AGU ADVANCING EARTH AND SPACE SCIENCES

Water Resources Research®

RESEARCH ARTICLE

10.1029/2021WR031919

Leakage Management Influence on Water Age of Water Distribution Networks

Orazio Giustolisi[1], Francesco G. Ciliberti[2], Luigi Berardi[2], and Daniele Biagio Laucelli[1]

[1]Department of Civil, Environmental, Land, Building Engineering and Chemistry, Technical University of Bari, Bari, Italy, [2]Department of Engineering and Geology, University "G. D'Annunzio" of Chieti Pescara, Pescara, Italy

Key Points:

- The analysis of real Water Distribution Network (WDN) shows that leakage reduction by pressure control and District Metering Areas design impacts on water age that need to be investigated
- A new Lagrangian scheme for water quality models is tested to overcome limited accuracy of former models, allowing unlimited parcels
- Water inflow and water paths variations due to leakage management activities influence water age as the field of velocity in WDNs is changed

Correspondence to:

D. B. Laucelli,
d.laucelli@poliba.it

Citation:

Giustolisi, O., Ciliberti, F. G., Berardi, L., & Laucelli, D. B. (2023). Leakage management influence on water age of Water Distribution Networks. *Water Resources Research*, *59*, e2021WR031919. https://doi.org/10.1029/2021WR031919

Received 30 DEC 2021
Accepted 28 DEC 2022

Abstract Leakage management of Water Distribution Networks (WDNs) is a relevant technical task worldwide. Background leakages along mains and connections to properties in aged infrastructures represent a significant portion of the WDN water outflow, sometimes exceeding customer demands. Pressure control and District Metering Areas (DMAs) design are two classic activities aimed at leakage management. The former reduces the background leakages, and thus the velocities in pipes, especially those along main paths for water transport across the WDN. The latter involves closed valves confining each DMA, which change network connectivity reducing redundancy and eventually pressure, that is, background leakages. Hence, both activities reduce water velocity in pipes because of leakage reduction (i.e., the overall WDN inflow), while DMAs could increase it in some pipes. Therefore, leakage management activities might affect the water age in WDNs, as a general water quality indicator, depending on the initial level of leakages, the WDN connectivity and topology, and the attained reduction. This work studies the effect of leakage management on water age for two real WDNs, which are characterized by different initial levels of leakages and expected reduction, after planning pressure control and DMAs. The advanced hydraulic modeling, employing pressure-driven leakage modeling at pipe level, is integrated with water age analysis, allowing unlimited number of parcels. The sensitivity of water age to the maximum number of parcels in the model is studied to identify the trade-off between accuracy and computational runtime. The variation of mean daily water age due to change of demand pattern is analyzed considering a 4-week operative cycle.

1. Introduction

Today, the natural deterioration of aged water distribution infrastructures asks for planning and designing technical activities spanning from operative (short), to tactical (medium) and strategic (long) horizon times. However, these activities influence the hydraulics of the Water Distribution Networks (WDNs) in terms of field of velocity and flow in pipes. Therefore, the water quality issue is related to the leakage management of WDNs.

It is recognized that leakage management affects several aspects of the WDNs such as the quality of service (Almandoz et al., 2005), increasing energy consumption and economic losses (Colombo & Karney, 2005), and exposing customers to potential health issues (WHO, 2017). Starting from the new millennium, the socio-economic awareness about the problem of water losses has motivated regulators and water companies to move from reactive to proactive maintenance approach (Thornton & Lambert, 2005). Among different classes of water leakages, the background and unreported leakages are those having the most important volumetric effect on the WDN management (Berardi & Giustolisi, 2021); thus, they are referred to as volumetric leakages in the following.

Leakages depend on pressure and deterioration of pipes because they can be considered as outflows working according to the Torricelli law, which orifice area varies with the pressure depending on pipe material. Therefore, the main design activities for leakage management in operative and tactical planning horizons are pressure control (e.g., Walski et al., 2006) and District Metering Areas (DMAs) design (e.g., MacDonald & Yates, 2005), together with active leakage control (Thornton & Lambert, 2005). Pipe replacement plans range from the tactical to the strategic time horizons because works are much more expensive in terms of time and direct and indirect costs.

Pressure control in WDN is mainly implemented using pressure control valves (e.g., Araujo et al., 2006) and should guarantee the correct pressure for supplying water to customers (e.g., Berardi et al., 2018). DMA design, which was originally conceived to monitor flows and pressure, also gives the opportunity to change the original WDN topology by closing sectioning valves at DMA boundaries, thus modifying the field of velocity and flow in

10.1029/2021WR031919

the hydraulic network, and possibly reducing pressure status and leakages in the WDN (e.g., Laucelli et al., 2017; Wright et al., 2015).

For example, today in Italy many water companies are designing activities such as pressure control and districtualization aimed at reducing the water losses in short-medium time horizons using huge investments from the national government. Therefore, together with designing activities, a new issue is raising about assessing the change of water quality in consequence of the reduced inflow in the network, due to the reduced leakages, and modification of hydraulics due to the DMA design, that is, the general change of fields of velocity and flow as average values and spatial distribution.

Daily average water age is considered as a general indicator of water quality (Grayman et al., 2009) since longer retention times turn on quality decay due to extended chemical and biological reactions along the network (Blokker et al., 2016) and set off potential health impacts (e.g., Säve-Söderbergh et al., 2017). Chemical disinfections of drinking water, both chlorinated and non-chlorinated, can expose consumers to potential health risks of organic and inorganic disinfection-by-products (DBPs) (Li & Mitch, 2018). The potential formation of DBPs in WDNs is related to several biological, chemical, and physical factors, which exacerbates as water age increases (Poleneni & Inniss, 2015). Hence, the evaluation and monitoring of impacts on water quality are crucial activities for WDN reliability (Wang et al., 2021) and asset management (Qi et al., 2018). More recently, some authors have considered keeping track of the minimum-maximum characteristics of water age (Braun et al., 2020; Matchell et al., 2009; Piller & Tavard, 2014) which, together with the average value, provides the "water age envelope."

The change of daily water age due to leakage reduction is assessed using standard tools developed during the last century, mainly based on EPANET (Rossman, 2000), sometimes with some modifications and improvements. The need of understanding the water quality behavior throughout WDNs motivated some recent works to support design and planning for WDN management (e.g., Farmani et al., 2006; Quintiliani et al., 2019; Vrachimis et al., 2021). The EPANET (Rossman, 2000) water quality analysis toolkit has been used by several authors for various purposes and some contributions were proposed to improve it over time.

The basic concept of the EPANET water quality solver is to evaluate a 1D advective-reactive transport equation scheme by assuming as boundary conditions a complete and instantaneous mixing of water at WDN junctions (Rossman, 2000; Rossman & Boulos, 1996) and different mixing schemes on tanks. As mentioned in EPANET 2.0 manual, internally, EPANET treats water age as a reactive constituent whose growth follows zero-order kinetics with a rate constant equal to 1 (i.e., each second the water becomes a second older). In earlier releases the adopted computational scheme was the Eulerian one. The Eulerian scheme deals with the approximation of the differential terms of the advection equation with finite-difference for a spatially and temporally grid or defining the transport and reactions conditions for discrete times along discrete, equally sized, and completely mixed volume parcels. Considering the minimum water quality time step as the minimum travel time along all the network, the Eulerian Discrete Volume Method (DVM) (Rossman et al., 1993) approach discretizes each pipe of the network in several equally sized and completely mixed volume segments and evaluates, at each water quality time steps, the water quality routing as a series of reaction, parcels transport along pipes, and volume and mass mixing at each node. To avoid shorter water quality time-steps, caused by *short pipes* with high velocities, or pipes with smaller travel times which can affect the mass advection, it sets a minimum allowable water quality time-step, which is equal to 1/10 the length of the hydraulic time-step. For very long pipes or for pipes with slow velocities the generation of many segments is avoided by imposing a maximum number of segments. Early EPANET releases adopted default value of the maximum allowed number of segments of 100. The DVM scheme accuracy depends on the water quality time step size, and it has been observed that the method is subject to phase shift errors (Rossman & Boulos, 1996).

Starting with EPANET 2.0 (Rossman, 2000) the solver implemented the Lagrangian Time-Driven Method (TDM): this method tracks the concentration and size of a series of nonoverlapping segments of water that fill each link of the network (Rossman & Boulos, 1996). The basic difference between DVM and TDM stands on the transport mechanism along the link. In TDM, as water quality time steps progresses, for each link, the upstream segment increases its volume by the same size of loss of the downstream segment. The generation of new segments in the upstream node of the link is controlled by creating new segments only when the existing downstream segment of a node differs in concentration/age by a specific tolerance. If the difference in quality is below the tolerance, then the size of the current last segment in the link is increased by the volume flowing from the upstream node over the time step, and the quality/age is a volume-weighted average of the node and segment

10.1029/2021WR031919

quality. Furthermore, the segments order along pipes is reversed when flow changes direction, from one hydraulic simulation time step to the next one. The accuracy of this method depends on the choice of the water quality time step and the tolerance for concentration/age used to control the generation of upstream segments (Rossman & Boulos, 1996).

However, the accuracy of water quality of EPANET solver is critical as recently discussed by Davis et al. (2018) who pointed out important limitations in the water-quality routing algorithm used in EPANET. In fact, the need of limiting the number of parcels inside the pipes, especially with high travel time, motivated an empirical way of fixing them based of the quality tolerance, as reported into the EPANET 2.2 user manual (Rossman et al., 2020, p. 78): "*Using too large a value for this tolerance might affect simulation accuracy. Using too small a value will affect computational efficiency. Some experimentation with this setting might be called for.*" Such limitations also motivated Munavalli and Kumar (2004) to propose an enhanced modified Event-Driven method, which limits artificial mixing of segments and loss of resolution in concentration as in the EPANET water quality simulation.

About the hydraulics of EPANET, it does not allow computing pressure-dependent leakages at pipe level, and, in practice, they are surrogated as fixed demand patterns at nodes. This limitation does not permit the prediction of leakage reduction, which occur at pipe level in consequence of changes of pressure status, during the design of pressure control, districtualization, and pipes replacement. In fact, classic demand driven model are known to be inappropriate to support planning leakage management activities. Some hydraulic solvers surrogate pressure-dependent leakages (e.g., Wu et al., 2009) as outflow concentrated at nodes and depending on nodal pressure.

We here calculate the pressure-dependent volumetric leakages at pipe level and their reduction due to management activities using the model proposed by Giustolisi et al. (2008). Such model provides accurate and technically consistent distribution of leakage outflows (Berardi & Giustolisi, 2021) and the assessment of changes in the velocity and flow fields in the network, which are the basis for any water quality assessment in consequence of whatever design activity. Furthermore, to control and overcome the possible lack of accuracy of the EPANET water quality solver, we here present and discuss a different way to simulate the water quality, which allows to limit or not the maximum number of parcels into pipes in the standard Lagrangian scheme. This allows to assess the accuracy of the solver using a variable maximum number of parcels, the effect of a time correction, and the computational runtime. The results provide technical-scientific indications on the accuracy of water age calculations and the effective evaluation of the influence of leakage reduction activities.

2. Assessment of the Influence of Leakage Management on Water Quality and Accuracy of Modeling

The purpose of the work is the accurate modeling of the water quality as consequence of leakage management activities, which are today planned in many water systems around the world. The water quality indicator, for example, for disinfection purposes, is the water age which depends on the fields of velocity and flow in pipes and influences the advection of any contaminant. Therefore, on the one hand, the hydraulic modeling of water leakages at pipe level, as function of the pressure, is relevant to predict their reduction as consequence of design activities (i.e., pressure control, districtualization, and pipe replacement) and the actual changes of flow and velocity fields. On the other hand, the accurate simulation of water age, which depends on such changes of flow and velocity fields, is mandatory to tackle related technical-scientific issues in an effective way.

This work aims at answering some technical-scientific questions related to the influence of the leakage management on water age in WDNs. The technical questions are:

1. is the impact of leakage reduction on water age relevant and does it depend on initial leakages rate?
2. is the calculation of the water age affected by the assumed demand pattern for hydraulic simulation (i.e., a 24-hr operating cycle vs. an operating cycle of 4 weeks)?
3. does the water age depend on daily customer demand or on the daily network inflow (i.e., including leakage outflows)?

Other questions refer to accuracy and runtime for water age calculation. They are:

1. what is the impact of limiting the maximum number of pipes parcels in water quality models in terms of computational runtime versus accuracy?

10.1029/2021WR031919

2. how a correction factor introduced to overcome the limitation of maximum number of parcels impact on the accuracy of the water age simulation?

The last two are open issues reported in literature and in EPANET 2.2 (Rossman et al., 2020) user's manual, as mentioned above.

To answer to the above technical questions in a scientifically consistent and robust way, we here use the advanced hydraulic modeling (implemented in WDNetXL software tool), which allows the simulation of the pressure-dependent pipes leakages (Giustolisi et al., 2008; Giustolisi & Walski, 2012). Section 3.1 reports the model equations and a brief explanation of the most relevant issues of hydraulic simulation. Sections 3.2–3.6 report and discuss the water quality modeling strategy, which starts from the velocity and flow fields obtained by the advanced hydraulic simulations. The water quality simulation uses the Lagrangian scheme, and each single section provides details on the complete modeling strategy.

The proposed water quality model enables to discuss the accuracy issues related to the effect of limiting the maximum number of parcels into pipes during water quality simulation (*long pipes*), that is, pipes for which the maximum number of parcels is not sufficient during the simulation because of the high travel time. Furthermore, we consider the possibility to adjust the travel time of the single parcel to correct the possible acceleration of advection due to the limited number of parcels. The correction makes consistent the overall travel time of substances with the high length or low velocity in *long pipes* during simulation but causes a lack of synchronization with other pipes joining the same ending node. In other words, the time the substance into a "long pipe" reaches the downstream node is different from the time the substances traveling into other pipes joining the same downstream node reach that node, that is, the correction creates inconsistencies among travel times into the parcels among pipes.

Therefore, we here present the innovative possibility of not limiting the number of parcels, which is offered by advances in programming languages, beyond the central processing unit (CPU) required memory issue. It is worth to remark that novel water quality analysis was developed without using EPANET source code since it does not allow evaluating the impact of water quality model accuracy using unlimited number of parcels, which is of direct relevance when WDN configuration changes due to leakage management activities.

3. Methods and Materials

3.1. Advanced Hydraulic Analysis

This section shortly recalls the mathematics of WDN hydraulics, which is the basis of the water quality analysis because it provides the field of the velocities and flows in pipes on which the substance diffusion and kinetic are performed in next water quality model. For the sake of simplicity devices are not considered here, although it does not impair the generality of the presentation. A hydraulic network is composed of p pipes with unknown flow rates, n nodes with unknown heads (internal nodes), and n_0 nodes with known heads, for example, reservoir levels or initial level of tanks. WDN hydraulic model solves the following nonlinear mathematical system based on momentum and mass balance equations (e.g., Giustolisi & Walski, 2012):

$$\begin{aligned} \mathbf{A}_{pp}\mathbf{Q}_p(t) + \mathbf{A}_{pn}\mathbf{H}_n(t) &= -\mathbf{A}_{p0}\mathbf{H}_0(t) \\ \mathbf{A}_{np}\mathbf{Q}_p(t) - \mathbf{d}_n(\mathbf{H}_n(t)) &= \mathbf{0}_n \end{aligned} \tag{1}$$

where $\mathbf{Q}_p = [p,1]$ column vector of unknown pipe flow rates; $\mathbf{H}_n = [n,1]$ column vector of unknown nodal heads; $\mathbf{H}_0 = [n_0,1]$ column vector of known nodal heads; $\mathbf{d}_n = [n,1]$ column vector of nodal demands generally depending on pressure; $\mathbf{0}_n = [n,1]$ column vector of null values; $\mathbf{A}_{pp}$ = diagonal matrix of size $[p,p]$, whose elements are based on pipes resistance and flow. The domain of the equations, that is, the network, is represented by $\mathbf{A}_{pn} = \mathbf{A}^T{}_{np}$ and $\mathbf{A}_{p0}$ = topological incidence sub-matrices of size $[p,n]$ and $[p,n_0]$, respectively, which are derived from the general topological matrix $\bar{\mathbf{A}}_{pn} = [\mathbf{A}_{pn} \mid \mathbf{A}_{p0}]$, that is, excluding the signs the structure of the edge-vertex (incidence) matrix from the graph theory standpoint, of size $[p,n + n_0]$. The solution of system (1), that is, the computation of the unknowns $(\mathbf{Q}_p; \mathbf{H}_n)$, requires the boundary conditions $(\mathbf{d}_n; \mathbf{H}_0)$, and the pumps curve or devices status over time, if any.

Note that $\mathbf{d}_n$ is the average demand in the snapshot t of the hydraulic network functioning, which represent a steady-state window Δt. Considering the definition of average value, it can write (e.g., Berardi & Giustolisi, 2021; Giustolisi et al., 2012),

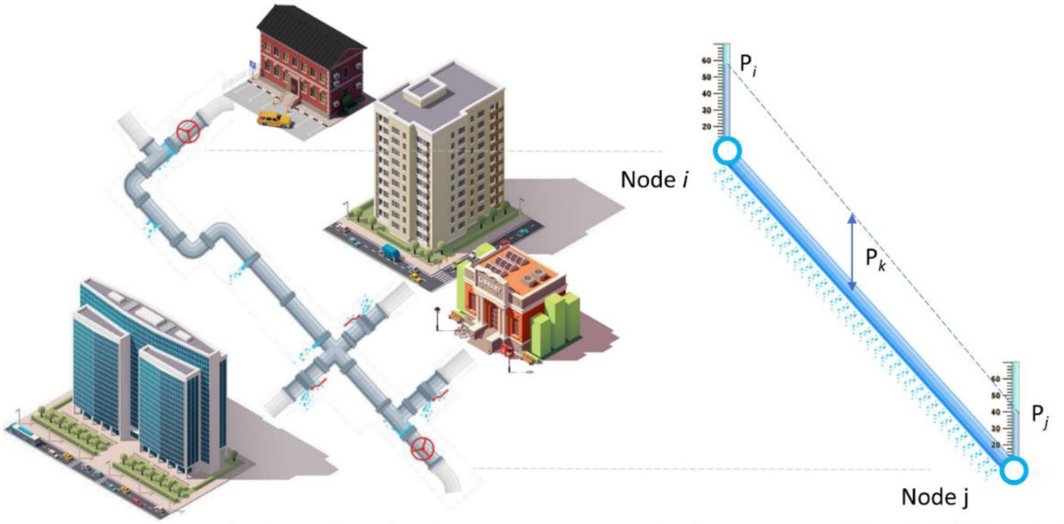

Figure 1. Representation of volumetric leakages in the advanced Water Distribution Network (WDN) model.

$$\mathbf{d}_n(\mathbf{H}_n(t)) = \frac{\int_t^{t+\Delta t} d_n(H_n(t))dt}{\Delta t} = \frac{\mathbf{V}_n(\mathbf{H}_n(t))}{\Delta t} \tag{2}$$

Then, we now focus on the second equation of system Equation 1, hence

$$\mathbf{A}_{np}\mathbf{Q}_p(t)\Delta t - \mathbf{V}_n(\mathbf{H}_n(t)) = \mathbf{A}_{np}\mathbf{V}_p(t) - \mathbf{V}_n(\mathbf{H}_n(t)) = \mathbf{0}_n \tag{3}$$

where $\mathbf{V}_p$ and $\mathbf{V}_n$ are the volume flow in the pipes or exiting/entering the nodes during time interval Δt at the steady-state snapshot t. Equation 3 recalls that the balance involves volume $\mathbf{V}_p(t)$ of parcels in pipes entering or exiting the node containing a certain mass of substance traveling in the network and the water volume $\mathbf{V}_n(t)$ of any demand component at node (Giustolisi & Walski, 2012).

Note that the volume $\mathbf{V}_n(t)$ is generally the summation of water volumes supplied to customer and leakage over Δt (Berardi & Giustolisi, 2021). Modeling pressure-driven leakages at pipe level (Giustolisi et al., 2008), see Figure 1, is mandatory to get accurate prediction of hydraulic status in terms of change of leakage volumes at each steady-state time step, especially while planning leakage management activities. In fact, using a fixed demand "leakage pattern" as in classic hydraulic modeling makes it impossible to predict the changes of leakages due to pressure control.

The water age analysis is based on the water velocities and flow fields, asking for a hydraulically consistent analysis within the same modeling assumptions which should be not affected by lack of modeling definition of demands (Giustolisi & Walski, 2012). In addition, to increase the accuracy of prediction of water pipe velocities (i.e., of water age analysis) the hydraulic analysis should accommodate actual topological changes due to closed valves at the boundary of DMAs (Laucelli et al., 2017).

Figure 1 recalls that real volumetric leakages, which are relevant for WDN management purposes, happen along distribution mains and connections. Most leakage outflows, which represent the highest amount of water volume at annual balance scale, are not reported to water companies or cannot be found by leakage detection campaigns; therefore, their location is unknown. For this reason, leakages are represented in the model as uniformly distributed outflows along pipes, depending on pipes deterioration and pressure P_k equal to the average between the pressure values at two ending nodes (Figure 1, right) (P_i and P_j) as computed by solving model in Equation 1. Such representation holds also when a gate valve is closed at one end of a pipe, that is, leaving the pipe under pressure, or to analyze the impact of pipe rehabilitation works, that is, by updating the deterioration parameter of each rehabilitated pipe.

3.2. Water Quality Analysis and Accuracy

The water quality analysis is here performed using a Lagrangian approach whose accuracy is controlled by the maximum allowed number of water parcels and exploits the effective programming using cell matrices data type. The parcels are moved downstream the pipes at each water quality step Δt_q which is assumed shorter than the hydraulic steady-state analysis step Δt. Therefore, the number of quality steps inside Δt should be an integer number,

$$m_p = \frac{\Delta t}{\Delta t_q} \geq 1 \tag{4}$$

to be consistent with the steady state assumptions about stationarity of the boundary conditions during the simulation (Giustolisi & Walski, 2012). In fact, if Equation 4 did not hold and m_p was not an integer, one Δt_q would belong to two different steady-state time steps Δt violating the hydraulic steady-state assumption.

The travel-time (T_r) to traverse a pipe of length L, depends on the actual velocity (v_{act}) varying over Δt_q,

$$T_r(t\Delta t + s\Delta t_q) = \frac{L}{v_{\text{act}}(t\Delta t + s\Delta t_q)} \quad \forall t = 0, \ldots, T_c \quad \forall s = 1, \ldots, m_p \tag{5}$$

where t and s are the variables referring to the time steps Δt and Δt_q, respectively, and T_c is the maximum t, generally referring to an integer number of hydraulic time steps over an operative cycle of the analysis (e.g., 24 steps of 1 hr each for simulating a daily cycle). Note that the initial condition for $t = 0$ and $s = 1$ is mandatory.

The number of parcels required at each water quality step is the ratio of T_r with Δt_q,

$$n_p(t\Delta t + s\Delta t_q) = ceil\left(\frac{T_r(t\Delta t + s\Delta t_q)}{\Delta t_q}\right) = ceil\left(\frac{L}{v_{\text{act}}(t\Delta t + s\Delta t_q)\Delta t_q}\right) \tag{6}$$

which must be an integer number. Therefore, the number of parcels is rounded toward positive infinity (*ceil*) meaning rounding at the real number toward the closest greater integer number.

Consequently, the Δx_q is the length of parcel varying over Δt_q given by:

$$\Delta x_q(t\Delta t + s\Delta t_q) = \frac{L\,\Delta t_q}{T_r(t\Delta t + s\Delta t_q)} \tag{7}$$

Nonetheless, a shorter parcel at the end of the pipe is necessary to accommodate the use of rounding by *ceil* to have an integer number of parcels n_p in Equation 6. Thus, the $\Delta x_{q\text{-last}}$ is the length of the last parcel varying over Δt_q given by

$$\Delta x_{q-\text{last}}(t\Delta t + s\Delta t_q) = \left[L - (n_p(t\Delta t + s\Delta t_q) - 1)\Delta x_q(t\Delta t + s\Delta t_q)\right] \leq \Delta x_q(t\Delta t + s\Delta t_q) \tag{8}$$

The actual velocity over the quality time-steps can be computed as linearization of the steady state velocities between two steps Δt:

$$\begin{aligned} v_{\text{act}}(t\Delta t + s\Delta t_q) &= V_{\text{act}}(t\Delta t) + \frac{s \cdot \Delta t_q}{\Delta t}(V_{\text{act}}((t+1)\Delta t) - V_{\text{act}}(t\Delta t)) = \\ &= V_{\text{act}}(t\Delta t) + s\frac{V_{\text{act}}((t+1)\Delta t) - V_{\text{act}}(t\Delta t)}{m_p} \end{aligned} \tag{9}$$

where $V_{\text{act}}(t\Delta t)$ is the pipe flow velocity at corresponding tth time-step. It must remark that the assumption of linearizing velocity between two hydraulic time step accounts for the increase in time sampling in water quality models, which is neglected in other water quality models that use the same value of velocity over multiple Δt_q in the same Δt (Davis et al., 2018). Alternatively, a hydraulic steady-state analysis using $\Delta t = \Delta t_q$ can be performed linearizing the boundary conditions (e.g., the customer water demands) consistently with the steady-state assumption of the hydraulic analysis. What is the best choice among those reported above and how far the linearization of velocity within the hydraulic time step is worth in terms of accuracy versus computational time still an open issue, which can be analyzed in future studies since it is beyond the main purpose of assessing the nexus between leakage management activities and water age of this work.

3.3. Short and Long Pipes

The DVM method for water quality/age analysis in EPANET assesses *short pipes* with high velocities issues, which leads to lower water quality time steps, by defining a minimum water quality time-step (Rossman & Boulos, 1996; Rossman et al., 1993). In the case of *long pipes*, implying the generation of many segments, EPANET 1.0 solver avoided the generation of many segments by assigning a default maximum number of segments, equals to 100 (Rossman et al., 1993), while further versions settle it by inserting a minimum tolerance value for the difference between nodal quality/age of downstream node and concentration/age of existing downstream segment (Rossman & Boulos, 1996).

In the proposed water quality analysis, which is different from EPANET, when $\Delta t_q > T_r$ the pipe is a *short pipe*, that is, $n_p = 1$. It occurs for short lengths (L) and/or high velocities (v_{act}). For *short pipes* we assume $\Delta x_q = L$ and $\Delta x_{q\text{-last}} = 0$ meaning that the advection of the substance decelerates, since it is assumed to take a longer time (Δt_q) to reach the downstream node than the actual travel time of water (T_r). It is worth noting that the proposed scheme evaluates pipes flow velocity across two water quality time-steps by linear interpolation as in Equation 9, therefore the maximum number of parcels generated along the pipe is variable.

The maximum number of parcels $n_{p\text{-max}}$ needs to be defined because of computational runtime and memory requirements. Therefore, we define *long pipes* those for which at any Δt_q the following inequality holds:

$$n_p(t\Delta t + s\Delta t_q) = ceil\left(\frac{L}{v_{\text{act}}(t\Delta t + s\Delta t_q)\Delta t_q}\right) > n_{p-\max} \tag{10}$$

Then, assuming $n_{p\text{-max}}$, the occurrence of a *long pipe* is determined during water quality analysis by its higher length (L) and/or smaller velocity (v_{act}). For *long pipes* we assume $\Delta x_q = \Delta x_{q\text{-last}} = L/n_{p\text{-max}}$ meaning that the advection of the substance accelerates since it is assumed to take $\Delta t_q \cdot n_{p\text{-max}} < T_r$ to reach the downstream node. Note that *short* and *long pipe* occurrence depends on velocity (v_{act}); consequently, any pipe could be short or long, during the analysis, with respect to Δt_q and $n_{p\text{-max}}$ assumptions. Overall, the water quality analysis asks for an optimal selection of Δt_q and $n_{p\text{-max}}$ considering a good trade-off between the accuracy of the analysis and computational runtime accounting for memory storage. In addition, the selection of Δt_q affects the accuracy of the calculation of the substance kinetics excluding the cases of water age and trace.

3.4. Null Velocities and Stopped Water

The EPANET water quality/age analysis is conditioned by numerical approximation of the hydraulic solver. In case of a pipe with flow rate close to zero, the water quality routing bypasses that pipe. Equation 10 shows that for null velocity the number of parcels is infinite. For null velocity pipes we can argue that the water in the pipe stops as for a closed/abandoned pipe. However, the difference between a closed/abandoned pipe and a pipe where the water stops is that in the latter the water can flow again over time (e.g., due to controls).

Therefore, in case of water stop it is necessary to modify the network topology during the water quality analysis to exclude null velocity pipes from the advection of the substance at a specific Δt_q, while allowing to perform kinetic reactions (e.g., the water age increases in that pipe since water stops). Finally, let us remark that null velocity might come from a real hydraulic condition or from a calculated hydraulic status (e.g., a closed pipe during hydraulic simulation because of controls). This means that the condition of null velocity for water quality analysis needs to account for the minimum velocity related to the accuracy of the hydraulic solver (e.g., 10^{-8} m/s). This is a relevant issue while designing leakage reduction activities, which might require closing sectioning valves at the boundaries of DMAs.

3.5. Mapping Parcels Over the Water Quality Steps

The strategy of mapping the mass in the parcels from water quality time step $s - 1$ to s is important for accuracy because it might determine a loss of information about the spatial distribution of substance masses. A way to limit such loss of information is to map them from $s - 1$ to s by preserving as much as possible the distribution of masses itself.

The proposed scheme implements the following strategy. When the number of parcels decreases, masses are reaggregated, see Figure 2a. For example, in parcel 1-new the mass $M_{1\text{-new}}$ is the sum of M_1, M_2, and a portion

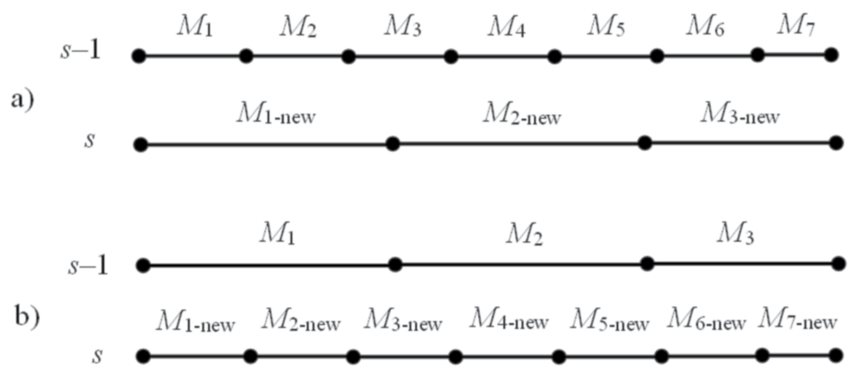

Figure 2. Mapping of parcels in pipes over the quality steps: (a) reduction and (b) increasing.

of M_3. Therefore, M_3 is divided between parcels 1-new and 2-new and M_5 between 2-new and 3-new depending on their overlapping position from $s - 1$ to s. When the number of parcels increases, masses are split, see Figure 2b. For example, parcels 1-new and 2-new have the masses reduced proportionally to M_1 as the reduction of Δx_q; in other words, they maintain the same concentration. The parcel 3-new takes the mass $M_{3\text{-new}}$ from parcels 1 and 2 depending on the overlapping position from $s - 1$ to s. This means that the new parcels generally maintain the concentration pattern while the ones across two new parcels interpolate the relevant concentrations.

However, the case of Figure 2a corresponds to an undersampling of the information about the pattern while the case of Figure 2b to an oversampling, meaning that the information about their spatial pattern results distorted. Indeed, considering Figure 2, if one assumes an initial mapping (at $s - 1$) with seven parcels, followed by a mapping (at s) with three parcels, and thereafter a new mapping (at $s + 1$) with seven parcels again, the final distribution of concentrations will be different from the initial one.

3.6. Accuracy Correction Versus Not Limited Number of Parcels

In the proposed scheme the accuracy of water quality calculations with respect to long and *short pipes* is affected by Δt_q and $n_{p\text{-max}}$. Differently, in EPANET solver numerical scheme for water quality analysis, the accuracy depends on the choice of the water quality time steps (Δt_q) and the concentration tolerance used to avoid the generation of new segments (Rossman, 2000; Rossman & Boulos, 1996; Rossman et al., 1993).

It is possible to develop strategies to correct the loss of accuracy due to *long* and *short* pipes. A simple strategy is to correct Δt_q based on the following ratio,

$$c(t\Delta t + s\Delta t_q) = \frac{n_p(t\Delta t + s\Delta t_q)}{n_{p-\max}} = \frac{T_r(t\Delta t + s\Delta t_q)}{\Delta t_q n_{p-\max}} \tag{11}$$

that is, ratio of the number of parcels required by the travel-time (as real number) and the maximum number of parcels allowed. The correction is greater than unit for *long* pipes, lower than unit for *short* pipes, and unitary for the others. Therefore, the quality time step in *short* and *long* pipes is $c\Delta t_q$ while in the normal pipes it is Δt_q. This creates a mismatching between water age (i.e., quality time step) and parcel (i.e., hydraulic simulation time step) arrival times in nodes where *long*/*short* pipes and *normal* pipes join. Thus, the correction results in a lack of synchrony of the mass/concentration arriving in a node from *short*/*long* pipes with respect to the other *normal* pipes joined at the same node.

Hence, we introduce an innovative way to increase the accuracy of the water quality analysis by removing the limitation of $n_{p\text{-max}}$. In fact, it can be argued that the assumption of minimum velocity to stop water in pipes according to the hydraulic analysis accuracy (e.g., 10^{-8} m/s), implies that $n_{p\text{-max}}$ cannot be infinity, and it depends on the velocity variability over T_c (e.g., 24 hr) and Δt_q,

$$n_{p-\max}(k) = ceil\left(\frac{L(k)}{\min\left[v_{\text{act}}(k, t\Delta t + s\Delta t_q)\right]\Delta t_q}\right) \tag{12}$$

10.1029/2021WR031919

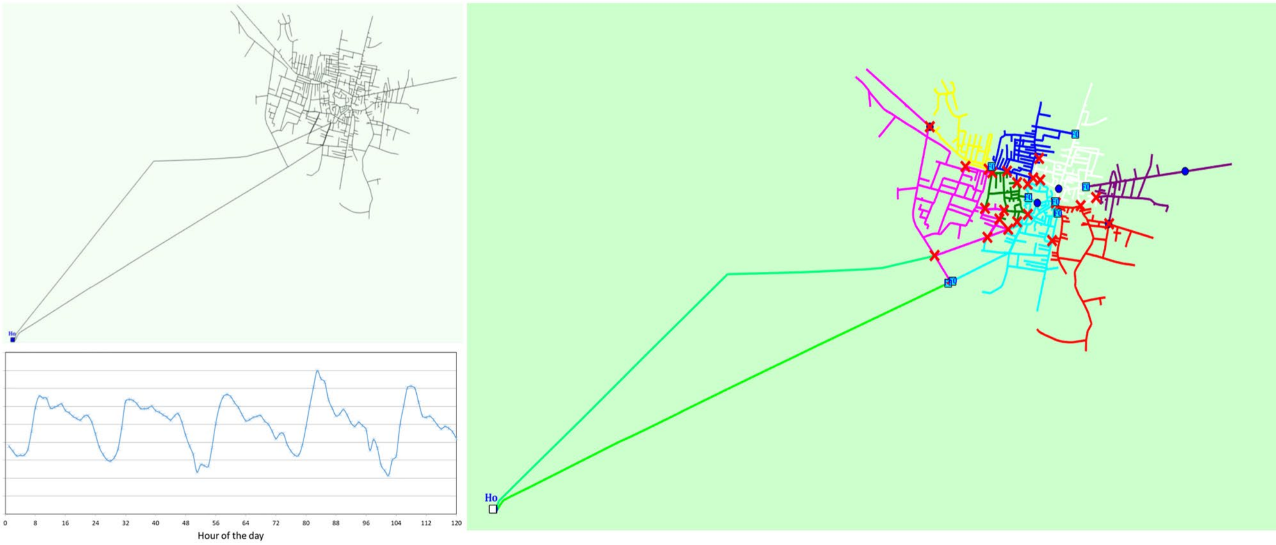

Figure 3. Sannicandro Water Distribution Network (WDN) and calibrated demand pattern (left) and DMA design (right).

This assumption asks for efficient data structures to optimize computational runtime and memory storage. For instance, to this purpose, data of parcels for each pipe can be stored, for example, in MATLAB programming environment, in a cell type matrix. This way the maximum value of parcels among pipes, $\max(n_{p\text{-max}}(k))$, does not constraint the size of a single matrix that is used to store data of all the parcel in classic programming approach. The maximum amount of data stored for parcels depends on the summation of $n_{p\text{-max}}(k)$ over all pipes, and the maximum value that we can use for $\max(n_{p\text{-max}}(k))$ depends on the CPU memory and indexing.

Finally, it is worth noting that as Δt_q decreases and m_p (the number of quality steps inside Δt) increases, the variation of the velocity, $v_{\text{act}}(t\Delta t + s\Delta t_q) - v_{\text{act}}(t\Delta t + (s-1)\Delta t_q)$, and the variation of the number of parcels, $n_p(t\Delta t + s\Delta t_q) - \mathrm{n_p}(t\Delta t + (s-1)\Delta t_q)$, decreases as well. This fact is beneficial for accuracy with respect to the mapping of the pipe from $n_p(t\Delta t + (s-1)\Delta t_q)$ to $n_p(t\Delta t + s\Delta t_q)$ at each quality step s. Equation 12 shows that the unbounded $n_{p\text{-max}}$, beyond the computer and programming language limitations in indexing, allows decreasing Δt_q thus increasing the accuracy of the substance kinetics calculation and overcoming *short pipes* issues. The accuracy of results is then conditioned by the memory storage and computational runtime capabilities, which is a minor issue considering the continuous increasing of the available computational power.

4. Case Studies and Results Discussion

4.1. Case Studies: DMA and Pressure Control Designs Versus Water Age

Two real WDNs in Apulia (Italy) are here used to study the change of water age due to pressure control and DMA designs for leakage management. The models of both networks were calibrated based on a mass balance approach allowing to separate consumers' demand from volumetric leakages (Berardi et al., 2017), using a hydraulic time step $\Delta t = 1$ hr. Model calibration provided the identification of a unique demand pattern, using flow and pressure measurements of five daily operative cycles corresponding to different functioning in summer/winter and working days/holidays (see left side of Figures 3 and 4). The calibration did not use data at DMA level because the DMA design was part of leakage management activities. Table 1 summarizes key indicators of leakage management and water age before and after pressure control and DMA design. The leakage rate is defined as the linear leakage indicator M1a [m^3/day/km], according to the Italian Regulation (ARERA, 2017).

The first WDN supplies 3,194 consumers of Sannicandro town from a single reservoir by means of two main water supply lines. The hydraulic network is about 40 km long and delivers an annual average of customer demand of 1,041 m^3/day. The ratio between M1a and the average pressure of the system P_{WDN} is 1.33, meaning that the leakages are caused by pipes deterioration and, possibly, by the excess of pressure (Berardi & Giustolisi, 2021). The WDN hydraulic model is composed of 1,050 pipes and 928 nodes. After pressure control and DMA design,

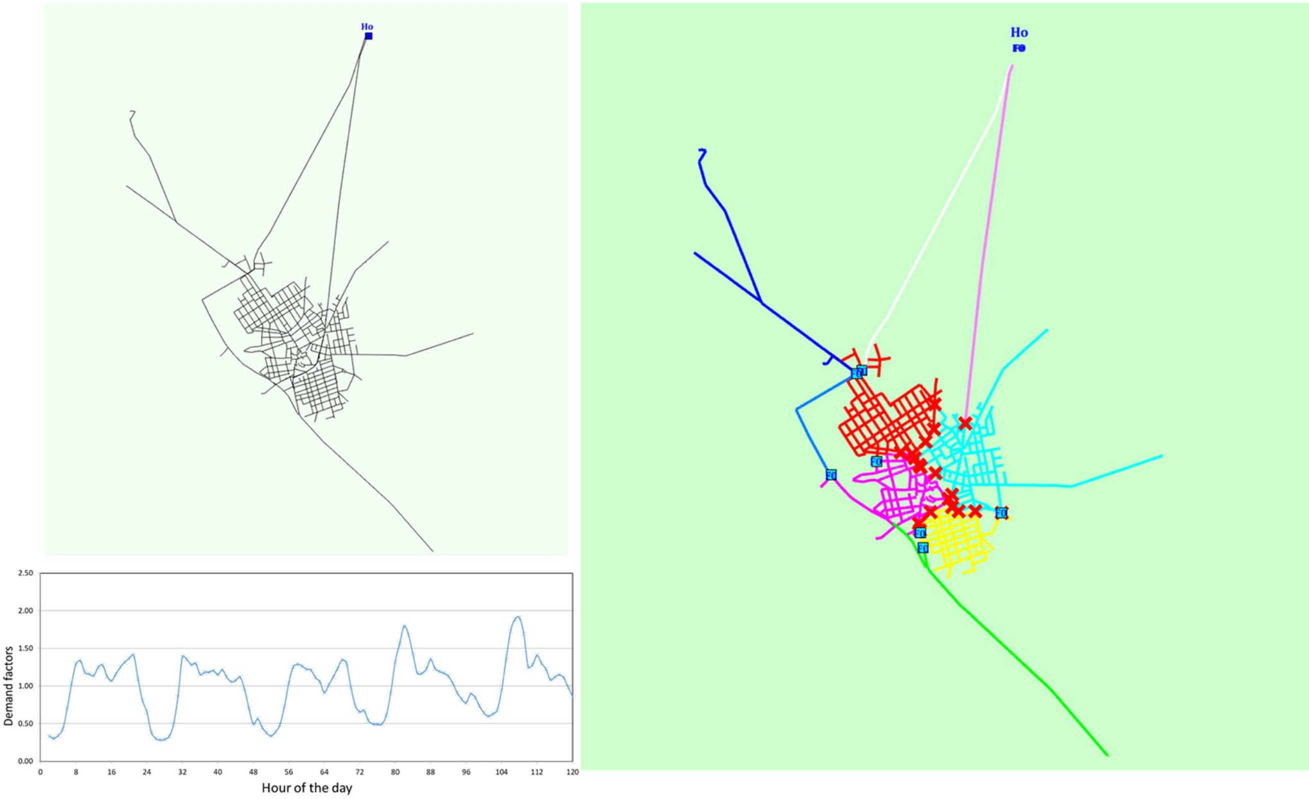

Figure 4. San Marzano Water Distribution Network (WDN) and calibrated demand pattern (left) and DMA design (right).

volumetric leakages are reduced of about 59%. This required a single pressure reduction valve and 10 DMAs identified by means of 23 closed gates and 10 flow meters at the boundary of DMAs (see Figure 3 right).

The second WDN serves 3,286 consumers of San Marzano town from a single reservoir by means of two main water supply pipes. The hydraulic network is about 51 km long and delivers an annual average of customer demand of 2,277 m^3/day. The ratio between M1a and P_{WDN} is 0.63, meaning that leakages, lower than the first WDN, are mainly caused by pressure excess. The WDN hydraulic model is composed of 989 pipes and 729 nodes.

After pressure control and DMA design, volumetric leakages are reduced of about 25%. The leakage management activities included a single pressure reduction valve and 10 DMA identified by means of 19 closed gates and 10 flow meters at their boundaries (see Figure 4 right).

To provide answers to the above mentioned technical/management questions (1), (2), and (3) that motivated this work, water age was computed in both WDNs on a 24-hr operating cycle, assuming the first 24 hr of the demand pattern identified during the calibration, the configurations before and after leakage management planning, and using the model described above with unlimited maximum number of parcels (i.e., see Equation 12). The values

Table 1
Indicators of Leakage Management and Water Age Before and After Pressure Control

	Sannicandro		San Marzano	
Property	Before	After	Before	After
Linear leakage indicator (M1a) [m^3/day/km]	53.4	21.9	18.6	13.9
Daily leakages [m^3/day]	2,111	865	959	717
Average pressure (P_{WDN}) [m]	40.3	17.5	29.4	22.9
Average water age [h]	11.18	17.71	21.83	27.32

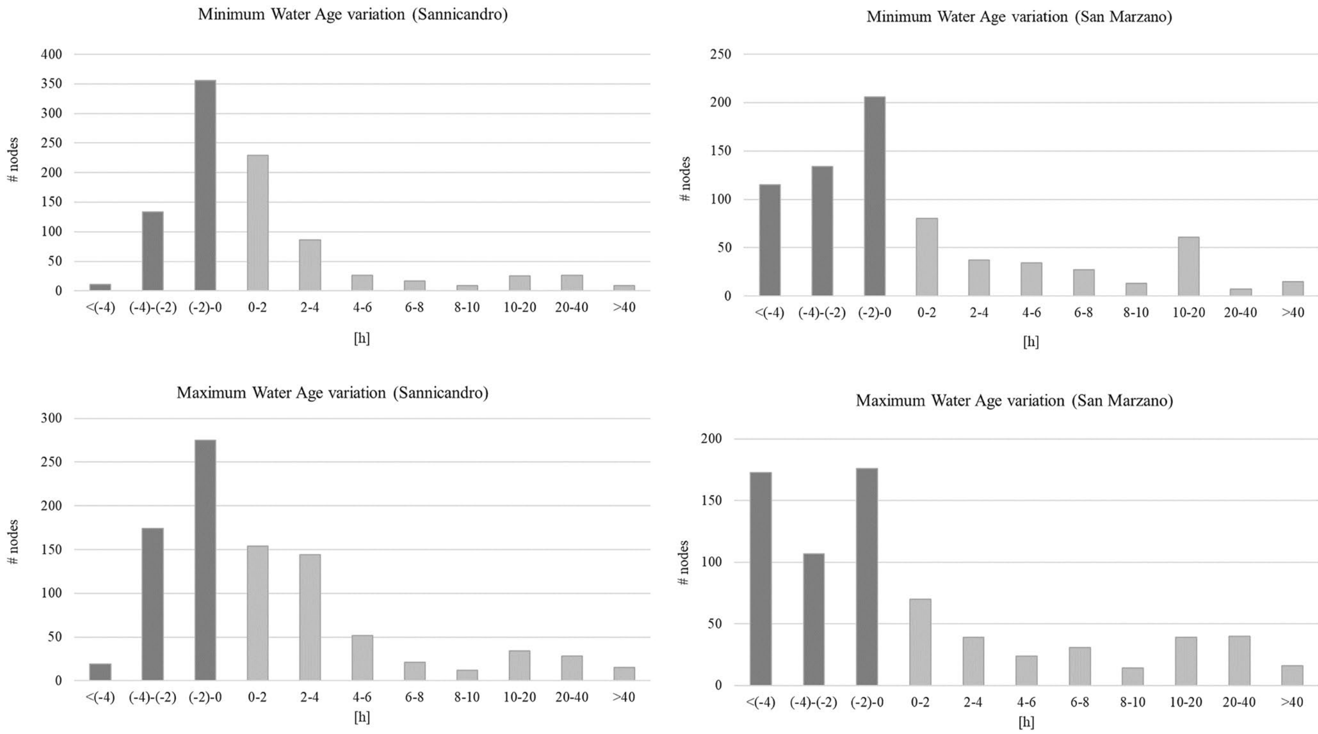

Figure 5. Minimum and maximum variation of daily water age at nodes: Sannicandro Water Distribution Network (WDN) (left) and San Marzano WDN (right).

of water age at network level in Table 1 show that leakage management activities cause an average increase of about 6 hr in both WDNs. However, for a careful evaluation the histograms in Figure 5 show the distribution of variations of the minimum and maximum nodal water age (i.e., as difference between values after and before the leakage management activities). It should be noted that negative values of the variation indicate a decrease in water age (dark gray histograms), while positive values indicate an increase in water age (light gray histograms).

It is worth noting that a massive increase (>40 hr) in the minimum daily water age is observed in a small number of nodes (nine in Sannicandro WDN and 15 in San Marzano WDN), that are almost the same experiencing an increase also in the maximum daily water age. Such nodes are the nearest to sectioning valves at DMA boundaries, that were closed to reconfigure water paths in the system or are ending nodes of branches with null to small water demands, that were at higher pressure in the initial configuration. In the latter case, the reduction of leakages along branched pipes (i.e., due to pressure reduction) reduces the water discharge and velocity, thus increasing the retention time.

Observing Figure 5 it appears that leakage management activities have different consequences, in terms of "water age envelope," on the two analyzed networks (Braun et al., 2020). Sannicandro WDN shows that about 50% of the nodes experience a reduction of the minimum and maximum water age (mostly between 0 and 4 hr), meaning that leakage reduction activities also improve the quality of the water supplied at those nodes. The other 50% of nodes shows the opposite phenomenon, but with an increase even higher than 40 hr. For San Marzano WDN, on the other hand, it is noted that two thirds of the nodes improve the quality of the water supplied, with many nodes (about 115) showing a water age decrease of more than 4 hr. These different consequences are undoubtedly due to the peculiarities of each WDN in terms of topology, distribution of consumers' demands, feeding pipes, etc. Therefore, the leakage management activities, which lead to water loss reduction on average, also act on the reconfiguration of the flows within the new DMAs, and this creates areas that reduce the water age (e.g., those along the new water paths where flow rate increases), while others (e.g., those close to the new closed valves) have an increase and must be carefully monitored by the water service.

In this regard, it should be noted that for Sannicandro WDN the pressure control by PRV is responsible for leakage reduction by 53% (out of a total reduction of 59%), while in San Marzano WDN there is a lower global reduction in leakages but 19% of leakage reduction (out of a total reduction of 25%) is achieved by reconfiguring water paths with DMA design.

10.1029/2021WR031919

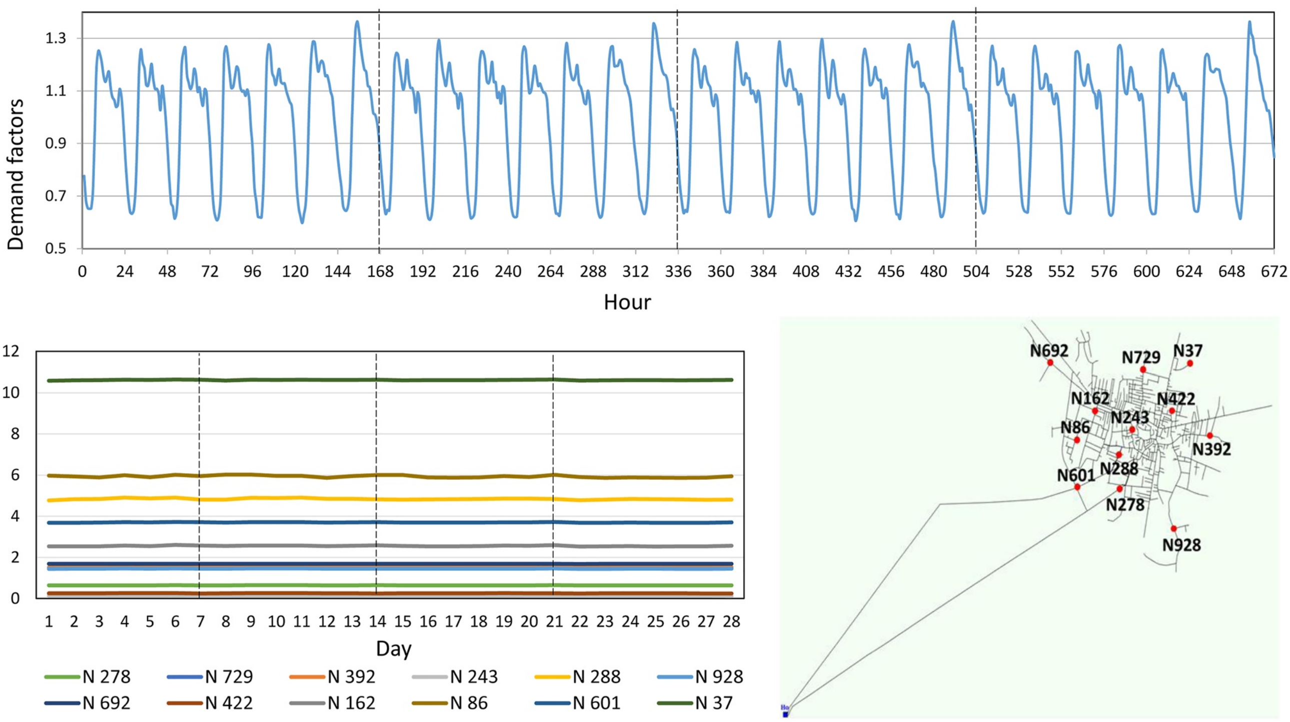

Figure 6. Sannicandro Water Distribution Network (WDN) after leakage management: demand pattern for 4-week operating cycle (top); average daily water age in some nodes (bottom left); location of nodes (bottom right).

Therefore, answers to the above mentioned technical/management questions (1), (2), and (3) can be given as follows:

1. The analyses above demonstrate that water age depends on leakage management activities and on the initial leakage rate which is non-trivial and depends on reconfiguration of water paths as well as on reduction of leakage outflows. Therefore, the effects of leakage management need to be carefully investigated using advanced hydraulic model accounting for pressure-dependent leakages along pipes.

Usually, water age analyses assume that the analyzed daily pattern reproduces the same as itself, although it might depend on initial conditions. Therefore, the impact of changing demand patterns on water age is here studied by running a 4-week extended period simulation in the case of Sannicandro WDN (after leakage management) only. Note that for the sake of clarity the following figures report the average daily value of water age. The upper diagram in Figure 6 shows the water demand patterns used as 4-week operating cycle, with different water consumptions in working days and weekend days (note that the demand patterns are identified from measured inflow data using the calibrated model). The diagram in the left lower part of Figure 6 shows the average daily water age for each node whose location is indicated with red dots on WDN map on right lower part of the figure. Therefore:

2. results show that the change in water demand over different daily operating cycles does not results into remarkable variations of the mean daily water age. This result is consistent with WDN hydraulics. In fact, the slight increase of user water demand in some days likely results into a reduction of pressure and thus into lower leakages. Vice versa, if user water demand decreases, leakages are likely to increase. Ultimately this mechanism does not substantially change the total network inflow, and the mean daily water age across the 4-week operating cycle as well.
3. This analysis on the Sannicandro WDN, in conjunction with the above discussed variations of minimum-maximum daily water age after leakage management, demonstrate that daily water age depends on total network inflow rather than on changes in consumers' demand.

10.1029/2021WR031919

4.2. Accuracy Issues in Water Quality Modeling

As mentioned above, a key issue in water quality modeling is the setting of water quality time step Δt_q and maximum number of parcels $n_{p\text{-max}}$. Indeed, setting $n_{p\text{-max}}$ determines an acceleration of the advection of the substance, that is, the transfer of masses is faster than it would be with unlimited number of parcels. In Section 3.6, two strategies are proposed to cope with this issue.

The first strategy consists in correcting Δt_q using the time correction c as in Equation 11: it allows setting $n_{p\text{-max}}$ to reduce the CPU time. However, such correction creates a lack of synchrony between the mass/concentration arriving to a node from *short* ($c < 1$) and/or *long* ($c > 1$) pipes and the other "normal" ($c = 1$) pipes, because water flow rates come from hydraulic simulation whose time step Δt does not change. The second strategy uses "unlimited" number of parcels for each pipe: it maintains the synchrony between mass/concentration transfer and hydraulic simulation, although it requires adequate CPU memory and indexing capabilities.

To investigate the impact of both strategies, the water quality was modeled in Sannicandro WDN and San Marzano WDN in both configurations (i.e., before and after leakage management). It was assumed $n_{p\text{-max}}$ in the set (60, 120, 240, 480, 960, 1920, 3840), with or without using the correction factor c. In each WDN water quality analysis, the number of pipes which required the maximum number of parcels over 24-hr operating cycle are:

- Sannicandro (Before leakage management) = (358, 295, 195, 95, 58, 15, 14).
- Sannicandro (After leakage management) = (426, 356, 292, 189, 100, 100, 17).
- San Marzano (Before leakage management) = (416, 328, 265, 211, 159, 99, 44).
- San Marzano (After leakage management) = (426, 332, 269, 219, 168, 123, 47).

This shows that the same value of $n_{p\text{-max}}$ has different impact on water quality modeling in different WDNs. For instance, for the configuration before leakage management, setting $n_{p\text{-max}} = 3{,}840$ results into 1.3% of *long* pipes (i.e., with $n_p = n_{p\text{-max}} = 3{,}840$) in Sannicandro WDN, while into 4.4% in San Marzano WDN. Moreover, reducing leakages generally results into a higher number of pipes requiring $n_{p\text{-max}}$ parcels, due to decrease in water flows and velocities, consistently with the results discussed in previous section.

All cases above are compared with water quality model using "unlimited" number of parcels. In Sannicandro WDN the number of parcels for a single pipe due to the "unlimited" assumption equals 419,817 and 15,348,855, respectively, before and after leakage management. In San Marzano WDN to maximum number of parcels for a single pipe equals 55,404,006 and 1,731,815 before and after leakage management, respectively. Assuming that the evaluation of water age with the unlimited parcel case is not affected by modeling errors, Figures 7 and 8 show on the x-axis the error (in hours) on average daily water age (calculated on a 24-hr operating cycle) with respect to the unlimited parcel case. For the sake of clarity, the x-axis is organized into intervals of errors. Note that the bar corresponding to null error include errors up to 5 min, assumed as technically negligible for the purposes of this analysis. The y-axis reports the number of nodes falling into error interval. For each network the water age was analyzed with the configuration before and after leakage management activities, and with or without using the correction factor c. All these analyses were carried out assuming $n_{p\text{-max}}$ in the set (60, 120, 240, 480, 960, 1,920, 3,840). All plots on the left side of the figures (i.e., no time correction) confirm that limiting $n_{p\text{-max}}$ to low values results in underestimating water age. Using higher values of $n_{p\text{-max}}$ results in globally increasing the accuracy of water age evaluation, that is, almost all nodes show null error. Accounting for time correction c (plots on the right) determines an overestimation of average daily water age regardless to the assumption of the network configuration with and without leakage management activities. It is worth noting that in San Marzano network, due to its topological characteristics, there are many *long pipes* (as mentioned above), and this affects the accuracy of the estimation of water age in some nodes. This situation tends to be reduced with the increase of $n_{p\text{-max}}$, but in a less evident way if compared to Sannicandro WDN.

Figure 9 further analyses the simulation of water age in terms of ratio between the mean water age (i.e., over 24-hr operating cycle and across all nodes) for various $n_{p\text{-max}}$ and the mean water age computed with unlimited number of parcels (i.e., assumed to be not affected by modeling errors). Figure 9 confirms that without accounting for time correction c (left plot) for both study networks there is an underestimation of the water age with limited $n_{p\text{-max}}$, while with time correction the water age is tendentially overestimated, as above mentioned by Figures 7 and 8.

In detail, it can be noted that, without time correction c, both networks show similar trends, whereas for Sannicandro the underestimation of the water age is identical before and after the leakage management activities,

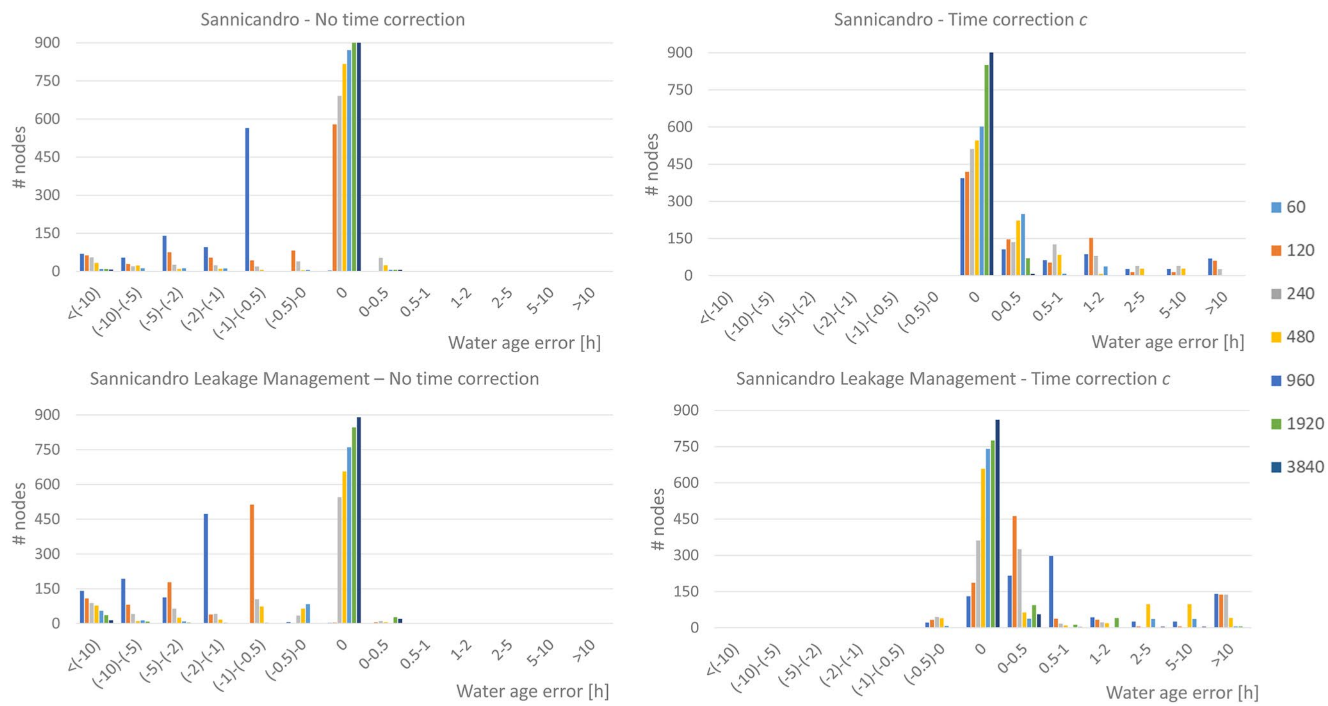

Figure 7. Sannicandro Water Distribution Network (WDN): error on average water age (left) no time correction; (right) time correction; (up) before and (bottom) after leakage management activities.

while for San Marzano, the analysis after the interventions shows greater underestimates than before. This could mean that the topological and hydraulic characteristics of the San Marzano network (e.g., the main role of water paths reconfiguration due to districtualization) further condition the water age modeling, strictly requiring the approach proposed here and therefore the use of unlimited quality parcels. The assumption of the time

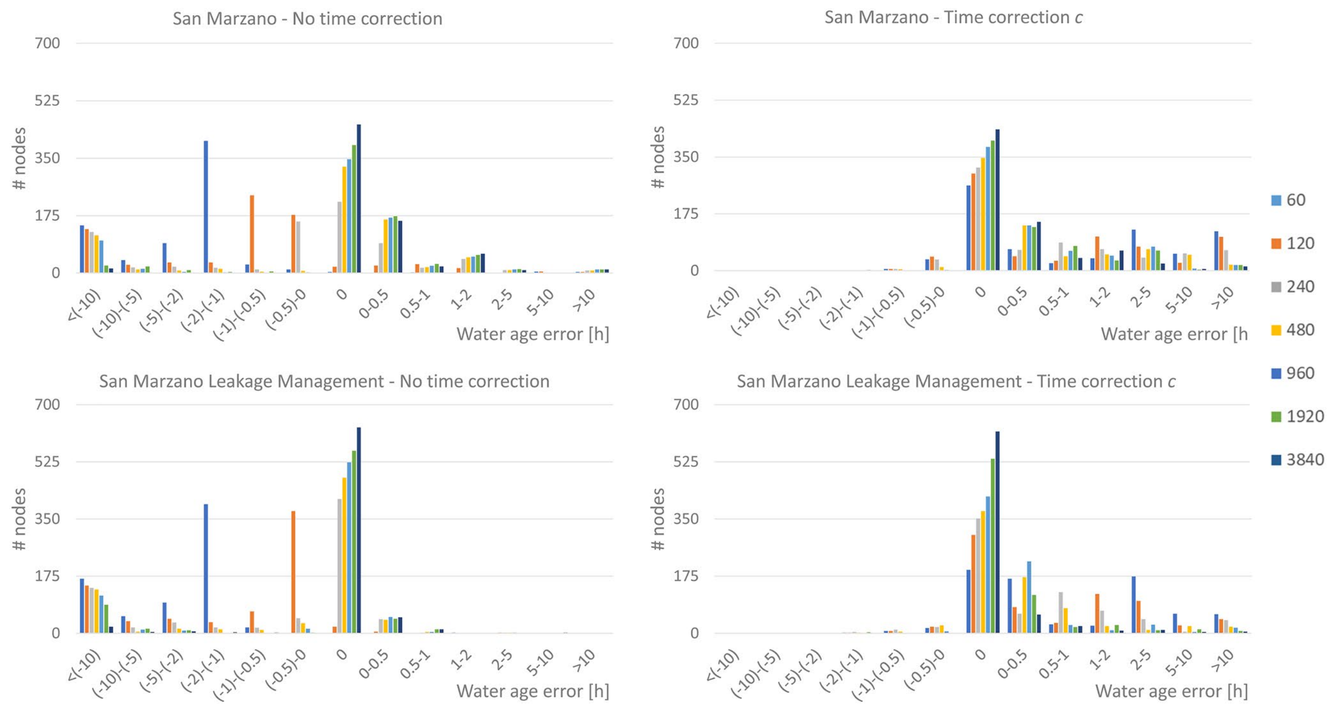

Figure 8. San Marzano Water Distribution Network (WDN): error on average water age (left) no time correction; (right) time correction; (up) before and (bottom) after leakage management activities.

10.1029/2021WR031919

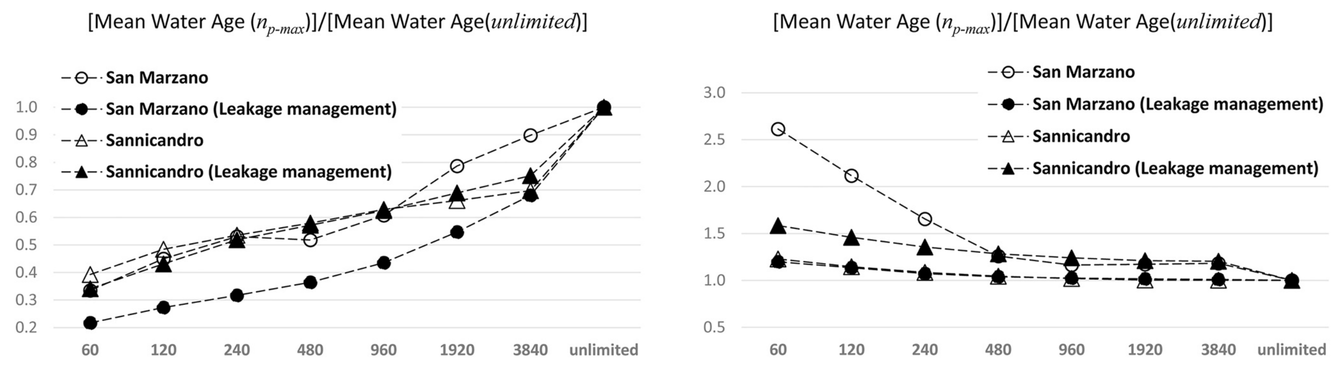

Figure 9. Ratio between the mean water age for various $n_{p\text{-max}}$ and mean water age with unlimited $n_{p\text{-max}}$: (left) no time correction; (right) time correction.

correction *c* (right plot) implies for Sannicandro a constant worsening of the water age estimate in the condition after the leakage management activities compared to before (where an $n_{p\text{-max}} = 480$ would be enough to have a precise estimate). On the other hand, for San Marzano, time correction induces significant overestimates of the water age in the pre-intervention condition, while after there are excellent estimates even for low values of $n_{p\text{-max}}$.

To analyze the CPU time performance in all model settings, all WDN configurations were run for 100 operating cycles and relevant average data are reported in Table 2. It is evident that the unlimited number of parcels requires about double time for each operative cycle with respect to simulations with limited $n_{p\text{-max}}$. Recall that using time correction *c* results into lack of synchrony with respect to hydraulic time step.

Previous results support answering the two questions related to water quality modeling issues, as follows:

4. Limiting maximum number of parcels allows reducing CPU time but might introduce large underestimation of water age. Such loss in accuracy depends on the specific WDN hydraulics behavior and the same value of $n_{p\text{-max}}$ can result into different accuracy in different WDNs. This result is also relevant when WDN hydraulic regime changes in consequence of leakage reduction. The proposed strategy for using unlimited number of

Table 2
CPU Time Performances for Sannicandro Water Distribution Network (WDN) and San Marzano WDN Assuming Δt_q = 1 min and Δt = 1 hr

Sannicandro WDN				San Marzano WDN			
$n_{p\text{-max}}$	Time correction	Run time/run time (unlimited)	Operative cycle run time [min]	$n_{p\text{-max}}$	Time correction	Run time/run time (unlimited)	Operative cycle run time [min]
60	Time correction	0.58	0.425	60	Time correction	0.39	0.364
120		0.59	0.435	120		0.41	0.377
240		0.60	0.438	240		0.42	0.390
480		0.61	0.445	480		0.43	0.401
960		0.62	0.457	960		0.45	0.416
1,920		0.63	0.459	1,920		0.48	0.447
3,840		0.63	0.462	3,840		0.53	0.489
Unlimited	No time correction	1.00	0.733	Unlimited	No time correction	1.00	0.925
60		0.57	0.421	60		0.39	0.364
120		0.58	0.423	120		0.41	0.379
240		0.60	0.442	240		0.41	0.383
480		0.60	0.442	480		0.44	0.406
960		0.61	0.451	960		0.45	0.416
1,920		0.63	0.465	1,920		0.49	0.449
3,840		0.65	0.473	3,840		0.53	0.492

parcels is only bounded by memory and data storage capabilities. Nonetheless, the absolute CPU time is lower than 1 min for each operating cycle (i.e., simulating 24 hr of WDN functioning) on the real WDNs used in this work.

5. Using factors to correct the limitation of number of parcels results into higher water age and introduces loss of synchrony between the mass/concentration advection and hydraulic simulation, which needs to be considered.

As final consideration, if one needs to set $n_{p\text{-max}}$, the percentage of pipes with number of parcels larger than $n_{p\text{-max}}$ seems to be a good driver to select the correction strategy. For instance, in Sannicandro WDN, where just few pipes reach $n_{p\text{-max}} = 3{,}840$, time correction does not improve convergence toward "true" water age (i.e., using "unlimited" number of parcels). Vice versa, in San Marzano WDN synchrony provides a faster convergence toward "true" water age, at least in terms of mean values (see Figure 9 right).

5. Conclusions and Final Remarks

Leakage reduction in WDN is of preeminent importance nowadays and various actions can be undertaken to accomplish such task. This work investigates the impact of leakage management activities, that is, pressure control and DMA design, on water age because of change of flow and velocity fields in the network.

For the first time, water quality modeling is based on advanced WDN hydraulic models accounting for pressure-dependent leakages, computed at single pipe level. The discussion above demonstrates the technical consistency of such hydraulic modeling with the purposes of accounting for water quality while planning leakage management activities.

A novel Lagrangian scheme for water quality model is presented, which overcomes some limitations of previous models, allowing *unlimited* number of parcels in *long/low velocity* pipes. It means that the maximum number of parcels is limited by memory and storage capability of the computing environment. The new water quality model is also used to investigate the impact of limiting the maximum number of parcels a priori, with or without introducing a time correction factor for *long pipes*. Extensive numerical examples on two real WDN, in their original configuration and after planning pressure control and DMA design, have been used to answer five main questions.

From leakage management perspective, the analyzed case studies showed that (a) the change in water age is affected by the leakage reduction caused by the considered leakage management activities (i.e., pressure control and DMAs), and by other factors such as the initial level of leakages, the new WDN connectivity, the customer demand distribution, and the attained reduction, as these reflect WDN specific hydraulic behavior. In addition, a 4-week long extended period simulation proved that (b) the mean water age in WDN nodes is not much affected by normal variation of demand pattern over different days. This happens because pressure-dependent leakages, as simulated in the advanced WDN model, tend to increase as pressure increase due to reduction of demands and vice versa. Such result suggests that (c) daily water age, as an indicator to drive water quality management activities, depends on daily network inflow rather than on daily customer demand. In order to better understand the nexus water age-leakage management activities future works could concern WDNs with different supply lines to see whether and how the planning and reconfiguration of flows can reduce water age; moreover, it could be interesting to study the evolution of water age at the single user level rather than at model nodes.

The sensitivity analysis of water age modeling for various maximum number of parcels allowed, unveils that (d) different WDNs would require different $n_{p\text{-max}}$, meaning that the loss of accuracy also depends on the WDN hydraulic, either before or after leakage management. In addition, (e) the correction of representation error in *long* and *short* pipes by using a time correction factor results into an increase in water age values which is higher for lower values of $n_{p\text{-max}}$. Therefore, such correction allows for a shorter CPU time than *unlimited* number of parcels, although it introduces a lack of synchrony between times when substances reach the same node where *long/short* pipes and *normal* pipes join. The analysis of results shows that the number of pipes reaching $n_{p\text{-max}}$ seems to be a good driver to choose about using time correction factor, meaning that the error in accuracy is negligible. As a final remark, results show that using the *unlimited* number of parcels results into double CPU time than other modeling settings.

Finally, future works could investigate whether, in water age modeling, it is better to linearize velocity between two hydraulic time steps or keeping v_{act} constant at every water quality time steps Δt_q within the same hydraulic time step Δt.

Data Availability Statement

Data from the academic spin-off company IDEA-RT s.r.l. archive were used in the creation of this manuscript. These data and related results are available at POLIBA (2022). Calculations and figures were made with WDNetXL version 5.0 (Spagnuolo et al., 2018), available under WDNetXL license at http://www.idea-rt.com/.

Acknowledgments
The authors thank the personnel of the academic spin-off company IDEA-RT s.r.l. (www.idea-rt.com) for their assistance in the implementation and use of the WDNetXL software, for providing pictures from WDNetXL and all the used data. The authors would like to thank the editor and three anonymous reviewers for their comments and suggestions that have helped us improve the final manuscript.

References

Almandoz, J., Cabrera, E., Arregui, F., Cabrera Jr., E., & Cobacho, R. (2005). Leakage assessment through water distribution network simulation. *Journal of Water Resources Planning and Management*, *131*(6), 458–466. https://doi.org/10.1061/(ASCE)0733-9496(2005)131:6(458)

Araujo, L. S., Ramos, H., & Coelho, S. T. (2006). Pressure control for leakage minimisation in water distribution systems management. *Water Resources Management*, *20*(1), 133–149. https://doi.org/10.1007/s11269-006-4635-3

ARERA. (2017). Delibera 917/2017/R/IDR – Regulation of the technical quality of the integrated water service or of each of the individual services that make it up) (in Italian). Retrieved from https://www.arera.it/it/docs/20/046-20.htm

Berardi, L., & Giustolisi, O. (2021). Calibration of design models for leakage management of water distribution networks. *Water Resources Management*, *35*(8), 2537–2551. https://doi.org/10.1007/s11269-021-02847-x

Berardi, L., Simone, A., Laucelli, D., & Giustolisi, O. (2017). Feasibility of mass balance approach to Water Distribution Network model calibration. *XVIII International Conference on Water Distribution Systems, WDSA2016, Procedia Engineering*, *186*, 551–558. https://doi.org/10.1016/j.proeng.2017.03.269

Berardi, L., Simone, A., Laucelli, D. B., Ugarelli, R. M., & Giustolisi, O. (2018). Relevance of hydraulic modelling in planning and operating real-time pressure control: Case of Oppegård municipality. *Journal of Hydroinformatics*, *20*(3), 535–550. https://doi.org/10.1007/s11269-021-02847-x

Blokker, E. J., Furnass, W. R., Machell, J., Mounce, S. R., Schaap, P. G., & Boxall, J. B. (2016). Relating water quality and age in drinking water distribution systems using self-organising maps. *Environments*, *3*(2), 10. https://doi.org/10.3390/environments3020010

Braun, M., Piller, O., Deuerlein, J., Mortazavi, I., & Iollo, A. (2020). Uncertainty quantification of water age in water supply systems by use of spectral propagation. *Journal of Hydroinformatics*, *22*(1), 111–120. https://doi.org/10.2166/hydro.2019.017

Colombo, A. F., & Karney, B. W. (2005). Impacts of leaks on energy consumption in pumped systems with storage. *Journal of Water Resources Planning and Management*, *131*(2), 146–155. https://doi.org/10.1061/(ASCE)0733-9496(2005)131:2(146)

Davis, M. J., Janke, R., & Taxon, T. N. (2018). Mass imbalances in EPANET water-quality simulations. *Drinking Water Engineering and Science*, *11*(1), 25–47. https://doi.org/10.5194/dwes-11-25-2018

Farmani, R., Walters, G., & Savic, D. (2006). Evolutionary multi-objective optimization of the design and operation of water distribution network: Total cost vs. reliability vs. water quality. *Journal of Hydroinformatics*, *8*(3), 165–179. https://doi.org/10.2166/hydro.2006.019b

Giustolisi, O., Berardi, L., & Laucelli, D. (2012). Generalizing WDN simulation models to variable tank levels. *Journal of Hydroinformatics*, *14*(3), 562–573. https://doi.org/10.1061/(ASCE)0733-9429(2008)134:5(626)

Giustolisi, O., Savic, D., & Kapelan, Z. (2008). Pressure-driven demand and leakage simulation for water distribution networks. *Journal of Hydraulic Engineering*, *134*(5), 626–635. https://doi.org/10.1061/(ASCE)0733-9429(2008)134:5(626)

Giustolisi, O., & Walski, T. M. (2012). Demand components in water distribution network analysis. *Journal of Water Resources Planning and Management*, *138*(4), 356–367. https://doi.org/10.1061/(ASCE)WR.1943-5452.0000187

Grayman, W. M., Murray, R., & Savic, D. A. (2009). Effects of redesign of water systems for security and water quality factors. In *World Environmental and Water Resources Congress 2009: Great Rivers* (pp. 1–11). https://doi.org/10.1061/(ASCE)WR.1943-5452.0000187

Laucelli, D. B., Simone, A., Berardi, L., & Giustolisi, O. (2017). Optimal design of district metering areas for the reduction of leakages. *Journal of Water Resources Planning and Management*, *143*(6), 04017017. https://doi.org/10.1061/(ASCE)WR.1943-5452.0000768

Li, X. F., & Mitch, W. A. (2018). Drinking water disinfection byproducts (DBPs) and human health effects: Multidisciplinary challenges and opportunities. *Environmental Science & Technology*, *52*(4), 1681–1689. https://doi.org/10.1021/acs.est.7b05440

MacDonald, G., & Yates, C. D. (2005). DMA design and implementation, a North American context. In *Leakage conference, IWA*.

Machell, J., Boxall, J., Saul, A., & Bramley, D. (2009). Improved representation of water age in distribution networks to inform water quality. *Journal of Water Resources Planning and Management*, *135*(5), 382–391. https://doi.org/10.1061/(ASCE)0733-9496(2009)135:5(382)

Munavalli, G. R., & Kumar, M. M. (2004). Modified Lagrangian method for modeling water quality in distribution systems. *Water Research*, *38*(13), 2973–2988. https://doi.org/10.1016/j.watres.2004.04.007

Piller, O., & Tavard, L. (2014). Modeling the transport of physicochemical parameters for water network security. *Procedia Engineering*, *70*, 1344–1352. https://doi.org/10.1016/j.proeng.2014.02.148

Poleneni, S. R., & Inniss, E. C. (2015). Small water distribution system disinfection by-product control: Water quality management using storage systems. *GEOMATE Journal*, *9*(17), 1365–1369. https://doi.org/10.4236/jwarp.2013.58A005

POLIBA. (2022). Leakage management influence on water age of Water Distribution Networks. Politecnico di Bari Onedrive Repository [Dataset]. Retrieved from https://politecnicobari-my.sharepoint.com/:f:/g/personal/danielebiagio_laucelli_poliba_it/EhBxuj74HCFKpIQnDXDv7RQBluazZSKyAM16DyF4aoQjoA?e=8XL8Na

Qi, Z., Zheng, F., Guo, D., Zhang, T., Shao, Y., Yu, T., et al. (2018). A comprehensive framework to evaluate hydraulic and water quality impacts of pipe breaks on water distribution systems. *Water Resources Research*, *54*(10), 8174–8195. https://doi.org/10.1029/2018WR022736

Quintiliani, C., Marquez-Calvo, O., Alfonso, L., Di Cristo, C., Leopardi, A., Solomatine, D. P., & De Marinis, G. (2019). Multiobjective valve management optimization formulations for water quality enhancement in water distribution networks. *Journal of Water Resources Planning and Management*, *145*(12), 04019061. https://doi.org/10.1061/(ASCE)WR.1943-5452.0001133

Rossman, L. A. (2000). EPANET user's manual. Retrieved from http://nepis.epa.gov/Adobe/PDF/P1007WWU.pdf

Rossman, L. A., & Boulos, P. F. (1996). Numerical methods for modeling water quality in distribution systems: A comparison. *Journal of Water Resources Planning and Management*, *122*(2), 137–146. https://doi.org/10.1061/(ASCE)0733-9496(1996)122:2(137)

Rossman, L. A., Boulos, P. F., & Altman, T. (1993). Discrete volume-element method for network water-quality models. *Journal of Water Resources Planning and Management*, *119*(5), 505–517. https://doi.org/10.1061/(ASCE)0733-9496(1993)119:5(505)

Rossman, L. A., Woo, H., Tryby, M., Shang, F., Janke, R., & Haxton, T. (2020). EPANET 2.2 user manual. Retrieved from https://epanet22.readthedocs.io/_/downloads/en/latest/pdf/

Säve-Söderbergh, M., Bylund, J., Malm, A., Simonsson, M., & Toljander, J. (2017). Gastrointestinal illness linked to incidents in drinking water distribution networks in Sweden. *Water Research*, *122*, 503–511. https://doi.org/10.1016/j.watres.2017.06.013

Spagnuolo, S., Perrone, G. C., Berardi, L., Laucelli, D., Simone, A., & Giustolisi, O. (2018). Practical application of WDNetXL system to DMA design of Apulian networks, In *WDSA/CCWI Joint Conference Proceedings 2018* (Vol. 1). Retrieved from https://ojs.library.queensu.ca/index.php/wdsa-ccw/article/view/12273

Thornton, J., & Lambert, A. (2005). Progress in practical prediction of pressure: Leakage, pressure: Burst frequency and pressure: Consumption relationships. In *Proceedings of IWA special conference leakage* (pp. 12–14).

Vrachimis, S. G., Eliades, D. G., & Polycarpou, M. M. (2021). Calculating chlorine concentration bounds in water distribution networks: A backtracking uncertainty-bounding approach. *Water Resources Research*, *57*(5), e2020WR028684. https://doi.org/10.1029/2020WR028684

Walski, T. M., Bezts, W., Posluszny, E. T., Weir, M., & Whitman, B. E. (2006). Modeling leakage reduction through pressure control. *Journal-American Water Works Association*, *98*(4), 147–155. https://doi.org/10.1002/j.1551-8833.2006.tb07642.x

Wang, S., Taha, A. F., & Abokifa, A. A. (2021). How effective is model predictive control in real-time water quality regulation? State-space modeling and scalable control. *Water Resources Research*, *57*(5), e2020WR027771. https://doi.org/10.1029/2020WR027771

WHO (World Health Organization). (2017). *Guidelines for drinking water quality*. World Health Organization.

Wright, R., Abraham, E., Parpas, P., & Stoianov, I. (2015). Control of water distribution networks with dynamic DMA topology using strictly feasible sequential convex programming. *Water Resources Research*, *51*(12), 9925–9941. https://doi.org/10.1002/2015WR017466

Wu, Z. Y., Wang, R. H., Walski, T. M., Yang, S. Y., Bowdler, D., & Baggett, C. C. (2009). Extended global-gradient algorithm for pressure-dependent water distribution analysis. *Journal of Water Resources Planning and Management*, *135* (1), 13–22. https://doi.org/10.1061/(ASCE)0733-9496(2009)135:1(13)