that can perform the proposed regulation.

63 Over the last years, several studies have focused on different aspects concerning the
64 optimal regulation of water supply systems. In this context, numerical simulations allow
65 a realistic representation of complex water systems, which involve economic, social and
66 engineering issues (e.g., Jain et al., 2005). Therefore, the vast majority of water system
67 planning and managing studies is based on numerical modeling approaches (e.g., Rani and
68 Moreira, 2010). A widely adopted approach (e.g., Lund and Guzman, 1999) is to define
69 operating rules based on engineering targets, and to check the response of the modeled
70 system. In this approach, a centralized regulation system is usually implemented and possible
71 coordination mechanisms for multiple water storing facilities at large scale are proposed by
72 various authors (e.g., Anghileri et al., 2013; Ficchi et al., 2016). In fact, coordination in
73 operations increases the system efficiency and resilience, especially in a context of adverse
74 conditions such as climate change and increasing water demand (e.g., Marques and Tilmant,
75 2013).

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

83 Referring to the specific case study, the hydraulic constraints and the regulation objectives
84 are formulated as a set of mathematical conditions (see the section “Hydraulic Constraints”).
85 The operating rules for the optimal hydraulic and energy regulation are then developed ac-
86 cording to these constraints (see the section “Management Rules”). Flow balance equations,
87 triggering thresholds for the tank levels and a centralized management approach are the
88 main tools for the regulation system. Beside the specific case study, the proposed solu-
89 tion approach provides guidance for the design of the control system for modern WSSs in
90 highly populated mountain regions, where the available storage volume constrains the sys-
91 tem reliability, several local water systems have to be networked, and energy recovery can
92 be performed.

93 **CASE STUDY**

94 The WSS will be in service along an Alpine valley in northwestern Italy with a total
95 population of 115 000 inhabitants distributed in 20 municipalities. These municipalities are
96 very different in terms of size, population, and economic activities. The upper valley is
97 characterized by small towns with a permanent population ranging between 300 and 3000
98 inhabitants. Tourism is the main economic activity and during the ski season population
99 can increase by one order of magnitude. The municipalities of the lower valley enclose
100 most of the valley permanent population and host a range of industrial and commercial
101 activities. In particular, the main town (marked with V in Fig. 1) hosts 50000 inhabitants
102 and several industrial activities. Currently, water is provided by local water supply networks

103 operating independently one from each other. A typical local network (see inset in Fig. 1)
104 is characterized by a local storage tank collecting water from springs or wells and supplying
105 the local population. Groundwater pumped from wells is often necessary to satisfy the local
106 water demand. For each local network, the amount and the temporal pattern of daily water
107 demand as well as the relative contribution of each local source (e.g., water wells) are known
108 from historical data (see Fig. S1 and Table S2 in the Supplemental Data).

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

126 **Characteristics of the hydraulic system**

127 The new WSS takes water from a 3 500 000 m³ reservoir located at an altitude of 1900
128 m a.s.l. This reservoir stores water with high physicochemical quality and is currently used
129 for hydroelectric purposes. Thanks to an agreement between the hydropower and the water

130 supply companies, the flow released from the hydroelectric power plant is constant during
 131 the week but varies in the range 250-500 l/s according to a monthly schedule (see Table S1 in
 132 the Supplemental Data). This discharge is collected downstream of the hydroelectric plant
 133 and is transferred to the water potabilization plant (WPP). After rather mild treatments,
 134 water is stored in a first water tank (S1 in Fig. 1) and then delivered to the municipal
 135 water systems of the valley by a pressurized water main. The water main is a 700 mm
 136 diameter ductile iron pipe with a roughness of 0.1 mm and with thrust-resisting joints. In
 137 the upper valley the water pipe is characterized by steep slopes (2% on average, with 10%
 138 in the steepest profile). Three intermediate inline tanks (S2, S3, and S4 in Fig. 1) split the
 139 water main in order to limit the static water pressure in the pipes. Downstream of the last
 140 tank (S4 in Fig. 1) the water main runs for 50 km along the lower part of the valley, where
 141 14 of the 20 supplied municipalities are located. A key feature of the system is that the
 142 topography of the valley severely constrained the construction of tank S2, that has an area
 143 of 80 m² and a height of 4 m. As a result, small changes in the inflow or outflow discharge
 144 induce large changes in water levels. The rate of level variations \dot{h} is given by

$$145 \quad \dot{h} = \frac{dh}{dt} = \frac{1}{\Omega} \cdot \Delta Q \quad (1)$$

146 where h is the level in tank, t is time, Ω is the tank area and ΔQ is the net flow to the tank.
 147 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
 148 5 minutes (that is required for adjusting the flow rate with the installed valves or turbines)
 149 the level of S2 varies as much as 40 cm which corresponds to 10% of the tank height.

150 The whole system is monitored and controlled by a supervisory control system (SCS).
 151 The flow rates into and out of the tanks and the flow rate delivered to the local water systems
 152 are measured by electromagnetic flow meters, whereas water levels in tanks are measured
 153 by ultrasonic level sensors. The acquired data are processed by a decision algorithm that
 154 calculates the target values for the control devices. The operating rules implemented in the

155 decision algorithm have been developed in order to optimize the operations of the WSS and
156 are presented in the next section “Management rules”. The active elements controlled by
157 the SCS are turbines and valves. Pelton turbines with electronically controlled Doble needles
158 adjust the flow that enters into the intermediate tanks. Needle valves with electric actuators
159 regulate the flow from the water main to the local water systems. The time interval for flow
160 adjustment is longer than 5 minutes. This slow regulation results in smooth transitions from
161 one steady state to another one, without large pressure and flow fluctuations (Boulos et al.,
162 2005). In this way, water hammer and hydraulic resonance (e.g., Riasi et al., 2010) in the
163 system are prevented.

164 **Characteristics of the energy recovery system**

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

$$178 \quad Q_T = k \frac{Q_N}{10 \cdot J}, \quad (2)$$

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

183 **Hydraulic constraints**

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

189 Water level h in tanks must satisfy three conditions

$$190 \quad h > h_{\text{MIN}}, \quad h < h_{\text{MAX}}, \quad h \rightarrow h_{\text{MAX}}, \quad (3)$$

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

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

204 must be as low as possible, namely

$$205 \quad n_T = \min[\mathbf{n}_T], \quad n_V = \min[\mathbf{n}_V] \quad (4)$$

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

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

$$215 \quad E = \min[\mathbf{E}], \quad \mathbf{E}_j = \sum_{i=1}^{n_L} e_i \bar{Q}_i \quad (5)$$

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

220 METHODS

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

226 The hydraulic model (see Fig. 2) is a in-house developed MATLAB code consisting of
227 a system of $(M+N)$ nonlinear equations, where N is the number of nodes and M is the

228 number of elements (i.e., pipes, valves, pumps, and turbines). The following equations are
 229 used to model the hydraulic behavior of the system. The *flow-head loss relation* in the m -th
 230 pipe that connects nodes n and $n+1$ reads

$$231 \quad h_{n+1} - h_n - \hat{r} \cdot |Q_m^{\hat{a}-1}| Q_m - \hat{b} |Q_m| Q_m = 0; \quad (6)$$

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

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

$$235 \quad c_{c,m} Q_m + c_{b,m} \cdot (h_{n+1} - h_n) + c_{f,m} \cdot (Q_m - \tilde{Q}_m) + c_{r,m} \cdot [h_{n+1} - h_n - f(Q_m, h_{n+1}, h_n)] = 0, \quad (7)$$

236 where the set of coefficients $\mathbf{c} = \{c_c, c_b, c_f, c_r\}$ indicates if the m -th element is closed ($\mathbf{c} = \{1, 0, 0, 0\}$),
 237 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 de-
 238 pending 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
 239 be specified as a generic nonlinear function that describes the hydraulic characteristics of
 240 the element (e.g., pump and turbine performance curves from the technical documentation).
 241 Moreover, *flow continuity* at nodes must be satisfied, namely

$$242 \quad \sum_{m=1}^{M_n} Q_{m,n} - Q_{d,n} = 0, \quad (8)$$

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

$$247 \quad h_n = h_s. \quad (9)$$

248 where h_s is the hydraulic head of the s -th tank. Inertial effects in the system are negligible
 249 due to slowly varying boundary conditions, i.e., long closure time of valves (5 minutes). For
 250 this reason, unsteady formulations of the pipe hydraulics (e.g., Nault and Karney, 2016) are
 251 not required and the time evolution of the system is modeled by a succession of steady-
 252 states with duration Δt and whose boundary conditions at each instant are obtained from
 253 mass balance equations for the tank levels (as in Rossman, 1993). In the studied WSS,
 254 tank level variations are negligible compared to the piezometric head in pipes. Thus, the
 255 evolution of level in tanks does not affect significantly the water heads and the flows in the
 256 system. For this reason, the constant tank level assumption in the steady-state solution is
 257 valid, and a more refined model formulation is not required (Todini, 2011; Giustolisi et al.,
 258 2012). Moreover, it should be noted that in steady state simulations Δt represents the
 259 time interval during which the boundary conditions are assumed to be stationary. When
 260 pulsating stochastic water demands are applied at nodes of water distribution systems, this
 261 assumption is valid for Δt of the order of minutes because with a lower Δt the average value
 262 of the demand would not be representative (Giustolisi et al., 2012). Differently, in the present
 263 study, water demands are applied directly at local tanks and the stochastic fluctuations are
 264 balanced by the water volume stored in tanks. Thus, shorter time steps, of the order of tens
 265 of seconds, can be used.

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

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

- 275 operating rules (see following sections), the input data are processed and the param-
 276 eters $\mathbf{c}=\{c_{c,m},c_{b,m},c_{f,m},c_{r,m}\}$ and \tilde{Q}_m in (7) are updated for the calculation at time
 277 $t_1 = t_0 + \Delta t$;
- 278 4. the level of the s -th tank (h_s) at time $t_1 = t_0 + \Delta t$ is updated as $h_s(t_1) = h_s(t_0) +$
 279 $\Delta h_s(t_0)$, with $\Delta h_s(t_0) = [\sum_{m=1}^{M_s} Q_{m,s}(t_0) + Q_{p,s}(t_0) + Q_{d,s}(t_0)] \Delta t / \Omega_s$, where Ω_s is the
 280 tank area, $Q_{m,s}$ is the flow from the m -th element into the s -th tank, M_s is the number
 281 of elements connected to tank s , $Q_{d,s}$ and $Q_{p,s}$ are the flow demand and the inflow at
 282 tank s from local sources;
 - 283 5. water demand (Q_d) and water inflow (Q_p) at local tanks are updated at time $t_1 =$
 284 $t_0 + \Delta t$, according to the data provided by the remote monitoring system;
 - 285 6. steps 2-5 are repeated for the solution of the hydraulic problem at successive time
 286 steps.

287 The key advantages offered by this numerical model are: (i) to define customized and
 288 time-dependent nonlinear functions that describe the system components (e.g., the term
 289 $f(Q_m, h_{n+1}, h_n)$ in (7)); (ii) to implement sensitivity and performance analyses for different
 290 sets of operating rules that enforce the hydraulic constraints (3)-(5); (iii) to implement an
 291 algorithm with varying time step. This last point is crucial for the correct computation of
 292 the timing of turbine operations that are triggered by target tank levels (see next section).
 293 In fact, due to the small area of the water tanks, fast variations of the water levels occur. As
 294 a result, the crossing of target levels can be detected with a sufficient precision only adopting
 295 small time steps. More in detail, the water level computed with a time step Δt results in
 296 a maximum error ϵ_h equal to $\epsilon_h = \int_t^{t+\Delta t} \dot{h} dt$. For the tank S2, $\dot{h} \sim 1$ mm/s (see Eq. (1))
 297 and thus the time step Δt must be shorter than 10 s to keep $\epsilon_h < 1$ cm. However, too
 298 short constant time steps would lead to long and unaffordable computation times and would
 299 be in conflict with the hypothesis of steady-state conditions. For this reason, the solution
 300 algorithm adopts a variable time step, whose duration is increased or decreased when tank
 301 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

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

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

328 in detail, the tank level is allowed to vary between two regulation thresholds $h_{U,1}$ and $h_{L,1}$,
 329 where subscripts “ U ” and “ L ” refer to upper and lower thresholds, respectively (see Fig. 3).
 330 The actual value of these thresholds is selected on the basis of technical considerations, and
 331 will be detailed in the following sections. Adjustments of the turbine opening are performed
 332 only when the water level in S1 exceeds $h_{U,1}$ or goes below $h_{L,1}$ (see Fig. 3a,b). When the
 333 water level goes below $h_{L,1}$, the value of Q_{T1} , calculated using (10), is rounded down to the
 334 nearest discrete value of discharge that can be regulated by the turbine (see Eq. (2)). In this
 335 way, the flow rate leaving S1 is slightly lower than the flow rate that precisely satisfies Eq.
 336 (10) and the water level in S1 slowly rises. On the other hand, when the level exceeds $h_{U,1}$,
 337 the value of Q_{T1} evaluated with (10) is rounded up to the nearest discrete value given by
 338 Eq. (2) and the level in S1 slowly decreases. The flow regulation from S2 to S3 is performed
 339 by T2. The control parameters are Q_{T1} (that plays the same role of $Q_{IN,S1}$ in the regulation
 340 of T1), Q_{T2} , Q_E , Q_F , and h_2 . The same operating rules described before are followed (see
 341 Fig.3c). Flow from S3 to S4 is regulated by T3. The involved parameters are Q_{T3} , Q_{LV} (i.e.,
 342 the flow to the municipalities of the lower valley) and h_4 . The upstream tank (S3) is nearly
 343 nine times bigger than the downstream tank S4 and therefore h_3 is less sensitive to flow rate
 344 fluctuations. For this reason, the updating of Q_{T3} is performed focusing on the level h_4 of
 345 the downstream tank, S4. As discussed in the next section, the water level in tank S3 is
 346 instead used to control the flow rate discharged to the lower valley, Q_{LV} .

347 **Coordination between the WSS and the local water systems**

348 Some local supply systems of the lower valley are affected by the following criticalities: (i)
 349 low quality of spring water (e.g., turbidity), (ii) low quality of groundwater (e.g., high con-
 350 centrations of sulphate), and (iii) high cost of pumping operations. It is therefore convenient
 351 for technical and economic reasons to replace these local sources with the water conveyed
 352 by the water main. This replacement can be done whenever the municipalities of the upper
 353 valley consume less water than the WPP production and, then, there is an excess of water
 354 available for the lower valley. The water availability for the lower valley is estimated by

355 monitoring the level of S3, which has by far the largest storing capacity of the system. More
 356 in detail, when the level of S3 exceeds the threshold $h_{U,3}$ (see Fig. 3e), the SCS performs
 357 the two following steps:

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

$$361 \quad Q_{SUR} = Q_{IN,3} - Q_{LV} \quad (11)$$

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

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

$$366 \quad \sum_{i=1}^{n_{OFF}-1} \bar{Q}_{P,i} \leq Q_{SUR} \leq \sum_{i=1}^{n_{OFF}} \bar{Q}_{P,i}, \quad (12)$$

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

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

$$372 \quad \sum_{i=1}^{n_{OFF}} \bar{Q}_{P,i} \leq Q_{SUR} \leq \sum_{i=1}^{n_{OFF}+1} \bar{Q}_{P,i} \quad (13)$$

373 Moreover, steps 1 and 2 are repeated every 6 hours even if no threshold is crossed. This
 374 time interval approximately corresponds to the time required for significant variations of
 375 local consumption. In order to select the number (n_{OFF}) of local sources to be turned off,
 376 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

404 water demand in the lower valley increases. Notice that the temporal evolution of water
 405 levels in S1 and S2 follows a daily recurrent pattern. In fact, the only parameters that affect
 406 the evolution of h_1 and h_2 are the flow rates delivered to the municipalities of the upper
 407 valley, that present a daily consumption pattern. Instead, daily patterns are not observed in
 408 tanks S3 and S4. In fact, water level variations in S3 and S4 depend on the flow discharged
 409 to the lower valley (Fig. 3d,e) which is updated when the regulation thresholds in S3 are
 410 crossed or when a period of six hours has passed without any crossings. In order to have
 411 a global view of the behavior of the 3-year-long system dynamics, the relative frequency
 412 (RF) of the water level in the four tanks is evaluated (Fig. 4b). The average water level
 413 in tanks is high and guarantees an average total water reserve of 14830 m^3 , 80% of which
 414 is stored in S3. Therefore, in case of a pipeline failure upstream of S3, the water stored
 415 in S3 can sustain the total downstream population ($\sim 100\,000$ inhabitants) for an average
 416 time of 8 hours. Actually, this duration underestimates the system resilience because most
 417 of the local sources of the lower valley are gradually reactivated (i.e., the well fields) when
 418 the water level in S3 decreases. In this situation, a higher cost for the water supply (i.e.,
 419 pumping and water treatment costs) must be taken into account. Levels in tanks S1, S2,
 420 and S4 are always restrained between the physical bounds of the tanks, thus avoiding empty
 421 and overflow conditions during the whole simulation time. The RF distribution tails often
 422 extend beyond the regulation thresholds (Fig. 4b) because of the delay in the response of
 423 the water level to the flow regulation performed by the turbine. However, the clearance
 424 between the regulation thresholds and the tank physical limits prevents the occurrence of
 425 overflow and empty conditions. Finally, the average value of h_3 is higher than $h_{U,3}$ because
 426 the water provided by the WPP satisfies the full water demand of the lower valley for most
 427 of the simulation time.

428 The second constraint (4) concerns the minimization of the number of operations per-
 429 formed by the turbines. The histograms in Fig. 5 report the daily average number of flow
 430 rate variations with magnitude ΔQ_T performed by the turbines. The size of the histogram

431 classes (12.5 l/s for T1 and T2 and 50 l/s for T3) corresponds to the smallest amount of
432 change that can be regulated by the turbine, that is 10% the nominal flow rate through a
433 jet. Turbines T1 and T2 display only mild flow rate variations ($\Delta Q_T < 150$ l/s and $\overline{\Delta Q_T} \simeq$
434 30 l/s) thanks to the high sensitivity of the four-jet Pelton turbine governing system. Differ-
435 ently, the flow rate through T3 is affected by larger variations ($\Delta Q_T < 400$ l/s and $\overline{\Delta Q_T} \simeq$
436 125 l/s). Finally, it should be noted that each turbine experiences less than 9 flow rate
437 adjustments over a day. As will be discussed in the following section, the number and the
438 extent of these variations are the compromise between the maximization of water storage in
439 tanks and the minimization of the number of turbine operations.

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

458 contribution decreases from 67% to 6%. This reduction results in a significant energy saving.

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

471 **SENSITIVITY ANALYSIS**

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

476 The preliminary step was to understand the effect of changes in the regulation thresholds
477 on the water volume stored in tanks and on the number of turbine operations. The upper
478 threshold (h_U) was fixed, while the lower threshold h_L was varied as $h_L = h_U - \Delta H$, with
479 ΔH taking values between 0.3 and 2.4 m. Fig.8 reports the number of turbine operations and
480 the volume of water stored in tanks S1, S2, and S4 as a function of ΔH . As ΔH increases
481 up to 1.5 m, the number of turbine adjustments decreases rapidly and for larger ΔH no
482 additional reduction is obtained (Fig. 8a). As to the effect of ΔH on the water volume
483 stored in tanks, the mean and minimum water storage volumes strongly decrease with ΔH
484 (Fig. 8b-d). As a consequence, high values of ΔH induce a reduction of the system resilience

485 in case of failures. In particular, when ΔH is larger than 1.2 m only half of the capacity
486 is exploited for tanks S2 and S4. After these analysis, ΔH for tanks S1, S2, and S4 was
487 set equal to 1.2 m as a compromise value between the optimization of turbines performance
488 and the optimization of the system resilience. A similar sensitivity analysis was performed
489 for the regulation thresholds in tank S3. However, the water volume stored in tank S3
490 presented no significant variations. According to the operations (11)-(13) the water released
491 to the lower valley from S3 depends on prescribed temporal variations of municipal water
492 demands and of water production at the WPP. Therefore, variations in the level thresholds
493 $h_{U,3}$ and $h_{L,3}$ have little effects on h_3 . Additional simulations were conducted to test other
494 combinations of ΔH for tanks S1, S2, and S4 (Table 2). Fig. 9 reports the total number
495 of turbine operations and the total minimum volume of water stored in all tanks for the
496 different investigated scenarios. These results are reported in relation to those obtained with
497 the adopted solution (Table 1 and scenario 2 in Table 2). The points lying on the dashed line
498 represent the Pareto front of the nondominated scenarios. The solutions lying on the Pareto
499 front are equivalent because none of the objectives can be improved without worsening the
500 other ones. Among these nondominated solutions, the scenarios included in Region I present
501 a greater water storage but a higher number of turbine operations, in relation to scenario 2.
502 On the other hand, the scenarios in Region II result in a lower number of turbine operations
503 but offer a lower water reserve. Among the Pareto optimal solutions, scenario 2 was selected
504 for its central position in the domain of the possible solutions.

505 CONCLUSIONS

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

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

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

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534 providing valuable information. The Editor and three anonymous Reviewers are acknowl-
535 edged for their suggestions that helped us to improve our work.

536 **NOTATION**

537 *The following symbols are used in this paper:*

A, B, \dots, 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 $[0, 10 \cdot J]$;
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;
$Q_{EX, A \dots R}$	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;
\bar{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 T_i ;
\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.

538

SUPPLEMENTAL DATA

539

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

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620 **List of Tables**

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623 Scenario 2 corresponds to the values in Table 1. 31

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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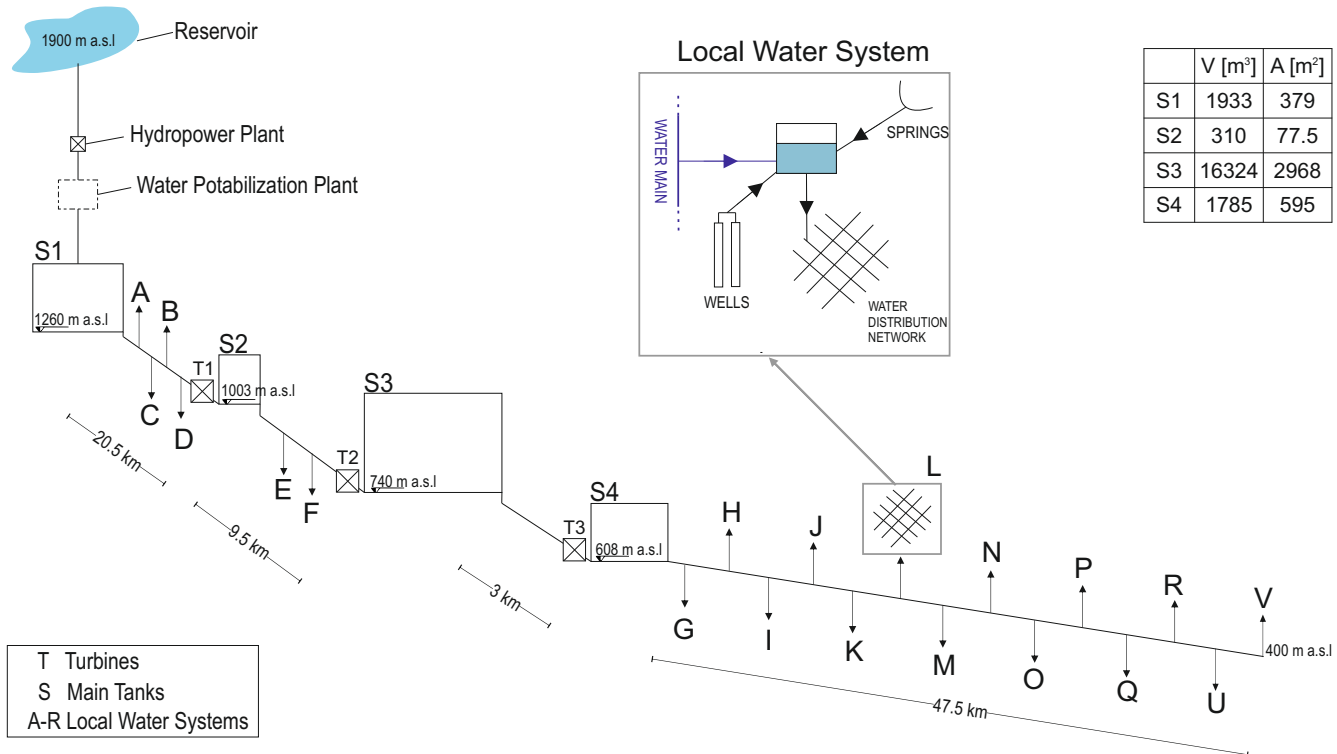


Figure 1. Scheme of the WSS. The capital letters (A,B, ... V) indicate the local water networks. The inset shows a typical local water system with a storage tank supplied by mountain springs, local wells and by the new water main.

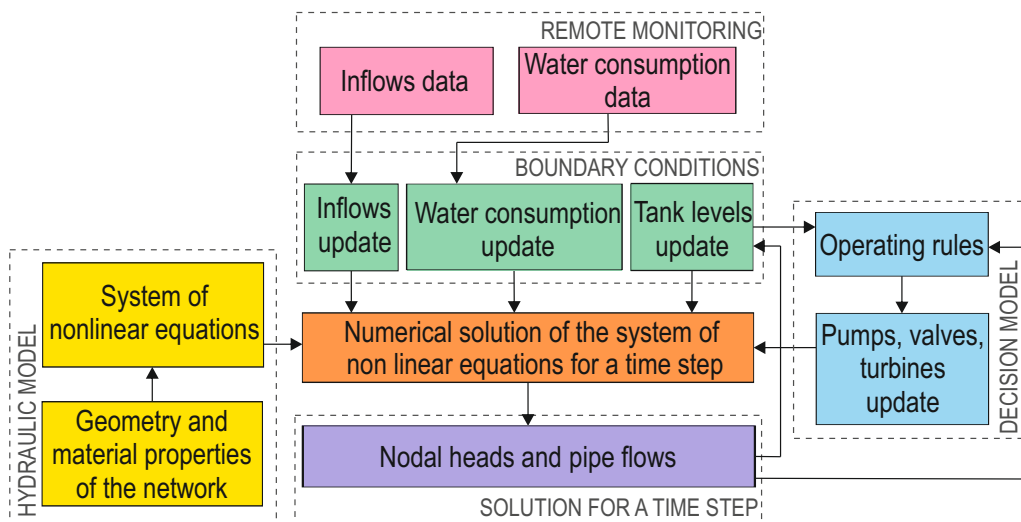


Figure 2. Scheme of the numerical model developed to simulate the operation of the new WSS.

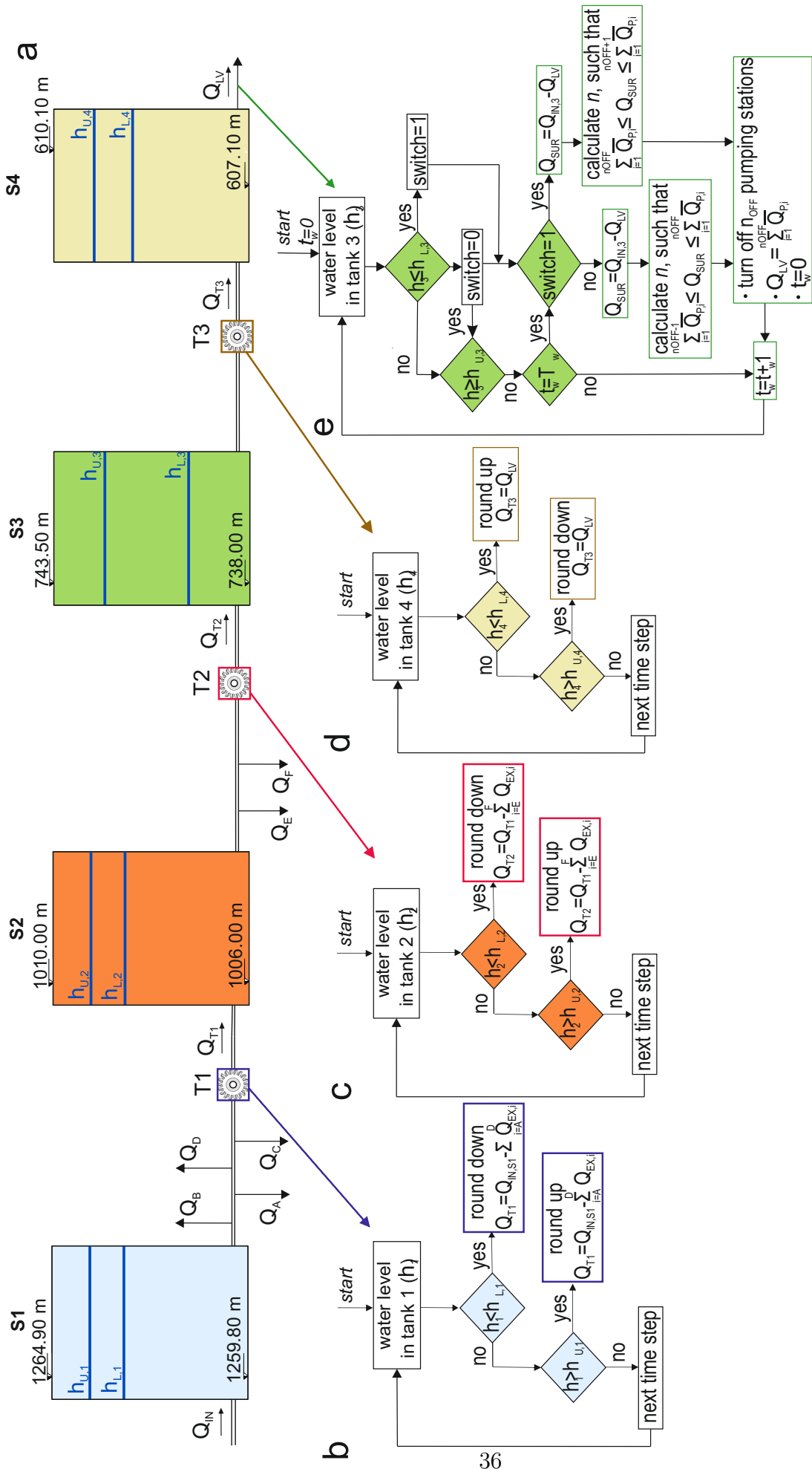


Figure 3. Scheme of the feedback-control algorithm of the new WSS. Panel (a) represents the scheme of the hydraulic system. The block diagrams report the operating rules for T1 (b), T2 (c), T3 (d) and for the control of the local sources in the lower valley (e).

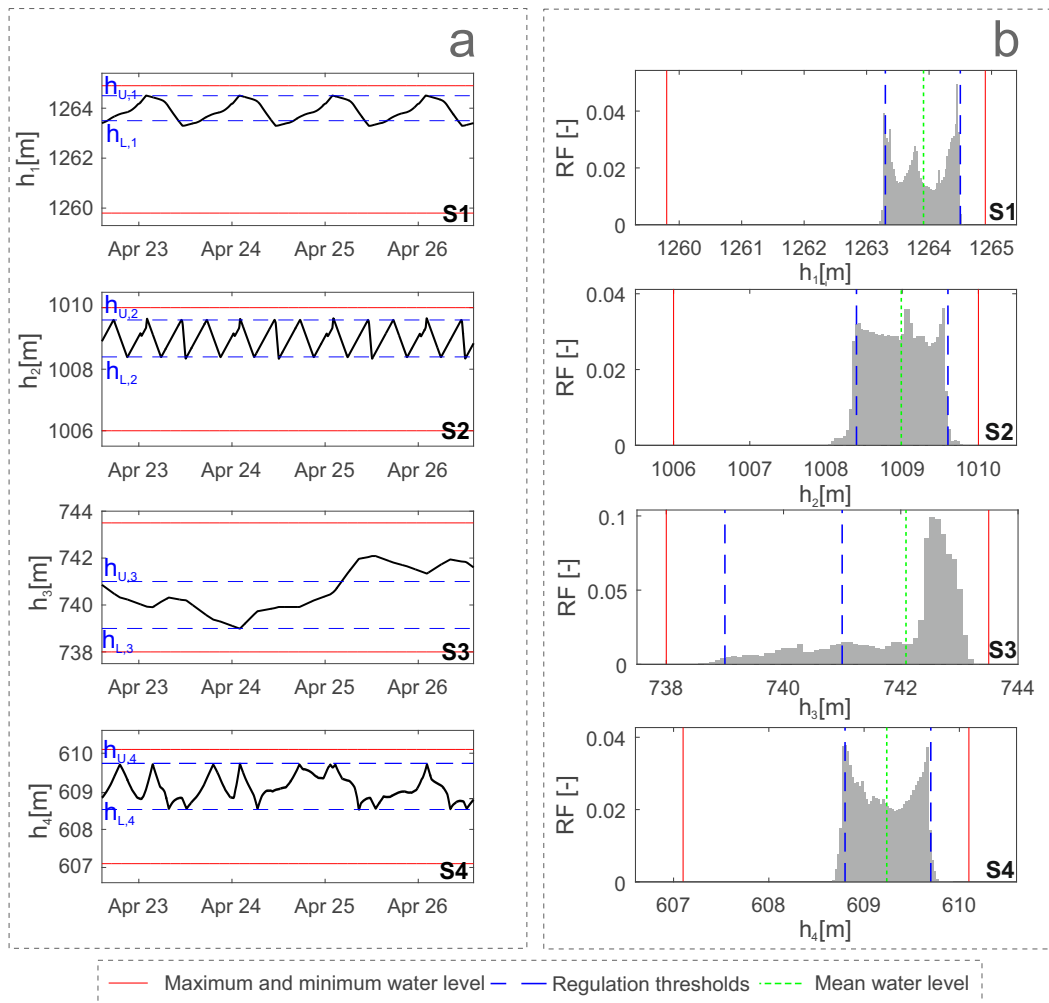


Figure 4. (a) Typical time series for a generic four days period. (b) Relative frequency of levels in tanks S1, S2, S3 and S4 for a 3-years simulation time.

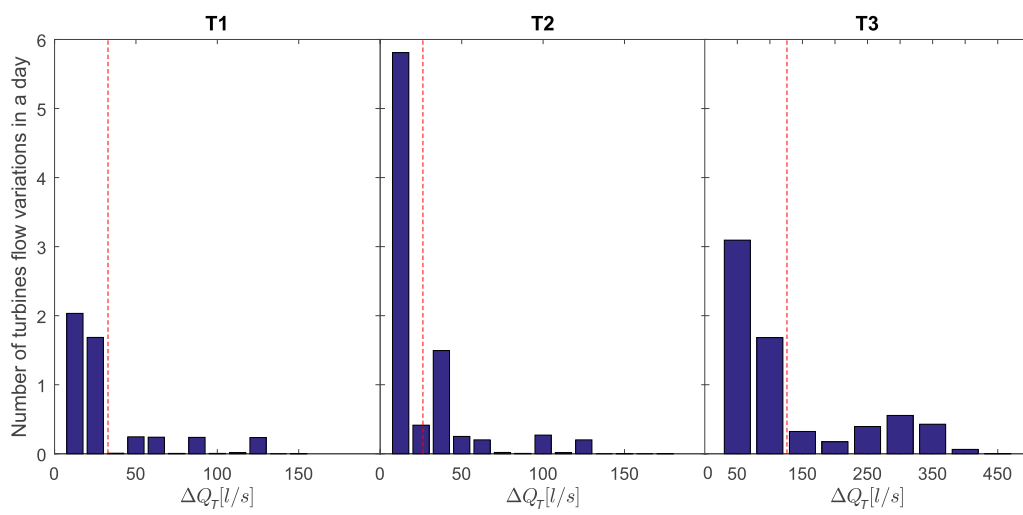


Figure 5. Daily number of flow rate variations with magnitude ΔQ_T for turbines T1, T2 and T3. The dashed line indicates the average magnitude of ΔQ_T .

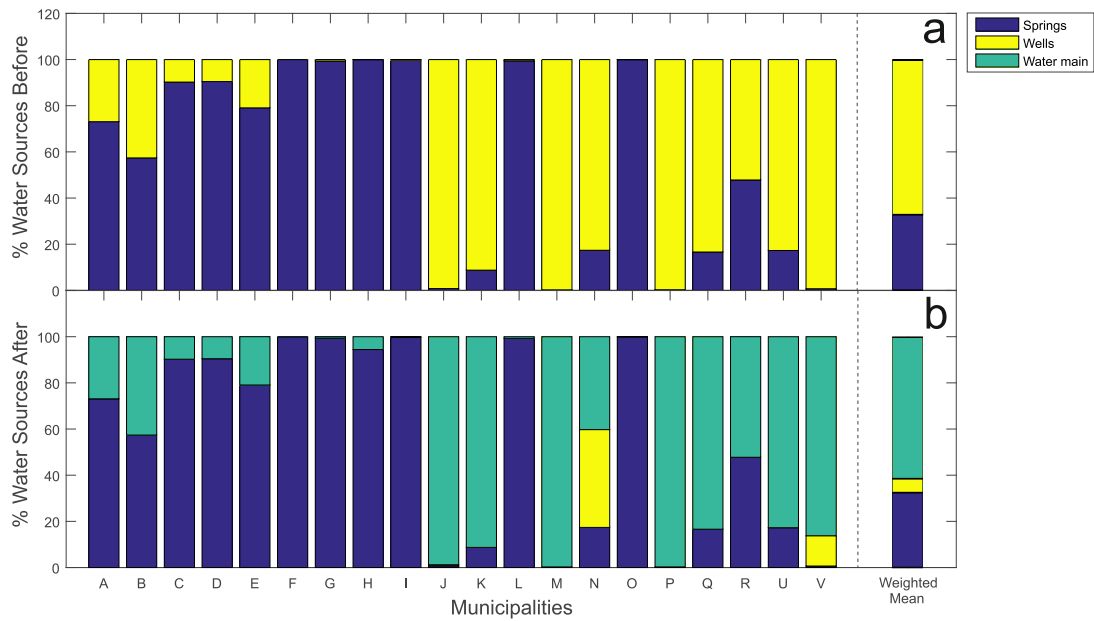


Figure 6. Composition of the water sources supplying each local network (a) without water from WPP and (b) with the water from WPP. The last bar on the right reports the mean composition in the entire valley.

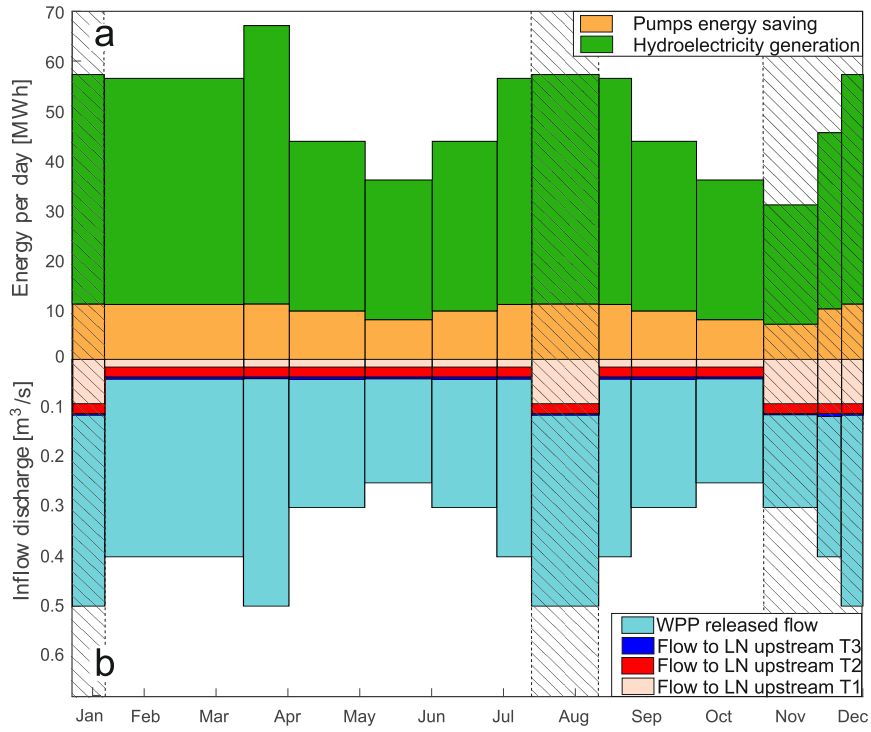


Figure 7. (a) Daily energy saving and hydropower generation over a typical year. (b) Released discharge from the WPP, flow delivered to the local networks upstream T1, T2 and T3. The hatched areas highlight touristic seasons.

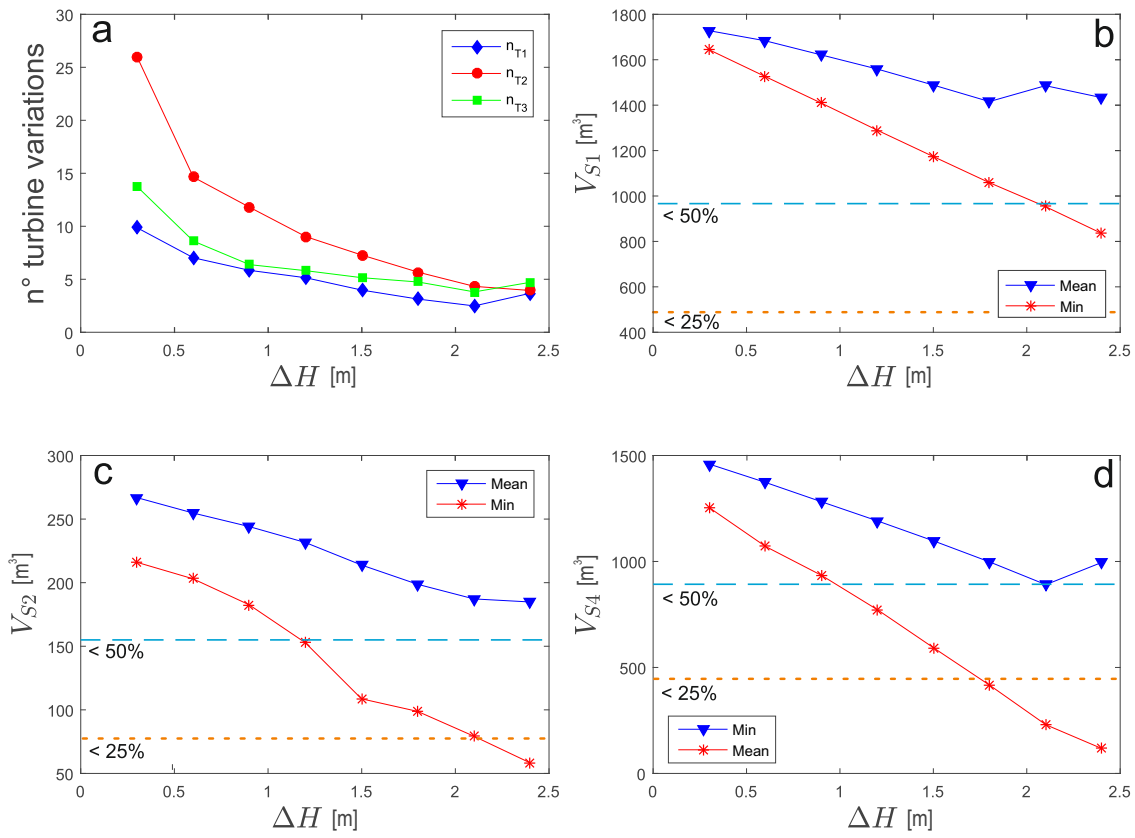


Figure 8. (a) Number of flow rate variations in a day for turbines T1, T2 and T3. (b)-(d) Minimum and mean water volume stored in tanks S1, S2 and S4 for different ΔH . The dashed and dotted lines mark the 50% and 25% of the total tank capacity, respectively.

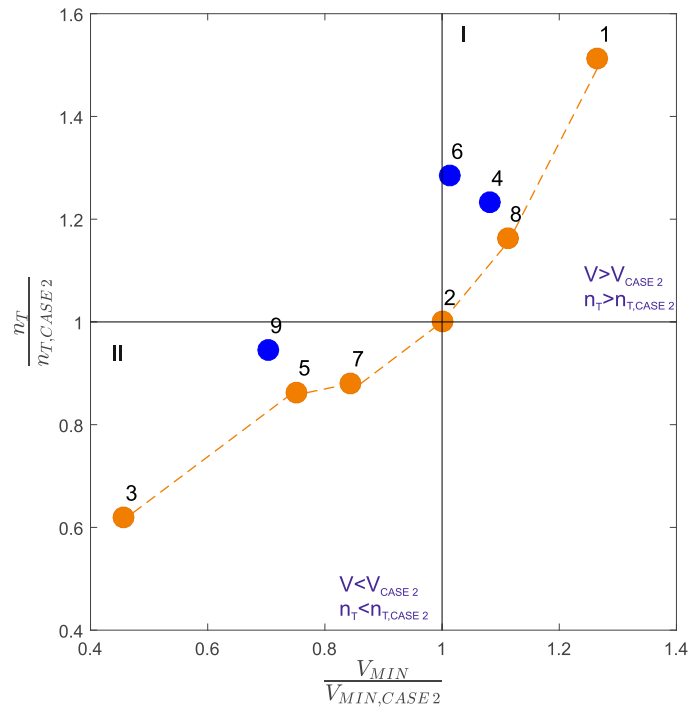


Figure 9. Number of turbine operations against minimum water storage in tanks for different combinations of ΔH_1 , ΔH_2 , ΔH_4 , in relation to the case of Table 1. The points lying on the Pareto front (dashed line) are the nondominated solutions. Notice that scenario 2 corresponds to Table 1