Implementation of pressure reduction valves in a dynamic water distribution numerical model to control the inequality in water supply

Gabriele Freni, Mauro De Marchis and Enrico Napoli

ABSTRACT

The analysis of water distribution networks has to take into account the variability of users' water demand and the variability of network boundary conditions. In complex systems, e.g. those characterized by the presence of local private tanks and intermittent distribution, this variability suggests the use of dynamic models that are able to evaluate the rapid variability of pressures and flows in the network. The dynamic behavior of the network also affects the performance of valves that are used for controlling the network. Pressure Reduction Valves (PRVs) are used for controlling pressure and reducing leakages. Highly variable demands can produce significant fluctuation of the PRV set point, causing related transient phenomena that propagate through the network and may result in water quality problems, unequal distribution of resources among users, and premature wear of the pipe infrastructure. A model was developed in previous studies and an additional module for pressure control was implemented able to analyze PRVs in a fully dynamic numerical framework. The model was demonstrated to be robust and reliable in the implementation of pressure management areas in the network. The model was applied to a district of the Palermo network (Italy). The district was monitored and pressure as well as flow data were available for model calibration.

Key words | dynamic model, intermittent distribution, method of characteristic, pipe-filling process, PRVs, water distribution network modeling

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INTRODUCTION

The distribution of water resources can be made through two different delivery methods: continuous or intermittent distribution. Continuous distribution ensures better management of the water network because the water demand depends only on user requests and the service quality can be better guaranteed. In a water scarcity condition, an intermittent system is used by the management authority for rationing the available water volume, for reducing real losses and/or for controlling consumption (Fontanazza *et al.* 2012).

Due to several detrimental aspects, this approach should be only applied if no other management choices are available. Despite this, it is broadly adopted not only in developing countries (Hardoy *et al.* 2001) but also in developed ones (Cubillo 2005). In a water scarcity condition, this practice reduces the background water losses with little financial efforts (Criminisi *et al.* 2009). Despite this, when the practice of intermittent supply is protracted over time, the effect could be opposite. Due to the water hammer induced by the filling process (De Marchis *et al.* 2010) a deterioration of the pipes occurs, thus increasing the rate of burst and increasing leakages, preventing achievement of one of the main objectives of the intermittent supply. Furthermore, discontinuous distribution presents several critical aspects, such as users' inequality in access to water resources and the presence of filling and emptying transient phenomena affecting the mechanical stability of the pipes, the durability of the network and water losses (Vairavamoorthy *et al.* 2001).

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Impact on water quality can be equally relevant because empty water pipes can be exposed to ingress of soil particles and contaminated water from the surrounding soil through leak openings. This means that the water quality integrity of the system is compromised and that users cannot be guaranteed a safe supply (National Research Council 2006).

Users try to adapt to intermittent distribution by installing local tanks, in order to collect water when the distribution service is available, and use them when the service is suspended (Arregui *et al.* 2006). Tanks are often oversized with respect to the users' real needs and their presence makes the network work in conditions that are quite far from the design ones: flows in the lower parts of the network are much higher than the design until the tanks are full and water resources can reach the tanks in the disadvantaged areas of the network; pressure on the network is generally lower than the design and it is controlled by the levels in the tanks (Giustolisi *et al.* 2012).

This configuration of the system reduces the applicability of common steady state models, because the private tank filling process creates continuous change in the hydraulic network behavior. To follow this constant change in network state variables, dynamic and pressure driven models are needed. Considering this aim, Giustolisi (2010) presented an extension of the pressure-driven analysis using a global gradient algorithm (Todini 2003; Giustolisi *et al.* 2008a, b) permitting the effective introduction of the lumped nodal demand while preserving the energy balance by means of a pipe hydraulic resistance correction. The model allowed the simulation of private tanks but tools for the regulation and control of network pressures could not be modeled.

Pressure control is one of the main technical options that a water manager can put in place to reduce the inequalities among users in such complex cases. Nevertheless, the low pressures and the complex and dynamic hydraulic behavior of the system with private tanks prevent a simple analysis of the effect of pressure control devices such as pressure reduction valves (PRVs) and pumps. Hydraulically controlled PRVs maintain a specified outlet pressure, irrespective of a higher fluctuating inlet pressure, and they are often implemented dividing the network into districts (Pressure Management Areas – PMAs). In intermittent networks, they may control pressures (and indirectly flows) in the advantaged parts of the network reducing the inequalities in water resource access among users.

Pressure transients caused by the combined behavior of a network and PRV propagate through a PMA and result in water supply problems, a higher number of pipe bursts, and premature wear of the pipe infrastructure. Since it is impossible to eliminate demand changes from a network, it is important to control PRVs appropriately to minimize their impact on the system. The interaction between automatic control valves and transients has been investigated in several publications. Bergant *et al.* (2001) investigated the effect of valve closing time on the transient response in a pipeline and compared measured data with a simulation model. The effect of automatic control valves in a real pipe network was shown by Brunone & Morelli (1999), and used to estimate the friction in a transient model. A model for analysis and control of PRVs was implemented by Presscot & Ulanicki (2008), using dynamic formulations and experimental analysis but the model was not integrated with a dynamic network modeling approach.

The analysis of the network during the filling process was carried out with a dynamic model, assuming that the air pressure inside the network is always equal to the atmospheric one and that the water column cannot be fragmented (De Marchis *et al.* 2010). A demand model based on the node pressure-consumption law defining flow draw from the network and filling the tank was previously integrated into the network model (De Marchis *et al.* 2011). In the present paper, a PRV module was integrated in the network model, following the dynamic approach proposed in Presscot & Ulanicki (2008), obtaining a fully dynamic model of the network filling process in the presence of PRVs. The model was calibrated and applied for the implementation of PMAs in one of the distribution networks of Palermo (Italy). The research proposed here starts from the preliminary finding presented by Freni *et al.* (2012).

METHODOLOGY

In this section the numerical model and the case study are presented. The model description is divided in two parts: the discussion of the network hydrodynamic model that was previously presented in De Marchis *et al.* (2010) and

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the detailed description of the PRV valve that was implemented and integrated in the present study.

The network model

In the proposed numerical model the transient in pipes is simulated using fast elasticity-demand pressure waves. In fact, the initial velocity of the water front, inside a previously empty pipe, can be quite large since the pressure gradient is relatively high due to the rapid change in pressure, which can be considered atmospheric at the water front. In water distribution networks, where the pipes are initially empty, different filling cases occur and must be simulated by the numerical models. The proposed numerical model is able to simulate the following cases, shown in Figure 1.

The first empty pipeline is connected to the network reservoirs and the filling of the network starts after the opening of the gates (Figure 1(a)). As the water front reaches one of the users' connections, tanks start to fill, with a discharge that depends on the geometric and hydraulic features of the diversion as well as on the pressure at the derivation point (Figure 1(b)). When the water front reaches the end of a pipeline (Figure 1(c)), water begins to flow inside the pipelines connected to it; the pressure inside the filled pipeline generally continues to increase until a steady-state condition is reached.

Since water distribution networks are generally looped to increase system reliability, both ends of a pipeline start to fill during the filling; as a consequence, two water fronts proceed inside the pipe from opposite directions (Figure 1(d)). Once they reach the same cross-section, the subsequent collision can cause an increase in pressure that is a function of the velocity propagation of the water front. The numerical model, using the method of characteristic, is able to take into account these relatively small water hammers.

Because of the complexity of the system, determined by the various possible filling conditions that may occur, it is necessary to make some simplifying assumptions. Based on the study conducted by Liou & Hunt (1996), it is assumed that the air pressure at the water front is always atmospheric and the wave-fronts are always perpendicular to the pipe axis and coincident with the cross-sections. For detailed discussion of the above hypothesis see De Marchis *et al.* (2010, 2011).

In this paper, the solution of hydraulic equations has been carried out by means of the Method of Characteristics (MOC), starting from the condition of an empty network.

The 1D unsteady flow of the compressible liquid in the elastic pipe is described by the following system of equations:

$$g\frac{\partial h}{\partial s} + V\frac{\partial V}{\partial s} + \frac{\partial V}{\partial t} + gJ + \frac{g}{c}V\sin\vartheta = 0 \qquad (1)$$
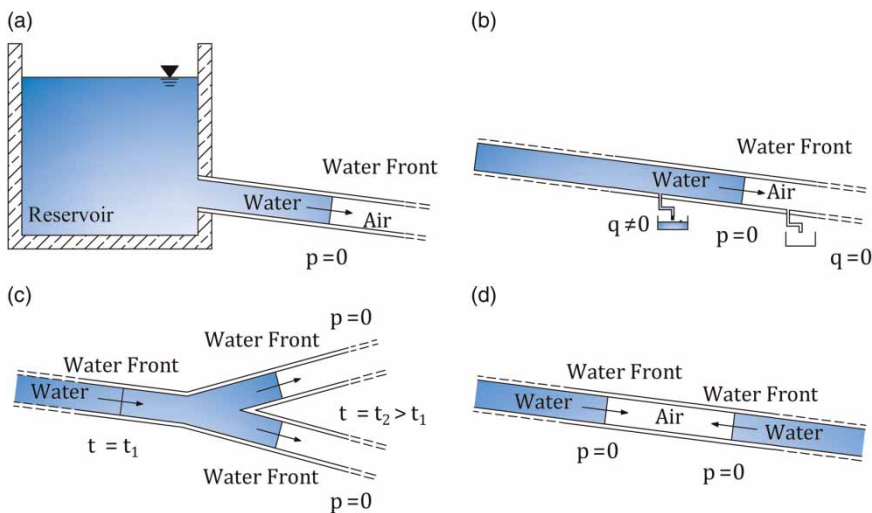
Figure 1 │ Hydraulic schematics of the network filling process: (a) initial phase of water front propagation; (b) water front reaches a user connection; (c) water front reaches the end of the pipeline; (d) two water fronts proceed inside the pipe in opposite directions.

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$$\frac{g}{c}V\frac{\partial h}{\partial s} + c\frac{\partial V}{\partial s} + \frac{g}{c}\frac{\partial h}{\partial t} = 0 \tag{2}$$

where t is the time, V is the velocity averaged over the pipe cross-section, h is the water head, g is the acceleration due to gravity, c is the celerity of pressure waves, ϑ is the slope of the pipeline, while $J = J_s + J_u$, represents the head loss per unit length due to steady and unsteady friction, respectively. The steady friction contribution is calculated according to the classical Darcy–Weisbach equation:

$$J_s = \frac{f}{D}\frac{V|V|}{2g} \tag{3}$$

where f is the Darcy-Weisbach friction factor, calculated dynamically at each time step. On the other hand, J_u, according to the formulation of Brunone *et al.* (1991), later modified by Vítkovský *et al.* (2006), can be calculated according to:

$$J_u = \frac{k}{g}\left(\frac{\partial V}{\partial t} + c\phi_A\frac{\partial V}{\partial s}\right) \tag{4}$$

where k is a coefficient obtained dynamically in the function of the flow regime, as will be shown in the following, while ϕ_A is a coefficient depending on the sign of the convective acceleration. Specifically, $\phi_A = +1$ if $V(\partial V/\partial s) \geq 0$, and -1 if $V(\partial V/\partial s) < 0$. Introducing Equation (4) into Equation (1) and applying the MOC, the momentum and continuity partial differential equations can be transformed into ordinary differential equations, known as compatibility equations:

$$(1+k)\frac{dV}{dt} + \alpha\frac{gdh}{cdt} + gJ_s + \frac{g}{c}\alpha V\sin(\theta) = 0 \tag{5}$$

$$(1+k)\frac{dV}{dt} - \beta\frac{gdh}{cdt} + gJ_s - \frac{g}{c}\beta V\sin(\theta) = 0 \tag{6}$$

where α and β are $(k+2-k\phi_A)/2$ and $(k+2+k\phi_A)/2$, respectively.

The compatibility equations are valid along the proper positive and negative characteristic lines of equation that, introducing the unsteady friction model, read:

$$C^+: \frac{ds}{dt} = +\frac{c}{\alpha} \tag{7}$$

$$C^-: \frac{ds}{dt} = -\frac{c}{\beta} \tag{8}$$

In the proposed numerical model the coefficient k was calculated at each time step toward the Vardy & Brown (2003) formulation, given by:
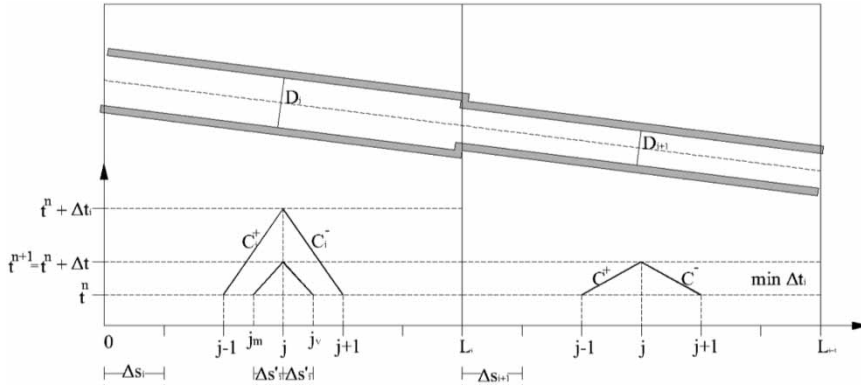
$$k = \frac{\sqrt{c^*}}{g} \tag{9}$$

with

$$c^* = \begin{cases} 0.0476 & Re < 2500 \\ \dfrac{7.41}{Re^{\log\left(\frac{14.3}{Re^{0.05}}\right)}} & Re > 2500 \end{cases} \tag{10}$$

In order to study the transient flow in the water distribution network, the MOC are combined with the proper boundary conditions. A constant water head is imposed to all the reservoirs feeding the network, thus water levels remain constant during the filling process. Coherently with the assumption of atmospheric air pressure in the pipelines network, the water head at the front face of partially filled pipes is equal to zero.

Equations (5) and (6) can be solved through the finite difference technique. Following the notation used in Figure 2, these equations read:

$$h_j^{i,n+1} - h_{jm}^{i,n} + \frac{1+kc}{\alpha}\frac{c}{g}\left(V_j^{i,n+1} - V_{jm}^{i,n}\right)$$
$$+ \left[\frac{c}{\alpha}J_{jm}^{i,n} + V_{jm}^{i,n}\mathrm{sen}\theta^i\right]\Delta t_i = 0 \tag{11}$$

$$h_j^{i,n+1} - h_{jv}^{i,n} - \frac{1+kc}{\beta}\frac{c}{g}\left(V_j^{i,n+1} - V_{jv}^{i,n}\right)$$
$$- \left[\frac{c}{\beta}J_{jv}^{i,n} - V_{jv}^{i,n}\mathrm{sen}\theta^i\right]\Delta t_i = 0 \tag{12}$$

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Figure 2 | Space-time scheme for two-pipes of different diameter with the same number of sections N and different time step Δt_i.

where $V_j^{i,n+1}$ and $h_j^{i,n+1}$ are the velocity and the water head in the j-th section (of abscissa $(j{-}1)L_i/N_i$) of the i-th pipe at the time step $t^n + \Delta t$; θ^i is the slope of the i-th pipe; and j_m and j_v are the sections upstream and downstream of the j-th section, respectively.

The time step advancement Δt_i^n, function of the length and of the celerity of the i-th pipe, is calculated for each pipe and then the minimum value is chosen as the unique time step integration:

$$\Delta t^n = \min_i \Delta t_i^n = \min_i (L_i^n/(N_i \, c_i)) \tag{13}$$

When the velocity of the water front $V_j^{i,n+1}$ is calculated, the filling process is updated according to:

$$L_i^{n+1} = L_i^n + V_N^{i,n+1} \cdot \Delta t \tag{14}$$

where L_i^{n+1} is the length of the water column inside the partially empty i-th pipeline at the time t^{n+1}.

The compatibility equations for the pipelines connected to the node are resolved together with the continuity equation at each junction node, and the discharge provided to user tanks is calculated as a function of the water head. Specifically, the discharge $Q_{j,\mathrm{up}}$ at the j-th node entering the tank connected to the node can be obtained as:

$$Q_{j,\mathrm{up}} = C_v \cdot a \cdot \sqrt{2g(h_j^i - h_{j,\mathrm{tank}})} \tag{15}$$

where C_v is the non-dimensional float valve emitter coefficient, a is the valve effective discharge area, g is the

gravity acceleration, h_j^i is the water head at the j-th node and $h_{j,\mathrm{tank}}^i$ is the height of the private tank.

Although more complex methods were considered in the past to relate coefficients C_v and a to valve-opening rates, here constant values were used for both of the coefficients (Criminisi *et al.* 2009) which have been calibrated experimentally, as discussed in the following paragraphs. Equation (15) can be used to calculate the discharge at nodes only when the floating valve is open, i.e., while the user tank is not entirely filled. Thus, this equation must be combined with the tank continuity equation, which can be written as:

$$\begin{cases} Q_{j,\mathrm{up}} - D_j = \dfrac{dW_j}{dt} = A\,\dfrac{dH_j}{dt} & \text{for} \quad H_j < H_{j,\max} \\ Q_{j,\mathrm{up}} = 0 & \text{for} \quad H_j \geq H_{j,\max} \end{cases} \tag{16}$$

where D_j is the user water demand at the j-th node, W_j is the volume of the storage tank connected to the node having area A, H_j is the tank water level, and $H_{j,\max}$ is the maximum allowed water level in the tank (before the floating valve closes).

Further details on the numerical model can be found in De Marchis *et al.* (2010, 2011).

The valve model

The dynamic analysis of PRVs follows the formulation and experimental analysis provided by Presscot & Ulanicki (2003, 2008), in which the derivative of the opening of the valve χ_m is proportional to the difference between a given

set point h_{set}, and the current outlet pressure h_{out} (Figure 3). The PRVs are located at some nodes of the network. Assuming an initial value of $\chi_m = \chi_{m0}$, the valve capacity Cv is calculated using the following equation:

$$Cv = 0.021 - 0.0296e^{-51.1x_{m0}} + 0.0109e^{-261x_{m0}}$$
$$- 0.0032e^{-683.2x_{m0}} + 0.0009e^{-399.5x_{m0}} \tag{17}$$

So, knowing the incoming flow (q_m) and the inlet head (h_{in}) of PRV, the PRV outlet head (h_{out}) can be determined with the equation:

$$h_{out} = \frac{h_{in} - q_m^2}{[Cv(x_{m0})]^2} \tag{18}$$

Equation (18) relates the flow through the PRV to the head loss across it and the opening. Valve opening and closing are controlled by the pilot circuit allowing a flow q_3 to fill the valve control space.

The inflow to the control space of the PRV is dependent on the difference between the outlet head and the valve setting. The valve is characterized by two parameters α_{open} and α_{close}, fixed to 1.1×10^{-6} and $10 \times 10^{-6}\,\mathrm{m^2/s}$ respectively in the present study following the experimental results of Presscot & Ulanicki (2008). The two parameters determine the opening and closing celerity of the valve and, at the same time, its sensitivity and reactivity to pressure fluctuation.

The inflow q_3 to the control space is calculated as follows:

$$q_3 = \begin{cases} \alpha_{open}(h_{set} - h_{out}) & \text{if } \dot{x}_m \geq 0 \\ \alpha_{close}(h_{set} - h_{out}) & \text{if } \dot{x}_m < 0 \end{cases} \tag{19}$$

where

$$\dot{x}_m = \frac{dx}{dt} \tag{20}$$

Another function of valve opening (χ_m) is the cross-sectional area A_{cs} of the control space, that is determined using the equation:

$$A_{cs} = \frac{1}{3700(0.02732 - x_{m0})} \tag{21}$$

Calculating q_3 with Equations (19), a new value χ_m can be estimated representing the adaptation of valve opening to seek the required set point h_{set}:

$$x_m = \frac{q_3}{A_{cs}(x_{m0})} \Delta t + x_{m0} \tag{22}$$

The system of Equations (19)–(22) requires an iterative resolution because inflow q_3 depends on h_{out} by means of Equation (19) that is dependent on valve opening condition x_m again dependent on q_3 according to Equation (22).
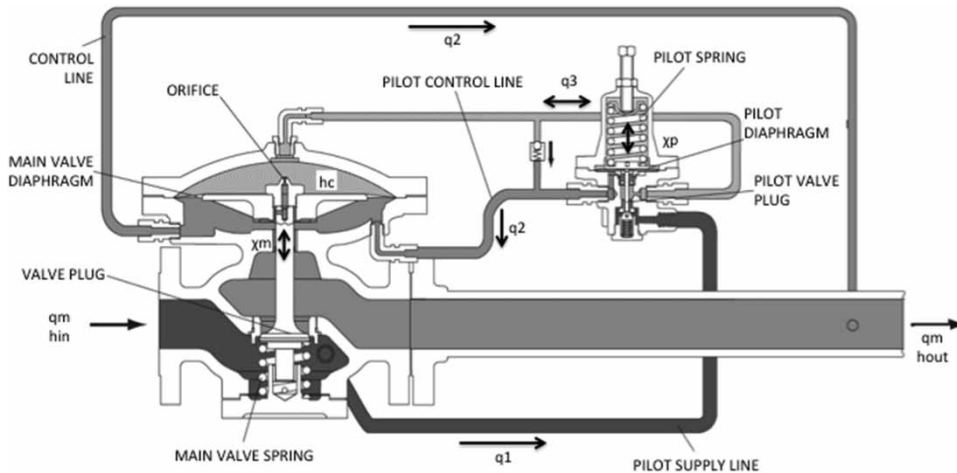

Figure 3 | The PRV scheme and governing flows.

The system has to be solved making an initial hypothesis on valve opening χ_{m0} and then solving the equations in order until a new value of χ_m is obtained in Equation (22). The iterations are continued until the difference between $q_{3,i}$ (with i being the i-th iteration) and $q_{3,i-1}$ is less of an established tolerance that was assumed equal to 0.1% in the present study.

The case study

The model has been applied on one of the 17 distribution networks of Palermo city (Sicily). The network is fed by two tanks at different levels, that can store up about $40,000 \, \text{m}^3$ per day, and supply around 35,000 inhabitants (8,700 users). It has been designed to deliver about 400 l/capita/d but the actual mean consumption is about 260 l/capita/d. Pipes are made of polyethylene and their diameters range from 110 to 225 mm (Figure 4). Additional details on the analyzed network can be found in De Marchis *et al.* (2010).

The system is supplied on a daily basis because the high level of leakages (around 25%) does not allow the manager to supply the network continuously with the available water resources. Considering that intermittent supply was historically a common practice, especially during summer, all the users are supplied via tanks having volume equivalent to two days' consumption. In the present paper, the aim was the evaluation of the impact of intermittent supply on

water resources distribution among users. For this reason, leakages were simply proportionally divided according to the node demand because exact positions of leakages were not known.

The system is monitored by six pressure cells and two electromagnetic flow meters (Figure 4). Data have been provided on an hourly basis almost continuously since 2001, and the network hydraulic model calibration is continuously updated when new data become available (Criminisi *et al.* 2009). The pressure data used for model calibration have a time resolution of five minutes and were taken from the period between June and October 2002, during which the network was managed by intermittent supply on a daily basis. The pressure time series were available at each of the six pressure gauges and used to represent the filling and the emptying processes. For the same period, flow data entering the network were available with the same temporal resolution.

The current configuration of the network is characterized by significant inequality in the distribution of water resources during intermittent supply. As demonstrated in De Marchis *et al.* (2010), the users in the lower part of the network, characterized by the lowest geodetic elevation, can access water resources soon after the beginning of a service period and they are able to fill up their tanks that were emptied in the period of service unavailability. At the same time, the users in the upper part of the network have to wait for the advantaged users to collect water resources and pressure over the network to rise in order to begin filling their tanks. As discussed in the introduction, the definition of PMA can help in the reduction of inequalities among advantaged and disadvantaged users. In the present study, two configurations were considered dividing the district into two and four PMAs. In Scenario A, the network was divided into two approximately equal parts (Figure 5(a)). In Scenario B, the network was divided into four PMAs increasing the number of valves introduced in the system and the number of closed pipes (Figure 5(b)). The two configurations were chosen based on the original design of the network in which the district is divided into four areas that can be insulated for maintenance purposes. In the present application, some of the existing static section valves are simply substituted by PRVs.
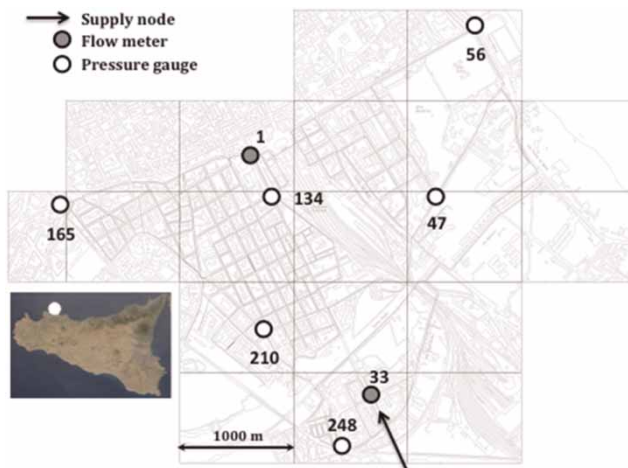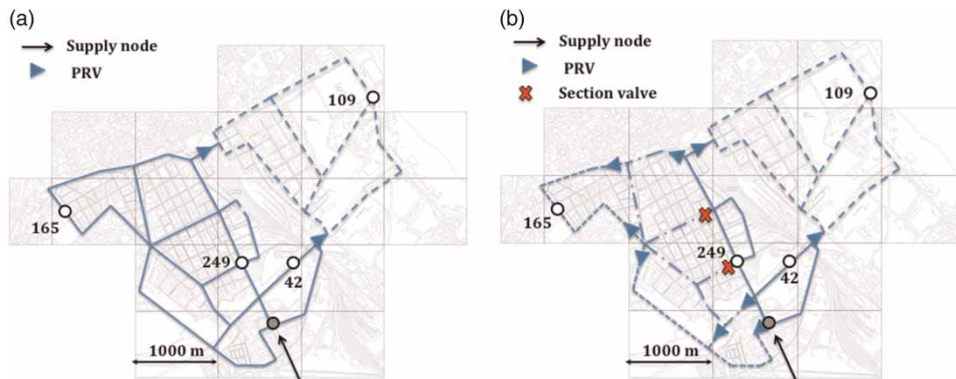

Figure 4 │ Case study network scheme.

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Figure 5 | Position of the PRVs and closed pipes on network mains: Scenario A (a) and Scenario B (b). The blue lines define the boundaries of the PMAs.

ANALYSIS OF RESULTS

The model was initially calibrated according to the pressure profiles available during the monitoring period from the six pressure gauges located in the network. The results, not shown here, can be found in De Marchis *et al.* (2011) where a section dedicated to the model calibration, in the same case of study, is reported. The presented model was used to analyze and compare different configurations of PMAs in order to reduce inequalities between user accessing and collecting water resources considering the relevant role of private tanks.
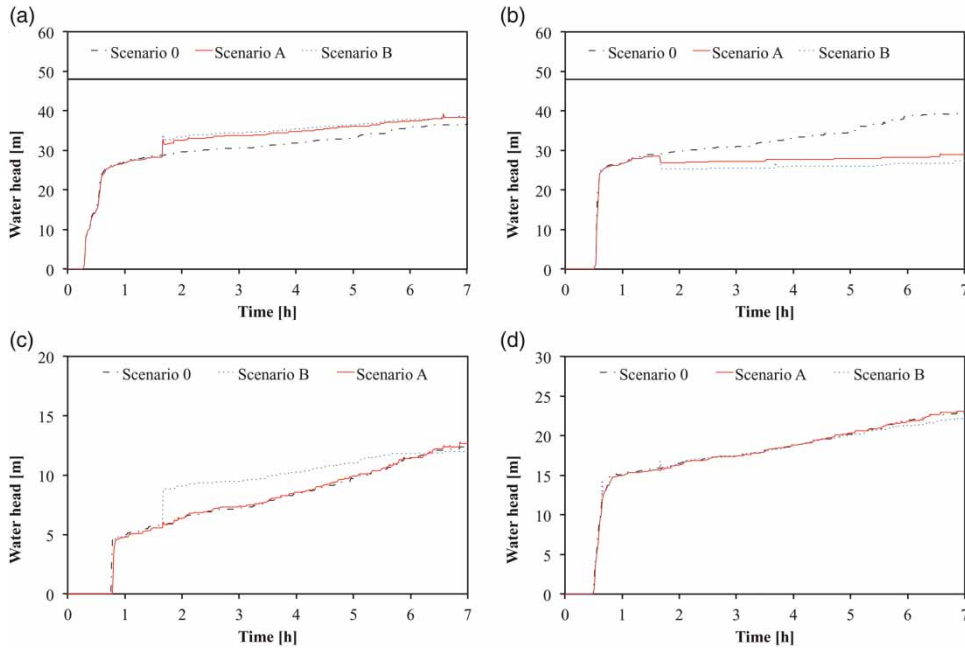
In the analysis of results, Scenario 0 (i.e. the current situation with one network district with no pressure control) was compared with the two proposed PMA scenarios. The effectiveness of district definition was evaluated by means of pressure levels in the network and by means of the water volume supplied to the users at different moments of the service day. In all the scenarios, the simulation starts with the reactivation of service during intermittent distribution on a daily basis. Because of user water consumption during the day before, at the beginning of the simulation, all the private tanks are almost empty and their supply valves are fully open.

Figure 6 shows the comparison between the water head variation in time obtained in the three different scenarios analyzed here. The first 7 h of the dynamic filling process are analyzed in four different nodes of the network. Specifically, the pressure in the nodes 42, 109, 165 and 249 were plotted. These nodes were chosen to be representative of

the effects of the PRVs in the different PMAs, as can be observed in Figure 5 where the nodes were shown to improve the clarity. Figure 6 shows pressure levels in four nodes of the network: initially the pressure is null and the pipes are empty. The time taken for the filling process is different for each of the four nodes monitored. In the disadvantaged node (Figure 6(c)) the transient period of the filling process can be protracted for almost 1 h from the beginning of the simulation. For details about these inequities see De Marchis *et al.* (2010). The static level of the supply tank is equal to 48 m above medium sea level.

Figure 6(a) shows that, when the PRVs are activated (Scenario A and B), in the upper part of the network an increase of the pressure is achieved, with respect to Scenario 0. Furthermore, the increase of the pressure is higher in Scenario B, where several PRVs were activated to reduce the inequality between the users. On the other hand, Figure 6 (b) shows the reduction of the pressure at node 109 located in the lower part of the network. Also at this node, in Scenario B the water head reduction is higher than that registered when two PRVs were activated (Scenario A). Due to the fact that a pressure driven model is used to calculate the discharge entering the users' tanks, the increase of the pressure in the disadvantaged nodes and the reduction in the advantaged ones reduces the inequalities in the water supply. Figure 6(c) shows that in the most disadvantaged nodes, located either in the highest part of the district or in the farther part of the network from the inlet node, in order to reduce the inequalities it is necessary to divide the district into four PMAs. The water heads obtained

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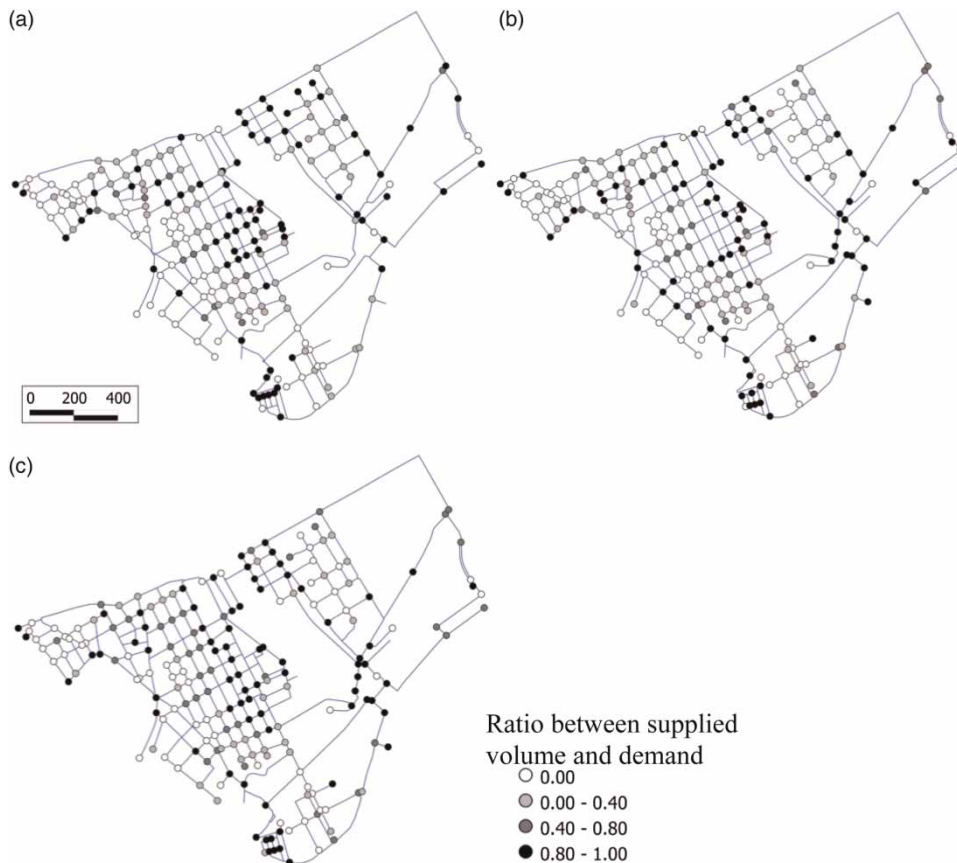

Figure 6 | Pressure level variation in time in four nodes: (a) 42; (b) 109; (c) 165; (d) 249. In figure (a) and (b) the horizontal line represents the static level of water supply equal to 48 m.

Water head [m]
○ 10.00 - 17.00
◑ 17.00 - 24.00
◕ 24.00 - 31.00
● 31.00 - 38.00
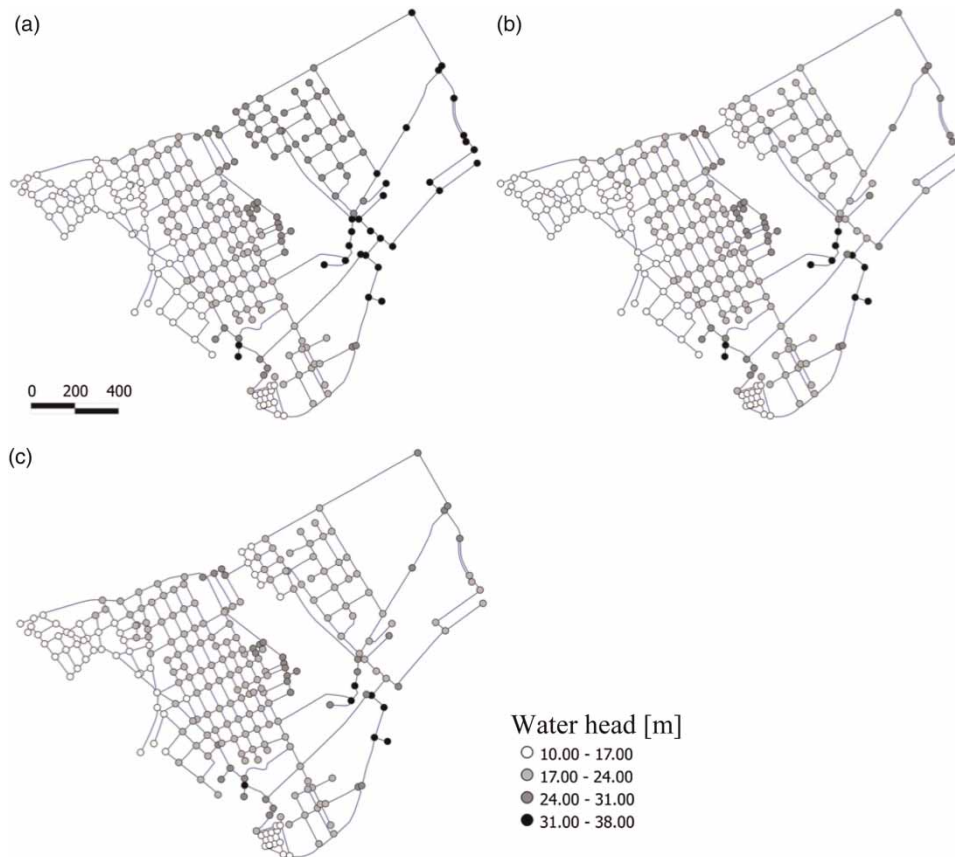
Figure 7 | Pressure levels in the network after 3 hours: (a) Scenario 0; (b) Scenario A; (c) Scenario B.

Figure 8 │ Volume supplied to the users after 5 hours: (a) Scenario 0; (b) Scenario A; (c) Scenario B.

in Scenario A, in fact, are equal to those in Scenario 0. Finally, Figure 6(d) shows that in the nodes located near to the inlet node the three scenario profiles are very similar, with negligible differences in water head distribution.

Figure 7 shows pressure levels in the network after 3 h in the three selected scenarios. The separation between the different PMAs is clear and the average pressure in the network progressively decreases by implementing two and four different districts: in Scenario 0, the average pressure head is 22.1 m, decreasing to 21.4 m in Scenario A and 20.7 m in Scenario B. More interestingly, the standard deviation drops from 6.5 m in Scenario 0 to 5.4 m and 4.6 m, respectively in Scenario A and Scenario B. This fact confirms a more uniform distribution of pressures over the network and, considering that the majority of the uses are head driven because of private tanks, a more uniform distribution of resources.

This consideration is confirmed by looking at water head after 3 h (Figure 7) and at supplied water volumes after 5 h (Figure 8). The percentage of users able to collect the totality of their daily demand after 3h drops from 14% to 10% and 4% in Scenarios A and B. After 5 h in Scenario 0, one quarter of the users' tanks were filled while only 20% and 13% have completed their supply in the two PMA scenarios. The implementation of PMAs has a more relevant impact on users unable to be supplied: after three hours, 45% of users are unable to be supplied in Scenario 0 and this number is reduced to 38% and 29% in Scenario A and B, respectively; after five hours, the number of non-supplied users is still high (39%) while it is reduced to 29% and 18% in the two PMA scenarios.

CONCLUSIONS

In the study, a dynamic mathematical model for intermittent networks was integrated with a PRV model in order to

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simulate management actions for reducing inequalities between users in their access to water resources. The model was demonstrated to be robust and to correctly represent the application of several valves in the network showing the impact of such choices on network pressure and on water supply distribution. From a practical perspective, the creation of PMAs has a relevant impact on intermittent networks helping the reduction of inequalities between users accessing and collecting water resources. The presence of private tanks helps advantaged users to collect as much water as possible in a few hours after the restoration of service; at the same time, several users are unable to collect water because pressure in the network is too low. The introduction of PMAs mitigates this problem by reducing the differences of pressure between different points of the network. The introduction of the valves reduces the differences between water collected by users in the first part of the service day even if inequalities still remain. The analysis demonstrated that PMAs can help move towards having equal distribution of water resources during intermittent service but further analyses are needed to implement an optimal distribution of valves in order to reduce the different distribution of water supply between users. The impact of valves on the network is not easily predictable without the use of dynamic models as the presented analysis has demonstrated. Some parts of the network are unaffected by the presence of the valves because they are dominated by the proximity of network inlets. The introduction of valves has a pervasive impact on the network, cutting pressure downstream of the valve but also increasing pressures in the upper part of the network due to the compensation of flow distribution in the network.

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