Modeling of distribution network filling process during intermittent supply

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ABSTRACT: The paper presents the modeling results of the filling process of a water distribution network subjected to intermittent supply. The local tanks built by users for reducing their vulnerability to intermittent supply increase user water demand at the beginning of the service period and the time required for completely fill the network. Such a delicate process is responsible of the inequalities taking part among users. Users located in advantaged positions can receive water resources soon after the beginning of the service period while disadvantaged users have to wait until the network is full. Such an highly dynamic process requires ad-hoc models to be developed in order to obtain reliable results. The paper reports a numerical model able to evaluate this complex process and its application to a real Italian case study. The model provided good agreement with calibration data and allowed for an interesting insight in network filling process.

1 INTRODUCTION

Intermittent water distribution is nowadays a widespread solution to cope with water shortage caused by drought periods or lack of network maintenance inducing high leakages. Intermittent supply reduces background losses and it leads users to believe saving water but it does not actually reduce user water consumptions.

This approach, widely adopted not only in developing countries (McIntosh 1993, McIntosh & Yñiguez 1997, Hardoy et al. 2001, Vairavamoorthy et al. 2001) but also in developed ones (Cubillo Gonzales & Ibanez Carranza 2003), has the advantage of requiring small financial efforts. However, it leads to network operating conditions that are very far from the usual design ones.

The water utility try to distribute as well as possible the limited water resources dividing the entire network in different zones (defined on uses numbers) and supplying each zone with a rate of the available volume for a fixed period of time shorter than 24 hours. So, each zone is subjected to cyclical filling and emptying process and users need to collect as much water as possible during the service period for covering their needs when supply service is not available. The users try to cope with water service intermittency by building private storage tanks that are often over-designed in order to take into account higher water consumption and possible leakages. So, the node water demand does not depend on the actual user consumption but on node water head.

Intermittency has as consequence a very remarkable management problem: the water utility can not guarantee to users the same accesses to the available water resource. The inequitable water distribution among users is caused by two aspects. Hydraulic conditions of an intermittent network is based on two transitional phases: network filling up and emptying process. In the first phase, the water volumes entering the network do not reach all the users at the same time, with obvious disadvantages for users located faraway from the supplying nodes. This process can last for a long time for networks where the pipe length is a few tens or hundreds kilometres. Furthermore, in distribution systems designed for continuous water supply, the users exposed to intermittency are likely to collect as much water as possible in their tanks whenever the service resumes (Totsuka et al. 2004). In this condition, the tanks are filled up once the supply has been restored and this contemporary use of water service generates larger peak flows than predicted in the network design process, increasing the pressure losses. Consequently, disadvantaged users located faraway from the supplying nodes or at higher elevation in the network will always collect less water than those nearer to the source. Intermittent distribution can also have a large impact on water quality allowing for the introduction of soil in the pipes when they are empty (Yepes et al. 2001).

Generally adopted network models are not able to analyze the complexity of the network filling process. An adequate assessment of the node water demand considering the private storage tank has to be done to better understand and model this process as well.

In the present paper, a model is reported able to analyze the network during the initial filling process making the hypotheses that air pressure inside the network is always equal to the atmospheric one and that the water column can not be fragmented. A demand model based on the node pressure-consumption law defining flow drawn from the network and filling the tank has been integrated to the network model. The proposed methodology has been applied to a part of Palermo distribution network (Italy) that has been monitored during both continuous and intermittent distribution thus allowing for having experimental evidences of unequal water supply among the users.

2 FILLING PROCESS NUMERICAL MODELING

The filling process of a water distribution network has been analyzed by the mean of the Method of Characteristics (MOC), starting from the condition of empty network to the steady-state pressure one. The numerical method is based on some simplified assumptions (Liu & Hunt 1996): the air pressure at the water front face is always equal to the atmospheric one (this hypothesis is ensured by the presence of air inlet/release valve and air vents), the water front is always coincident with the pipe cross-sections, the resistance can be calculated with the classical relationship valid for steady state flows.

The MOC approach transforming the momentum partial differential equations into ordinary differential equations coupled with the hypothesis discussed above allows to solve the filling transient by the mean of the compatibility equations:

$$\frac{dV}{dt} + \frac{gdh}{cdt} + gJ + Vsen\theta = 0 \tag{1}$$

$$\frac{dV}{dt} - \frac{gdh}{cdt} + gJ - Vsen\theta = 0 \tag{2}$$

where 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 velocity of the pressure wave or celerity; J is the head loss according to Darcy-Weisbach equation; t is the time; and θ is the slope of the pipe. The Equations 1 and 2 are respectively valid along the positive and negative characteristic lines defined as:

$$C^+ : \frac{ds}{dt} = +c \tag{3}$$

$$C^- : \frac{ds}{dt} = -c \tag{4}$$

where s is the linear abscissa along the pipe.

Integrating the compatibility equations along the characteristic lines applying the finite difference method, Equations 1 and 2 can be rewritten in the following algebric form:

$$V_j^{i,n+1} - V_{jm}^{i,n} + \frac{g}{c}\left(h_j^{i,n+1} - h_{jm}^{i,n}\right) + \left[gJ_{jm}^{i,n} + V_{jm}^{i,n} sen\theta^i\right]\Delta t_i = 0 \tag{5}$$

$$V_j^{i,n+1} - V_{jv}^{i,n} - \frac{g}{c}\left(h_j^{i,n+1} - h_{jv}^{i,n}\right) + \left[gJ_{jv}^{i,n} - V_{jv}^{i,n} sen\theta^i\right]\Delta t_i = 0 \tag{6}$$

where $V_j^{j,n+1}$ 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 $t^n + \Delta t$ respectively; θ is the slope of the *i-th* pipe; *jm* and *jv* are the upstream and downstream sections of the section *j* at the previous time step t^n, given by the positive and negative characteristic lines respectively, as Figure 1 shows.

The integration time step Δt_i is equal to:

$$\Delta t_i = L_i^n / (c_i \cdot N_i) \tag{7}$$

where L_i^n is the length of the *i-th* pipe filled at the time t^n; c_i is the pipe celerity; and N_i is the number of segment in which L_i^n is divided (N_i is fixed equal to 10). At each time advancement, the integration time step Δt is calculated in every filled or partially filled pipe and then the minimum value is chosen as the only value for the equation system.

In order to study the transient flow in 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.

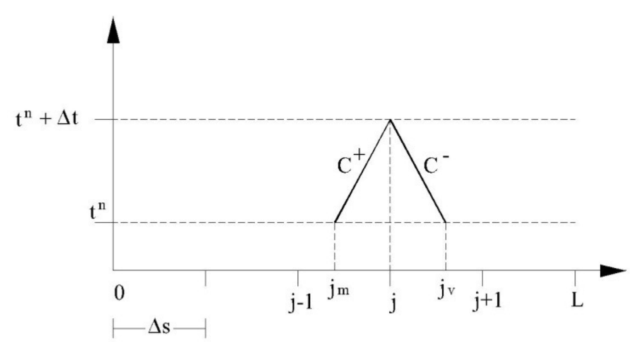

Figure 1. Sketch of the interpolation of the hydrodynamic variable (V and h) varying the time step Δt

The compatibility equations are solved jointly with the continuity equation and taking into account the float valve emitter law for each network node.

The solution of the compatibility equations (Eqs 5-6) and along the characteristic lines (Eqs 3-4) allow to assess the velocity V and the water head h at each time step and in those points in which has been discretized the public network. The water head are evaluated by the mean of a node demand discussed afterwards, then the node water drawn from the distribution network to the private storage tank is updated. The filling process is updated as well at each time step as follows:

$$L_i^{n+1} = L_i^n + V_N^{i,n+1} \cdot \Delta t \tag{8}$$

where $V_N^{i,n+1}$ is the velocity calculated at the water front face; and L_i^{n+1} the length of the *i-th* pipe partially filled. When the length of the water column is equal to the pipeline one, the update stops while the connected pipes start to fill according to equation (8). The filling process continues until all pipelines are completely filled.

The proposed analysis of the filling network process based on the MOC is particularly complex since the system of equations has to take into account the different conditions of each single pipeline of the network. Four different cases can occur: empty pipe, pipe partially filled from one side, pipe partially filled from both sides an full pipe. The numerical model proposed is thus a powerful tool for studying the intermittent distribution of public network since it can be used for an empty, partially filled or completely filled pipeline system and the solution is iterated until the steady state is obtained.

The network filling process is controlled by the presence of local tanks inside buildings. Such presence is taken into account in the model by the definition of a specific head – discharge law at network nodes. Such node law is based on the combination of the tank continuity equation (Eq. 9) and the float valve emitter law (Eq. 10). Main equations may be summarised as in the following:

$$Q_{up} - D = \frac{dV}{dt} = A \cdot \frac{dh}{dt} \tag{9}$$

$$Q_{up} = C_v \cdot a \cdot \sqrt{2g(H - H_r)} \tag{10}$$

where D and Q_{up} are respectively the user water demand and the discharge from the distribution network to the local tank; V is the volume of the private tank having area A and variable water depth h; C_v is the float valve emitter coefficient, a is the valve effective discharge area, H is the hydraulic head over the distribution network, H_r is the height of the floating valve supplying the tank and g is the gravity acceleration. Float valve emitter coefficient C_v and the effective discharge area a depend on the floater position and thus on water level in the tank according to the following empirical laws:

$$C_v = \begin{cases} if\ h < h_{min} \Rightarrow C_v = C_v^* \\ if\ h > h_{min} \Rightarrow C_v = C_v^* \cdot \left(\frac{h_{max}-h}{h_{max}-h_{min}}\right)^n \end{cases} \tag{11}$$

$$a = \begin{cases} if\ h < h_{min} \Rightarrow a = a^* \\ if\ h > h_{min} \Rightarrow a = a^* \cdot \left(\frac{h_{max}-h}{h_{max}-h_{min}}\right)^m \end{cases} \tag{12}$$

where $h_{\min}$ and $h_{\max}$ are respectively the water depths at which the valve is fully open and fully closed, C_v^* and a^* are the emitter coefficient and the effective discharge area of the fully open valve and m and n are shape coefficients usually ranging between 0.5 and 2 to be experimentally estimated.

The node head - discharge model parameters have been set to the average obtained in a field campaign that has been carried out in the same network in 2007. Several users with local tanks have been monitored in order to investigate apparent losses due to water meter under-registration. In that occasion, data have been collected to calibrate the add-on model. Details are provided in Criminisi et al. (2009) and Fontanazza et al. (2009). C_v^* has been set equal to 0.57, a^* has been set equal to 2.8 cm^2, m and n have been set to 0.85 and 0.78 respectively. Pump output has been set to 0.72 in the present study.

3 THE CASE STUDY

The model has been applied to one of 17 supply networks of the city of Palermo (Italy). Figure 2 shows the scheme of the network adopted in the study.

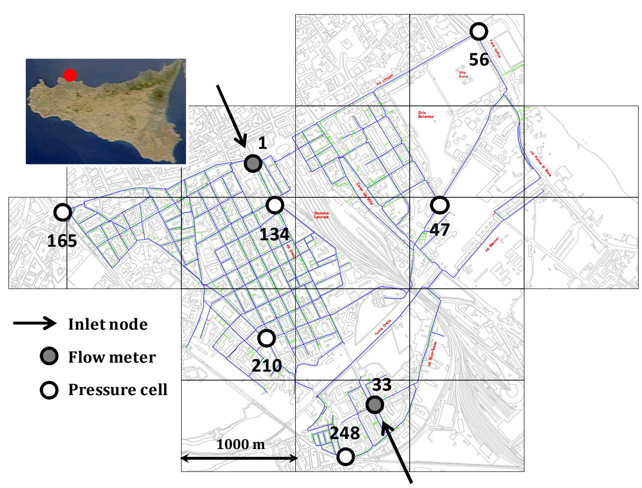

Figure 2. Case study network scheme

This network has been chosen because it has been recently rebuilt and all its geometric characteristics are precisely known as well as the number and the distribution of user connections, the water volumes delivered and measured, the pressure and flow values in a few important nodes. The renovation process has regarded the sole distribution network keeping in service the old cast iron feeding pipes.

The network is fed up by two tanks at different levels, that can store up to 40,000 m^3 per day, and supply nearly 35,000 inhabitants (8700 users). The network is about 40 km long and the pipes are made of polyethylene, with diameters ranging between 110 and 225 mm.

The system is monitored by 6 pressure cells distributed over the network and by 2 electromagnetic flow meters put in the network inlet nodes (Fig. 2). Data is provided almost continuously on hourly basis since 2001 and the network hydraulic model calibration is constantly updated once new data are provided (Fontanazza et al. 2007, Fontanazza et al. 2008).

For the present study, some pressure data were available at a resolution of 5 minutes for a period between June 2002 and October 2002 in which the network was managed by intermittent supply on daily basis. 78 pressure timeseries were available at each of the 6 pressure gauges representing the filling process (one every two days). In the same period, flow data entering the network were available with the same temporal resolution.

Network node (user connection) elevation ranges between 3 and 47 m above the sea level while building height between 3 and 50 m.

The network has been designed to deliver about 400 l/capita/d but the actual mean consumption is about 260 l/capita/d. As consequence, in ordinary conditions, the network is characterized by low water velocities and correspondently high pressures causing in the past high leakages. This condition together with the recurrent lack of water resources has not permitted to maintain continuous distribution in the last 5 years (at least in summer period) and intermittent distribution on daily basis has been introduced as a common practice convincing the users to build up local private tanks. Furthermore, because of the great amount of water losses occurring in the pipe connecting the tanks with the network, the water utility has decided to reduce the pressure level on the network thus inducing the users to maintain the storage tanks in order to prevent temporary interruptions of water supply and to adopt local pumping system to feed the tank even if network pressure is not sufficient. Typically, private tanks have specific volumes equal to 200 – 250 l/capita and this assumption has been adopted in the present application.

For the following model application, for sake of simplification, the average pressure profile and the average network inflow patterns have been computed between the 78 available daily time series.

4 MODEL APPLICATION AND RESULTS DISCUSSION

The model has been calibrated according to the pressure profiles available in the 6 pressure gauges present in the network. Figure 3 shows the good agreement between the simulated and measured pressure time series.

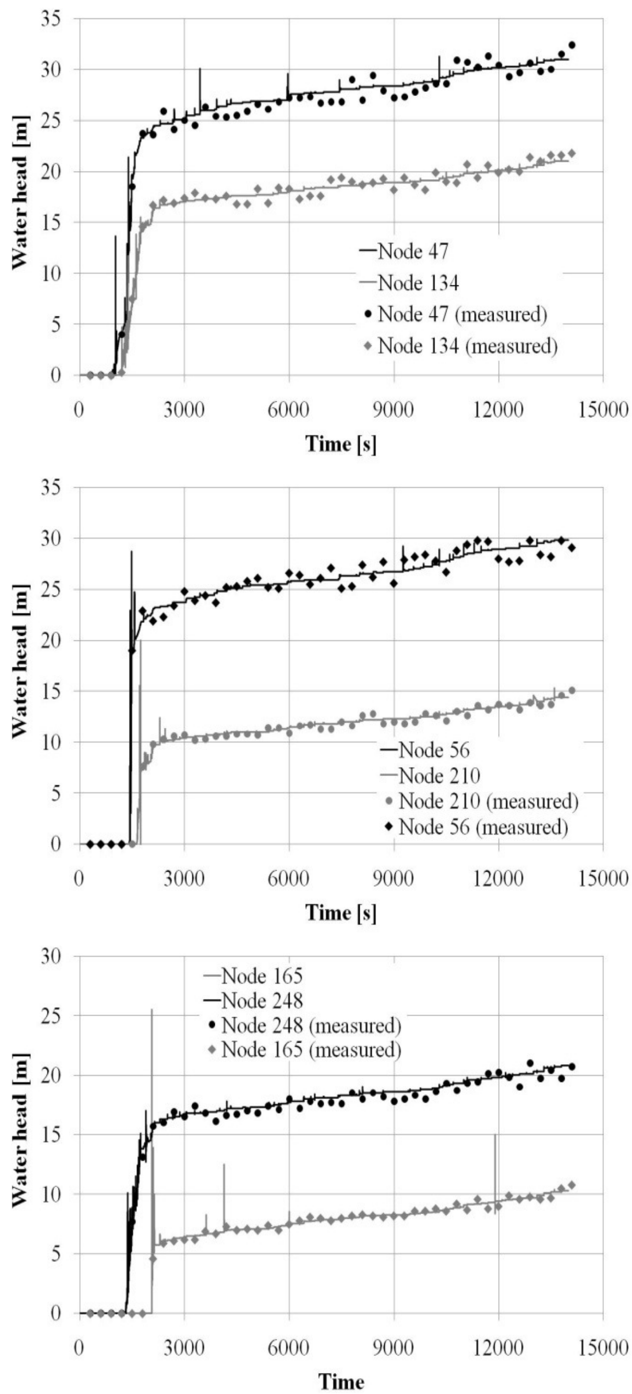

Figure 3. Comparison between simulated and measured pressures in the pressure gauge nodes

Some of the monitoring nodes start receiving water approximately 10 minutes after network public reservoirs start supplying the network (nodes 47 and 134). Disadvantaged nodes start receiving water after more than 30 minutes. Unfortunately, due to the low pressure (less than 10 meters), users do not receive water supply for a much longer time (node 165). As it can be seen in Figure 4, where the discharges from the nodes 47 and 165 to the local tank is reported, many users (node 47) start to receive water supply within the first hour of service, while others (node 165) have to wait up to 7 hours to start receiving water resources from the network. Moreover the disadvantaged users receive a very lower discharge than the users located in the advantage position.

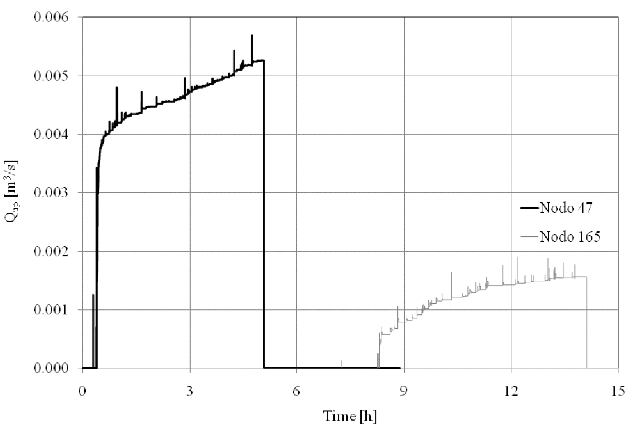

Figure 4. Discharge from the local network to the local tank in nodes 47 and 165.

The disadvantaged condition of several users is well showed by Figure 5 where the time needed to begin the filling up of the local tanks of the highest users at each node is reported. Finally, in Figure 6 the time needed for the highest users at each node to complete the filling up of the local tanks is showed as well.

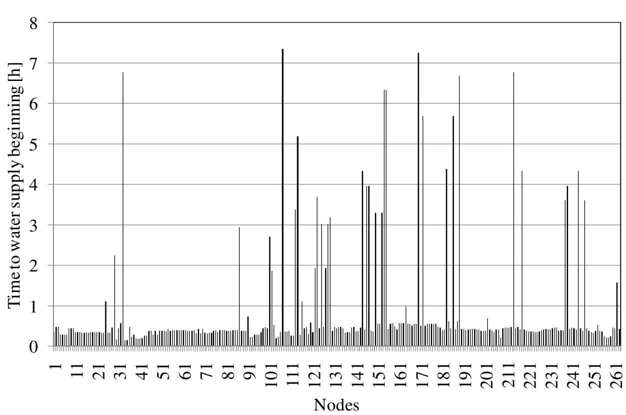

Figure 5. Time to water supply beginning in the network nodes

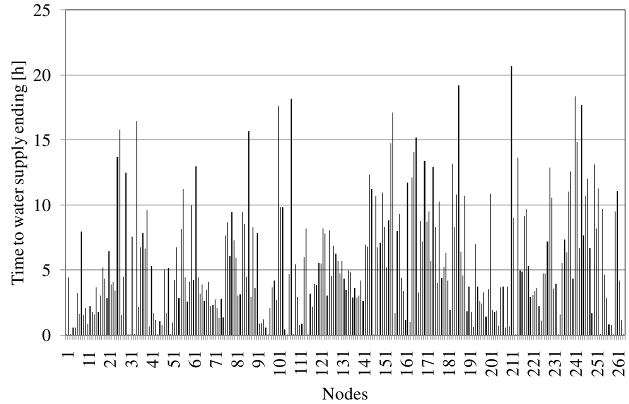

Figure 6. Time to water supply ending in the network nodes

A few users complete to fill up the tanks after 15 hours, confirming the high vulnerability to intermittent distribution. Lower distribution period may lead to an insufficient water supply for the users connected with the disadvantage nodes. This result may be an useful indicator to evaluate the most efficient strategies to manage the network in order to reduce the high inequalities due to the intermittency of the water supply service.

Figure 7 shows the dynamics of pipe filling that clearly point out the location of the disadvantaged users in the left corner of the network (Figure 7-b).

Network mains start filling quite immediately after the water supply has started while the upper part of the network is still empty after 30 minutes. The service has started 45 minutes after, the network is entirely full but the pressure levels over several users are still inadequate for service.

This result confirms that intermittent distribution generates higher competition among users and consequent inequalities without providing relevant advantages in terms of water demand reduction. In fact, water demand in the whole monitored period remained sensibly constant and ranging between 190 and 230 l/capita/day. The same period (June - October) of the following four years, in which water distribution was continuous, reported an average water demand equal to 221 l/capita/day.

Disadvantaged users usually install boost up pumps in order to compete with advantaged ones. Such behaviour has not been considered in the present simulations but it does not seem to have a great impact on the analysed network.

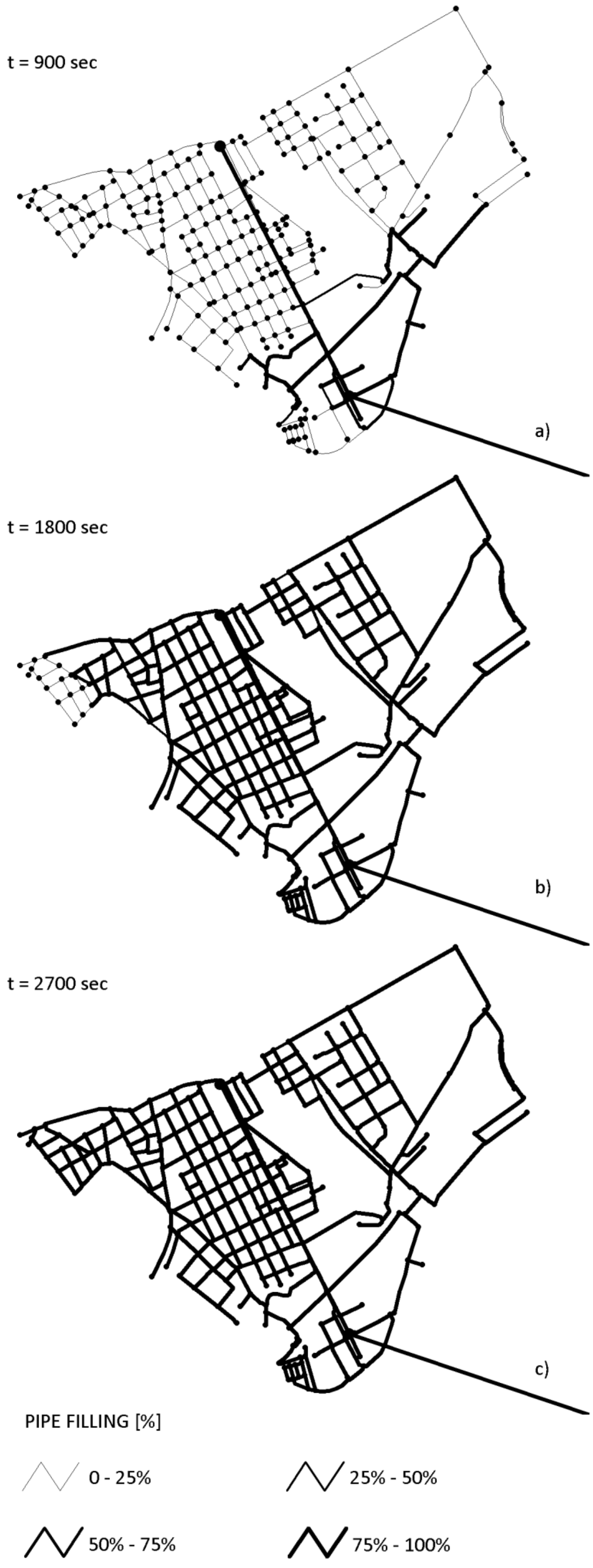

Figure 7. Dynamics of pipe filling in the network

5 CONCLUSIONS

The present paper discussed the development and the application of a numerical model able to simulate the initial filling process of a distribution network managed via intermittent supply. The problem is complex because of the highly dynamic conditions of the network and the dependence of network behaviour on the filling process of local reservoirs installed by the users. Such tanks are used for reducing the vulnerability of the users to intermittent distribution but they are also the main cause that does not allow the network to fill up unless some of the users have stopped filling their local tanks.

The paper has showed the reliability of the proposed model by means of an application to a real monitored case study.

The model application demonstrated the relevant negative impacts that depends on intermittent supply and the inequalities that intermittent distribution generate among users.

Future steps of the research can be aimed to the model application in order to evaluate network management practices alternative to intermittent distribution, such as Pressure Management Areas, that can solve the problem of network management under water scarcity without the negative effects provided by service intermittency.

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